TUBULAR STRUCTURES

4th INTERNATIONAL SYMPOSIUM, DELFT 1991

Edited by: J. Wardenier and E. Panjeh Shahi

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Subcommission XV-E

Comité International pour
le Développement et l'Etude de
la Construction Tubulaire (CIDECT)

Delft University of Technology
Faculty of Civil Engineering

INO Building and Construction Research
TUBULAR STRUCTURES

4th INTERNATIONAL SYMPOSIUM, DELFT 1991
This volume consists of papers presented at the 4th International Symposium on Tubular Structures held in Delft, The Netherlands, 26, 27 and 28 June, 1991, organised under the auspices of the Department of Steel and Timber Structures, Faculty of Civil Engineering, Delft University of Technology.

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TUBULAR STRUCTURES

The 4th International Symposium

Edited by

J. Wardenier
E. Panjeh Shahi

Department of Steel and Timber Structures
Faculty of Civil Engineering
Delft University of Technology
Delft, The Netherlands

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PREFACE

The structural hollow section has proved to be an excellent element for a wide range of applications e.g. industrial buildings, towers, masts, jibs, offshore platforms, barriers, and also for other fields such as mechanical equipment, agricultural applications, etc. Initially, its use was restricted due to the limited evidence regarding the complicated behaviour of the joints. Over the last 25 years many research programmes have been carried out. As a result the use of hollow sections has been increased enormously. Nowadays current design codes for steel structures nearly all include a chapter on tubular structures. The research has evolved from experiments and analytical models for simple joints to advanced numerical simulation for joints which have an extremely complicated load transfer.

Research is concentrated on fabrication and protection friendly designs. Due to the interaction between research and practice new applications are possible. The use of robot welding in future and the increased interest of architects for steel structures will create new designs and applications.

There is excellent exchange of information and international cooperation in research between members of Subcommission XV-E "Welded Joints in Tubular Structures" of the International Institute of Welding (IIW) and the members of the Comité International pour le Développement et l'Étude de la Construction Tubulaire (CIDECT).

Subcommission XV-E concentrates on the exchange of information, international coordination and the development of international design recommendations, whereas CIDECT initiates and sponsors many research programmes in the field of Tubular Structures.

Since its inception as a study group in 1964 and as a Subcommission in 1968 under the chairmanship of Prof. Toprac from the USA, a worldwide collection of data on circular hollow sections joints has been made. The information flow can be recognised in the earlier API and AWS recommendations.

Under the chairmanship of Prof. Hauk from the FRG (1973-1981) a direct coordination was made with CIDECT since he was also chairman of the Technical Commission of CIDECT. In this period, all test evidence on joints in rectangular hollow sections and on joints between hollow section braces and I, H and channel section chords was collected and analysed.

Under my chairmanship (1981-1991) the subcommission has presented the "Design Recommendations for Hollow Section Joints under Predominantly Statically Loading" and a draft for the "Recommended Fatigue Design Procedure for Hollow Section Joints".

These recommendations have served as a basis for uniform design recommendations in Eurocode and many national standards.

After this symposium Subcommission IIW-XV-E will be chaired by Prof. Packer from Canada. Work will concentrate on multiplanar joints under static and fatigue loading, concrete filled chords, etc.

Since its foundation in 1962, CIDECT, as an international organization of major
hollow section manufacturers has initiated and sponsored many research programmes
in the field of stability, fire protection, wind loading, composite structures and the
static and fatigue behaviour of joints.
In recent years under the chairmanship in the Technical Commission by Mr. Dutta
and the presidency of Mr. Ehlers from the FRG, CIDECT emphasises on the
dissemination of information to architects, structural engineers and constructors.

After the first international IIW Conference on Tubular Structures in Boston, 1984
it was felt that there was a need for such a conference every two or three years, in
combination with the Annual Assembly of the IIW.
Prof. Kurobane was the stimulator to have a second successful international meeting
on Safety Criteria of Tubular Structures in Tokyo, 1986. A third successful
Symposium was organized by Prof. Niemi and Prof. Makelainen in Lappeenranta,
1989.
This fourth symposium on Tubular Structures in 1991 is located in Delft, in
connection with the IIW Annual Assembly in The Hague.

As a result of the excellent cooperation between IIW-XV-E and CIDECT and the
common interests, this fourth symposium is organised by both organizations in
combination with Delft University of Technology and TNO Building and Construction
Research.
With this combination, the programme committee of this symposium has provided an
attractive programme with a balance between research and application.
From the 71 abstracts received, the international programme committee accepted 48
papers for presentation.

As chairman of the National Organising Committee, I wish to express my
appreciation to the authors, session chairmen and programme committee members
for all their hard work.

Further, I would like to express my sincere thanks to the opening speakers i.e.
Mr. Boereman, Chairman of the Executive Board of Delft University of Technology,
Mr. Salkin, Past President of the IIW and Mr. Ehlers, President of CIDECT.
Last but not least, I like to express my sincere appreciation to the sponsors of this
symposium and to Mr. Panjeh Shahi and Mrs. Van Paassen who did an enormous
amount of work in the organization of this symposium.

Prof. dr. J. Wardenier
Chairman of the National Organising Committee
Delft, June 1991
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Load carrying structures made of hollow sections for the free port in Duisburg

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Abstract

The construction of a free port in Duisburg led to an intensive building activity. This paper reports on the erection of two large warehouses and the buildings for customs clearance. The steel structures consist mainly of hollow sections. The dimensioning and design of the hollow section structures as well as the reasons for their selection and other possible alternatives are described in detail.

1. Introduction

The first European inland free trade port is presently being built in the port of Duisburg-Ruhrort. In the framework of this building project two large warehouses with an intermediate passage for customs were erected. The whole structure is about 400 m long and 52 m wide. Fig. 1 gives a general view of it.

As the buildings are to represent a landmark for the city of Duisburg, situated on the rivers Rhine and Ruhr, special requirements had to be fulfilled with respect to the architectural design. Besides, the economy in fabrication and maintenance, erection time as short as possible and flexible use of the structures had high priorities.

2. Warehouses 1 and 2

Seven pylons were erected in imitation of the bridges over the Rhine in Duisburg and the roofs of the warehouses were hung on them with inclined tension members. Fig. 2 shows warehouse 1 with its four pylons, guys and the arrangement of roof girders.

The pylons are made of circular steel tubes of 1.50 m diameter and panels of 20 mm thickness. Their height is 28 m in total, hereby exceeding that of the roof ridge by about 15 m (fig. 3). For guying, steel rods of 63 mm diameter made of St 52 are used. The roof truss girders have inclined upper chords. They are made of hollow sections. The lattice girders will be dealt with later in detail (fig. 4, 5). A plate girder was arranged in longitudinal direction at the roof ridge for the purpose of connecting guys with trusses (fig. 5). The outer columns and bay rails are made of open rolled sections. In total 1300 tons of constructional steel have been used for the warehouses 1 and 2. The panels for the roofs and walls are made of trapezoidal steel plates with additional thermal insulation.
3.1. Statical concept and load transmission

The roof construction is made of trapezoidally profiled plates (TS 131 x 0.88), which rest directly on the lattice girders with a spacing of 5.5 m without purlins (fig. 4). They transmit the vertical loads at a maximum overall height of 3.0 m over a span of 40 m in total into the outer columns and nearly centrically into the guyed ridge beam. For the trusses of hollow sections, this results in a double-span-system (15 and 25 m span) with a cantilever of 11.0 m over the loading ramps (see fig. 3, 4).

Due to the soft guying of the ridge beam the middle support of the trusses has to be taken as weak elastic. An effort was made to use identical trusses for a production in series, thus design calculation could be reduced to that for a single truss. In doing so, two extreme values for the spring rigidity (max./min. cv) were chosen to cover the varying stiffnesses of the middle support due to the different courses of the guy-ropes and the non-linear effects of these ropes. Based on these values the cross sectional forces were calculated. The forces were determined assuming an idealized truss (pin joints) with a continuous chord. The maximum forces in each single member of a truss had to correspond to the design forces derived in accordance with the loadcases given in DIN 1055 (dead weight, installation p = 0.10 kN/m², snow, wind pressure and wind undertow). A complete multi-planner calculation was only done after the determination of the stiffnesses of all structural members (truss, ropes, pylons) in order to check the assumed spring rigidities.

3.2. Lattice trusses for the roof

3.2.1. General

Dimensioning of the truss members made of rectangular hollow sections [4] was done based on the following aspects:

- minimum weight of the roof construction
- filigree construction should feature a high inherent stability (lateral torsional buckling and overall buckling)
- Simple fabrication with low effort of shop welding
- Less expense for prime coat and final painting (latter to be done on site)
- Optical and architectural aspects.

The dimensioning for the lattice trusses of hollow sections led to about 30 % less dead weight than for those with standard open rolled sections (the dead weight of the roof construction is 0.3 kN/m²). Application of the steel St 52 was also more advantageous than doing so with open sections due to the better slenderness ratio of the hollow sections. The resulting weight of about 2.2 t of a 25 m long truss was easy to lift during the erection. The chords of the trusses of square hollow sections 160 x 6.3 in St 52-3 were made
without graduated dimensions. The diagonals had two different cross sections (MSH 60 x 4 to MSH 100 x 4) corresponding to the design forces and moments.

Since the both values considered for the spring rigidity led to relevant design forces with different signs in the diagonals, the hollow sections were also suitable to take up low compression in members, which are mainly designed for tension. The design with square hollow sections under compression is economically efficient (lower cross sectional area and less weight) due to better cross sectional values of hollow sections (larger radius of inertia in both axes). In addition, it is to be mentioned that the hot formed rectangular hollow section used feature higher static values with lower corner radius than the cold-formed hollow sections [4]. Moreover, the hot formed hollow sections have lower residual stresses and are therefore classified for the more favourable buckling curve (a) according to Eurocode 3 draft [1] and DIN 18800, part 2 [2].

The Fabrication (= welding) of the lattice trusses took place on a shop board after the chords and bracing were cut to the given sizes. The welding of the bracings to the chords was done with weld thicknesses according to the static requirements, i.e. thicknesses of 3.5 mm and 4.5 mm were used corresponding to the small wall thicknesses of the hollow sections (see fig. 6).

The surface area of the hollow section structure to be painted was by 55 % lower than that for a comparable construction made of open cross sections. This led to remarkable savings in costs, especially because of the customer's demand for the high quality type of prime coat and final paint. Protection against corrosion on the inner surface of the hollow sections is not necessary in case of airtight welds, since the oxygen within the hollow sections wears off and corrosion cannot continue further. The airtightness was guaranteed by welding end plates to the hollow sections. The problem of the assembly of dirt in the edges and corners of the open cross sections, which was of concern to the customer, is not significant for hollow sections.

An opposed to these advantages in selection and application of rectangular hollow sections, the material purchasing costs are higher. On the German market the price of the hollow sections is higher by about 50 % than that for open profiles. Still the shown roof construction with hollow sections was more economical due to the addition of the factors - less material weight, easier fabrication and lower painting costs. Besides by the intended rationalization through production in series, the fabrication costs were mainly determined by the joint design of the structure. Regarding this a number of considerations made in the course of the execution of this job may surely be applicable to other similar constructions.
3.2.2. Connections and constructional aspects

Diagonals on chords

The welded joints of rectangular hollow sections were designed following DIN 18808 [3], the German standard for the design and calculation of hollow section structures under predominantly static loading which appeared in 1984. In order to attain a simple and economical fitting cut of the diagonals (single inclined saw-cut) overlap of the diagonals was avoided. The simple and rapid manual welding of the diagonals to the chord required moreover, a systematic gap of at least 20 mm between the toes of the diagonals (see fig. 7).

This resulted in an eccentric intersection of the diagonal axes to the chord axis. The verification of the unstiffened welded joints is made according to DIN 18808 by the determination of the required wall thickness ratio of the chord to the diagonal. In doing so, the maximum eccentricities of 1/4 of the hollow section depth (here 40 mm) was also taken into account. In some cases, the designed gaps led to exceeding this eccentricity, so that an additional moment bending the chord (maximum was about 750 kN/cm) had to be considered (additional bending stress: 40 N/mm²). (Distribution of the moment on the diagonal and chord corresponding to their stiffness ratios would have brought a small share to the chord. Therefore, this moment was neglected for this construction).

The fillet welds were applied according to DIN 18808 with a throat thickness $a = t$ (wall thickness of hollow section) and $a = \text{reduced } t$ for not exploited hollow sections. A number of bracing joints was also verified by directly reducing the weld-thickness $a$ according to the transversal force and the effective area of the weld seam of section.

Bracing connections to the chords

The design of rectangular hollow section bracings jointed to chord, as shown in fig. 8, is a bit more complicated that that with open sections, as in general, additional moments occur due to the eccentricities. They have to be taken into consideration in the design, which means that reserve load bearing capacity must be available to take this additional moment.

The centrical connection (fig. 8a) with a slotted rectangular hollow section and a gusset plate punched through it, is expensive and time consuming in fabrication. Gusset plate in the middle of one side of the chord (fig. 8b) can only transfer shear forces and lead to transverse bending stresses. More efficient connections (fig. 8c) lead to additional bending or torsional stresses in the hollow sections, depending on the forces in the bracing. In general the eccentricities required while using hollow section have to be considered in the static calculation and the effects have to be verified.
as being covered by the existing stress reserves.

Rectangular hollow section welded to end plate
(especially axial force transfer through the end plate)

The means of transmission of axial force by a end plate with high strength bolts can also be applied to rectangular hollow sections. In the ideal case, the end plates protrude to all sides, so that the forces from all four walls of a rectangular hollow section can be transferred directly (similar to a pipe flange connection). In the construction here, the end plates could only protrude to two sides, so that end plate and bolts could transfer the forces in the webs of the hollow section. Verification for the end plate and bolts were made based on the plastic design given in the "DStV-Ringbuch" [5] for rigid connections of end plates to open cross sections. In doing so, the tension flange of the open section (flush end plate) could be compared to the web of the rectangular hollow sections (see fig. 9).

Application of the method described in the "DStV-Ringbuch" for any profile and other than St 37 material was made taking the following into consideration:

- No increase of the plastic moment by 10 %
- Limitation of the bolt strength according to DIN 18800
- Determination of the centre of the normal force by means of the stressed area of the hollow section (fig. 9)

4. Summary

The construction of the warehouses at the free trade port in Duisburg using rectangular hollow sections provides a filigree and aesthetical structure, which moreover fulfills the modern requirements on fabrication and maintenance. Therefore this concept of using lattice girders of rectangular hollow sections was also realized for the roof construction of the customs area (fig. 10, 11). The statical and constructional requirements are in no way more complicated than those for conventional truss constructions.

5. References
fig.1: general view of the warehouses and the between custom area

fig.2: warehouse no.1

fig.3: cross section, static system
fig. 4: total view

fig. 5: inside view during erection
fig.6: fabrication (shop welding)

fig.7: connection diagonal - chord
a) centrical by punched through plate

b) excentrical (and not suitable for normal forces)

c) excentrical (or centrical)

fig.8: Connection of bracings' to the chord

fig.9: End plate connection

fig.10: Detail of roof construction in custom area
fig. 11: roof over custom area
AN ECONOMIC COMPARISON OF SINGLE CHORD AND DOUBLE CHORD RHS WARREN TRUSSES

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Summary

The separated double chord RHS Warren truss architecture is demonstrated to be a cost effective configuration for use in the design of medium and long span roof and floor truss systems. The design of 57 RHS trusses of single, back-to-back double, and separated double chord architectures has been undertaken using current, state-of-the-art, computer software packages available for use within the Canadian structural design community and a summary of these designs is presented. The designs are based on typical floor and roof loadings for Southern Ontario, Canada, and all member and joint detailing is carried out in a manner consistent with current Canadian design codes using the limit states design philosophy. Individual truss designs are categorized based on design parameters of interest including the depth of truss, the span-to-depth ratio, the bay width, and the number of panels. Material and fabrication cost estimates for the construction of these trusses are incorporated into an economic analysis and a comparison of the cost effectiveness of the various configurations is presented. Summary level output including the number and type of joints, the net steel mass of the truss, maximum live load deflections, and estimated truss costs are reported for each truss considered. The results indicate that for spans of 40 metres and longer, and span/depth ratios of 15 or higher, the separated double chord architecture provides superior economy and structural performance. In many longer span, or higher load, applications, the separated architecture is shown to be the only configuration capable of carrying the factored dead and live loads applied in the current investigation.

1. Introduction

All designs presented and analyzed in the paper are based on designs produced using the Canadian Institute of Steel Construction’s (CISC’s) PCTRAD program and the Comité International Pour Le Développement et l’Étude de la Construction Tubulaire’s (CIDECT’s) DCTRUSS package. The PCTRAD program is a general purpose steel truss design and analysis package capable of generating designs using single chord or back-to-back double chord RHS configurations. DCTRUSS (release 1.2), by comparison, is a special purpose package designed specifically to assist the practising engineer in the design and analysis of separated double chord RHS Warren trusses. A brief review and comparison of these two software packages is presented in the next section and their design capabilities and limitations are outlined.

Following an introduction of the software packages used, the methodology employed in the current study is discussed. In particular, this section of
the paper introduces the 13 geometrical truss configurations, three (3) loading conditions, and four (4) connection details employed in the design and analysis of the 57 RHS Warren trusses considered in this study. The 57 trusses consist of 19 designs for each of the single chord, back-to-back double chord, and separated double chord architectures. Following this discussion of study methodology, the results of the various truss designs are reported and conclusions are presented.

2. The PCTRAD and DCTRUS Software Packages

The PCTRAD package has been described as an interactive, user-friendly, general purpose steel truss design program. Of particular interest to the designer of HSS trusses, the PCTRAD package includes the capability to design and analyze trusses of single and back-to-back double-chord HSS construction. Additionally, the package will investigate the adequacy of the connections in single chord HSS truss configurations for those joints which conform to Group 2 truss connections as defined by CIDECT Monograph #6 and the Canadian Implementation of CIDECT Monograph #6 (See also CIDECT report No. JAJ-84/9.E). Group 2 connections are defined as being gussetless welded HSS connections occurring in trusses with rectangular HSS chord members and rectangular or round HSS web members. The package does not, at present, have the capability to check the adequacy of the connections in back-to-back HSS double chord truss designs (C.S.C.C., [1]).

Developed under sponsorship from CIDECT (Project #5AT), the current version of DCTRUS is specifically tailored to the design of separated double chord RHS Warren trusses using rectangular and square HSS sections available from Canadian mills. As such, the package complements the capability of the PCTRAD package to produce designs of single chord and back-to-back double chord geometries using HSS sections. As a specific purpose program, however, the DCTRUS package provides more extensive design guidance to the practising engineer than does the PCTRAD package. The single chord joint check algorithm employed in the PCTRAD package provides for verification of the suitability of a given connection based on optimized member selection procedures. At present, a warning is issued by PCTRAD if the joint requires stiffening but no further capability is present within the package to allow for a redesign based on an optimization of the connection (C.S.C.C., [1]).

By contrast, the DCTRUS package provides for both member and joint design optimization. In the event that a joint capacity is insufficient to carry the desired loads based on an optimized member selection procedure, a message is provided to indicate that joint stiffening will be required and additional cycles of design and analysis are undertaken to produce a design which has sufficient joint strength. As the joint strength will be dependant on the members present at the connection, this additional optimization will result in a new designation of members. Thus, the DCTRUS output will allow the practising engineer to consider the relative costs of reinforcing the connections at an optimized member sizing and of increasing the member sizings to provide adequate strength at the connections (Luft, [2]).

Both DCTRUS and PCTRAD employ a graphic user interface to assist the user in the generation of input data. This interface has been designed to ensure that the two packages have a consistent look and feel to them. Unlike the PCTRAD package (which was originally designed to run on main frame computer systems), the DCTRUS package has been designed specifically to operate on
personal computers. As a result, the DCTRUSS package runs at speeds approaching 15 times those of PCTRAD. Both software packages require only basic loading and geometry inputs in order to generate member (and joint in the case of DCTRUSS) designs for Warren truss architectures. The resulting designs are optimized based on the member weights (i.e. the lightest sections available are selected) with all sections selected from the RHS tables provided in the Canadian Institute of Steel Construction’s (CISC’s) Handbook of Steel Construction ([1] and [2]).

The two packages are based on the CAN3-S16.1 Limit States Design Code and permit the designer to check the ultimate strength limit of members and joints, and verify that serviceability is satisfied. The packages accept specified and factored dead and live loads, and uplift wind loads. All loads can be identified as point, uniformly distributed, or uniformly varying loads applied to either the top or bottom chords. Both DCTRUSS and PCTRAD automatically generate simple geometries of simply-supported and statically indeterminate Warren trusses with parallel chords or pitched top chords. Basic geometry input such as truss span, depth, and number of panels is required. Program output includes critical load cases and combinations for both member and joint designs, critical design forces, interaction ratios for members (and joints in DCTRUSS), and members selected for all locations. Joint welding specifications are also provided by the DCTRUSS package ([1] and [2]).

3. Methodology

Having reviewed the capabilities of the PCTRAD and DCTRUSS software packages, the discussion will now focus on how these packages have been employed in the current study to compare, on a cost basis, the relative capabilities of single, back-to-back double, and separated double chord Warren trusses to carry typical floor and roof loadings when applied in medium and long span applications. In order to provide a basis for comparison, the investigation was limited to single chords employing gapped or stiffened joints, back-to-back double chords employing stiffened gap joints, and separated double chords. Figure 1 shows a typical Warren truss along with the four connection details of interest in the current study.

For cost estimation purposes, designs of the four typical joints (with the dimensions shown in Figure 1) were submitted to three (3) local fabricators and quotes were requested for these isolated joints. Additionally, a median cost for 350W Class H HSS steel was obtained from a local steel mill. The resulting cost estimates are reported in the “Notes to Tables 2, 3, and 4”. Cost estimates were obtained for each truss by multiplying the number of joints of each type by the appropriate typical joint cost and adding to this total the cost of the steel in the truss (based on the net steel mass of the truss).

Table 1 lists the 13 truss geometries designed as a part of the current study. As a basis for discussion, a base case truss was selected (truss 94-B7-SD20-P10 in Table 1) and designed in each of the three (3) configurations of interest: single chord, back-to-back double chord, and separated double chord. The base truss is simply supported at the extremes of the top chord member, has a span of 40 metres, a depth of 2 metres (span/depth ratio = 20), a bay width of 5 metres, 10 panels along the top chord, out-of-plane supports at 10 metre intervals, and unfactored dead and live loads of 2.4 kiloPascals each. All loadings are applied as uniformly distributed loads.
along the top chord of the truss. The bay width parameter was allowed to vary between 2 metres and 10 metres for the three truss configurations indicated in Table 1 in order to assess the impact of changes in loading on the choice of truss architecture. This variation in effect allowed for a comparison of a lightly loaded structure (bay width = 2 metres), a typically loaded structure (bay width = 5 metres), and a heavily loaded structure (bay width = 10 metres). As a result of the 13 geometrical configurations, and the additional six loading conditions (3 each for the lightly and heavily loaded cases), a total of 19 designs for each truss were attempted.

Referring to Table 1, it will be noted that three spans are considered (20 metres, 40 metres, and 60 metres), span/depth ratios are permitted to range between 10.0 and 40.0, and the number of top chord panels may vary from 5 through 15 (with an accompanying change in web angle between 2:1 and 1:2). Of particular note, a series of seven of the truss configurations has been designed to assess the impact of the span/depth ratio on the choice of truss. For this series (refer to Table 1) each truss has a span of 40.0 metres, 10 panels, and a 5.0 metre bay width. Only the depth, and consequently the span/depth ratio, is permitted to vary within this group. This series then provides a good cross-section of data on which to base conclusions as to the impact of the span/depth ratio on the choice of architecture in medium to long span RHS Warren truss configurations.

4. Results

Summary level output from the 19 designs attempted are presented in Tables 2, 3, and 4 for the single chord, back-to-back double chord, and separated double chord trusses, respectively. In reviewing these tables, it should be noted that only those trusses which were technically feasible are included. Trusses for which members could not be designed due to member size limitations or local (member) instability are not reported. In particular, it should be noted that only 9 of the 19 truss designs attempted using the single chord architecture were successful. By contrast, 15 of the 19 back-to-back trusses were successfully designed, and 17 of the 19 separated double chord truss designs were successful. In the latter case, one of the two failures was due to inadequate joint strength (D6-B5-SD30-P15) as opposed to the member failures typically witnessed in the single chord and back-to-back double chord trusses. Of interest is the fact that no single chord truss design employing a span/depth ratio of greater than 20 could be successfully implemented (and those successfully designed with a span/depth ratio of 20 were all lightly loaded). By contrast, the back-to-back truss failures occurred only for the most extreme span/depth ratio (span/depth = 40), loadings (bay width = 10 metres), and spans (span = 60 metres).

Figures 2 and 3 plot the output from the special series of seven trusses which were designed to analyze the impact of the span/depth ratio on the truss design. Figure 2 plots the net steel mass of the truss versus the span/depth ratio while Figure 3 reports the estimated truss cost versus span/depth ratio results. In examining the results, the inability of the single chord truss to resist loadings for span/depth ratios of greater than 20 is clearly seen (only two of the seven single chord truss designs attempted were successful). For short to medium span trusses, or lightly loaded trusses, however, the single chord truss appears to be very competitive both in terms of structural performance and cost. For longer span trusses, or heavier loads, the separated double chord truss appears to
be a more economical and structurally more efficient architecture than does the back-to-back double chord architecture. Having said this, however, it should be noted that a review of Tables 3 and 4 reveals that the back-to-back double chord architecture does appear to be 10 to 15 percent stiffer than its separated double chord counterpart. This is likely due to the very conservative adjustment due to the presence of flexible joints which the DCTRUS program employs in calculating the maximum deflection of the truss (Luft, [3]).

A review of the remaining data in Tables 2, 3, and 4 reveals that the cost advantage of the separated double chord truss is magnified for heavily loaded structures while there is very little advantage associated with the selection of a particular architecture for lightly loaded structures. Similarly, the number of panels in the top chord does not appear to be a significant factor in the choice of architecture. Rather, there is a cost tradeoff between increased member sizes when fewer panels are selected and increased joint costs when many panels are selected.

5. Conclusions

In conclusion, the separated double chord Warren truss architecture outperforms the single and back-to-back double chord types at high span/depth ratios and under severe loading conditions. The separated architecture appears to be more robust and more economical than its back-to-back counterpart. Additionally, the separated architecture enjoys an innate out-of-plane stability of the compression chord which minimizes the need for lateral support, although this parameter was not considered in the current study. The only potential problem with this new architecture is its flexibility, although estimates of local deformation effects are likely conservative. It may be, in fact, that the separated double chord configuration is as stiff as the back-to-back configuration, particularly for long span applications. More experimental data is, however, required before a reduction in the joint adjustment factor (currently at 15%) can be implemented within the DCTRUS program.

In summary, the separated double chord architecture performs in a superior fashion when compared to other hollow structural section alternatives for medium to long span applications with truss span/depth ratios of 15 or higher. Similarly, the separated architecture is the most economical solution for heavily loaded structures. By contrast, the single chord truss configuration appears to perform as well or better than the other configurations considered for shorter spans or more lightly loaded configurations. Due to member strength considerations, however, a single chord architecture is not likely viable at span/depth ratios above 20.

6. References


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Notes to Table 1:

1. The truss designation reflects the span, loading, span/depth ratio, and the number of panels in the truss configuration. The first letter in the designation (shown as a '?' above) will be either a 'S', a 'B', or a 'D'; representing a Single chord truss, a Back-to-back double chord truss, and a separated Double chord truss, respectively. The 2, 4, or 6 which follows the first letter represents the span/10. The second term in the configuration refers to the loading on the truss and represents the assumed bay width. Three bay widths were considered in the current study, 2m, 5m, and 10m; B? indicates a truss type for which all three loadings were tested. B5 signifies a bay width of 5m. The SD## identifies the span/depth ratio while the P## indicates the number of panels in the top chord of the truss.

2. Unfactored dead and live loads of 2.4 kPa were used as a typical loading for all trusses considered in the current study. These loads have been multiplied by the bay width of the truss, factored using the appropriate load factors and combinations from the National Building Code of Canada, and applied as a uniformly distributed loading (UDL) across the top chord of the truss.

3. All trusses considered in the current study have been designed as being simply supported at the extremes of the top chord.

4. All trusses are assumed to be supported out-of-plane at 10 metre intervals.
### TABLE 2: Single Chord Truss Results

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<th>Cost-Mat'1 ($)</th>
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### TABLE 3: Back-to-Back Double Chord Truss Results

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<td>19282</td>
<td>20441</td>
</tr>
</tbody>
</table>

**Notes to Tables 2, 3, and 4:**

1. Joint costs are calculated based on fabricator's estimates for the four (4) joints shown in Figure 1. The gapped single chord joint cost used is $40/joint, the reinforced single chord joint cost is $95/joint, the back-to-back reinforced double chord joint cost is $120/joint, and the separated double chord joint cost is $61/joint.

2. Material costs are based on a quoted estimate of $900/ton, or approximately $0.89/kg, for 350W Class H steel.

3. The delta parameter in the span/delta deflection ratio reported is the maximum deflection which occurs along the truss chord based on the unfactored live load only. It is assumed that the deflection due to unfactored dead loads will be offset by camber during the construction of all trusses.

4. Nineteen (19) trusses of each type were designed. Those truss results reported in Tables 2, 3, and 4 are for the truss designs which were technically feasible. Single, back-to-back, and separated double chord truss failures are not reported.
(a) TYPICAL WARREN TRUSSES
- 203x203x10 H.S.S. WEB MEMBERS
- 254x254x12 H.S.S. CHORDS

(b) SINGLE CHORD UNREINFORCED K-JOINT
- 203x203x10 H.S.S. WEB MEMBERS
- 250x20 PLATEx650
- 254x254x12 H.S.S. CHORDS

(c) SINGLE CHORD REINFORCED K-JOINT
- 203x203x10 H.S.S. WEB MEMBERS
- 275x12 PLATEx700
- 2-152x152x6 H.S.S. CHORDS

(d) DOUBLE BACK TO BACK CHORD REINFORCED K-JOINT
- 203x203x10 H.S.S. WEB MEMBERS
- 2-152x152x10 H.S.S. CHORDS

(e) SEPARATED DOUBLE CHORD K-JOINT

FIGURE 1: TYPICAL STUDY CONFIGURATIONS
For a 5m bay width, 10 panels
And a 40m span

FIGURE 2: STEEL MASS VS. SPAN/DEPTH RATIO

FIGURE 3: TRUSS COST VS. SPAN/DEPTH RATIO
Summary

The CUBIC Composite Floor is a development of the CUBIC Space Frame in which the upper grid of chords is embedded in, and strengthened by, a layer of concrete, thereby permitting composite action without the need for conventional shear connectors.

The paper introduces the concept and discusses the structural action, including the force transfer between the rectangular hollow section posts and the concrete layer. The results of a test to destruction on a full scale prototype floor panel, which was carried out at Nottingham Polytechnic, UK, are also discussed.

1. Introduction

The CUBIC Space Frame is a modular structural steel frame without diagonal bracing members. The frame has been used in the construction of buildings with a wide range of spans and loading conditions [1][2]. Generally it comprises an upper and lower grid of I section chords which are connected together by rectangular hollow section posts (Figure 1). The behaviour of the connection between the chords and the posts was the subject of a paper in the Third International Symposium on Tubular Structures [3].

In a development of the original idea, the upper grid of chords has been embedded in, and strengthened by, a layer of concrete, thereby permitting composite action without the need for conventional shear connectors [4]. The resulting composite action enables savings to be made in the weight of the steel, whilst minimising the structural depth and maximising the available space for service installation. These features are particularly relevant to the highly serviced, minimum depth floors which are frequently sought for commercial buildings today.

Recommendations for the design of steel/concrete composite beams are given in design codes such as the recently published BS 5950: Part 3: Section 3.1 [5]. Not only are the recommendations restricted to beams, but they also assume that the concrete slab is located on top of the steel beam. The slab is positively connected to the steel by some kind of welded shear connector. In order to assess the relevance of such recommendations to the CUBIC Composite Floor and to provide greater understanding of the structural behaviour of this type
of construction, an extensive test programme was initiated at Nottingham Polytechnic, including the large scale prototype testing which is the subject of this paper.

2. The CUBIC Composite Floor

The CUBIC Composite Floor is a three dimensional structure, which uses the typical prefabricated steel modules shown in Figure 1 as its basic building blocks. The modules are bolted together to form a steel structure which is capable of supporting all construction loads. Galvanised steel decking is then positioned on the bottom flange of the top chords to act as a permanent shutter for the concrete slab (Figure 2). A light reinforcing mesh is laid over the top chords to provide crack resistance. Finally, concrete is cast, embedding the top chord and providing a concrete cover of typically 30mm to the top of the steel frame. No shear studs are necessary, all interaction between steel and concrete occurring at the embedded nodes.

During construction the steel structure has to support the decking, reinforcement and wet concrete loads, in addition to the usual construction imposed loads. However, once the concrete has hardened, all subsequent loads which are applied to the floor are resisted partly by the steel and partly by the concrete. Since the concrete only enhances the strength of the top chord, the greatest savings in steel weight can be achieved by adopting a smaller section size for the top chord of the steel structure than for the bottom chord.

3. The Test Frame

The test frame represents a full size structure of a typical commercial floor with a grid size of 1.5m, an overall depth of 890mm, and a span of 6.75m x 6.75m between supports (Figure 3). It was designed as a space frame for an unfactored inclusive load of 3kN/m² at the casting stage of the concrete slab and 8.7kN/m² in service when acting compositely with the hydrated concrete slab.

To investigate the behaviour of different types of edge conditions, the test frame was built with only one axis of symmetry along one of the diagonals (A1 to F6). Two of the edges are trimmed by Vierendeel girders while the other two edges are formed by standard CUBIC Space Frame side modules, (Figure 3).

The frame was supported at each corner on 200kN load cells with a ball joint between the frame and load cell. After erection of the steelwork the Lytag reinforced concrete slab was cast on profiled permanent steel decking to a maximum depth of 130mm. The concrete had a target 28 day cube strength of 30N/mm² and a dry density of 19kN/m³, although the actual 28 day cube strength was found to be 37N/mm². The top chords of the two sides trimming the structure with the CUBIC Space Frame side modules were embedded in concrete, the top chords of the two Vierendeel girders were not embedded.
The test frame was instrumented with strain gauges (see Figure 4) and linear voltage displacement transducers (LVDT). Optical markers were provided for precision optical surveys to be carried out during the testing of the structure.

4. Test Procedure and Results

The frame was loaded with a layer of sand bags onto which 1 metre long cast steel billets, each weighing 100 kg, were laid in a chequer board pattern (see Figure 5). The load was monitored using the load cells at the points of support.

The test programme consisted of four stages which are described below. Stages 1, 2 and 4 were in accordance to BS5950: Part 1 Section 7 [6], and represent respectively a stiffness test, a strength test and a test to failure.

Stage 3 was included to record the effects of inducing negative bending moments into the frame as could occur at internal column positions.

**STAGE 1.** 1.0 Dead load + 1.25 Imposed load. The 35 tonne applied load was maintained for 24 hours and then removed, in accordance with the procedure in BS5950 [6]. Since the deflection of the frame failed to recover by the 80% required in BS5950, the same load was applied again for a further 24 hours. On removal of this load a recovery of 95% was recorded in deflection.

**STAGE 2.** 1.4 Dead load + 1.6 Imposed load. The load was applied in stages to obtain some load-deflection curves. After the maximum applied load of 48 tonnes had been maintained for 24 hours the frame was unloaded in a single step. A typical load-deflection graph for node C3 is given in Figure 6, with further results given in reference [7].

**STAGE 3.** Reverse load application through upward jacking at node C3. For this test the frame was loaded as in Stage 2, with weights in the form of sand bags and cast steel billets. An hydraulic jack positioned under node C3, was then used to apply an upward force to the frame until the deflection at C3 was zero. The upward load of 25 tonnes was then removed. Some residual deformation of the frame in the region of node C3 was evident from the deflection measurements.

**STAGE 4.** Imposed downward load to failure of the structure. An initial load of some 65 tonne was applied in stages using sand and billets. The additional load which was required to produce failure was applied using nine hydraulic jacks positioned above the frame. The frame failed at a total applied load of 91.5 tonnes, as a result of ductile splice plate failure in the bottom chord at grid location A/3-4. The bottom chord splice bolts at grid locations B/3-4 and C/3-4 also failed.
5. Structural Behaviour of the Test Frame

The test results have demonstrated that the concrete contributes significantly to both the stiffness and the strength of the CUBIC Composite Floor. For example, the recorded deflection of the composite floor at C3 under the first application of stage 1 loading was 21mm, whereas a value at least 40mm would have been expected if the steel frame alone had supported the load. Also, the applied load which was required to produce failure in a bottom chord splice joint was 91.5 tonnes, whereas the maximum plastic capacity of the steel frame using measured yield stresses would have resulted in failure at an applied load of no greater than 49 tonnes.

To better understand the behaviour of the composite floor under applied loads, the structure has been analysed using conventional space frame software, assuming a skeletal frame in three distinct ways:

(a) Steel frame resists all applied loads even after concrete has hardened.

(b) Part composite frame in which the concrete resists a proportion of the compressive load in the composite top chord only, all shears and moments in the top chords being resisted by the steel.

(c) Fully composite in which the concrete resists a portion of the shear, moment and compressive load in the composite top chord based on the gross concrete section (uncracked). This therefore results in a complex force transfer between the concrete slab and the steel framework, particularly at the embedded nodes.

In (b) and (c) a modular ratio of 15 has been assumed, along with an effective concrete width for each top chord member from centre of grid to centre of grid (typically 1.5 metres). Table 1 summarises the results of the analysis, from which it can be seen that the fully composite model most closely predicts the actual frame deflection. This conclusion is also supported by the preliminary comparisons of measured and predicted strains, which also suggest that the fully composite model gives close agreement to the actual results although this work has not yet been completed.

Based on experience from CUBIC Space Frame applications [2], the above calculations also include allowance for the effect of bolt embedment in the clearance holes of the splice plates, which primarily affects the frame deflection. The effect of this embedment was seen during the first application and removal of load in the Stage 1 test, when less than 80% recovery was obtained. Since this embedment had occurred during the first load application the subsequent load application and removal resulted in a 95% recovery. Figure 6 also illustrates this effect when the applied load was increased to 48 tonnes and the structure was unloaded.
As mentioned earlier the measured ultimate strength of the frame is significantly greater than the strength of the steel frame alone. The contribution of the concrete in resisting shear was therefore considerable, necessitating both force and moment transfer to the SHS posts.

A simple analysis to determine the limiting moment transfer between the concrete and SHS posts suggests that the complex stresses adjacent to the embedded nodes allows the concrete to exceed its normal compressive strength - otherwise there is theoretically insufficient capacity within the steel and the concrete to resist the load which was actually applied. Moreover, although some redistribution of load was observed during the Stage 4 test, there was no obvious distress to the concrete prior to failure of the bottom chord splice joints. Further investigation of the force transfer at these embedded nodes is currently being carried out by component testing.

6. Conclusions

The CUBIC Composite Floor is a development of the CUBIC Space Frame, which enables savings to be made in the weight of steel whilst minimising the structural depth and maximising the available space for installation of services. It is self supporting during construction and requires no added shear connectors.

A full scale, well instrumented, prototype frame has been extensively tested at Nottingham Polytechnic. The testing has been carried out to the recommendations of BS5950, with a final test to destruction. The results have demonstrated that the concrete contributes significantly to both the stiffness and strength of the CUBIC Composite Floor.

The test frame has been modelled analytically in a number of ways. Closest correlation with the experimental results has been found when the concrete is assumed to resist a proportion of the top chord shear, moment and compressive loads, based on the gross concrete section.

Further component tests are being carried out to examine the force transfer at the embedded nodes.

7. Acknowledgements

The research into the CUBIC Composite Floor is being carried out by the first author at Nottingham Polytechnic, UK, and is being sponsored by ASW-CUBIC Structures Ltd. The authors wish to thank Mr M L Kubik for the significant contribution which he has made to this research project.
8. References


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<td>(b) Part composite frame</td>
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<td>Test result</td>
<td>21.0</td>
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Table 1. Deflection at C3 node under 35 tonne applied load (First Application)

26
Figure 1. CUBIC Space Frame Module Types

Figure 2. Typical Section Through Test Rig
Plan of Frame

Vierendeel Girders

<table>
<thead>
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<th>1.5m</th>
<th>1.5m</th>
<th>1.5m</th>
<th>1.5m</th>
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890mm 740mm

O/ALL CRS

Bottom Chord

Top Chord

Elevation of Frame

- Splice Joints
- Vertical Members

Figure 3. Plan and Elevation of Steelwork
Plan of Floor Steelwork Showing Position of Strain Gauges

Enlarged Section Showing Strain Gauges

Figure 4.
Figure 5. Photograph of Test Frame Under Load

Figure 6. Graph of Load vs Deflection at Nodes
LONG SPAN BOOMS IN TUBULAR CONSTRUCTION

FOR CONVEYOR TRANSPORT OF OVERBURDEN

Diethelm K. Feder
Consulting Engineer

Summary

In recent years two long span tubular trusses have been built as structural components of spreaders which convey overburden across lignite mines in Texas.

The discharge booms of these machines cantilever more than 200 m. They consist of a tubular space lattice girder cable-stayed vertically and horizontally to a hoistable portal shaped mast located at the center of the spreader. The loading of the booms is governed by the compressive forces from the cable-staying, and bending moments in two planes from dead load and wind. Fluctuating stresses are caused by the changing material load on the belt conveyor, and vibrations induced by the travelling of the crawler mounted spreader.

The contribution informs on loading conditions and design load combinations, describes structural details of the tubular joints, and discusses the approaches used in the design calculations of the tubular joints for static and fatigue stresses.

1. Introduction

Lignite, also known as soft coal or brown coal, is a fuel that is mined in many parts of the world. The overwhelming portion is mined in open pit mines. Since the calorific value of lignite is much lower than that of black coal it is usually not shipped over long distances but used to feed power stations located close to the mines. The deposits of lignite originate from a geological period 30 to 50 million years ago, and are covered by overburden layers of Tertiary soils up to several hundred meters thickness. Deposits with thin covers of overburden having been exploited first, the overburden to coal ratio for many mines nowadays is in the order of magnitude of 5:1 and higher. To make mining the lignite still economic necessitates the use of very large machines which are able to move overburden masses between 100000 and 250000 cubic meters per day.

A very economic way is to transport the overburden directly across the pit, which in these cases has the cross-section of a ditch. Sometimes bridges on crawlers or rails are used for this purpose, but a big problem for them is the limited carrying capacity of the unconsolidated freshly dumped soil on the depleted side of the mine. Therefore a mining company in Texas has chosen to use so-called "cross-pit-spreaders" fig. 1 to move the overburden. Because of
the depth of the pit of up to 30 m and the additional use of draglines to move the lower banks of overburden, and the required safe slope angles, the booms of these spreaders must have a length of more than 200 m. Compression and bending in the booms is taken by a tubular space lattice girder, which is the subject of this paper.

2. Structural components of the discharge boom

Two cross-pit-spreaders have been designed and built in the periods 1982 - 1985 (A) and 1989 - 1991 (B). As may be seen from fig. 2, the boom consists of 4 major structural components: the hoistable portal shaped main mast, the vertical cable staying with its intermediate auxiliary masts to support the cables, the horizontal cable staying with cross girders to support the horizontal cables, and the main girder. The whole boom can be considered as a huge space truss with a length of over 200 m, a depth of about 50 m, and a width of about 30 m where all tension members are cables. In the elevation, the cables are loaded by the dead weight of the structure, in the plan view stiffness is achieved by pretensioning the cables. It may be mentioned that also other structural components of the spreaders, like the masts, the central derrick-like structure, and the receiving booms are built up of tubular members. In spreader A the horizontal bracing has one plane, in spreader B there are two parallel planes of bracing at a distance equal the depth of the main girder.

3. Loads and loading combinations

Long span structures are loaded above all by their dead weight. The total weight of these booms, including auxiliary equipment like walkways, the conveyor support structure, and the mechanical parts of the conveyor, is up to 800 metric tons. The second major load is the live load of the transported material, although it is much smaller than the dead load. This load depends on the planned capacity of the machine, the width and troughing angle of the conveyor belt, and the specific gravity of the transported material. A typical value for the machines discussed here is around 4 kN/m or roughly 1/10 of the dead load. Additional loads are local live loads on the walkways and dirt accumulation ("incrustation") from material that is carried off the conveyor belt by wind or vibrations and settles on the structure.

An exceptional load is ice accumulation on the structural members. Although a rare event in Texas, it may happen once every few years that after a spell of very cold Canadian air the weather changes to a drizzling rain coming from the Golf of Mexico. The mining company therefore specified an ice loading corresponding to a 1 in. (= 2.5 cm) layer of ice on all structural parts. This is quite a severe loading which has governed the design in several places.

So far, only vertical loads have been discussed. Horizontal loads arise from wind, and in view of the danger of tornados, a maximum wind velocity of 160 km/h was specified. The low wind resistance factor of tubular sections was one of the major reasons to choose a tubular construction. A further reason was the smooth surface which offers advantages with respect to corrosion protection and incrustation. A type of horizontal load specific to mining equipment is inclination, which may occur in any direction, since the machines are travelling over uneven ground and planned grades to get to different levels of the mine. Required values depend on the mining conditions and also on whether the machines are operating or just
travelling from one location to another. A normal value is an inclination of 1:15 which has to be superimposed with a lower wind of e.g. 70 km/h.

Further horizontal loads are caused by dynamic excitation due to travel movements. With the increasing ease of application of finite element methods, nonlinear analysis and dynamic vibration analysis have become more or less standard for the design of such slender structures, at least to check the influence of these effects. For normal design calculations their application is too time consuming because superposition of different loadings is not possible.

Loading cases and required superpositions are defined in special codes dealing with bulk handling and open pit mining equipment [1, 2], which also give the safety factors for different loading cases. Three [1] or four [2] loading cases have to be considered, where each can be made up of a group of loading combinations depending on the operating situations of the machine and special events like overloading of the belt, clogging of a chute, travelling on steeper inclines, strong wind etc., or the a.m. formation of ice on the structure.

4. Tubular construction of main girders

Fig. 3 shows part of the main girders in tubular construction. The girders consist of a space truss with four continuous chord members and bracing members in all four walls for torsional rigidity. The shape is quadratic to rectangular with distances of chord centerlines between 3.2 and 4.5 m. Whilst spreader A has a Warren type bracing with verticals in all four walls, spreader B in the web walls has verticals and parallel diagonals arranged to get mainly tension. In the horizontal walls it has a K-bracing. The interior space of the girder contains the belt conveyor.

For aesthetic reasons, and for ease of drawing documentation and fabrication, the designers have tried to keep the outer dimensions of the tubes the same over long parts of the girder and adjust the members to the acting section forces by changing wall thicknesses. A typical chord tube outer diameter is 244.5 or 298.5 mm, bracing members range from 101.6 to 177.8 mm diam. The steel quality is StE690 for the chord tubes and StE360 for the bracing members. High strength steel chord members were required to accommodate the stress of the extreme loading combinations with ice or high wind, or partial ice load with low wind and reduced live load. Fig. 4 shows typical tubular joints between chord and bracing members in two planes. At the sections where the bracing cables are attached to the girder, special joints and frames are necessary to introduce the high concentrated forces. Fig. 5 shows some cases which illustrate the complexity of these welded connections.

5. Calculation methods employed

Since no generally accepted code for tubular structures existed in the design stage of the cross-pit-spreaders, the designers have used various approaches to do their job. The major problem of course is to find a safe fatigue design. As mentioned earlier, fluctuating stresses are caused by varying live loads, changing inclinations, and vibrations. For simplicity of calculations, the dynamic vibrational loads are usually represented by an additional +/- static load on the dead weight and on inclination. Typical dynamic factors for crawler mounted spreaders in open pit mining are between 5% and 10% vertically, and 2% to 10% horizontally. So
basically the stress range for fatigue loading becomes

$$\Delta S = f (1 \times \text{live load} + 2 \times \text{dynamic load vertically} + 2 \times \text{dynamic load horizontally}).$$

Tubular joint design calculations for spreader A were based on CIDECT Monograph No. 7 [3] and on DIN codes where applicable. With respect to section forces from static loading cases no specific tubular joint design formula has been used, since diameter ratios and thickness dimensions stayed within the limitations recommended in DIN 18808 [4], and full penetration welds have been used throughout. The designers tacitly assumed that these limitations are valid also for the high strength steel used for the chords. Moreover, static design was governed by the buckling load under compressive forces, which requires permissible stresses to be reduced by the buckling factor, so that the general stress level at the joints is correspondingly lower. Incidentally, the high buckling strength of tubes as compared with open shapes was an additional reason to choose a tubular construction in this application.

Since the main girder for the total system had been idealized by one continuous beam, part of it was modelled in a separate system as space truss with rigid joints in order to check the influence of that idealization. The analysis has given evidence of secondary bending stresses of appr. 6.5 kN/cm$^2$ in the chords, whilst the corresponding number for the bracing was 3 kN/cm$^2$ under dead loads. These values were added to the maximum stress in the fatigue verification for the member normal forces, which in their turn had been derived from the section forces acting upon the idealized cross-section.

For the fatigue verification of the diagonals the diagrams for K-type joints (Fig. 123/126) from [3] have been used. The effect of the R-ratio was taken into account as recommended in [3] by formulae based on DIN 15018.

The fatigue strength of chord members also was judged by the nominal stress concept (Fig. 114 in [3]) considering axial stress from the normal force and secondary bending stress with the global value of 6.5 kN/cm$^2$ as mentioned above.

Because of the positive effect of an overlap on joint load capacity all K-joints were designed with an overlap aimed at 50% of contact length between diagonals and chord. As is well known, this results in an eccentricity of the intersection of the centerlines of the diagonals with respect to the centerline of the chord. This eccentricity was judged according to DIN 18808, Sect.5.3, which gives a limit of $e/d_o = 0.25$ up to which secondary stresses may be neglected. All the joints in this application were below this limit.

The verification for the T-joints was done using Stress Concentration Factors according the DNV formula recommended in Tables 17 and 18 of [3].

For lack of pertinent information, interaction between section forces from bracing members in the vertical and the horizontal planes was ignored for all joints.
The tubular construction was completely pre-erected in Germany and shipped to the USA. Because of highway transports involved, sections were limited to a length of 16 m. Erection joints between chord members and between diagonals were made by pretensioned bolted flange connections. The flange to tube connection is known as a very unfavorable detail. It was assigned a permissible stress range of 4.5 kN/cm², according an earlier draft edition of Eurocode 3.

For spreader B the tubular joints have been designed according to the AWS Structural Welding Code [5]. Allowable stresses in general were assumed at 0.6 the yield limit for the loading case "main loads", so that the following values resulted:

<table>
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<tr>
<th>Material</th>
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<tr>
<td>ST52-3</td>
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<tr>
<td>TStE690V t ≤20 mm</td>
<td>41.0</td>
</tr>
<tr>
<td>TStE690V t &gt;20 mm</td>
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Safety under static loading was judged by Sections 10.5.1 "Local Failure" and 10.5.2 "General Collapse" of [5]. This verification for local failure is based on the punching shear criterion, for which the chord ovalizing parameter is important. Since all joints are multiplanar, the ovalizing parameter was computed on the basis of eq. 1, the formula given in the commentary of [5] which incorporates the influence of the other bracing members in the vicinity of the one for which punching shear is verified (see also Marshall [6], Fig. 11).

\[
\alpha = 1.0 + 0.7 \left\{ \left( \sum P \sin \theta \cos 2\phi e^{-\left(0.67r^2\right)} \right) / P \sin \theta \right\}
\]

In some instances this formula has delivered much higher ovalizing parameters than the "normal" formula for K-connections per eq. 2,

\[
\alpha = 1.0 + 0.7 \frac{g}{d}
\]

obviously because the beneficial influence of small g/d ratios resp. high d/D ratios is not properly reflected in eq. 1.

For the fatigue loading case the hot spot stress concept was used in conjunction with the allowable fatigue stress curve \( X_1 \) of the AWS code. According to the usual practice for open pit mining equipment, the stress range at \( 2 \times 10^6 \) cycles was taken as the permissible stress range value. To account for the influence of the multiplanar joints, the Kellogg formula (Fig. 12 in [6]) has been used to compute the hot spot stress originating from the section forces of the brace members. High ovalizing factors computed by eq. 1 directly raise the hot spot stress. In cases where this increase did not seem justified, fatigue resistance has been verified by applying stress category DT of the AWS code.

6. Conclusions

The long span booms described give an example of tubular construction in open pit mining equipment. The inherent advantages of tubular shapes - smoothness of surface and high buckling resistance - were the decisive factors for using them. Although the average
structural steel designer is not familiar with the design rules for tubular structures, the recommendations or codes available in Europe and the U.S.A. have given sufficient guidance to enable the design and manufacture of these long span tubular structures. There is one point of insufficient information which has led to guesswork to a certain degree. That is the interaction of section forces in multiplanar joints. One has to face the fact that the majority of joints in practice will be multiplanar. Therefore research and codes should provide more positive guidance in this respect in order to give reassurance to the designer who is working with tubular construction.

7. Acknowledgements

The author thanks the following companies for the permission to publish technical details and for giving access to drawings, calculations and slides:
Texas Utilities Mining Company, Dallas (Texas) as owner and operator of the machines, Krupp Industrietechnik, Rheinhausen (Germany) as successor to Demag Lauchhammer Division, manufacturer of spreader A, VOEST-ALPINE Bergtechnik, Zeltweg (Austria), manufacturer of spreader B.

8. References


Figure 1. System of overburden removal and dumping

Figure 2. Cross-pit-spreaders in Texas mines
Figure 3. Sections of main girders in tubular construction
Figure 4. Typical tubular joints
Figure 5. Special joints at cross frames
Towards the end of the 19th Century some outstanding examples of tubular construction were built. The stresses, sections used and methods of fabrication were greatly different from contemporary methods and materials.

Although SHS is now used world-wide it has enjoyed but 50 years of life so far. Its acceptance and rapid development helped by improved welding techniques are part of the structural revolution of the 20th Century. Computer analysis in like manner aided the development of Space structures.

The combination of these factors has produced an ever widening range of aesthetically pleasing structural arrangements and wide spans which, as each year passes, provide high quality structures for the widest variety of arrangements and enable the user to anticipate greater things to come.

1. **1890-1950 The early days**

It is well known that 100 years ago the world's largest clear span bridge was built from curved plates riveted to form tubular steel members.

That bridge, the Forth railway bridge, still stands today. What is not so well known is that a bridge across the Channel between England and France was also proposed by those same British Engineers working in association with a French Company. It would have used tubular steel tapered triangulated bow-string girders carrying suspended spans, similar in style to the Forth Bridge.

Fifty years ago, as an experimental officer on military bridging one of my duties was to load-test the "Inglis" bridge, a tubular steel bridge developed in the 1914-1918 war and still in use in 1940. This remarkable bridge with minor modifications successfully carried a forty tonne tank on 40m span under the most adverse dynamic loading condition and at one stage was modified for service as an assault-bridge capable of being launched by one man under fire. It was a 'piece-small' type of fabrication with screwed ends and pinned connections into prefabricated nodes. Its successor, the Bailey Bridge, built with high carbon steel, 340N/mm² yield point, was difficult to weld.

Problems in the use of welding have been overcome progressively during the last fifty years and then, as now, were successfully dealt with in such a way that the bridge panels became the first ever mass-production standard modular units for structural use. Most of that work was done by female welders quite new to industrial fabrication methods.
Welding however by that step had begun its big strides and rapid progress.
Its acceptance however was not so rapid and many otherwise experienced engineers were loathe to accept the load-bearing capacities of the "new-fangled" method. "How do you know the welds are safe?" they would ask.

2. 1951-1970 Post war pioneering

Doubts that existed were answered dramatically by the construction of the world's largest walking dragline with a jib some 90m long carrying a loaded bucket of 40 tonnes per minute and doing so for millions of dynamic repetitions over its working life. Built from CHS 'Chrome Molydenum' steel with a 340N/mm² yield point the jib was mainly assembled using two welders assisted by fitters and was lifted into position forty years ago in 1951.

With such an example and others of the same type it was subsequently easier to argue the case for welding.

It is not so easy to sell the world's largest walking dragline in quantities sufficient to develop a market in tubular steel fabrication.

It was necessary to demonstrate the feasibility of use of SHS in a wide variety of structures and the best way to do that was to take a prestige job using the tube against competition in rolled sections.

That was simply accomplished at Haslar where a series of 67m span girders were built over a tank for testing wave action on shipping models.

They, too, were site fabricated owing to their outsize dimensions and many lessons were learned particularly regarding fit-up and tolerances in welding. At the same time, in Australia, a 73m clear span tubular steel roof was being erected over the Melbourne Olympic swimming pool. A wide variety of structures followed, standard buildings with spans of 37m, towers up to 50m high followed by a radio-tower in Gibraltar 183m high.

Not all "wide spans" are horizontal.

Many of the large 19th Century structures were built for use by railway transport. In a similar way, the development of air transport necessitated the construction of wide span hangars.

The introduction of the Boeing 747 gave rise to the need for hangars far greater than any previously built.

During the same period the rapid development of computers facilitated analysis of three dimensional space structures.

Having examined in great detail ten prime alternatives, and four sub-divisions it was decided to use a tubular steel space frame with 138m clear span for the first specially designed hangar for two B747 aircraft. A new quality of steel was used for the first time in a British structure - steel that we now know as Grade 55C. The weight saving in the main girders exceeded 50% as compared with the lower grade of normal steel.

Another novel decision was taken on the same contract - in 1968 - to build the complete roof at low level and raise it in one piece to its final position, an achievement that succeeded with but minor difficulty and which, in principle, is now relatively commonplace.
A further novel decision was to apply an overload to the completed structure to examine its performance. 1100 tonnes of ballast were applied one weekend, supported by the main girders and resulted in most satisfactory comparisons with the calculated stresses and deflexions being reasonably less in all cases.

This first hangar "01" and its successor "02", built and tested similarly, were believed to be the world's largest clear span space frames, the largest tubular steel buildings and the first among many to be lifted as complete structures. It was also some 50% lighter than more orthodox schemes but its performance was 100% better.

3. 1970-1990 Rapid development

Having built such clear spans it is easier to sell the idea of lesser spans for other uses, such as football grandstands. The grandstand for Bristol City FC, 100m, was originally planned to have three intermediate supports holding up the main-roof girders. It was finally built as a 100m clear span supporting secondary trusses carrying metal cladding. Fig. 1. The sight-lines permitted a girder depth of 5m and that was accomplished by a single-plane latticed girder using Grade 55c 457 dia. CHS with simple Warren or V bracing. Supplied in four pieces joints were made by butt welding at site with special attention at end connections of the lower chord to permit initial stretch.

On this job most fabrication was done in workshops. In Glasgow, Celtic FC were next in line for a new grandstand. Initially this too was intended to have internal supports at roughly 18m centres. After Bristol, however, it was not difficult to suggest a clear span.

What England could do Scotland could certainly do. In this case the 100m girder supported 4400m2 of roof and an extensive press/TV box at its centre. Depth was restricted owing to site and sight conditions. 5m deep was possible and could be achieved by double or box girders in high yield CHS with 457mm main chords. To facilitate fabrication, improve working conditions, speed up site construction and, in general, to produce an economic structure the girder was designed using a cross-bracing system. This was deliberately chosen to permit it to be divided into top and bottom halves suitable for transport and reassembly at site. The cross-over joints were carefully considered with a view to completion by the simplest of site welding. The outcome was eminently successful. The girder weighed 100 tonnes only and compared favourably with one supporting a similar roof in lower grade steel but 109m span 8m deep weighing over 300 tonnes. Fig. 2.

The complete girder, detailed in that way, was laid out in the workshop with temporary bolted connections dismantled and taken by road transport in 'saw-tooth' shaped subdivisions less than 3m deep to site where it was reassembled using the same bolts.
Shortly after the Heathrow 01 and 02 hangars were completed it was decided to investigate the possibility of building a 3 bay hangar within the limits of space and overall height permitted. Using Grade 55c steel it was just possible to accommodate the framework. With any lesser quality the job was not possible as the increase in girder self-weight gave rise to the inevitable increase in bending moment, increasing the self-weight, increasing the moment and so on ......

Eventually, due to availability and programming of space usage, it was decided to build three extensions onto an existing older hangar and postpone the clear span job. Fig. 3.

The first two extensions to Technical Block A at Heathrow were each 103m clear span separated by a high level plantroom giving an overall length exceeding 240m. Each extension consisted of an outer periphery frame of 7.5m deep box girders in CHS Grade 50c with shorter span girders connected at right angles. The box girders, as for Celtic FC, were split at mid-depth, transported to site, reassembled and completed by site welding as before. When the third extension was considered agreement was reached to allow one metre extra overall height. The hangar was designed using clear-span girders running parallel to the door. Including the fascia girders supporting the door tracks, all were made in simple plane lattices 5m deep.

The steel was designed so that it could be built at ground level or "in the air". Fig. 4.

The fabricator chose the ground level method which was accomplished by using the main box stanchions as lifting points with the gable girders, supporting the long spans, passing up the centres of the tri-part box columns. The 800 tonne lift was achieved in a few hours in one afternoon. This roof, in tube, was placed in competition with the previous box-girder type and also a further design in rolled steel. The interesting tender showed the latest tubular design to be cheapest, the previous tubular design next and rolled steel in third place. The differences were, however, within 10% of cost. Fig. 5.

The first Boeing 767 aircraft was due for delivery to Britannia Airways at Luton and their existing hangar was too small to accept anything except the fuselage.

The new hangar was to be built in two stages. Stage one as half-a-hangar extension to the existing one and stage two to replace the existing when full operation of stage one was complete. Stage one was fabricated with camber suitable for double the span and propped temporarily at the centre of the finished hangar. Just before stage two was added it was decided to increase the clear span by 7m. This, it was determined, could be achieved by providing additional centre space restraint but of course the camber would now be wrong. The problem was overcome by joining the main girders together by site welding in such a way that the 'new' end was set 560mm below level so that on jacking up the gable the centre came back to just 15mm above its previous present level on the clear span. The centre-prop was 'self-removed' by burning away some of its steel so that it gently yielded under the load.
The fashion created by the 01, 02 hangars at Heathrow, so successfully built and efficiently operating, gave rise to a similar style for the SIA Changi Hangar in Singapore. The SIA hangar, 218m span, was designed also as a double layer diagonal grid using CHS members. It was built in four huge sections on an island offshore from site, shipped across, joined by site-welding and lifted in one piece to its final position. It is an unusual hangar having no doors - an economy made possible by the benign climate and ideal site location. Whereas the 01, 02 hangars had considerable repetition in the high and low level roofs the SIA hangar had a tapering rear section with haunched ends giving little repetition. In the first case the repetitive nature was desirable to reduce costs and increase production whereas in the second case the labour was cheap and the climate such that almost all the work took place outdoors with no cover. These points must be considered in the design processes for any hangar in different world locations.

Generally speaking, the use of spaceframes in double layer grids for the main roof areas of hangars, although usually slightly more costly, has such benefits that they should always be considered as one of the alternatives in any hangar design. With simple front to back secondary spans it is necessary to use generous wind bracings in the roof with ties to the secondary girder lower chords against wind uplift. With fewer main bracings the end connection forces are significant greater than those whereby with the use of diagonal grid separate wind bracings are not required and the forces are distributed through a number of shear-resisting members. The added advantage of the two way grid or spaceframe is its ability to share applied load and thereby minimise the vertical elastic movement. Subsequent modification to craneage or imposed loads is more easily permitted or catered for.

This advantage accrues mainly because the outer peripheral zones of such grids are stressed less than the heavier modules since it is normal practice to keep to common external dimensions for ease of detailing and standardisation of fabrication. Hence these members are often understressed by normal imposed vertical loads and can thereby accept 'extra' load when resisting wind forces. It is worthy of note that although there are a variety of alternatives for the main roof areas they are unlikely to have great influence on the final overall weight provided of course they are competitively designed in the first place. The design of the main girders is therefore of paramount importance. The main girders, however, have a significant affect on site time, cost and performance. One of the great failures of engineering design for wide spans is that of accepting or creating an arrangement which can only be built slowly and tediously by site-welding individual members. It is compellingly easy to design any large span without consideration of the supply, delivery and erection procedure. In adverse climates site-welding is to be avoided where possible. In benign climates it is, in any event, just as important to provide protection from the wind even in 'nice weather' and inspection no less costly than for better conditions. It is, in any country in the world, preferable
to fabricate under cover in controlled conditions and to that end
the design arrangement should be chosen.
The careless selection of girder depths, widths and bracing
arrangements which prohibit shop fabrication should be avoided.
Great care should be taken eventually to 'build the girder at site'
envisaging the difficulties and then on return to the design
office to choose an arrangement minimising those difficulties and
maximising shop fabrication. This may involve a greater amount
of welding and fitting but almost inevitably saves time and cost
overall.
Where a final choice, for whatever reason, requires modules too
large for transport then the problem is little different. It is
obviously simpler to fabricate a large number of standard pieces
or modules than it is to make non-standards with little repetition,
even if fabrication is done on site. Such a case occurred in
Santiago where a Spacegrid was chosen for LANCHILE Airlines for a
75 x 75m hangar roof. It was decided to use a 10 x 10 modular
diagonal on square roof with a module 7.5m corner of a square
based pyramid. This replaced an alternative two-way square
grid that had previously been proposed and subsequently proved
to be unbuildable. Two simple jigs were made at site and under
temporary cover modules 7.5 x 7.5 x 4.5m deep were fabricated.
As each one was made it was shifted by a light mobile crane to
its position on site. That part of the roof was designed and
detailed in less than two weeks, against several months that had
been taken up for the previous design and detail.
Spacegrids are beneficial for operating conditions in all
structures where vertical movement is to be kept to a minimum.
Hangars like this had previously been used for helicopter
 housings at Gatwick and Aberdeen and for the same reason.
A different use was found for FINNAIR in Helsinki where the new
hangar for wide bodied aircraft has just been completed. The main
frame designed as bent-knee portals supported at their inner ends
by a central block and tied laterally by plate girders received a
spacegrid infill using the SIRIUS system supporting the roof
craneage and includes some 32 grids - 30 x 30m maximum on plan.
This was the first use of the system in Finland and achieved
with elegant simplicity.
Not all wide spans are hangars although they are by number
probably the most common.
Spacegrids for a variety of buildings have been used and are
becoming not only more popular but also much bigger.
At Trondheim in Norway the roof over a large tank for wave-testing
of model ships extends 81m x 54 and in addition to heavy
snowload carries three 10 tonne underslung cranes from the roof.
The Spacegrid in that case employs universal column top chords,
HHS bracings and rolled steel channel bottom layer.
Similar flat roofs have been designed including a recent case where
the 150 x 100m roof is divided into two 100 x 75 sections which
slide apart to give a 150 x 100m clear opening.
In Austria, near Vienna, is an elegant pyramid built with a 65m x 65m base and glazed internally and out. So accurate were the prefabricated diamond-pyramid modules that the double glazing was applied progressively as the steel erection moved up the slopes. Accuracy in fabrication is of paramount importance. Any error repeated 600 times is to be avoided if possible. Dimensional control is not difficult to achieve given intelligent forethought of workshop methods. Fig. 6 & 7.

In Norway a new 90m arched roof has been completed at Ostfoldhallen. With arches at some 12m centre to centre the infill is achieved by metal roofing without purlins. Football will be played there under cover. A similar need in another Scandinavian country gave rise to the design of a 90m spaceframe arch. The point of interest arose when the calculated weight of the spaceframe proved to be almost identical with that of the triangulated girders. In the spaceframe there are some 600 repetitive modules each with four identical top sides and bracings, in other words 2400 repetitions. Fabrication would be more expensive but cladding much cheaper because of the smaller spans for the decking. Preference for either of the finished alternative appearances is a matter of taste.

The "O2" hangar at Heathrow has an office block at each end and a central screen dividing it into two halves. Painting is carried out in one half and it was previously taking several weeks to refurbish one B.747. To speed up the work it was required to add 8 or 10 overhead telescopic cranes to that half of the roof, each weighing approximately 20 tonnes. Due to the original design arrangement and the newer loading the existing roof could not cater for the new loads. After considering many alternatives the problem was solved. A hole was knocked at centre span over the existing screen and another at the end by the office block. Stanchions were passed through the holes leaving some seven metres projecting above the roof. A 70m clear span girder using SHS was fitted over the roof onto the protruding stanchions and three other girders fitted to it at their inner ends. The outer ends were supported on a triangulated portal supported at ground level. Holes were cut through the cladding and slings passed through the holes connected to the secondary girders at the top and carrying the crane beams at the bottom. Seals were fitted around all of the slings. All of the cranes are now supported externally without any load on the original structure. A seemingly impossible job was accomplished with the only 'complication' of employing a special high-jib crane for a very short time. Again, the 70m girder 6m deep was centre depth split for economy and site erection. Fig. 8 & 9.

Not all structures are simple to build. Among the most difficult, not to design, but to clad, are domes. The need for very accurate fitting in three dimensional modes in closed circles places a heavy responsibility on the draughtsman and fabricator.
Cladding alone can often cost more than the dome structure. To overcome this a Spacegrid dome is arranged to permit the use of normal "flat roof" cladding and insulation applied laterally or longitudinally depending on the module employed. In principle, the modules for arch, dome, pyramid, flat roof are identical differing only in their regularity or polygonal shape. Spans of 200m are under consideration with this sort of arrangement. Domes, however, are not cheap and undoubtedly arches remain the most competitive way of efficiently using SHS provided clearance height and head room are satisfied. As long ago as 1960 an 800 feet (243m) clear span arch was fully designed in such a way that 90% of the fabrication was workshop based and repetition of detail was enormous. Taking the idea first used for the Celtic FC girder, using deeper girders, an arch of some 600m is easily attainable and yet could be mostly shop fabricated. With new ideas of prefabrication flat roofs of 400m or so clear both ways could also be built with repetitive fabrication. Fig. 10. Undoubtedly covered recreational areas will be commonplace in large urban environments to provide unnaturally all that nature can provide, but at a distance. Certainly the athletics for the "Olympic 2000" meeting should be completely enclosed with an adjustable roof.

The covered environment, controlled, with the absence of wind and weather will permit the use of a wide variety of forms of construction and materials. Structures in reinforced concrete built to last a thousand years (subject to maintenance and deterioration) will be a thing of the past. Buildings capable of quick assembly and rapid dis-assembly will permit re-location to suit changing circumstances.

Leisure activity, mountaineering, rock climbing, pot-holing, ski-ing, cascade canoeing, sailing and other outdoor sports will be possible in all major cities where activities could not otherwise take place without travelling considerable distances in a variety of directions. All this has been possible for over twenty years with the use of SHS and welded construction. Fig.11 & 12. Perhaps with the demise of the Channel Tunnel through financial failure, accident or sabotage, we may yet see the world's greatest engineering achievement, the Channel Bridge, in SHS with spans as great as 300.0 m. That, too, has been possible for some time, certainly for the past twenty years, and needs no greater engineering knowledge or ability than presently exists but rather a greater political interest and appreciation of the very obvious advantages.
FIG 1. BRISTOL CITY FOOTBALL CLUB.

FIG 2. CELTIC FOOTBALL CLUB.
FIG 5. TECHNICAL BLOCK A. EAST PEN AT LOW LEVEL.
FIG 6. CONSTRUCTION OF AUSTRIAN PYRAMID.

FIG 7. COMPLETED PYRAMID.
FIG 8. EXTERNAL SUPPORT OF INTERNAL CRANES.

FIG 9. INTERNAL SLUNG CRANES.
FIG 10. MODEL OF WIDE SPAN ARCH.
FIG 11. INDOOR RECREATION.

FIG 12. RUGBY MATCH.
TUBULAR STRUCTURES ADOPTED FOR SUPPORTING MEMBRANE

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Summary

For these several years buildings which have permanent membrane structures have been built in Japan and tubular structures have been used to support the membranes as structure members in almost all these kinds of the membrane structures. In this report we will explain the history of membrane structures and their kinds, and after that we will explain how and why tubular structures are used and finally we will introduce to the readers the actual membrane structures built with this method.

1. Explanation of membrane structures

1.1 History of membrane structures

Membrane structures resist external loads with tensile stresses and keep stability, transmitting the tensile stresses to membrane supporting structure, under part of structure and foundation structure.

Membrane structures have a long history. They were used in ancient times for simple structural type in such a way that support push up animal skin. At that time, the membrane was used as finishing material rather than structural material. Since then there was no radical change in the structural type, and mainly used as simple structure such as sunshade eaves and circus tent.

In 1950's, Prof. Frei Otto of West Germany invented new membrane structures to which initial tensile stresses were applied. Adoption of this method changed the membrane structures from mere tent to permanent structures.

In Japan, various kinds of membrane structures (mainly pneumatic membrane structures) were used as pavilions in Exposition '70 held in Osaka, and they became widely known as temporary membrane structures. In 1988, the large pneumatic membrane stadium was built in Tokyo as the first building of this type in Japan and this is known as a permanent membrane structure. After this progress, in 1987 "Technical Standards for Specific Membrane structures" based on Article 38 of the Building Standards Act was published and more membrane structures have been built up.

1.2 Classification of membrane structures

The membrane structures are classified according to structural types as given below.
To any structure initial tensile stresses are applied.
1- Frame membrane structure. (Fig. 1.1)
Membrane is covered on the shape made of frames. The membrane surface is flat or primarily curved.

2- Suspension membrane structure. (Fig. 1.2)
In principle, this is suspension structural type, using membrane as the main structure member. The membrane surface is secondarily curved and its Gauss curvature becomes negative.

3- Pneumatic membrane structure. (Fig. 1.3)
Difference of air pressure supports the structure, sending air tight space covered with membrane. The membrane surface is secondarily curved and Gauss curvature is positive.

2. Membrane structure and tubular structure

2.1 Membrane fixed details of the structure

Generally, a permanent membrane structure is composed of several membrane panels and at the time of construction, they are brought to the construction site from the manufacturer's factory rolled on core pipes and fixed them on the beams spreading them at the construction site.

Regarding the fixing devices of the permanent membrane structures, they are such types as given below.

A. Cheap fixing device of membrane. (Fig. 2.1)
The cheapest device type which can be fixed with hardware among this types, but this type has possibility of rainwater leakage. Therefore, they are usually used for only small structural types for outdoor use.

B. Popular fixing device of membrane. (Fig. 2.2)
This type prevents rainwater leakage, comparing with the cheap fixing device (A type) and faces to the room. Since this type is equipped with gaskets, its price is higher than that of A type.

C. Fixing device which has tensile stress of membrane. (Fig. 2.3)
This is the type the tensile stress device is installed in popular fixing device (B type) and has the most expensive devices. This type enables small adjustment of the fixing device in the process of spreading and fixing. Furthermore, restraining of initial tensile strength is possible, of which strength deteriorated by creep of the membrane, lengthening the bolts. However, designing that bending stress does not affect the bolts is necessary.

D. Binding type. (Fig. 2.4)
This type is different from others. This type has cuff made of the same material as the main membrane on the back side of surface of the membrane. This cuff is bound on the frame under the membrane. The cost is cheaper than that of A type, but the strength is insufficient and in the case when applied to arch members, the cuff makes wrinkles and lacing ropes are exposed, therefore, these may be the problems from the point of view of design.
2.2 Design of membrane structures

There are few different points in designing membrane structures from other structures. First of all, all the possible curved surfaces shall be summed up, and since the deformations of the membrane structures are larger than those of other structures, such design plans which can allow deformations are required.

The designing of membrane structures must be done as the chart shown in table 1.

1. Designing and the structural plan
   Analyze the membrane surface shape according to design plan. Generally numerical analysis is done to seek curved surface of equal tension.
   For the answered membrane surface shape, check whether it is workable or not, whether it makes depressions which collect rainwater or not and so on.
   Proceed with the structural plan of tensile stress generated in the membrane materials by external loads.

2. Stress and displacement analysis of the membrane structure
   The membrane material has such texture woven with glass fiber or polyester fiber and on which resin is coated. Analyze this material as anisotropically elastic material.
   In the analysis of the membrane structure, assume the material to be linear and consider the non-linear shape.
   According to "Technical Standards for Specific Membrane Structures", it regulates stability of the membrane materials 8 for long time loading, 4 for short time loading, and regarding the cables, 3 for long time loading, 2.2 for short time loading.

3. Designing of the membrane supported structure
   Analyze it as boundary reaction load gained by the stress and displacement analysis of the membrane structure.

2.3 Tubular structure support for the membrane structures

Generally tubulars are used for the membrane support of the membrane structure. The main reasons of the application are as follows.

1. For the membrane structure the direction it receives exterior force and the direction it affects structural members are different from those of conventional structures. Therefore from dynamic point of view, tubulars are advantageous which have less directional property. (Fig. 2.5)

2. For the membrane structure since deformations of the membrane materials are larger than those of conventional roof materials, the direction of force which acts on the membrane supported structure largely changes. Therefore, tubulars are used, which have less dynamically directional property as (1). (Fig. 2.6)

3. Generally since the membrane fixing devices hold up membrane fixing base plates from the supports, load acts on it, deflecting from the center of the support structure. This causes generation of twisting moment in the support structure. For disposition of this twisting moment, like tubulars which have strong resisting
section performance against the twisting stress are advantageous.

For the above reasons, tubulars which have the same section performance in any direction and strong resisting capability against twisting are more advantageous than structural member of H-steel which have dynamically directional property, and from the point of view of the design, for the membrane structure which has smoothly curved shape, tubulars (especially pipes) are very suitable.

3. Membrane structures, in Japan

Among the membrane structures in Japan, we introduce two buildings built in Osaka.

3.1 Tennis court roof

Location: Suita City, Osaka Prefecture. Year built: 1988
Area of the membrane structure: 1858.6m²
Span: 35.2m x 52.8m Height (membrane roof): 15.5m
Use: Sports activities
Membrane material: Glass fiber cloth coated with ethylene tetrafluoride resin

Photo 1 shows the membrane structural building built for the roof of tennis court. The structural characteristic of this building is that it was constructed by tilting arch shaped tubulaires alternately and the openings were covered with the membrane. Around the external perimeter, for disposition of the membrane tensile stress, lattice beams composed of tubulars were fixed and tilted to let them meet at the direction of the membrane tensile stress. At the membrane fixing areas, membrane pulling device are fixed and this in enabling to cope with creep of the membrane material.

Confortable environment is made with mild sunshine which is getting into the inside space through the membrane. (Fig. 3.1)

3.2 Swimming pool roof

Location: Osaka City, Osaka Prefecture. Year built: 1990
Area of the membrane structure: 6216m²
Span: 48m x 129.5m Height (membrane roof): 31.79m
Use: Swimming pool
Membrane material: Glass fiber cloth coated with ethylene tetrafluoride resin

Photo 2 shows the membrane structural building built for the roof of indoor swimming pool. the structural characteristics of this building is that the membrane has unequal tensile stress ratio of 1:2 (warp direction: fill direction), and this is particular membrane structural building, the membrane surface is composed of this texture. Since the size of each membrane panel is very large as 16m x 48m, when a load is imposed, its tensile stresses are very strong. For disposition of the membrane tensile stress, tubulars SM50 were used as arch material. In addition, holes were made on the membrane and the supports were erected through the membrane from the under part of structure and cables were fixed on tops of the supports and three points of each arch so that the arch can be strong enough. The arch of gable side is unable to be reinforced with cables, therefore, it is
reinforced with lateral ties against the arch. Since the inside of this building is used for swimming pool, the membrane has partly internal membrane to prevent dripping of condensed water from falling on the floor directly. (Fig. 3.2)

4. Rating of tubular structures for the membrane structures.

Currently in Japan many membrane structures, including small ones, are now being built and planned, but still not easy to be built like conventional buildings, and for further development of the membrane structures, the surfaces of which are supported with tubular, clear settlement of the following problems may be necessary.

1. In case of connecting tubular to tubular, since the thickness of tubular is thinner than that of H-steel, there is some risk of local buckling, and in case of welding, the welding parts shapes a cubic curve surface and this makes the welding difficult.

2. Since the membrane fixing device has a complicated structure, when shaping tubulars it increases the number of frame shaping work and sometimes the difficulties in frame shaping and rust prevention work.

3. For both the backing frames and main frames, rust prevention is required, because they are affected by dew condensation. Inspections after the completion of the building are also difficult due to its complicated structure.

For the above problems, in Japan enterprises and colleges have been performing various studies and experiments and trying to materialize new ideas, therefore, we can expect the problems will be solved in the near future.

For the engineers directly in charge of the designing, if they design the building fully understanding the advantage and disadvantage of the membrane structure, they will be able to create new space which has never been created by conventional buildings.
Figure 1.1 Frame membrane structure.

Figure 1.2 Suspension membrane structure.

Figure 1.3 Pneumatic membrane structure.

Figure 2.1 Cheap fixing device of membrane.

Figure 2.2 Popular fixing device of membrane.
Figure 2.5 Difference of membrane structure and conventional structure.

Figure 2.6 Deformations of the membrane structure.

Figure 2.7 Generation of twisting moment in the supported structure.
Figure 2.3 Fixing device which has tensile stress of membrane.

Figure 2.4 Binding type.

Table 1. The designing of the membrane structure.
Photo 1. Tennis court roof.

Figure 3.1 Details of arch and lattice beam of tennis court.
Photo 2. Swimming pool roof.

Figure 3.2 Elevation of swimming pool.
LONG-SPAN AND HEAVY-LOADED TUBULAR STRUCTURES
- Examples for design -

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1. General

For the design of structures with long spans, truss systems or arch structures are selected and hollow sections (semi-finished products) are chosen. The reasons for this are:

- more favorable load bearing behaviour,
- adjustment of the necessary member cross sections to the member forces through variation of the wall thickness,
- smaller surface and thus, more favorable measures for corrosion protection,
- relatively low dead weight,
- and others.

In this work, four different structures are presented in which the hollow sections represent the main supporting structure. In addition to the description of the structure, the quality control during production as well as experimental investigations carried out on the finished structure are illustrated.

2. Changi-Hangar Singapore

Dimensions: 218.4 x 92.4 m; height: = 11.00 up to 15.00 m;
surface = 20,000 m²
Type of supporting structure: tubular truss grid (diagonal grid)
Steel weight roof structure: 2.750 t = 137 kg/m²
Initiation: 1983
Client: SIA (Singapore International Airlines)
Consultant Engineer: Dr. Y.S. Lau, Singapore
Steel construction: Mc Dermott South East Asia Pte. Ltd.

The hangar has the capacity to service simultaneously three B 747 Jumbo aircrafts and two narrow bodied aircrafts.

The new Changi-Hangar with a column free front opening and without gates is enclosed on three sides by an U-shaped nine-storey service building with a height of approximately 35 m on the internal edge of which the big roof structure is connected on 12 supporting points. The structure has been designed as diagonal grid which runs between edge supports and the 218 m long gate truss (multi-stayed rectangular girder) in the projection (fig. 4). The chord members of this gate truss (Appron truss) have an outer diameter of 813 mm. The wall thickness of the tubes has been adapted to the existing member forces (t_{max} = 60 mm). The distance of the trusses among each other is almost 12 m. Following the course of the moment and offering a roof drainage around the structure, the height of the truss grid amounts to 10 m all around and increases to 15 m to the middle of the Appron truss. The roofing is supported on a purlin system which divides the fields of the girder diagonally four times and vertically to the gate line.
On a small island (island Batam) 20 km away, the supporting structure was prefabricated in four components of the same size with a weight of approximately 700 t, which were lugged on the beach near the airport by cargo boats and from there, suspended on an insulating air cushion, to the place for installation (fig. 2). Figure 3 shows the support of the roof structure during transport. On the ground, the four components (92.0 x 55.0 x 12.0) were connected to a complete roof plate (fig. 4) and completed with a roof skin, and numerous crane rails, sprinkler systems and others were pre-assembled. With the VLS-lifting methods, this 3.850 t roof package was then brought to the final headway of 25 m by means of 24 special equipments with cable suspensions on the 12 points of support. During the lifting process, deformations occurring in the points of support and in the main supporting members were recorded continuously. Fig. 5 shows the suspension point of the Appron truss during lifting. In a height of about 4 m above the ground, the lifting process had been interrupted for some weeks in order to complete the installations and to readjust the crane systems. After reaching the final position, the loads in the lifting cables were transferred to bolted connections with straps (fig. 6). The supports (fig. 7) are developed in a way that on the one hand the movements of the roof caused by thermal effects can be balanced, and on the other hand the horizontal and vertical forces can be carried off into the surrounding frame building with 8 developmental cores. The maximum support reaction amounts to approximately 1.000 t (points of support of the main girder, Appron truss), and the smallest support reaction are 180 t.

In the middle of the main girder, the roof is stilted by 0,70 m in order to compensate deflection during lifting and to provide a slope for the roof skin.

Three crane systems with two bridges each, on which the teleplatforms were installed with all supply units, pass over the complete surface of the hangar. The crane system is designed in a way that the teleplatforms can pass over from one field to the other. Thus, the teleplatforms can be applied on any place of the hangar depending on the demand.

Over the complete time of construction, elongations, deformations and vibrations were recorded in order to check the assumptions of statics and, on the other hand, to determine the characteristic frequency of the roof. Figure 8 shows the finished roof structure.

Many tests, measurements and investigations verifying the safety of the Changi hangar roof structure were carried out. The following investigations going beyond normal measurements for quality safety have been performed:

- Strain and stress measurements on Appron truss of Changi hangar roof during the lifting procedure.
- Investigations on prestressed bolted connections in the area of the supports of Changi hangar roof structure.
- Strain measurements on the straps of the supporting points during load transmission.
- Strain gauge measurements to determine mean stresses of members and hot spot stresses in characteristic and critical structural joints.

The strain gauges were used for strain measurements and stress calculations for the following types of loads:

- 11 different load configurations,
- temperature fluctuations (24 hour reading),
- 8 different concentrated single-bay loadings,
- dynamic loadings to determine the natural frequency,
- crane runs to determine the influence lines for different roof sections.

- Wall thickness measurements
Investigations of the dynamic behaviour of the roof structure

- deflection measurements
- strain gauge measurements
- accelerometer measurements

Fatigue tests on specimens of the roof structure material including the investigation of the effects of arc strikes, temporary attachments and weld defects, and COD-tests.

During supervision, for almost all joints arc strikes were ascertained. The costs for repairing these thousands of arc strikes would be too high. A cheaper solution was decided which included that the arc strikes for all members with a high load are ground and that a subsequent magnetic particle inspection should confirm the crack free situation. Experimental investigations carried out in parallel showed the following influences on the service-life of the material used which is illustrated in fig. 9.

3. Hangars of the Airbus Industry in Bremen-Lemwerder

Accommodation for aircrafts: 4 aircrafts of the size Airbus A 300 / A 310
Dimensions: 233,0 x 56,5 m; height: 21,34 m; surface = 13.000 mm² per hall
Type of supporting structure: arc truss designed as triangular girder made of circular hollow sections

Initiation: 1984
Client: Airbus Industry
Design and statics: Comp. Bachmann S.A., F-94200 Ivry sur Seine
Steel construction: Service Commercial, F-93350 Le Bourget

Both hangars of the Airbus Industry in Bremen-Lemwerder (fig. 10) offering space for four aircrafts of the size Airbus A 300/A 310 one behind each other, have, in addition to the panel frames, 19 vertical trusses made of circular tubes designed as welded triangular girders and a final bevelled girder which is also made of circular hollow sections. The structure is cased with plastic sheeting, and it is designed and calculated as "temporary structure".

Figures 11a and b show the structure during erection and figure 12 shows the hangar after completion. The design of the low ends can be taken from figure 13.

In the joint area, the hollow sections are flattened so that complicated penetration lines could be avoided (fig. 14), and simpler welds (fillet welds only) were sufficient for the joints. A further advantage of flattening can be seen in figure 15. At the joints of the arc segments, the bolts could be arranged so close to the tubular chord that thin chord plate thicknesses were sufficient. This is especially advantageous for temporary structures that have to be erected several times (saving in weight).

During the supervision of the manufacture quality, unacceptably high lacks (lacks of fusion) were ascertained on the butt welded parts of the chord sections. According to the valid quality specifications, these welds should be ground and rewelded again.

These time-consuming repairs on the building site could be avoided, however, by transferring the tensile force through 2 semi-shells arranged above the butt weld. Figure 16 shows this semi-shell solution.
Before installing the semi-shells, the welder had to give proof of their skill by means of applied welder tests under siteshop conditions. At the same time, these investigations served for determining the scatters of the bearing load. The scatters were less than $\pm 10\%$ of the mean bearing loads.

4. WDL-Hangar in Mülheim-Ruhr

Accommodation of 2 Zeppelin-airships

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Static system</th>
<th>Girder dimensions</th>
<th>Biggest tube dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>42,0 m x 25,7 m; total length: 92,0 m</td>
<td>triangular girder as arc</td>
<td>height 2,0 m, width 1,5 m</td>
<td>$\varnothing 168,3 \times 8,0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\varnothing 139,7 \times 6,3$</td>
</tr>
</tbody>
</table>

Initiation: 1988
Client: WDL-Mülheim
Design and statics: Comp. Bachmann S.A., F-94200 Ivry sur Seine
Steel construction: Comp. SCMV, F-27150 Etrepagny

The WDL-hangar (fig. 17) is to offer space for two Zeppelins. The girders are designed in the same way as the hangars for the Airbus industry; e.g. with flattened tube ends. Figure 18 represents the dimensions of the girders with the most important details. The shape and dimensions of the bolted connections can be taken from figure 19. In the support area, some members were designed with angles $\angle 35^\circ$. Since $35^\circ$ are regarded as lower limiting value for the application of the welding method used (inert gas, MAG), some test specimens were produced under siteshop conditions. Then, they were sawn up into single plates (fig. 20) and checked with regard to lacks of fusion and effective weld thickness by means of macro sections. Despite some occasional lacks of fusion and pores, these welds could be accepted for the field of application.

For a more accurate control of the seams on the top ($0^\circ$) and in the area of the fillets ($180^\circ$), both parts in the middle were sawn up a second time. Figure 21 shows the investigated positions after etching. As it becomes evident from figure 21, the weld on the top is designed optimally where, according to figure 21 below, the theoretical weld point was not reached as expected. Despite lacks of fusion of the first layer (due to the small angle between bracing and chord member), for the second layer the welder reaches a good fusion of the parts to be connected. Since this seam in the structure is often stressed by shear forces, and for hollow section joints the lowest part of the load is transferred in the critical area, the performance of this welding detail can be regarded as sufficient.

After completion of the truss girder it was stated that the wind load assumptions had to be reconsidered. This resulted in a reinforcement of the chord members of the truss. Fig. 22 shows the position of reinforcements. They were made from $\Box$ steel with the dimensions $30 \times 30$ and were welded to the chords by means of long discontinuous seams. The photographs of figure 23 demonstrate these reinforcement measures.

5. Lottery Trust Company, Wiesbaden

Two new storeys, which have been suspended by two triangular girders made of circular tubes, were mounted on a building classified as a historical monument.

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Type of supporting structure</th>
<th>Total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>length: 36,0 m; width: 6,0 m; height: 3,0 m</td>
<td>Two triangular girders</td>
<td>220,0 t</td>
</tr>
</tbody>
</table>
Biggest tube dimension: Ø 406,4 x 36,0 mm
Initiation: 1989
Client: Lottery Trust Company Hesse, Wiesbaden
Design: Dipl.-Ing. Arch. E. Schwarz, Taunusstein
Statics: Grebner Consulting Engineer, Mainz
Steel construction: Stahlbau Schäfer GmbH, Ludwigshafen

The supplementary building of the main house of the Lottery Trust Company, Hesse, is suspended by two triangular tubular girders above the old building (fig. 24). For reasons of relief, an additional supporting structures is arranged as plane frame-structure in the middle of the building, which is connected with the triangular girder via an additional girder. Both frame structures (triangular girder and plane frame structure) take the total load jointly.

During the erection period the service in the old building was continued. The assembly, however, was very difficult. The triangular girders were completed in the workshop (fig. 25). Due to the location of the properties, they could not be lifted in one piece by means of mobile crane systems. A single crane with a lifting capacity of 1000 t (the biggest mobile crane in West-Germany) could not bring the respective triangular girder into the final position since the distance from the location of the mobile crane to the final position of the structure was too big. For this reason, a scaffolding was built next to the house for the pre-assembly. The complete tubular trusses divided into three parts were supplied to the siteshop, welded together (fig. 26), and the butt joints were tested on 100% US and 100% X-ray. Then, the triangular girders together with the plane frame girders (weight approximately 300 t) were shifted up to 30 m. For shifting, Teflon-bearings and soft soap were needed. Fig. 27 shows the steel structure after completion of the steel construction work.

For assembling reasons, the braces of the lower base surface have been connected to the tubes by means of steel sheets. Since the load bearing capacity of such joints is not yet regulated, investigations on the load bearing capacity have been carried out. Fig. 28 shows the test specimen in the 3 MN-tensile testing machine. On some places of the lattice trusses smaller overlaps occurred between the web members, since the slopes of the braces were given. For welding reasons, a segment of one sheet steel, t = 20 mm, has been arranged between the chord members. Fig. 29 shows one of these points. This problem could be avoided if e.g. a certain eccentricity would be accepted as it is allowed in all calculation rules [DIN 18808, Eurocode 3 etc.].

Conclusions

The present paper tries to present the most interesting structures of the last years and shows the problems that occurred. Due to the time available and the restricted number of pages, a more detailed report could not be given. Nevertheless, the authors hope that the defects mentioned do not appear for newer structures; or that they can be reduced. Most of the defects are put down to the pressure of schedules and to the less time envisaged for the design.

References

Ohlemutz, A.: Überland-Transport einer Stahlkonstruktion
Der Stahlbau 51 (1982), S. 89

Mang, F.: Ermüdungsverhalten geschweißerter Hohlprofil-Fachwerk- knoten aus Stahl
IABSE (IVBH) Colloquium 1982, Lausanne
Mang, F.; Wolfmüller, F.; Yoon, W.Y.

Theoretical and Experimental Investigations on the Tubular Structure of Changi Hangar Roof, Singapore
Welding of Tubular Structures, Boston, 1984, proceedings

Fig. 1a   Dimensions of the hangar roof in Singapore

Fig. 1b   Assembly of the single parts of the hangar in a height of 5.0 m above the ground
Fig. 1c  View through the main girder (Appron truss with chord dimension Ø 813 mm)

Fig. 1d  K-joint no. 4 from the left, sight from the inside of the rectangular girder
Fig. 2  Transport of a structural member to the final place for installation

Fig. 3  Support of the roof structure during transport
Fig. 4  Complete roof plate before lifting

Fig. 5  Suspension point of the Appron truss during lifting
Fig. 6 Bolted connections with straps

Fig. 7 Bearing bracket welded together from steel sheets with a thickness of 100 mm. Supporting point of the main girder
Fig. 8  Changi Hangar structure

Fig. 9  Results of fatigue (CA) tests on specimens with arc strikes, welding point and base material
Fig. 10 Ready assembled girder of hall 2 of the Airbus-Industry, Bremen-Lemwerder
Status: January 1984

Fig. 11 Vertical girder no. 9 and 10 of hall 2
Fig. 12  Internal view of hall 1. In the front (right) the final bevelled girder with special pedestals can be seen.

Fig. 13  Low end design of the vertical girder no. 14 of hall 2.
Fig. 14 Joint with semi-flattened hollow sections

Fig. 15 Connection of the chord members, semi-flattened joints of circular hollow sections due to reasons of space can be well seen. Assembly of the single segments to the arc girder was realized by high-strength bolts.
Fig. 16  Arrangement of semi-shells on the joints of the tensile chords

Fig. 17  General view of the WDL-hangar in Mühlheim, Ruhr
Fig. 18  Triangular girder of the WDL-hangar and details of the supports

Fig. 19  Shape and dimensions of the bolted chord connections
Fig. 20  Test specimen after cutting to check the welding failure

Fig. 21  Detail plan of the critical position; the theoretical weld point has not been reached as expected
Reinforcement measures on the triangular girders No. 2, 3, 8 and 9
WDL-hangar in Mülheim/Ruhr

Fig. 22  Position of reinforcements for the chord members

Fig. 23a  Reinforcements additionally arranged to the tensile chords by means of welded steel with the dimensions 30 x 30
Fig. 23b  Welds at the end of the reinforcement chords on the tensile chords

Fig. 23c  Reinforcement chords with discontinuous welds after applying the coating.
Fig. 24  New building of the Lottery Trust Company in Wiesbaden, Hesse

Fig. 25  Triangular girder in the sitehop shortly before completion
Fig. 26  First triangular girder after completion of the mounted butt welded chord members

Fig. 27  Complete structure after assembly of the steel structural members
Fig. 28a  Detail plan - connections of the members on the lower total plane

Fig. 28b  Test specimen in the 3 MN testing machine
Fig. 29 Stiffening plate arranged in the joint area due to the smaller overlap of the web members
An increasing number of membranes are used as space frame roofing in Japan these days. A space frame is a very light roof structure. Excelling in design essentially, a space frame does not require ceiling in general. When used as light and highly translucent roofing, a space frame will drastically reduce the roof weight, forming bright open space thanks to natural illumination. Reduction in roof weight is advantageous in designing a large span structure. And combination of a space frame and a membrane that can be freely shaped brings about a considerable effect. Such space frames are used for a variety of applications including large assembly halls, sports hall and shopping centers as well as sheds over passageways and entrance canopies.

However, the peculiarity of a membrane causes not a few problems. In view of the "design" of a space frame with a membrane, attention must be paid to the following two points.

First, a space frame is a structure in which a load is transmitted mainly by an axial force. In order to bring about the structural advantage to a maximum extent, a membrane and a space frame must be so jointed that bending stress due to the tensile force of the membrane will not occur in the space frame. Secondly, the direction of a load applied to a space frame is different from ordinary roofing since increase and decrease in the tensile force of a membrane transmit external force to the space frame. Therefore it is important to analyze the membrane section for correct structural analysis of a space frame.

1. Introduction

Space frame can form structures on lightweight and high rigidity by dynamical rationality that can be obtained by combination of members in three dimensions. The number of space frames is increasing in various places as typical construction forms of large space structures in recent years. Also in Japan, standardization and industrialization of members and joints have been advanced with the improvement of structural analyzing methods by the progress of computers, and the space frame mass-production system has been established. Remarkable things among them are various system trusses. By the comprehensive management of design, analysis, and
production system of trusses by computers, works at construction sites have been simplified and accuracy has been greatly improved. On the other hand, membrane materials have been used for small structures such as tents and sunshades since ancient times. They have not been used for permanent structures because they lack reliability and stability as materials. However, materials of superior strength and durability have appeared in recent years, and they are used as construction materials for large-scale permanent structures. With the establishment of related laws and regulations, membrane materials now can be used comparatively easily for structures in Japan also. From such a background, membrane materials are considered as new construction materials, and can be seen frequently in Japan now. Of course their structural rationality by the combination with the space frame is evaluated, but they are highly evaluated for their productivity of space, their response to complicated shapes, etc. Following are the major characteristics of a space frame with membrane roof.

1) It is advantageous for larger space structure to become lightweight structures by the combination of membrane and space frame.
2) Although space frame is a structure, it has high design possibilities, and generally no ceiling materials are required. Therefore, when it is combined with translucency of membrane materials, an open and individual style space can be composed.
3) Joint detail of a space frame is simple compared to that of a steel frame structure, and its shape free degree is high. Possibly it will become better to make more varied shapes by the combination of membrane materials whose flexibility is good and whose responsiveness to complicated curved face, etc. are excellent.

As mentioned above, the combination of the space frame and membrane has structural rationality and possibility for unique design to cope with diverse needs for construction space.

2. Various Types of Space Frame with membrane Roof

Examples of typical space frames with membrane roofs are introduced in the following. One of Japan's typical system trusses "TM truss" is used in these examples. TM truss is a system to form a space frame by joining ball joints with bolt holes for joining members and end bolt set pipes. (See Figure 1.) Membrane materials are all Teflon coated glass fiber.

![Figure 1. Outline and Cross-section Detail of TM truss](image-url)
Photo 1. Osaka International Trade Fair Square
Cylinder Shape 3024 m² Pipe: Φ 89.1~Φ 165.2

Photo 2. The Nakayama Turf Entrance Canopy
Saddle Shape 3700 m² + 1900 m² Pipe: Φ 76.3~Φ 190.7

Photo 3. Orio Sports Center (Gymnasium)
Curved Surface 1976 m² Pipe: Φ 89.1~Φ 190.7
Photo 4. Kaiseizan Outdoor Stage
Double Arch Shape 760 m² Pipe: Ø 76.2–Ø 139.8

Photo 5. Toyota Tokyo Auto Showroom
Circular Dome Shape 200 m² Pipe: Ø 89.1

Photo 6. "LAPIA" Hachinohe Shopping Center.
Circular Dome Shape 1385 m² Pipe: Ø 76.3–Ø 190.7
3. Design Process of Space Frames with Membrane Roof

Actual design process of a recently constructed space structure is introduced in the following. This structure was constructed as a part of a shopping center in Hachinohe City, Aomori Pref. and it has various playing facilities inside for children. It was planned in compliance to the shop owner's idea to accommodate a large number of customers even in the wintertime in cold Hachinohe City. It can be said the example sufficiently displayed characteristics of a space composed by membrane and space frames. (Photo 6.)

![Figure 2. Skeleton Drawing of "LAPIA"](image)

The shape of the space frame is a circular dome whose diameter is 42m, rise 10.337m, and truss depth 2m. (Figure 2.) Lower structure is of SRC, and the space frame is installed on the rooftop (GL+24.6m) with 16 supporting nodes. Supporting conditions are shown in Figure 3 and loading conditions are as follows:

**Loading Condition:**

1) Dead Load: 105 kg/㎡
2) Snow Load: (whole surface) 154 kg/㎡
   (half surface) 169 kg/㎡
3) Wind Load: 238 kg/㎡
4) Seismic Load: coefficient of story-shearing force for roof=0.75

The TM Program was used for analysis, and each load was made to act on each joint as concentrated load.
Figure 3. Supporting Conditions of Space Frame

Figure 4. Mechanism of Load Transmission

P : Nodal Load
T : Membrane Tension
Figure 5. Member Stress Diagram
Figure 6. A-A Section Truss Deformation

Structural Analysis

Usually, the acting loads at the time of analysis are; dead load and snow load become vertical forces, and wind load becomes a force of perpendicular to the space frame surface. However, since the membrane transmits its load to the lower structure by the increase and decrease of its tensile force, the force acting on the space frame has components different from ones in normal conditions. (See Figure 4.)

It is necessary to evaluate the force acting on each joint of space frame from the result of membrane analysis.

In the following result of analysis, a comparison of ones by normal load input and by the acting load from membrane are indicated. (They are indicated the results during snow Loading which was critical in this time.)

a) Stress (Figure 5.)

Large tensile stress occurs in the direction of circumference of the peripheral sections of both upper and lower chords, and members in the central part become compression members inversely, which indicate general dome structural behavior. No significant change is observed in the case of a normal load, but the value of stress becomes 60-90%.

b) Deformation (Figure 6.)

Deflection in the central part of the dome is about 1/3000 of span and spread to the outside of the peripheral part is about 1/6500. It can be said the rigidity of the dome itself is very high. In normal loading conditions, deflection in the central part becomes about 85% and spread to the peripheral part about 80%.

As mentioned above, it is understood that the values of both stress and deformation when the load obtained from the membrane analysis is input becomes larger than those when normal load is input.
Joining the space frame and the membrane

The space frame and the membrane are joined through the secondary of steel frame. The space frame was analyzed by evaluating the force obtained by the membrane analysis as mentioned above, and, however, as indicated in Figure 7, the force in the horizontal direction (to the truss surface) that is offset to each other in general section becomes very strong at the ends, which generates large bending stress to the space frame joints. Since all joints of the space frame are theoretically pin joints and space frame is the structure to transmit load by axial force, it cannot resist bending stress. Therefore, as indicated in Figure 8, it was designed to form only axial force acts on space frames by forming latticed girders setting the members resist bending stress in secondary steel frames.

![Figure 7. Bending Moment Caused by Membrane Tension at the Ends](image)

![Figure 8. Joining Detail of the Space Frame and the Membrane](image)
4. Conclusion and Problems in the Future

1) It is necessary to take into consideration dynamic characteristics of membrane materials in the analysis of a space frame with membrane roofs. When the input load at the time structural analysis is made the acting force from the membrane, it is highly possible that stress and deformation become larger than those in normal conditions. It is, therefore, necessary to design each section by sufficiently evaluating the result of analysis of the membrane.

2) It is necessary to make the joint of membrane and space frame as the bending stress generated by membrane tension will not act on the space frame. As an example introduced in this paper, a form must be taken in which bending stress is disposed by the secondary steel frame used between membrane and space frame and only axial force acts on the space frame. However, the current method also has a big problem. Secondary steel frames which are usually light and small members for fitting roofs become almost the same size as the members for the space frame, and their quantity is also large. It is difficult to obtain the designer's agreement on these things, and it is also a big economical issue. It is necessary to improve these point in the future.

The Tension Truss Dome shown in photo 7. is a special case where such a problem of joining of the membrane and the space frame has been solved. In contrast to all the above examples that membrane roofs are installed on a space frame, the roof of this dome is formed by membrane hung inside. Therefore, it will require no secondary steel frames and there is no occurrence of bending stress. Although its free degree for shaping is low, it is a typical case that the makes the best use of the characteristics of a space frame having a membrane roof.

Photo 7. The Tension Truss Dome - Bajikoen Indoor Horse Riding Field

Note (Architect,Structural Engineer/General Constructor)

Photo 1. Osaka Urban Development & Housing build Dept. • K.Tohata & Associates, Architects / Obayashi Corp. • Matsumura-Gumi Corp.
Photo 2. Japan Racing Facilities Co., Ltd. • Matsuda, Hirata & Sakamoto Architects Planners & Engineers / Mitsui Construction Co., Ltd.
Aoki Construction Co., Ltd. • Muramoto Construction Co., Ltd.
Morimoto-Gumi & Co., Ltd.
Photo 3. YAS & Urbanists / Sato Kogyo Co., Ltd. • Oishi Kosuten Co., Ltd. • Yasui Gumi Co., Ltd.
Photo 4. Environmental Engineering Consultants Co., Ltd. / Ito Kensetsu Co., Ltd.
Photo 5. General Koei Co., Ltd. / Asahi Construction Co., Ltd.
Photo 6. Ishisato Architecture & Engineering Firm Inc. / Kajima Corp. • Taisei Corp. • Fujita Corp. • Shimizu Corp. • Obayashi Corp. • Toda Corp.
Photo 7. Toba Planning Co., Ltd. / Shimizu Corp.
1. Introduction

The structures made of Structural Hollow Sections (SHS) described in this paper are part of the giant Olympia & York Canary Wharf Development in London, England. They include a High Roof and two Low Roofs over the Canary Wharf station of the Docklands Light Railway (DLR). The structural form, including high glazed arches, evolved from the Architect's desire to reflect the tradition of London's great, older railway stations. Along with the adjacent retail and parking structures they form the centre of the Development as illustrated in Fig.1 and in a view of the DLR station shown in Fig.2.

2. Description of Space Trusses

The station roof consists of the High Roof over the central hall of the railway station measuring 38.0 m by 47.6 m and two Low Roofs extending some 32 m north and south. Fig. 3 shows an elevation of the roofs and Fig. 4 an inside view of the station, as seen from a model. The plan of the High Roof framing is shown in Fig. 5. It consists of seven identical units 6.7 m wide with a pitch of 15.5 m which are designed to act as two hinged arches. Each unit is made up of two curved trusses leaning towards each other and interconnected by horizontal tie members creating a trapezoidal cross-section. The sides of the units are glazed, while the valleys between adjacent units form the gutters. The top and bottom chords are made of Ø 219 Circular Hollow Sections (CHS) of varying thickness with the diagonal and horizontal members using Ø 114 CHS. One-half of the double truss unit is illustrated in Fig. 6. Most joints are multi-planar with 3 to 5 members meeting at
a node. The trusses are supported on exposed cylinder shaped steel bearings seated on concrete filled steel clad trapezoidal buttresses which are protruding from the adjacent roofs high above the railway tracks. The endwalls of the High Roof are louvred with glass slats with the vertical mullions suspended from the exposed 3 plate steel arch which forms the edge of the High Roof.

The Low Roofs consist of two 11.3 m shallow arched spans and one centre span of 13.5 m between deep plate girders supported on concrete columns, as shown in Fig. 3. Top and bottom chords of the curved trusses are 100 x 100 x 5 Rectangular Hollow Sections (RHS) while the diagonals are 70 x 70 x 5 RHS. The trusses are spaced at 1900 mm with pitched glazed skylights alternating with solid roof.

3. Analysis and Design

The analysis of the SHS trusses were carried out in the elastic range for various load combinations in the north/south and east/west directions with both symmetric and non-symmetric load distributions considered. The roof system is subject to its self-weight, wind and snow loading with allowances made in the valleys for snow build-up. The wind load, acting not only on the end walls but also on the sloped glazing of the roof, was based on the existing British Standards and compared with 50 year return pressures obtained from wind tunnel measurements carried out for the entire Canary Wharf development. These measurements show that there was no funnelling effect due to the supporting structures on either the High or Low Roofs. Temperature effects and the predicted displacements of the surrounding buildings are all taken into account. The system of 7 double trusses, together with the interconnecting struts and the end arches was analyzed as one unit taking full advantage of the high statical indeterminacy present (see Fig. 11). The maximum total unfactored compressive force in the bottom chord is around 350 kN with forces in diagonals typically around 150 kN mark. The layout of typical joints is illustrated in Fig. 7 with the support joints shown in Fig. 8. All buttresses typically support two end joints of the neighbouring units and share in the transfer of the total north/south wind (see Figs. 9 and 10).

The SHS roof elements were designed to BS 5950 and British Steel Corporation’s manual for design of SHS joints [1]. Additional guidance was provided by CIDECT Monograph No. 6 [2]. The strength of multiplanar joints was reduced by 10% as recommended by Makino et al [3]. Gap joints were selected to avoid overlapping of bracings. Full strength butt joints were provided in the top and bottom chord members at each change in curvature. Two sets of CHS sizes were used with wall thickness varied between 8 and 12.5 mm for top and bottom chords to suit the design forces. Flange type bolted splices were introduced by the erector in the centre section of the truss arch units to facilitate transportation and field assembly.

4. Special Features

The double truss units (as shown in Fig. 6) act as 2 hinged arches spanning 38 m between pairs of bearings. Because of the exposed position of the bearings and the fact that the end rotations due to imposed loadings are very small, it was decided not to develop machined pin type bearings but to use a pair of elastomeric bearing pads inside an architectural plate enclosure (shown in Fig. 9) to transfer the end thrusts and shears as shown in Figs. 8 and
9. This arrangement also provides adjustability during erection to suit fabrication tolerances. The concrete filled steel sheet buttresses shown in Figs. 9 and 10 are shaped to satisfy aesthetic requirements. The curved exposed roof surfaces and the gutters of the high roof (see Fig. 11) are made of stainless steel sheet material with the trusses and buttresses painted red.

The plate girders of the low roof are supported on top of the concrete columns on 50 mm thick cylindrical elastomeric bearing pads designed to reduce sound transmission.

5. Fabrication and Erection

Prototype connection units were fabricated for general approval of the welding and appearance. Finished double truss units were completely shop assembled and match marked in the shop for erection.

The erection of the double truss units was very difficult logistically, due to the need to keep the DLR track system operational and protected at all times, except for a few hours in the early morning. The erection of the units was accomplished by a mobile tower crane positioned beside the low roof. Temporary central scaffolding support towers (see Fig. 12) with suspended working platforms were constructed to support the half arch units marked 1 and 2 with a central closing element marked 3 bolted into place. After completion of all connections, including those between adjacent units, the scaffolding towers are lowered thus transferring the load fully onto the elastomeric bearings. The architectural plate closure units are then installed.

Acknowledgements

The following were the firms involved in design, fabrication and erection of structures described:

Adamson Associates, Toronto, Canada, Architects
Frederick Gibberd Coombes & Partners, London, U.K., Associate Architects
M.S. Yolles and Partners, Toronto, Canada and London, U.K., Structural Engineers
Mowlem Civil Engineering, Project Managers
Blight & White Ltd., Contractors

References

Fig. 1 South Elevation of Canary Wharf Development
Fig. 2 Canary Wharf Station.
Fig. 3 East-West Elevation of High and Low Roofs.

Fig. 4 Inside View of Canary Wharf Station.
Fig. 5 A Quarter of High Roof Framing Plan.

Fig. 6 Half of Double Truss Unit.
Fig. 7 Selected High Roof Joints.

Fig. 8 High Roof Support Joint.
Fig. 9 High Roof Buttress Details.

Fig. 10 High Roof Support System.
Fig. 11 High Roof Detail.

Fig. 12 Erection Diagram.
EXPERIMENTAL STUDY ON EXPOSED STEEL SQUARE TUBULAR COLUMN BASES USING A SPECIAL GROUT METHOD

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Summary

The column base is one of the most essential parts in designing earthquake-resistant structures. In our study, bearing experiment and experiments with a reduced model subjected to bending and shearing force under constant axial compression as well as with a full-scale model were carried out in order to clarify mechanical characteristics of the exposed type column base of a steel square tubular column. In constructing exposed type column bases, the most crucial problems are how to secure anchor bolt position accuracy and to keep the base plate's bottom surface and the leveling mortar in full contact. In our experiments, a special grout method was adopted to deal with those problems. Specific procedures of this method are to enlarge bolt hole on the base plate for absorbing some horizontal displacement error of the anchor bolt, and, with the use of special washer, to fill well-flowing non-shrinking grout mortar in the clearance between the base plate bottom surface and the foundation concrete upper surface as well as in the clearance between the anchor bolt and the base plate for ensuring excellent adhesion.

1. Introduction

Our study deals with bearing experiment and experiments with a reduced model subjected to bending and shearing force under constant axial compression as well as with a full-scale model, and describes the mechanical characteristics respectively.

The outline of the bearing experiments is as follows:

It is known that the ultimate strength of an exposed type column base greatly depends on the bearing strength of concrete under the base plate. Experimental studies on concrete bearing strength have been carried out for many years, but they mostly deal with reduced size models which have no reinforcement, and rarely use full-scale models. Our study, however, achieved a life-size under-stress representation of the bottom surface of the exposed type column base plate, and evaluated its bearing strength.

The experiments using a reduced model subjected to bending and shearing force under constant axial compression are outlined as follows:

A steel square tube was used as a column and deformed bars were used as anchor bolts. The size is one third the real one. The experiment dealt with the situation that the specimen of column base is subjected to bending force and shearing force under constant axial compression and its mechanical characteristics are studied, using magnitude of axial compression and size of the foundation concrete column as experimental variables. At the same time, review was also given to the bearing force...
evaluation formula proposed in consideration of bearing experimental results, and to evaluation of rigidity considering axial compression. The outline of the experiments using large steel square tubular columns is as follows:
A large steel square tube was used as a column member. This experiment dealt with the case that the exposed type column base with anchor bolts made of deformed bars was subjected to bending and shearing force, and aimed at verifying the evaluation formula proposed for the reduced model experiments.
The special grout method used in the reduced model and full-scale model experiments is outlined in Fig. 1

2. Bearing experiment of the foundation concrete

(1) Experimental plan

The specimen is shown in Fig. 2. In the actual experiment, as shown in Fig. 3, the specimen placed a loading plate as a rigid body on top was pressurized by a hydraulic jack to measure maximum compressive load in order to obtain its bearing strength. The bearing condition shown in Fig. 3 was a simple representation of an assumed uniaxial eccentric bearing condition of compressed concrete at the exposed type column base. Bearing area ratio, edge distance dimension and eccentric distance were factors considered in this experiment. Table 1 shows the experimental condition.

(2) Experimental results and discussion

Figure 4 shows the relation between bearing strength ratio and the edge distance. The figure generally shows the following tendency:
If the edge distance remains the same, as the bearing width decreases, the bearing strength ratio increases. If the bearing width remains the same, as the edge distance increases, the bearing strength ratio increases. The bearing strength is generally given in the following formula, [1]

\[
\frac{\sigma_b}{\sigma_c} = \alpha \left( \frac{A_b}{A_e} \right)^{0.86} \quad (1)
\]

\( \sigma_b \) Bearing stress
\( \sigma_c \) Cylinder compressive stress of concrete
\( A_b \) Bearing area, \( A_b = b \times X \)
\( A_e \) Effective supporting area, \( A_e = (b + 2e) \times (X + 2e) \)
\( \alpha, \beta \) Coefficients determined by experiment

Figure 5 shows the relation between the bearing strength ratio and the bearing area ratio. The mean value of coefficient (\( \alpha \)) in this experiment was thus calculated to be 0.86 (\( \alpha = 0.86 \)). This calculation supported the assumption that the bearing strength formula can be arrived at by eq. (2) under the conditions of this experiment.

Figure 5 shows eq. (2) in a solid line.
On the other hand, compressive stress distribution under the base plate bottom surface is assumed to be rectangular, and eq. (2) was then used to obtain a relation between the axial compression on the bearing area and the bending moment due to eccentricity of the gravity of them. It is shown in Fig. 6. The experimental values corresponded relatively well to the calculated ones.

3. Reduced model experiment subjected to bending and shearing force under constant axial compression

(1) Experimental plan

The specimen is shown in Fig. 7. In this experiment, a column base about one third the actual size with a riser foundation concrete was used. The focal point of the experiment was how the magnitude of constant axial compression and the cross sectional dimension of a foundation concrete column would influence mechanical characteristics of the column base. Alternating positive-negative loading was carried out under retaining required axial compression and by providing lateral force at a point 750 mm from the base plate bottom surface. Displacement of various parts of the specimen was measured with displacement gauges, and strain at each component was measured with strain gauges. No tension was introduced to anchor bolts.

(2) Experimental results and discussion

i) Experimental results

Table 2 shows the experimental results. As the cross sectional dimension of the foundation concrete column increased, the maximum bearing strength also increased. When the axial compression was 882 kN, the foundation concrete collapsed before the anchor bolt reached the yield point.

ii) Relation between load and displacement

Figure 8 shows a relation between lateral load (Q) and displacement of loading positions (Ø). The dashed line shows N-Ø effect. Figure 9 shows the envelope curves of relation between the moment (M) allowing for the N-Ø effect and the rotation angle of member (R). Figure 8 shows a tendency that the hysteresis curves of the specimen under axial compression being 0 kN to 588 kN changes its shape from a reversed S form with a slip to that with bulges as the axial force increases. This result deduces a supposition that elongation deformation of anchor bolts would become a dominant factor as the base plate was considered sufficiently rigid in this experiment. Figure 9 shows that the maximum strength increased with the increase of axial compression, and that, however, when axial compression was very great, the strength reached its maximum and suddenly fell considerably due to the collapse of the foundation concrete.

iii) Ultimate bending strength

Among the two moments, one at the time of tensile side anchor bolt yield and the other at the collapse of the foundation concrete, the smaller one was selected as an experimental value of ultimate strength. For this experiment, a formula based on a strength cumulatively added by concrete and anchor bolts was applied to calculation of the strength of the...
exposed type column base subjected to axial compression and bending moment. In this respect, it is necessary to modify concrete compressive strength ($F_c$) by a bearing coefficient ($\lambda = \delta b / \delta c$) as the concrete at the bottom surface of the base plate is subjected to a local compressive force. From the bearing experiment, the bearing coefficient ($\lambda$) was obtained from eq. (2), and the M-N interaction of concrete and anchor bolts was given from reference.[2] The M-N interaction was arrived at as shown in Fig. 10.

For reference, the curve with the case of $\lambda = 1.0$ was drawn in the figure. In Fig. 11, the vertical axis was given to the ratio ($e_{Mu}/\mu_{Mu}$) between the experimental values of ultimate bending strength ($e_{Mu}$) and its calculated values ($\mu_{Mu}$), and the horizontal axis was given to the ratio of axial compression to maximum compression of the foundation concrete column ($B_x D_x F_c$) shown in Fig. 3.

When the axial compression is from 0 kN to 588 kN, the evaluation formula based on the cumulative strength gives values well corresponding to the experimental ones as yield of the tension side anchor bolts comes earlier than the collapse of the foundation concrete. For cases where the foundation concrete collapse precedes, the experimental values and calculated ones corresponded relatively well in our experiment, but further study on them is necessary.

iv) Evaluation of rigidity

As shown in Fig. 9, when axial compression is applied to the column member, the rigidity of the column base varied depending on that. When no axial compression is applied to the column member, the rotational rigidity ($K_{g}$) of the column base is proved to be obtained by eq. (3). In this experiment, the rotational rigidity of the column base with axial compression acting on is assumed to be evaluatable from eq. (4).

\[
K_{g} = \frac{dM}{d\delta} = \frac{M(dt + dc)}{\delta a + \delta b}
\]

\[
\delta a \quad \text{Elongation of the anchor bolt}
\]

\[
\delta b \quad \text{Bending displacement of the base plate}
\]

\[
K = \sqrt{1 + \beta} \cdot K_{g}, \quad \beta = \frac{N \cdot dc}{n \cdot Ty(dt + dc) + N \cdot dc}
\]

\[
dc \quad \text{Distance between the center of the steel column cross section and the steel column flange}
\]

\[
dt \quad \text{Distance between the center of the tension side anchor bolt group and the steel column flange}
\]

\[
n \quad \text{Number of tension side anchor bolts}
\]

\[
Ty \quad \text{Yield strength of the anchor bolt}
\]

The experimental values were arrived at by allowing for bending and shearing displacement of the column member from loading position displacement and for additional bending due to the N-\delta effect. Comparison between the experimental value ($e_{Kg}$) and calculated one ($c_{Kg}$) resulted in Table 3. It shows a relatively good correspondence.

4. Experiment with full-scale large steel square tubes

(1) Experimental plan
The specimen is shown in Fig. 12. Table 4 shows the experimental conditions. Factors in the experiment are the anchor bolt diameter, rigidity of the base plate, and the cross sectional dimension of the foundation concrete column. Based on the assumed position of an inflection point, alternating positive-negative loading was carried out by applying lateral force at a point two meters height from the base plate bottom surface. Displacement of various parts of the specimen was measured with displacement gauges, and strain of each component was measured with strain gauges. No tension was introduced to the anchor bolts.

(2) Experimental results and discussion

i) Relation between load and displacement

Figure 13 shows typical hysteresis curves representing the relation between bending moment (M) and the displacement (\(\delta\)) of the position one meter height from the base plate bottom. Its experimental results are shown in Table 5. For specimens with greater base plate rigidity, 35-22-1 and 35-22-3, elongation of the anchor bolts became a dominant factor, changing shape of hysteresis curves into a slip form due to plastic elongation. On the other hand, specimens, 35-22-2 and 40-25-1, have lower base plate rigidity, and their hysteresis curves were changed into a spindle shape.

Figure 14 shows typical examples representing yield conditions of the anchor bolt and the base plate. Yield of anchor bolts always occurred after yield moment of the steel column, and no drastic rigidity fall was seen.

Yield of the base plate, except for the specimen 35-22-2, followed the yield of the anchor bolt as shown in Fig. 14 and like anchor bolts, the partial yield of the base plate gave no influence on the hysteresis curve of the column base connection until whole part came to a yield point.

ii) Ultimate bending strength and rigidity

Table 6 shows comparison between the values calculated by using a bearing force evaluation formula and a rigidity evaluation formula in the reduced model and the experimental values. They corresponded with each other relatively well.

5. Conclusions

The above experiments revealed the following:

Cross sectional dimension of the foundation concrete column is a crucial factor governing ultimate strength of the exposed type column base. Originating from this fact, a new method to evaluate the ultimate strength of the exposed type column base, based on the experimental results is proposed. Values obtained using this method closely match the experimental values.

As axial compression increases, the ultimate strength and rigidity increases. But the ultimate strength of column bases with smaller cross sectional dimensions of the foundation concrete column falls considerably after reaching the maximum load, so it is necessary to maintain an appropriate edge distance.

6. References

Figure 1. Special grout method

Figure 2. Test specimen

Figure 3. Test set-up

Figure 4. Relation between bearing strength ratio and edge distance

Figure 5. Relation between bearing strength ratio and bearing area ratio
Figure 6. M-N interaction Curve

Figure 7. Test specimen

Figure 8. Q-δ curve
Figure 9. M-R related envelope allowing for N-δ effect

Figure 10. M-N interaction curves

Figure 11. Comparison between experimental values and calculated values

Figure 12. Test specimen

Figure 13. M-δ curve

Figure 14. Yield conditions of anchor bolt and base plate
### Table 1. Symbols of specimens and conditions

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### Table 2. List of experimental results

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A: Yield of anchor bolts  
B: Collapse of foundation concrete

### Table 3. Comparison between experimental values and calculated values

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### Table 4. Symbols of specimens and conditions

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A: Yield of anchor bolts
B: Yield of base plate
C: Collapse of foundation concrete
S: Local buckling of steel column

Table 6. Comparison between experimental values of ultimate bending strength and rotational rigidity and their calculated values

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SPACE FRAMES FOR THE OCEAN PAVILION ON AWAJI ISLAND, JAPAN

Katsuyuki Nakajima
Design & Engineering Dept.
Engineering Div.
Taiyo Kogyo Corporation, Japan

Summary

The structure discussed is the roof of a steel space structure constructed at the Onokoro "AI"land Park Ocean Pavilion on Awaji Island, Japan. With a dome-shape gathered from three parts of the sphere having about 30m in radius, its respective spans are 38m in its long side, 33m in its short side, 22m in the height from the ground, and its surface area is 1,079m².

The dome consists of seven kinds of members, the number of which is 1,374. The average in length is about 3m and the diameters of the member cross-sections vary from 76.3mm with thickness 3.2mm to 216.3mm with thickness 4.5mm. The sizes of ball joints are from 110mm to 260mm in seven kinds and the total number is 340.

The structure is covered by a membrane roofing and an old wooden ship is displayed in the inner space.

1. Introduction

"Shinetsumaru" which was a "Kitamaebune" (mainly used for ocean shipment in the later half of 19th century) that was built by Takadaya Kahei who was an ocean merchant active in that period and came from Awaji Island of Hyogo Prefecture, Japan. Shinetsumaru was restored in 1985 and displayed. Then, it was presented at The International Transportation Convention held in Vancouver, representing Japan, and attracted many visitors.

It is decided that the ship is transferred to the center of the Onokoro "AI"land Park in Awaji Island, and will be preserved permanently. The visitors can climb up the deck of the restored ship from the sailing ship gallery and it can be seen from the roof of the gallery. By utilizing its gap of floor, it can be used as a circular theater for events.

The big roof covering it gives an image of a large classic Japanese ship which proceeds by catching the fair wind. An open type membrane structure by space structure is adopted for a clear frame. Through the open portion, the heroic Shinetsumaru with its sail hoisted can be seen. (Picture 1)

The decision of space structure dimensions, analysis, and construction are described here.

2. Dimensions

The structure is constructed by the following steps:

* A part of sphere with an approximate diameter of 30m is taken out.
* Its center is displaced.
It is rotated 120 degrees. Basically, it is a space structure using a triangular cone unit. The distance between the columns is about 38m. It is about 22m high. The length of the upper chord is about 3m. (Fig. 1) At the initial stage, all the upper chord nodal points are on one sphere. But it is far from the image that the ship catches wind. The image can be realized by displacing the spherical center of the 1/3 size model as shown in Fig. 2. TM TRUSS is assembled with the truss members and spherical nodal joints. At the end of the steel pipe truss member, a cone is welded to insert a bolt. The head of the bolt is supported inside the cone and the tensile force of the bolt is transmitted to the steel pipe. A wapper with a slit hole is placed in the bolt screw section which is produced from the cone. When the wapper is turned, its turning force is transmitted to the bolt and the globe and members are assembled. The wapper used between the globe and member transmits the compressive force exerted on the member to the globe. (Fig. 3) Table 1 lists the materials used.

3. Load

For the structural analysis, the following loads are taken into consideration:
- Dead load
- Snow load
- Wind load
- Seismic load

The cross-sections of the members are determined by the dead load and wind load. These two types of loads are described as follows. Membrane is used as the roof material and is set via the purlin to the globe. (Fig. 4) The dead load is assumed to be 50kg/m². The wind load is assumed to act in three directions +X, +Y, and -Y. The calculation formula for the wind load complies with the Construction Law's Ordinance as follows.

\[ w = c*q*s \]  \hspace{0.5cm} (1)  
\[ w \] Wind load (kg/m²)  
\[ c \] Coefficient of wind force  
\[ q \] Velocity pressure (kg/m²)  
\[ s \] Area (m²)  
\[ q = 60*\sqrt{h} \]  \hspace{0.5cm} (h ≤ 16m) \hspace{0.5cm} (2)  
\[ q = 120*4*\sqrt{h} \]  \hspace{0.5cm} (h > 16m) \hspace{0.5cm} (3)  
\[ h \] Height (m)  

Coefficient of wind pressure: c is set for each direction as shown in Fig. 5.

4. Analysis

The structure is supported at six points. The boundary conditions are assumed to be pin support. Fig. 6 shows the stress diagram. Fig. 7 shows deformation diagram. When a dead load is applied, the maximum compression force occurs at
the upper chords with the magnitude of 6.7 ton and the maximum tensile force occurs at diagonal members with the magnitude of 5.5 ton. The stress becomes smaller as it comes nearer to the center and is less than 1 ton. At this time, the deformation is greatest in the vertical direction, which 2.2mm. At the most nodal points, it is smaller than 2mm. The deformation in the horizontal direction is minimal.

When the fixed load plus wind load (-Y direction) are applied, a big stress that determines the cross sectional area is produced in the member at the supporting point. The maximum stresses are as follows:
* 62.0 ton (compressive) in the upper chord
* 45.3 ton (tensile) in the lower chord
* 58.2 ton (compressive) in the diagonal member

At this time, the deformation of 46.1mm at maximum is produced in the vertical direction and over 70mm in the horizontal direction.

Since the supporting points are not fixed by welding during construction before the truss is assembled, the maximum compression is 0.49 ton/cm² and the maximum tensile stress is 0.42 ton/cm² for the yield point of 2.4 ton/cm² is roller supporting is assumed for the fixed load in the boundary condition.

5. Production

Fig. 8 shows the production process of the TM TRUSS. Following the structural analysis and parts design, a computer system (CAD/CAM/CAE) is used for the machining data creation of pipes and globes and assembling drawings to be used in construction. The globe and cone which are the major parts are made by forging. For the other major parts, bolt, the head is made by forging, and the screw is made by rolling. The pipe and cone are welded by a dedicated auto-welding machine. The parts for the TM TRUSS are produced and assembled in compliance with the standard specifications, production criteria, and quality control criteria.

The parts are painted as follows:
Pipe:
1) The surface is cleaned and smoothed.
2) Zinc shop primer and epoxy resin primer are applied once.
3) Urethane is applied.
4) Finish painting with urethane.

Globe, wapper, bolt:
1) Antirust treatment by CZ COAT
2) Urethane is applied twice on site.

Table 2 shows the anti-corrosion performance of the CZ COAT.

6. Construction work

The truss was constructed by the entire scaffolding. On the scaffolding built beneath the truss framed, The members are directly assembled on the end of the truss framing surface. (Picture 2)

For checking the level during assembling, the nodal points are temporarily supported by the jacks to tighten the bolts at the coupling portions. (Picture 3) Table 3 lists the tightening torques for the bolts.

When the truss is completed, the supporting points are welded (Picture 4) and temporary supporting is removed and the membrane is set on the roof. Then, the scaffold is disassembled. Fig. 9 shows the construction process.

Truss assembling took 22 days, steel structure construction for membrane setting took 12 days, and membrane setting took 9 days.
Picture 1. General view of the Ocean Pavilion

Figure 1. Framing plan, elevation and section
Figure 2. 1/3 model

Figure 3. Outline and cross-section detail drawing

Figure 4. Joint of globe and purlin
Figure 5. Distribution of coefficient of wind pressure

Figure 6. Stress diagram
Dead load + Wind load (-Y direction)  Upper chord

Figure 7. Deformation diagram

Figure 8. Production process of TM TRUSS
Table 1. List of using materials

<table>
<thead>
<tr>
<th>Part name</th>
<th>Specifications</th>
<th>JIS number</th>
<th>Plate name / Yield point</th>
</tr>
</thead>
<tbody>
<tr>
<td>Globe</td>
<td>5T</td>
<td>JIS G 3106</td>
<td>Rolled steels for welded structure ( \sigma_y = 33 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>9T</td>
<td>JIS G 4051</td>
<td>Carbon steels for machine structural use ( \sigma_y = 35 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>JIS G 4105</td>
<td>Chromium molybdenum steels ( \sigma_y = 80 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td>Pipe</td>
<td>-</td>
<td>JIS G 3444</td>
<td>Carbon steel tubes for general structural purposes ( \sigma_y = 24 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td>Cone</td>
<td>-</td>
<td>JIS G 3101</td>
<td>Rolled steels for general structure ( \sigma_y = 25 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td>Bolt</td>
<td>5T</td>
<td>JIS G 4051</td>
<td>Carbon steels for machine structural use ( \sigma_y = 35 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>9T</td>
<td>JIS G 4105</td>
<td>Chromium molybdenum steels ( \sigma_y = 80 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>JIS G 4103</td>
<td>Nickel chromium molybdenum steels ( \sigma_y = 90 \text{kgf/mm}^2 )</td>
</tr>
<tr>
<td>Wiper</td>
<td>7T</td>
<td>JIS G 4051</td>
<td>Carbon steels for machine structural use ( \sigma_y = 35 \text{kgf/mm}^2 )</td>
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<tr>
<td>Pin</td>
<td>-</td>
<td>JIS G 4314</td>
<td>Stainless steel wires for springs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>JIS G 4105</td>
<td>Chromium molybdenum steels ( \sigma_y = 80 \text{kgf/mm}^2 )</td>
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</table>

Table 2. Anti-corrosion performance of CZ COAT and other surface treatment

<table>
<thead>
<tr>
<th>Sample</th>
<th>Salt water spray test (hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>White rust</td>
</tr>
<tr>
<td>Z COAT(130mg/dm²)</td>
<td>24 (Small amount)</td>
</tr>
<tr>
<td>CZ COAT(130mg/dm²)</td>
<td>1000 (Almost nothing)</td>
</tr>
<tr>
<td>Zinc plating(6µm) + C COAT</td>
<td>96 (Small amount)</td>
</tr>
<tr>
<td>Zinc plating(6µm, shiny chromate)</td>
<td>48 (Small amount)</td>
</tr>
<tr>
<td>Zinc plating(6µm,Yellow chromate)</td>
<td>72 (Small amount)</td>
</tr>
<tr>
<td>Hot-rolled zinc plating(150g/m² on one side)</td>
<td>24 (Large amount)</td>
</tr>
</tbody>
</table>

Table 3. Tightening torques for the bolts

<table>
<thead>
<tr>
<th>Bolt</th>
<th>M12</th>
<th>M16</th>
<th>M20</th>
<th>M24</th>
<th>M27</th>
<th>M30</th>
<th>M36</th>
<th>M42</th>
<th>M48</th>
<th>M56</th>
</tr>
</thead>
<tbody>
<tr>
<td>5T</td>
<td>4.3</td>
<td>12.0</td>
<td>23.5</td>
<td>41.0</td>
<td>60.0</td>
<td>70.0</td>
<td>80.0</td>
<td>80.0</td>
<td>80.0</td>
<td>80.0</td>
</tr>
<tr>
<td>9T</td>
<td>3.4</td>
<td>9.7</td>
<td>19.0</td>
<td>31.0</td>
<td>45.0</td>
<td>53.0</td>
<td>60.0</td>
<td>80.0</td>
<td>80.0</td>
<td>80.0</td>
</tr>
</tbody>
</table>
Picture 2. Entire scaffolding

Picture 3. Tighten the bolt
Figure 9. Construction process
STRUCTURAL DESIGN AND DYNAMIC CHARACTERISTICS OF TELECOMMUNICATION TOWERS IN NTT

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Summary

Microwave antenna towers are one of the most important facilities in a telecommunication network. More than 2,500 of these towers have been constructed so far in Japan, mostly the angle steel truss towers. But the steel towers using steel tubular members or structures have often been adopted in large scale towers. This paper describes features of the towers, the structural design method and the dynamic characteristics of the towers subjected to vibration tests.

1. Introduction

Microwave antenna towers differ from other kinds of structures in design requirements. Towers have usually been built on top of mountains and on buildings in cities, because microwave transmission requires unobstructed views. Furthermore, microwave transmission requires security in large earthquakes and typhoons, which often attack all areas of Japan. Considering the social importance of telecommunications facilities, the above-stated design requirements, and structural features of towers, NTT has established its own structural design method.

NTT has constructed more than 2,500 steel antenna towers throughout the Japanese islands. Latticed steel towers using structural angle steel (called "angle steel truss tower") are adopted in about 90 percent of all towers. But angle steel truss tower is not suitable for large scale towers structurally. The microwave antenna towers, using steel tubular members or structures, have often been adopted in large scale towers. This paper describes features of the towers, the structural design method and the dynamic characteristics of the towers subjected to vibration tests. Moreover, the dynamic characteristics of the towers determined by these tests were compared with those predicted by lumped mass models, and with those of angle steel truss towers[1].

2. Features of microwave antenna towers

The typical types of steel towers in NTT is shown in Figure 1 with the name of tower parts. Features of NTT towers are as follows:
(a) Microwave antenna towers have antenna decks on which many large scale and heavy antennas are installed. Towers have additional small steel towers on the topmost antenna deck. The small steel towers are built to be equipped with small antennas, lighting rod and aircraft warning facilities.
(b) Most of towers are constructed on buildings to improve utility value of land.
(c) In order to keep antennas sufficiently stable to meet the microwave transmission requirements under external forces, specific limits are given to the deflection and torsional angle of towers.

NTT has the following various types of microwave antenna towers now: ①
Angle steel truss type; ② Steel pipe truss type; ③ Rigid framed steel type; ④ Cylindrical steel plate type. The angle steel truss types of ① are adopted in about 90 percent of all towers. The types of ① are constructed out of angle steel members jointed together by gusset plates and bolts.

But angle steel truss type is not suitable for large scale towers structurally. The types of ② to ④ above mentioned have often been adopted in large scale towers atop buildings of about 100 meters height above the ground level. The types of ② used steel pipe with field-welded joints for main members and steel pipe with bolted joints for horizontal members and diagonal members. The types of ③ used steel pipe with field-welded joints for main members and H-steel with high-strength bolted or welded joints for horizontal members. The types of ④ are solitary large-diameter steel cylindrical shafts by welding cylindrical tubular sections. Photograph 1 show respectively the types of ① to ④ above mentioned. In addition to the above mentioned tower types. NTT has some reinforced concrete towers and some highrise buildings with antenna decks in its uppermost parts.

3. Structural design

basic requirements for structural design

Japan is located in an active earthquake belt, which surrounds the Pacific Ocean. Large scale earthquakes have caused damage every several years. And large scale typhoons often attack all over the country every year. The following common design principles have been established by NTT against earthquake or wind force.

Earthquake: (a) To be able to continue to provide proper service when subjected to STRONG earthquakes of seismic intensity V on the Japan Meteorological Agency Intensity Scale.(See Figure 2) (b) To continue to operate, though the quality is reduced by SEVERE earthquakes of seismic intensity VI. (c) To be able to prevent total destruction and maintain necessary effectiveness when subjected to DESTRUCTIVE earthquakes of intensity VII.

Structural design requirements for steel towers are shown in Table 1.

Wind: The deflection of structural members by the design wind force should not exceed the elastic limit. The design wind force is provided by the Building Standards Law of Japan, and calculated by following Eq. 1.

\[ P_w = C_w \cdot q \cdot A \]  

where \( q = 60 \sqrt{h} \quad h \leq 16m \)  
\( q = 120 \sqrt{h} \quad h \geq 16m \)  

(1)

The velocity pressure was decided on the basis of observation records, maximum wind velocity \( V=63\text{m/sec.} \) at Muroto Typhoon in 1934.

Other design principles: Besides those mentioned above, the towers must be designed in consideration of the live load (weight of antennas), snow load and the durability required by the expected service life. All steel surface must be galvanized, in principle, with the high quality specification. And, where required by law, the aircraft obstruction painting with the specification guaranteed for a long time, must be provided for all steel members.

structural planning

In order to construct the towers with adequate structural performance, appropriate structural planning must be worked out in full consideration of
the following points.
(a) A plan and elevation of a tower should be simple in principle, and a structure should be of balanced framing.
(b) Generally the structural system should be of square truss type consisting of angle steel with bolted joints, because of being most economical and most reliable in construction work. But angle steel truss type is not suitable for large scale towers structurally. The types of to above mentioned have often been adopted in large scale towers atop buildings of about 100 meters height above the ground level and have been built so as to match the local conditions.
(c) Aseismic response of tower atop building varies with period ratio of tower to building, mass ratio of tower to building and 1st period of coupled Tower-Building system, as shown in Figure 3. Therefore, the tower atop building should be planned in consideration of the vibrational characteristics of the tower and building, so as to make the period out of a resonance range.

structural design method
The following are the categories of structural design methods established by NTT for reasonable and efficient structural design.

(A) Category 1: The tower design job in this category is done by merely selecting a suitable type from the standard steel towers. The standard steel towers are classified into 30 types, with 10-40 m in height and 1 - 3 antenna decks. These standard steel towers are square in plan and composed of steel angle section forming trusses. All necessary drawings and documents including structural calculation sheets for several towers and for the relay stations in the list are in readiness for construction work.

(B) Category 2: The structural design of towers and buildings is carried out through the static structural calculation method. The static calculation method for the aseismic design of tower in this category is developed according to the results from several structural experiments and theoretical studies as well as NTT's own technical information accumulated from the tower's design in Category 3. We intend to expand the range of the design job in Category 2 by introducing some technical methods further obtainable through design jobs in Category 3.

The design seismic force for steel towers is obtained from the following Eq. 2.

\[ q_{D(t)} = \sum_{j=1}^{n} C_{0} \cdot \alpha_{j} \cdot Z \cdot w_{j} \]  

Seismic coefficient for a tower on the ground \( C_{0} \) (See Figure 4) are determined by the earthquake response analysis. Seismic coefficient for a tower atop a building \( C_{0} \) (See Figure 5) are based on the single-degree response spectrum to response waves at the top of the buildings. The primary natural period of a building and a tower can be obtained from the experimental formulas[2].

(C) Category 3: The structural design method for this category shall be applied to large structures, new structural design concepts as well as to new structural materials and construction methods. The process of this design method consists of the first phase that is carried out by the same method as is specified in Category 2, and the second phase wherein careful investigation and research are made for the aseismicity of structures, including dynamic response analysis. However, the design seismic force of the following towers atop buildings (a - d) should be obtained from plain
vibration analysis described below.
(a) Truss type steel towers on the ground over 80 meters high
(b) Truss type steel towers atop building over 60 meters high
(c) Steel towers with 5 or more antenna decks atop the building
(d) Rigid framed, cylindrical towers etc., excluding truss type towers

The design shear force for large scaled steel towers atop buildings is calculated by using the acceleration spectra on the top of the building. These spectra were obtained from the ground vibration characteristics and the primary natural period for steel towers, as follows in Eq. 3.

\[ Q_r = \sqrt{\sum_{j=1}^{n} \left( \sum_{i=1}^{n} P_{ij} \right)^2} \]  

Steel tower design acceleration spectra \( S_a \) is calculated by the procedure shown in Figure 7, using simulated earthquake waves having dynamic characteristics shown in Figure 8. The maximum response velocity spectra is shown for each kind of ground. Figure 9 indicates an example of design acceleration spectra for a steel tower atop the building on hard ground.

4. Experimental and analytical study on dynamic characteristics

NTT has carried out many experiments and analyses in order to make clear the structural performance of the steel towers. Some of these experiments and analyses are introduced as follows. Several of the towers, the type of (1) to (4), were subjected to vibration tests. The dynamic characteristics of the towers determined by these tests were compared with those predicted by lumped mass models, and with those of angle truss steel towers[1].

overview of the towers studied

The tests were applied to 16 towers as shown in elevations and plans in Figure 10. The towers ranged in height from 10 to 80 meters, and consisted of the types of (1) to (4) above mentioned: A-H are truss towers, angle steel truss towers(5) and steel pipe truss towers(6), I-L are rigid framed steel towers(6), and M-P are cylindrical steel plate towers(6).

Tower K with reinforced concrete surrounding the lower section to provide support was only erected on the ground; all the others (as shown in Table 2) were constructed on top of buildings that ranged from 2 to 9 stories in height.

E was built on top of a 3-floor penthouse that was itself constructed on top of a large-scale building. F was built on top of a 2-story small steel-frame prefabricated building. G and H on a 2-story small reinforced concrete building. N and O were both built on 3-story small reinforced concrete buildings, but N was set down into the building (in effect, the building surrounds the tower). The rest of the towers were built atop fairly large-scale reinforced or steel-framed reinforced concrete buildings. The cylindrical steel plate towers employed support members around the bottom section for additional reinforcement: W was reinforced with a pipe truss structure surrounding the tower, while O and P were both supported by 4 steel pipe members.

test procedures

Three basic testing procedures were conducted: (A) microtremor measurement, (B) free vibration testing, and (C) forced vibration testing. The free vibrations were produced either by manually swaying the towers(6-1) or with a vibration generator(6-2). Forced vibrations were produced by mounting a vibration generator (maximum vibrational force 13.8 kg) on the uppermost antenna deck, then using it to apply sine wave excitation.
analytical results and discussion

Record of free vibration test for tower F is shown in Figure 11(1). Records of free vibration test for tower P are shown in Figure 12(1). An example microtremor Fourier spectrum at the topmost antenna deck of tower J is shown in Figure 13(1). A typical resonance curve derived from the forced-vibration tests for tower J is shown in Figure 13(2). Natural periods, damping ratios, and ratios of primary natural period to tower height are summarized in Table 3. The relation between primary natural period and tower height for the various towers is shown in Figure 14.

The natural periods and mode shapes are computed from the lumped mass model. The computed natural periods and mode shapes of tower F and P are shown in Figure 11(2) and 12(2).

The following conclusions can be drawn from the data obtained.

(a) Concerning the relation between primary natural period and tower height, the results for the 8 truss towers agree very closely with previous data for angle steel truss towers[1] while the results for the 4 rigid framed and 4 cylinder towers diverged from the available data somewhat.

(b) Damping ratio for the types of 2 to 4 used steel pipe with welded joints for main members were quite low, ranging from 0.2 to 0.9%, though it might be possible to evaluate those larger than the test values considering larger amplitude during a big earthquake. This contrasted with damping ratios obtained for the angle steel truss towers with bolted joints, which yielded comparatively high values ranging from 1 to 3%. It was found that the diagonal and horizontal members with bolted joints had little impact on the damping even for the steel pipe truss towers(F,G and H).

(c) The natural periods and mode shapes computed from the lumped mass model are reasonably good estimates of these measured for the actual structure, in the first two translational modes.

5. General remarks

NTT has developed its own structural design method in order to ensure the safety of microwave antenna towers, and constructed more than 2,500 towers throughout the Japanese islands during the past thirty years. In order to achieve this enormous construction work, NTT has established its design standards of steel towers, while large scale towers have been individually designed so as to match the respective local conditions. NTT has carried out many experiments and analyses in order to make clear the structural performance of the steel towers.

Features of the towers, the structural design method and the dynamic characteristics of the towers subjected to vibration tests were introduced in this paper.

6. Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_w )</td>
<td>Wind pressure ((\text{kg}))</td>
</tr>
<tr>
<td>( C_w )</td>
<td>Wind force coefficient</td>
</tr>
<tr>
<td>( q )</td>
<td>Velocity pressure ((\text{kg/m}^2))</td>
</tr>
<tr>
<td>( h )</td>
<td>Tower height above ground level ((\text{m}))</td>
</tr>
<tr>
<td>( A )</td>
<td>Projected net area ((\text{m}^2))</td>
</tr>
<tr>
<td>( i\sigma_0 )</td>
<td>Level 1 design shear force</td>
</tr>
<tr>
<td>( \zeta_0 )</td>
<td>Seismic coefficient</td>
</tr>
<tr>
<td>( z )</td>
<td>Seismisity reduction factor for the district</td>
</tr>
<tr>
<td>( t\alpha_i )</td>
<td>Level i shear coefficient ratio (See Figure 6).</td>
</tr>
<tr>
<td>( \sum_{j=1}^{n} W_j )</td>
<td>Sum of weights from tower top to level i</td>
</tr>
</tbody>
</table>
Design shear force for floor $r$

Seismic force in degree $j$ for level $i$ of a steel tower

Seismic force in degree $j$ for level $i$ of a steel tower

Design acceleration value obtained from the response spectra corresponding to the natural period in degree $j$ for a steel tower

Modal participation factor to degree $j$ for floor $i$

Floor $i$ mass quantity

$(i,L)$ element of horizontal stiffness matrix for a steel tower

Floor $L$ displacement from the tower base when subjected to a seismic force of degree $J$

7. References


8. Tables and figures

Figure 1. Steel tower parts

Photograph 1. Microwave antenna towers in NTT
Photograph 1. The microwave antenna towers in NTT

Figure 2. Classification of seismic intensity

Table 1. Aseismic design requirements for steel towers

<table>
<thead>
<tr>
<th>Earthquake Intensity Scale</th>
<th>Human Lives</th>
<th>Damage to Tower</th>
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</thead>
<tbody>
<tr>
<td>V (0.08~0.25G ground acceleration)</td>
<td>Safe</td>
<td>Structural member: No damage (Elastic)</td>
</tr>
<tr>
<td>VI (0.25~0.40G ground acceleration)</td>
<td>Safe</td>
<td>Structural member: No damage (Elastic)</td>
</tr>
<tr>
<td>VII (≥0.40G ground acceleration)</td>
<td>Safe</td>
<td>Structural member: Avoid collapse</td>
</tr>
</tbody>
</table>

Figure 3. Earthquake response characteristics of tower atop building

Figure 6. Shear coefficient ratio distribution (t/α)
Figure 4. Seismic coefficient for a tower on the ground

\[ t_{Co} = 0.6 \cdot \left(1 - 0.2(0.6 - 1) \right)^2 \]

Figure 5. Seismic coefficient for a tower atop a building

\[ t_{Co} = 0.6 + 1.2 \left\{ 2 - \frac{t_{T1}}{t_{bT1}} \right\}^2 \]

Figure 7. Procedure for determining steel tower design acceleration spectra

1. Generating various simulating earthquake waves
2. Modeling the building
3. Calculating the roof motion by elasto-plastic response analysis
4. Calculating the response spectra to the roof motion
5. Establishment of steel tower design response spectra (acceleration)

Table: Kind of Ground vs. Ground Detail vs. Tc (sec)

<table>
<thead>
<tr>
<th>Kind of Ground</th>
<th>Ground Detail</th>
<th>Tc (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Ground consisting of rock, or sandy hard gravel</td>
<td>0.3</td>
</tr>
<tr>
<td>II</td>
<td>Ground consisting of sandy gravel, sandy hard clay, or loam</td>
<td>0.5</td>
</tr>
<tr>
<td>III</td>
<td>Ground except I, II, and IV</td>
<td>0.8</td>
</tr>
<tr>
<td>IV</td>
<td>Ground consisting of soft deep alluvium</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Figure 8. Maximum response velocity spectra (damping ratio = 0.05)
Figure 9. Steel towers design acceleration spectra example

Figure 10. Elevations and plans of the towers

Figure 11. Dynamic characteristics for tower F(2)
Figure 12. Dynamic characteristics for tower P(1)

- 1st: 0.92 sec
- 2nd: 0.14 sec
- 3rd: 0.61 sec

Figure 13. Dynamic characteristics for tower J(2)

- 1st: 1.2 sec
- 2nd: 4.0 sec
- 3rd: 1.6 sec

Figure 14. Relation between natural period and tower height

Table 3. Vibration characteristics and structural features of the towers

<table>
<thead>
<tr>
<th>Tower</th>
<th>Type / Category</th>
<th>Height (m)</th>
<th>Installation</th>
<th>Natural period (sec)</th>
<th>Damping ratio (%)</th>
<th>Tr/H</th>
<th>Test method</th>
</tr>
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<tbody>
<tr>
<td>A</td>
<td>①/3</td>
<td>70.0</td>
<td>on 6-story building</td>
<td>T1: 1.00</td>
<td>T2: 0.56</td>
<td>H1: 1.30</td>
<td>H2: 0.45</td>
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<tr>
<td>B</td>
<td>②/2</td>
<td>30.0</td>
<td>on 8-story building</td>
<td>T1: 0.46</td>
<td>T2: 0.20</td>
<td>H1: 1.30</td>
<td>H2: 0.45</td>
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<tr>
<td>C</td>
<td>③/2</td>
<td>30.0</td>
<td>on 2-story building</td>
<td>T1: 0.25</td>
<td>T2: 0.10</td>
<td>H1: 1.30</td>
<td>H2: 0.45</td>
</tr>
<tr>
<td>D</td>
<td>④/2</td>
<td>20.0</td>
<td>on 2-story building</td>
<td>T1: 0.12</td>
<td>T2: 0.05</td>
<td>H1: 2.00</td>
<td>H2: 0.60</td>
</tr>
<tr>
<td>E</td>
<td>⑤/2</td>
<td>18.0</td>
<td>on 2-story building</td>
<td>T1: 0.12</td>
<td>T2: 0.05</td>
<td>H1: 2.00</td>
<td>H2: 0.60</td>
</tr>
<tr>
<td>F</td>
<td>⑥/2</td>
<td>46.5</td>
<td>on 6-story building</td>
<td>T1: 0.70</td>
<td>T2: 0.40</td>
<td>H1: 1.40</td>
<td>H2: 0.60</td>
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<tr>
<td>G</td>
<td>⑦/2</td>
<td>30.5</td>
<td>on 5-story building</td>
<td>T1: 0.41</td>
<td>T2: 0.25</td>
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<td>H</td>
<td>⑧/2</td>
<td>70.0</td>
<td>on 6-story building</td>
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<td>T2: 0.50</td>
<td>H1: 1.40</td>
<td>H2: 0.60</td>
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<tr>
<td>I</td>
<td>⑨/3</td>
<td>65.0</td>
<td>on 3-story building</td>
<td>T1: 1.25</td>
<td>T2: 0.50</td>
<td>H1: 2.00</td>
<td>H2: 0.60</td>
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<tr>
<td>J</td>
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<td>92.0</td>
<td>on 7-story building</td>
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<td>H1: 2.00</td>
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<tr>
<td>K</td>
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<td>80.0</td>
<td>on the ground</td>
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<tr>
<td>L</td>
<td>⑫/3</td>
<td>40.0</td>
<td>on 8-story building</td>
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<td>M</td>
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<td>on 9-story building</td>
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<td>H1: 2.00</td>
<td>H2: 0.60</td>
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<tr>
<td>N</td>
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<td>H2: 0.60</td>
</tr>
<tr>
<td>O</td>
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<td>55.0</td>
<td>on 2-story building</td>
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<td>H2: 0.60</td>
</tr>
<tr>
<td>P</td>
<td>⑯/3</td>
<td>50.0</td>
<td>on 6-story building</td>
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<td>T2: 0.50</td>
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DESIGN OF FOUNDATION SYSTEM FOR THE JOLLIET TLWP IN 1760 FEET WATER, GULF OF MEXICO

W. J. Wang, A. F. Hunter, J. L. Mueller, Conoco Inc.
J. A. Denis, C. J. Adams, Earl & Wright Consulting Engineers

Summary

Conoco Inc. has recently completed the design and installation of the first Tension Leg Well Platform (TLWP) for the Jolliet Field in the Gulf of Mexico in 1760 feet of water. The foundation system of the TLWP consists of a drilling template and a foundation template, both of which are tubular structures. These two templates were fixed to sea bed using steel tubular tension piles. This paper describes the salient features and governing criteria of this unique foundation system for the world's tallest structure and deepest production platform. Also addressed are some of the challenging design problems and associated innovative solutions.

1. Introduction

The Jolliet Field Tension Leg Well Platform (TLWP) was installed in the Gulf of Mexico at a site near the southeast corner of Green Canyon Block 184 in the fourth quarter of 1989. The site is about 170 miles southwest of New Orleans, Louisiana, U.S.A. in 1760 feet of water. The TLWP consists of four major structural components: the drilling template, the foundation template, the tendons (tension legs), and the TLWP (hull/deck), as shown in Figure 1.

The Jolliet TLWP was installed by Conoco Inc. as the operator for a group of three producing companies. Conoco's partners are OXY USA, Inc., a subsidiary of Occidental Petroleum Corporation, and Four Star Oil and Gas Company, a subsidiary of Texaco Inc.

A set of technical papers (References [1] through [5]) pertaining to the Jolliet Project was published in Offshore Technology Conference (OTC) in 1990. This paper describes governing criteria and salient features on the design of the foundation system comprising of the drilling template and the foundation template.

2. Governing Criteria and Development Plan

Governing Criteria

The following governing criteria were considered in the development of the foundation system for the TLWP:
- Development plan
- Site conditions
- Functional requirements
- Fabrication method
- Procurement schedule
- Installation equipment/procedures
- Maintenance/inspection requirements

The design of the foundation system was controlled by both the pre-service design conditions and the in-service functional design conditions. Table 1 summarizes typical loading conditions considered in the design.
Site Conditions

To assure the safety of installation and production, an intensive and integrated site investigation was conducted to collect and scrutinize vast amount of geophysical, geological, and geotechnical data. For the foundation systems, drilling, sampling, and in-situ testing of four borings was carried out near each corner of the foundation template. In addition to three conventional borings carried down to 300 feet penetration, one boring was drilled to more than 600 feet to investigate the existence of gas hydrates in the vicinity. Because shallow depth shear strength controls the design of template settlement and associated mudmats and leveling system, shallow in-situ remote vane tests were also performed.

The seabed at the TLWP site is fairly smooth and has a slope of 3.66 degrees (6.4 percent). The soils at the site are normal to slightly overconsolidated clays. The clays range from very soft at the seabed to very stiff below 190 feet.

Development Plan

In order to optimize the overall project schedule, the two-piece template system was selected for the project development. The installation of the drilling template in 1987 enabled the drilling activities to commence using a semisubmersible derrick vessel (SSDV) while design, fabrication and installation of other components of the TLWP could proceeded in stages without jeopardizing the targeted first oil date of 1989. Using a combined drilling/foundation template, on the other hand, would delay the production by at least one year.

The foundation template was first designed as a buoyant and launchable template (2400 ton) to facilitate installation by a number of available smaller conventional derrick barges. Investigation of the market situation and available installation vessels led Conoco to decide, at the design stage, to proceed with a parallel design of a non-buoyant light weight template (1400 ton), which could be installed by only a few heavy lift semisubmersible crane vessels (SSCV). Through competitive bidding, the light weight template option was adopted for the project, resulting in a saving of several million dollars as compared with the original template concept.

Cost and schedule, as demonstrated by the above two examples, are the two major factors affecting the development plan and consequently the design of the final foundation system.

3. Design of the Drilling Template

Configuration

The drilling template (Figure 2) is a tubular steel structure consisting of the following major parts:

- A 350-ton supporting frame which has an overall plan dimension of 53 ft by 78 ft and vertical dimensions ranging from 13 ft to 23 ft to fit the seabed slope. The functions of the supporting frame are to support and locate all the well pods and provide temporary supports for the docking pile guide frames. The supporting frame is supported by four 30 inch casing drilled pin piles.

- Six well pods each of which weighs 35 tons and contains 4 wellhead receptacles to accommodate drilling and production risers.
Two 48 inch docking piles which were used to guide the installation of the foundation template. They were installed using a retrievable guideframe indexed to the drilling template.

Pre-Service Conditions

The drilling template was designed for installation by SSDV and was analyzed to ensure that the structure was adequate to withstand all the pre-service design conditions. These design conditions resulted from activities involved in the marine operations, which included all operations necessary for the transportation and installation of the template from the time the template left the fabrication yard until it was fixed in place at the Jolliet TLPW site.

Pre-service conditions, which included launch, lowering, pin pile installation, leveling, and well conductor installation, in conjunction with hydrostatic pressure controlled the design of the supporting frame. Since the installation SSDV was not yet selected at design stage, the frame was designed for flexibility of installation by various types of SSDV's. The launching operation of the template required that the template float and thereby required additional buoyancy. The frame required three auxiliary buoyancy tanks to maintain stability during launching. Also, the soft bottom conditions at site demanded that the template's on-bottom weight be minimized. The on-bottom weight was important for the initial settlement of the frame prior to the installation of the pin piles. The required surface buoyancy was balanced against the desired on-bottom weight considering the loads of the 1762 ft hydrostatic head plus the installation loads. The use of ring stiffened tubulars proved to be the most economical design from a weight saving point of view.

In-service Conditions

After the template was fully flooded and all well pods were installed, the conductor installation began. The maximum loading for the template and individual well pods occurred during the installation of the first conductor. The other significant load applied to the conductor during this phase was an upward force to test that the conductor was properly latched to the well receptacle. These pre-drilling loads governed the design of some of the members.

The detailed design of the well pods was controlled primarily by the drilling and production riser loads. The drilling loads included the drilling riser loads combined with BOP stack weight, guideline tensions and well kick applied non-concurrently to various well locations. The production loads included the production riser loads under extreme environmental conditions applied to all 20 well locations concurrently.

The fatigue strength of the template was assessed using riser response amplitude operators (RAO's) in conjunction with directional wave height exceedance. The minimum fatigue life for the critical joints investigated was 125 years as compared to the service life of the template of 20 years.

Foundation Design

The template's initial on-bottom buoyant weight plus the impact loads from the pin pile installation controlled the mudmat design. Using the ultimate soil bearing pressure at different penetrations together with appropriate factors of safety, the mudmat was designed to minimize initial settlement.
The pre-service loads for the drilling template controlled the design of the 30-inch pin pile design while the docking loads of the foundation template dictated the design of the 48-inch docking piles. Both were casing drilled to design penetration by the installation SSDV. The 48-inch pile required special handling due to its size and was a record setting installation by the casing drill technique.

**Mechanical Systems**

The mechanical systems for the drilling template included a hydraulic sling release system, leveling systems for the supporting frame and the well pods, and flooding systems for ballasting operations and pressure equalization. The template was protected against corrosion by a combination of coating and sacrificial anodes.

**Installation Procedures**

After completion of fabrication, the supporting frame was lifted onto a launch barge at the fabrication site. Once the template was properly seafastened to the barge, it was towed to the Jolliet Site and launched. Upon being ballasted and located under the moonpool of the SSDV, the frame was lowered to seafloor by marine riser using a bullnose/sling arrangement. The position and orientation of the frame was set by manipulating either the mooring system or by rotating the riser. The exact location of the frame was monitored by acoustic positioning system.

Upon landing on the seafloor within positional and orientational tolerances, the slings were released hydraulically by a remotely operated vehicle (ROV). Next, four 30 in pin piles were installed along the perimeter of the frame and secured to the pile sleeves. Since the frame was installed within the specified tolerance, further leveling was unnecessary.

After the frame was level and supported by the pin piles, two docking piles were installed using a retrievable guideframe.

The six well pods were installed next. The well pods were designed to be lowered on the drill string using a J-tool. Screw jacks were provided on the well pods to allow a final adjustment to be made within 0.5 degrees prior to installing the well conductors. Upon completion of the post-installation checks, drilling operations commenced.

**4. Design of the Foundation Template**

**Configuration**

The 1400 ton foundation template (Figure 3) serves to anchor the 12 TLWP tendons at their lower end and to transmit the tendon loads to 16 tension piles, four at each corner of the template. These piles are grouted to pile sleeves which are integral with the template.

The foundation template is a three-dimensional structure consisting primarily of tubular steel members. Overall plan dimensions of the template are approximately 200 feet by 200 feet, and the depth varies from 17 feet to 22 feet to suit the existing on-bottom slope. The framework consists of eight intersecting vertical trusses, with horizontal bracing in both the upper and lower plans. Each corner of the template has three tendon receptacles which are vertical tubular members containing load rings to support the tendon loads. The pile sleeves are located in each corner of the template such
that the centroid of the three tendons and their receptacles coincide with the centroid of the four piles and their sleeves.

**Pre-Service Conditions**

The various pre-service conditions which were analyzed include loadout, transportation, lifting, lowering, docking, leveling and foundation pile installation. Appropriate design load cases in the form of applied forces were developed for each of these conditions. These load cases included the effects of the template's weight, buoyancy, and inertia, as applicable, as well as hydrostatic collapse pressure where appropriate. Results of analyses indicated that, except for the corner framing areas, the design of the template tubular framing was generally governed by the pre-service conditions.

**In-Service Conditions**

The foundation template was analyzed for a number of global in-place design load cases investigated in the overall global response analysis of the TLP.

The foundation template design load cases were selected on the basis of those global cases which produced maximum tendon forces on the template. The maximum tendon forces involved cases with all tendons intact as well as one case with one tendon missing due to damage. Also, the design load cases included a case in which none of the 20 risers had been installed yet.

The structural response of the foundation template to the design loads was determined by performing a non-linear space frame analysis incorporating the soil-pile-structure interaction. For critical connections and tendon receptacle areas, finite element analyses were also performed to verify structural integrity. In addition, the fatigue capability of various template nodes were investigated and found to be adequate.

Installed in a water depth of 1760 feet, the foundation template can only be inspected and repaired by ROV's. Design for redundancy and reserve strength to minimize inspection requirements constitutes one of the most important design criteria. For the foundation template, these desirable designed features were fortunately achieved by meeting various pre-service and in-service conditions.

**Components Design**

The design of the foundation template's local details included the tendon receptacles, pile sleeves and the plate girders connecting them. It also included the lifting/lowering system design and the highly loaded leveling sleeve connection to the structure as well as the mudmats.

The leveling system for the template is a set of gravity slips acting against the leveling piles, thereby allowing the template to be raised and leveled by the derrick vessel.

The template grouting system consists of an ROV assisted grout distribution system with secondary and tertiary backups.

The template was flooded at the sea bottom to equalize the pressure of 780 psi. This was done first at the corner upper framing to preclude collapse in the event of an accidental pile impact and, then later for the entire template.
Foundation Design

The leveling piles are 60 in. diameter by 250 ft. long. The pile diameter was chosen to utilize the same hammer as the foundation piles while their penetration was selected based upon the pre-service design loads for the template. Their design penetration was 230 ft.

The foundation piles are 60 inches in diameter to allow installation using the underwater hammers most widely available. A driveability study performed indicates that they can easily be driven to the design penetration of 250 ft. The design pile penetration was determined based on API RP 2T [6] recommended factor of safety for tension piles. Results of a comprehensive research project on tension piles sponsored by Conoco and other participants were utilized to optimize the pile design.

Installation Procedures

The foundation template was loaded out, transported and lifted in a manner fairly conventional for offshore jackets. The template was lowered to 1760 ft below sea level and docked over the drilling template. The leveling piles were installed and the template were found to be within the specified tolerances. The foundation piles were driven to their final penetration with an underwater hammer after which they were grouted to the template.

Due to the nature of installing the foundation template in 1760 ft water depth, extensive underwater intervention studies were performed. During most of the installation phases, ROV's were utilized to assist the derrick vessel in installing the foundation template and piles. They provided visual contact with the template during the template and piling installation as well as manipulating the sling release system and the various grouting and flooding valves. The final grout density was also monitored at the seafloor by ROV using a manipulator held densitometer.

More details of the actual installation procedures are discussed by Wybro et al [4].

5. Concluding Remarks

Design of the foundation system for a world record deepwater tension leg well platform by the effective utilization of tubular structures was demonstrated in this paper.

Acknowledgements

The authors would like to acknowledge Conoco Inc. and their joint interest owners in the Jolliet Project, Four Star Oil and Gas Company, a subsidiary of Texaco Inc., and OXY USA, Inc., a subsidiary of Occidental Petroleum Corporation, for their permission to publish this paper.

References


### Table 1. Design Conditions

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<th>Drilling Template</th>
<th>Foundation Template</th>
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<td>Production Risers</td>
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**Figure 1. Jolliet TLWP**
Figure 2. Drilling Template - Jolliet TLWP

Figure 3. Foundation Template - Jolliet TLWP
TUBULAR AND GLASS STRUCTURES

Mick Eekhout

ABSTRACT

This contribution describes the development process from tubular structures covered with glass to real structural glass as performed at Octatube of Delft NL. The described process includes several methods of structural use of glass panels in spatial structures, ranging from non-loadbearing glass panels in conventional skylights around space frames, via structurally sealed glass panels with an additional stabilising function to structurally load-bearing glass panels, where metal elements are only used as connection elements and additional stability. The point of view of the author is that of a product-architect, who combines the profession of an architect, a structural engineer, an industrial designer on the one hand, and a specialist-producer on the other hand.

1 WHAT IS STRUCTURAL GLAZING?

The description 'Structural Glazing' has a number of different interpretations in the architectural profession, departing from the architectural or the structural point of view.

1.1 Architectural meaning

In the first place 'structural' is used as the abbreviation of the building technical word 'structurally sealed' meaning a method of attachment of glass panels to the sub-structure in curtain walling. In the early days of curtain walls it indicated anything else than fixing by screw strips. The growing tendency amongst architects to design in an abstract and non-material-bound way has strongly stimulated the use of glass as a cladding material for exterior walls and roofs. This tendency was reinforced shortly after the first oil crisis in 1973 when in European glass industries a search began for glass panels with grossly improved building physical quality in order to beat the enlightened energy costs of glass-clad buildings. The demand for slick building surfaces and low maintenance costs led to the application of glass panels that were structurally sealed to the aluminium sub-structure with silicone and were provided with a silicone watertight seal between the glass panels. These techniques were exploited on a large scale in the USA even before 1973, forced by the implications of high-rise building technology and imported into Europe only later. In fact, even after 20 years of experience in
the USA, structurally sealed glass is still not permitted in some countries like West-Germany. In those cases the silicone sealing and glueing techniques still have to be combined with mechanical screwing techniques that have a conventional safety. These descriptions all regard curtain walling and skylights. Loadings are only external loadings acting on one glass panel. No further structural loadings are taken. 'Structural' refers only to the mode of connection to the subframe: with sealant. The confusion grows when people commonly refer to any type of cladding in which sealant is used for watertightness only as 'structural'.

1.2 Structural meaning

Quite different and scientifically more interesting, is the line of thought following the structural engineering meaning of 'structural glazing': glass structures that bear external loadings over more than one glass panel, and contain bending and normal stresses. The strength properties of heat-strengthened or tempered glass challenge some designers to see where the limits of suitability are for glass panels to be used as load bearing structural elements in structures that only contain an absolute minimum of metal components. This article explains the development process of glass panels from the starting point of a non-structural via a half-structural to a current state-of-the-art of completely structural use in the structural engineering sense. In particular those cases of an increasing degree of experimentation and difficulty are revealed as they were processed in the Company of Octatube Space Structures bv in Delft NL during the last few years. The state-of-the-art is March 1990. This process of Design + Development + Application is described from a broader point of view of the author being a product-architect combining the skills of an architect, a structural designer and an industrial designer, combined with the possibilities and responsibilities of the specialist-producer, rather than that of only a broad vision of an architect, a more deep and narrow interest of the structural researcher or the economical interest of the producer.

2 THE PRODUCT-ARCHITECT

In the Netherlands the term 'product-architect' has been known ever since November 18, 1988 when the author proposed it in the first Booosting congres in Rotterdam [ref 1]. It was there described that the function of the product-architect is to design, research and develop components of buildings, independant from the actual design of buildings by project-architects, and to apply these building products in the overall-design of buildings that are normally designed by project-architects.

The product-architect tries to complete the potentialities of new materials (Material Science), production techniques (Material Processing) and of application systems (Structural Engineering and Architecture) with the analytical approach of the industrial designer, and the know-how on the architectural building site of the architect, and the creativity of both. So the field of action and also the
Organigram of a Design and Development Process for new products in the building industry.

Phase 1: 'Orientation and Product Concept' for a normal design + development process for new products in the building industry as a preliminary design phase with provisional market evaluation.

Phase 2: 'Testing Market on Product Concept' showing the market research on the first product concepts.

Phase 3: 'Techniques and Costs of Prototypes' showing the necessary in-house mainly technical developments to complete the prototype.

Phase 4: 'Prototype and Market' showing the confrontation in the market with the developed prototype and its evaluation.

Phase 5: 'Launching of Product' showing the process of production of the first application, with evaluation for duplication and further standard production.
abilities of the product-architect combine that of the architect, the industrial designer and the structural engineer.

3 DESIGNING + EXPERIMENTING + BUILDING

The position of the product-architect is producer-bound rather than consumer-bound. A large range of subsequent applications every specialist-producer normally is working on, enables him to design and develop a building component product into maturity within a reasonable short time without the danger that developments stop after the completion of only one prototype building, because one client stops a project or because a project has been completed.

The described process is a result of a slow step-by-step method. In detail and more generalised such a 'Design + Research + Development' process of building products has been worked out in fig 1. The Organogram in fig 1 gives a typical design and development process for building products and components (from 'Architecture in Space Structures' [ref 1]). This scheme is the result of analyses by the author of several product development processes. It has the advantage of visual communication for designers, needing only a brief comment:

- Phase 1: Orientation and product Concept showing a preliminary design phase with provisional market evaluation.
- Phase 2: Testing Market on Design Concept showing the market research on the first product concepts.
- Phase 3: Techniques and costs of prototypes showing the necessary mainly technical in-house developments to complete the prototype.
- Phase 4: Prototype and Market showing the confrontation in the market with the developed prototype and its development.
- Phase 5: Launching of a product showing the process of production of the first application, with evaluation for duplication and further standard production.

This process scheme can be used for a wide range of new products. The 'smell' of materials and the physical presence of it, has always been the source of know-how of any specialist, and has proven to be very inspiring for designers. New opportunities begin and end with materials and production processes. Furthermore, the real building opportunities of a number of successive building products enables continuous feedback and product improvement.

4 MATERIALS + TECHNIQUES + SYSTEMS

4.1 Material Properties

For an architecturally trained designer like the author, the numerical side of the Science of Materials is not the most compelling aspect of the profession. Yet the relative large differences in properties of the different building materials provide a reasonable indication whether combinations of these materials are desirable, possible or whether they are not. By just comparing these properties, some basic and very logical conclusions can be drawn, that usually are never done because scientists hardly ever step over the borders of their specialist territory.
2. Strain-stress diagram of Glass, compared with Aluminium and Steel

4.2 Young's modulus of elasticity

Approximate E-values in N/mm² are (see fig 2):
- Steel 210,000
- Aluminium 67,000 to 73,000
- Ordinary Glass 73,000 to 75,000
- Heat-strengthened glass 73,000 to 75,000
- PMMA 3,200
- PC 2,300
- Wood 14,000

Out of all transparent building materials glass is the best suited for structural purposes. Both polycarbonate (PC) and acrylate (PMMA) have a far less favourable modulus of elasticity (2,300 / 3,200 N/mm² compared with 75,000 N/mm² for normal / toughened glass), meaning that the stiffness of these materials is far less; so deflections under external loadings applied on PMMA and PC are much larger.

4.3 Tensile strength

The maximal tensile strengths in N/mm² of the different building materials are:
- Mild Steel 360
- Alu alloy AlMgSi 0.5 215
- Ordinary glass 40
- Heat-strengthened Glass 200
- PMMA 70 to 110
- PC 60 to 100
- Wood 100

The maximal tensile strength of glass is 40 N/mm², but that of heat-strengthened glass goes up to even 200 N/mm², while that of PC is 60 to 100 N/mm² and PMMA 70 to 110 N/mm². More noticeable: the maximal compressive strength of glass is in both cases 800 N/mm² (without the influences of buckling).
4.4 Brittleness

The most deviant property of glass applied as an element of a primary structure, compared with other structural materials, is its brittleness, which turns out to be most decisive for the use of glass as a structural material. The achilles heel, the weakness of glass (and to a far lesser extend also cast metals like some aluminium alloys), is caused by this brittleness. Very generally put, there are two fracture mechanisms competing to break a material:
- Plastic Flow
- Brittle Cracking.

The material will succumb to whatever mechanism is weaker. If it yields before it cracks, it is ductile. If it cracks before it yields, it is brittle. The potentiality of both forms of failure is always present in most materials. Yielding is a safe and much desired property, spontaneous cracking is an undesirable property for a structural material.

4.5 Time and creep

Glass is a solidified liquid and not a crystallised solid. However, the tendency towards crystallisation is present, and given time, glass will crystallise. This is known as divitrification. It involves shrinkages, the glass is often weakened and sometimes falls into pieces during the process. It will always fracture in the same brittle way. Creep in glass and heat-strengthened glass is practically nil, but for PC and PMMA is fairly high (although no exact figures are known). And from here on only hearsay can be noted. Some glass industries tell us that glass in time will deform (liquid will flow): hence many old mirrors have become bobbly by now and old window panes are never flat. Only data about the time scale in which this happens are completely unknown. Some glazers state that old large and thick window panes always are difficult to remove because the lower end appears to be thicker than the upper end that sticks in the glass groove.

Wood creeps, as we all know from heavily loaded tiled roofs built from timber beams.

4.6 Thermal conductivity

The respective values for the respective materials in W/mK:
- Steel 40-50
- Aluminium 200
- Glass 0.80
- PMMA 0.07 to 0.21
- PC 0.12 to 0.19
- Timber 0.12 to 0.16

4.7 Thermal expansion

This thermal expansion is important for the mutual differences between structural and cladding materials and between co-operating
materials in structure or cladding. Values are given in 10^-6 m/mK:

- Steel 12
- Aluminium 24
- Glass (all types) 9
- PMMA 70
- PC 65
- Timber 3 to 5

There will be trouble when values of co-operating materials differ too much. PMMA and PC expand 7 times as much as glass. The combination of steel and glass causes less problems than aluminium and glass.

4.8 Specific gravity

Values in kN/m3:

- Steel 78
- Aluminium 27
- Glass 25
- PMMA 12
- PC 12
- Timber 7

4.9 Behaviour at high temperatures

The maximal workable temperature in degrees Celsius:

- Steel 550
- Aluminium 250
- Ordinary Glass 60-110
- Heat-strengthened Glass 270
- PMMA 80
- PC 115

These figures show why fire-resistant panels have to be made of steel and steel elements (wire mesh in glass). No ordinary transparent material will have a good fire rating. Only borosilicate glass has a better fire rating, but is much more expensive.

4.10 Production techniques

Since we are interested to see in how far glass could be used as a real structural material, the failure mechanism for glass can be seen as follows. Glass is cooled during fabrication so fast that the molecules do not have time to sort themselves out into crystals. So cooled glass is a solidified liquid, not a crystalised solid. However, the tendency into crystallisation is present, and given time, glass will crystallize. This is known as devitrification. It involves shrinkages, the glass is often weakened, and sometimes even falls into pieces during the process. It will always fracture on the same brittle way. In fact, if we want to prevent the glass from cracking, we have to put it under compression. This can be done by heating the glass panels again, and chilling the two outsides of the hot glass panels during fabrication very fastly, so that the two outsides form with the core a compression + tension mechanism. When the outer surfaces are cooled, they solidise, while the core is still hot.
(700°C). The shrinking during cooling of the core causes the outside skins to be compressed, while the central core will remain under tension. So the outside skin of heat-strengthened glass is under compression, the core is under tension. This is a mechanism to avoid surface cracks, but also hides internal cracks. The mechanism is the same for nodular iron and cast aluminium: the outside surface can be very smooth, while irregularities can be hidden inside the material.

After strengthening the outcome is a glass panel with higher tensile strengths, also a higher impact strength. Great care has to be taken that the glass surface is not scratched by a sharp tool, because then it will crack into thousands of small bits. Try to bring glass panels out the reach of vandals. Or reversedly put, sharp glass hammers are a safety tool to break out of an all glass cage. (For use in structural glass applications like the Glass Music Hall in Amsterdam, see par 12).

In heat-strengthened glass panels possible cracks are avoided by the compression mechanism. Using connections of the bolt-and-hole type in the glass can, consequently, be done best by a pretensioning type of bolt connection: in that case not the hole edges are loaded by the bolt on flush, (with the inherent danger of enlarging the micro cracks around the bolt hole by drilling), but the pretensioned bolt will compress the two outside metal rings or components on the glass where the mutual friction will bring over the connection force. The friction force can be enlightened by grinding or blasting the surface around the bolt hole. Alternatively, A flush-type connection will have to contain an intermediate material between bolt and glass hole to avoid local toptensions in the hole, from which micro cracks can lead to serious cracks. But another idea might be to fill in the irregular left-over spaces in the holes between the outsides of the holes and the bolts by liquid epoxy, in order to get a firm abrasion connection. This is a technique developed to renovate old nailed railway bridges with slotted bolt holes. In this case the method is a means to isolate metal from glass and to adjust glass panels exactly to the required sizes. The type of connection will decide on the vulnerability of the irregularity of the bolt shaft and the bolt hole.

On scale of the glass panels, a possible compression (normal) force introduces the danger of buckling the panel so that the largest commercially available panel thicknesses (12, 15 or 19 mm) will have to be used that are quite expensive per volume. It would be better then - if at all possible - to load the glass panels under tensile (normal) force in stead of compression: that is to suspend rather that to stack them.

The thermal-hardening process is one of the interesting production techniques of structural glass. It was invented decades ago, but is still the basis of the current structural glass types. Another remark has to be made on installation of glass panels: although glass panels are very strong they are likely to splinter up into thousands of small pieces for example when not full attention is paid during hoisting and installation.
4.11 Statical systems

The statical systems applicable for glass structures all will depart from glass panels. Glass bars do exist but the nature of these glassfiber bars filled with epoxy is not translucent. Moreover the connections are still quite laboursome, so in this article glass is only regarded in panelform, and not in glass bars. The most simple solution is the classical vertical glass panel, able to make a vertical span with or without metal or glass ribs. One of the first architectural design priorities set by the author, and also by a great number of his colleague-architects, is to develop structures with minimal visual disturbances. As a consequence thereof the visual minimal 2-dimensional structures (because of the flat plates) like guyed structures are the most logical structures. Figure 3 with the derivation of the guyed structure principle is self-explanatory. The principle forming the base of guyed glass structures is that short and slender metal compression bars are used, with long thin metal tension bars and glass plates where invisible normal stresses in the form of tension- or compression stresses are included. Fig 3 also indicates the difference between Open and closed structural schemes, that is important in regards to the connections with the substructure. Also an indication is given of the single-sided and the double-sided schemes arising out of architectural reasons.

3. Derivation of guyed structures in cross section applicable to space structures with glass panels as the main structural elements and cross bars plus tensile rods as auxiliaties

Although the structural principle is fixed now, it seems advisable to develop new aspects and difficulties only at a modest speed: step-by-step, where every step means only one or two new aspects compared with the in-house technical state of the art. Thinking and developing in this way has led to the rest of the description of this article. The step-by-step-method is the only logical way of developing a new technology above the level of the state-of-the-art. Only after establishment of this technology the respective standards will usually be developed. The involvement in an earlier phase of positive product-research by the official Building research Institutes would be most welcome.
5. SPACE FRAMES WITH SEPARATE GLAZING SYSTEMS

Fitting in the whole constellation of the building industry, the decision for a separate glazing system around or on a space frame is an accepted solution, ready for sub-contracting. Standard solutions are suited for normal applications, but often fail when geometrical complications arise. In fact very soon it appears that standard solutions and more experimental (or non-standard) solutions are a world apart and show large differences in approach. For example suspended glass surfaces, irregular geometrical surfaces and facets usually require non-standard solutions. Apart from that, the resulting optical doubling of structural bars and glazing mullions can work very confusing. Along these considerations the author developed a new complete glazing system for the Raffles City Glass Atriums in Singapore (1983).

In case of the Raffles skylight system, there was no other way than to develop a separate skylight system on a separate space frame, as this space frame was very heavily loaded (high upward and downward windforces caused by the four large towers), and permitted only to rest on 3 or 4 points, to allow the towers to move freely without the danger of crushing the space frame. Figure 4 however, shows by the graphical play of lines that the doubling of space frame lining and skylight lining works a little confusing visually.
6. SPACE FRAMES WITH INTEGRATED GLAZING

When the glazing system is not seen as an independent or semi-dependant system on the space frame, the next step toward structural glass is to design and development an integrated system in which the glazing mullions do coincide with the space frame bars. A distinction has to be made here between space frames with square or rectangular modules and space frames with trapezoidal or triangular modules. Attention has to be paid to the fact that a definitive disadvantage is that the price of (trapezoidal but even worse:) triangular glass panels is more than double the price of square or rectangular panels. But by integrating the skylight mullion and the space frame tube, the visual aspect has been improved 100%. Following this route, in the Tuball space frame system a new line of profiles has been developed called the OT-profiles in which the functions of structure and cladding are clearly readable: the circular section carries normal forces, the T-flange on top carries the glass (or cladding) panel. The total system is called the Tuball-Plus system.

5 and 6.
Overall and inside view of the music pavillion geodesic dome in Haarlem, 9 m diameter, 7.5 m height. A 3-frequency icosahedron. Rib length 1.7 m, covered with laminated clear glass in the Tuball-Puls system.
7. Detail of the Tuball-Plus system.
8 and 9. Arcade in Tuball-Plus for the shopping centre 'de Amsterdamse Poort' in Amsterdam Zuidoost.

The music dome in Haarlem NL, designed by town-architect Prof Wiek Röling in co-operation with the author as a product-designer has been built in 1984 as the first application of a Tuball-Plus system of integrated structure and cladding elements. See fig 5 and 6. This dome is a geodesic dome with a 3-frequency icosahedron subdivision. All glass panels have the same triangular form as the structure. The design of the Tuball-Plus node with machined-out ends of the bars started as a brainwave behind the drafting table, strongly influenced by the possibilities of a prototype laboratory. For all completeness one should mention two typical technical contradictions, giving a clue of the immanent battle between the designer and the structural engineer behind the system. Firstly it is not a custom to introduce bending stresses in the space frame bars apart from the normal stresses. Secondly on the place where the largest shear forces are acting because of the bending moments, the most material at the end of the OT-bars has been machined away (see fig 7). These interventions are calculated out in the stress analysis of the different elements, but visually for the structural engineer they seem illogical. It is the product-architect/industrial designer who decided here.

The arcade of the Amsterdam shopping centre 'De Amsterdamse Poort' has been designed and built in the same Tuball-Plus system in square panels. See fig 8 and 9. The arcade is composed of delta trusses supported on each lower node, covered with laminated glass. Module of space frame abnd glazing: 1.7 x 1.7 m. Span 11 m, length 34.7 m, height 5.5 m. Designed by architect Ben Loerakker and the author.
7. SPACE FRAMES WITH STRUCTURALLY SEALED GLASS

A next step forward is to change the mechanical screw connection between glass and structure into structural sealant with silicone. The fixation is by glueing instead of screwing, while the watertightness seam is again a separate silicone seal instead of a rubber strip. So in this case there are two types of sealant: structural adhesive and weather seal, separated by a foamband so that structural movements in two directions are independent. The result is a flush outside surface without any screw strips, giving a dome even more a crystalline character. Irregularities that cause filthiness on the surface like screwstrips have been removed.

Example of this is the canopy of the Raffles City Hotel Complex in Singapore, where flat laminated clear glass panels were sealed directly onto the aluminium Tuball-Plus profiles. See fig 10 and 11. The flat roof plane has only a camber of 1%. On top of the space frame square panels were used; on the sides triangular panels. The outside of the triangular glass panels form one flush glass surface 1.5 x 42 m long. Module length 1.9 m.

10 and 11.
Pictures of the Raffles City entrance canopy in Tuball-Plus with laminated glass sealed with silicone sealant on top of the OT-profiles
Glueing glass panels with structural sealant on the building site is not a real gain in assembly technology as it is very sensible for low temperature and humidity / wetness. Therefore, further development led toward glass elements provided with glued-on aluminium profiles in the factory (under ideal climatic conditions) with a structural sealant, that can be screwed with mechanical means on the structure on the building site, and subsequently weathersealed with silicone when the humidity allows it. Alas in this case the visual profile thickness is larger, and the detailing has not a minimal slender-ness any more.

8. SPACE FRAMES AND STABILISING SEALED GLASS

The next step after the structural sealing of glass panels seems logical now. The idea is to give the glass panels also a stabilising function in the form of addition shear strength. At this moment is should be possible to design a rectangulated hinged space frame (actually a space "truss"), where the horizontal stability can be taken by the glass panels sealed with structural sealant on the metal frame, preventing horizontal deformations. The consequence of this is that not only triangulated dome and saddle-shaped structures can be built, but also rectangulated or trapezoidal subdivisions, that are cheaper than the triangulated structures by the cladding. (Triangulated glass panels are 2.6 times as expensive as rectangular or square panels, and so are more decisive in price that the structure itself. Also in general one could state that the covering skylights are more expensive that the space frame underneath). Using the shear strength of glass panels is an idea that, unconsciously, was used in the last century in glass houses. These glass panels ensured the majority of the horizontal stability because they were fixed in rigid putty. Detail study of these glass houses like the Crystal Art Palace in the Botanical Garden in Glasgow show by the curvatures in the domes with their horizontally twisted lines, that the shear resistance came from the glass panels and not from the metal structure below. See fig 12. There are no metal wind bracings in these structures.

12. Picture of the Crystal Art Palace in Glasgow, designed by Kibble in 1873, where glass panels in rigid putty were used unconsciously as an extra stabilisation for the entire structure.
One example of a stabilised spatially curved roof is given in the saddle shaped roof of the Entertainment Hall in Zandvoort designed by architect Sjoerd Soeters and the author in 1987, that with a possible infill of structurally sealed glass panels could have been stabilised in any horizontal direction. See fig 13. The actual roof was designed as a double prestressed membrane, and as a single layered space frame with plywood infill panels, but structurally sealed glass panels would have been possible, too.

13. Isometry of the anti-clastical roofs of the Entertainment Hall in Zandvoort: 15 x 45 m

Another example is a tent-like polygonal atrium skylight that is pretensioned by a central hanging mast and guying cables, as designed by the author in 1989 (fig 14).
9. SPOKED GLASS PANELS: OSAKA PAVILIONS

It is only a very simple step forward from glass panels stabilising space frames to the stabilisation of glass panels by metal components. But in doing so the importance of the two elements are interchanged: glass plates get the primary function, while metal components get the secondary function.

So this step is the most crucial one in the process described in this booklet as it turns the normal way of thinking upside down!

The most simple form is an assembly of 4 glass panels in a metal frame that are stabilised in the centre by two cross-bars, stabilised on their turn by 2 x 4 tensile bars to the 4 corners. The whole assembly resembles a square bicycle wheel with 2x4 spokes. The central nave really consists of 2 half elements compressing the glass panels in between together. The seams between two adjacent panels are formed by acrylate H-profiles that even do not give a shadow line and are almost invisible. (Fig 15). This system has been designed as the glass facade of a modular exposition pavilion for a Dutch pavilion built in Osaka in March 1989 for the EVD (a service of the Dutch ministry of Economic Affairs), by project-architect Frans Prins and the author. A similar pavilion was later built for Heineken Japan, (see fig 16) in a slightly modified form in Osaka in March 1990.

15. Exhibition mock-up of one fascia unit of the Osaka-pavilion, displaying the 2 cross bars and 8 tensile spokes.
16.
The Osaka pavilion consisting of a tunnel shaped space frame clad with prestressed membrane elements and stabilised glass facades as a tribute to traditional Japanese Architecture; the fascia elements are composed of 4 stabilised glass panels (1.2 x 1.2 m) each with 2 cross bars and 2 x 4 spokes. Designed by architect Frans Prins and the author.

The original design consists of a modular Tuball space frame in portal form 4.8 m high, 9.6 m deep and 19.2 m long. All three space frame planes are covered with white prestressed PVC/FS membrane elements, while the long fascia's are covered with the above presented stabilised glass system. When the system should be built without the surrounding metal frame, the glass panels would be compressed in their plane towards each other, with the acrylate H-profiles in between. The Osaka pavilion has been designed as an ode to traditional Japanese modular architecture, in a modern form, with modern materials and modern techniques: with a touch of western High-Tech. The requirement of quick site assembly and demountability has led to the specific choices of materials and components. The module had to be 2.4 m all over, as the Japanese are very strict in the use of regulations. As the shipping container size is outward 2.4 m but inward 2.2 m, glass panels sized 2.4 x 2.4 m could not be transported but had to be divided into 4 parts. So the size of the glass panels is 1.2 x 1.2 m, in demountable window frames of 2.4 x 2.4 m. In consequence of the design of the Tuball space frame, the nodal point of the top of the cross bars had the hollow sphere form in which the 4 spokes run and can be prestressed internally without visible nuts. This underlines the abstract character of the design.
10. PRESTRESSED GLASS PANELS

The next step in the development process is to give glass panels a real primary structural function, by letting them function to pass normal forces. For that aim a thorough theoretical study has been made in 1988 in-house by Rik Grashoff, a student of Civil Engineering at the TU Delft. The initiative was a result of writing the dissertation 'Architecture in Space Structures' by the author, started in Jan 1988, and published in May 1989. (Ref 2). The aim of this structural feasibility study was purposely kept on technical aspects, not financial or building-physical, as these aspects were supposed only to restrict and endanger a possible technical step forward. The material investigation showed indeed that glass is still the only appropriate structural transparent material for structures of the above described kind. (See par 4). For safety reasons in roofs heat-strengthened glass panels could be laminated and used as structural plates, although the lamination layer weakens the structural capacity by 30% in strength. Single heat-strengthened glass is not safe (depending on the estimation of the danger of vandalism or mechanical loadings and fall height). Duplex strengthened glass has problems with size accuracies of position of bolt holes and panel sizes. Laminated normal glass is less expensive but not as safe, but less expensive and cannot have bolt holes. The investigation has taken thick heat-strengthened glass panels as a base that are laminated with thin normal glass panels for minimal security reasons. The study resulted in a system of laminated heat-strengthened panels in square sizes from 1.2 to 2.1 m, with thicknesses of 8, 10, 12, 15 and 19 mm.

First Mock-up of the prestressed connection of 9 heat-strengthened glass panels, 10 mm thick, as a first step into the direction of load bearing glass structures.
The first prototype connection in the material-structural feasibility study was made with 4 (instead of glass) plywood panels and double-sided 4 turnbuckles, to prestress the connection so that both compression and tension forces could be transferred. In order to avoid the danger of asymmetrical prestressing (with bending moments on the glass panels), in the prototype at the end of the 3-months study equilibrium saddles were built in, in order to obtain even under strong asymmetrical pretensioning only normal forces in the glass plates. The total size of this second prototype was 1.9 x 1.9 m composed of 3 x 3 glass panels 630 x 630 mm (so in scale 1:3) to be able to transport the prototype as a whole, and to use it for exhibition purposes, while the mechanical connection nodes and auxiliaries were made on real scale 1:1. See fig 17. The whole assembly gave as a result of the mixture of scales a rather mechanical outlook, asking for further refinement in design. Another disadvantage of the 1:3 scale of the glass plated was their relative stiffness, allowing loadings that normally could not have been taken because of the high torsion rigidity of the scale model.

The internal prestress-method had the aim to overcome the cutting tolerances of the glass. In practice these glass tolerances seem quite satisfactorily now (See par 12: Glass Music BOX), so that the tolerances in the metal components are most decisive now.

11. STRUCTURAL GLASS WALL: COOL CAT, GRONINGEN NL

The first contract for a prestressed structural curtain wall was designed in 1988 by project-architect Paul Verhey for a fashion shop of the Cool Cat concern in Groningen NL. See fig 18. The design consists of 6 panels sized 2 x 2.25 m, in the total size of 4.5 m high and 6 m long, as a suspended glass curtain on the first story of the shop front, as shown on the perspective drawing nr. 18. The detail of the joint is given on adjacent photograph 19. For this first application the whole assembly has been built up in the Octatube factory. The evaluation of this mock-up in the laboratory in which both the glass panels as the joints are on real scale, proved the visual correctness of the Minimal-Material hypothesis. The six panels have been assembled into one consistent whole by a doublesided guyed bar system on 2 x 3 cross bars. The size of the short cross bars is 20 mm, the tensile guy bars 8 mm. The size of the glass panels is so large compared with the joints that these joints have been enlarged in the design phase out of visual considerations, in order to obtain a visually credible structure from 40 mm to 50 mm props. The structural assembly is a structural entity in the sense that this closed system contends the necessary tensile and compressive elements to form an independent whole. It could also work in space. However, to function the structure has been suspended from a steel portal frame. Windforces are taken to the 4 sides of the structure; on the lower side a metal bridge takes over the horizontal windforces of the both sides, but vertical forces are not taken over due to the vertical sleeve holes. In this way also the extra loads on the bridge can not cause extra vertical stresses in the curtain wall. The detailing of the nodes, the cross bars and the guy bars is
18. Perspective view of the Coll Cat shop facade in Groningen NL designed by Paul Verhey. The curtain wall of the first floor (4.5 x 6 m) is suspended from the steel portal and brings over horizontal wind forces to the 4 sides of the glass wall.

19. Detail of the Cool Cat node, designed by the author.

Very functional, but in its design also abstract. See fig 19. The glass plates are under compression by the prestress mechanism, while the wind loads result in extra compression forces in the glass plates as well as extra tension in one of the tensile elements. These are two different mechanisms acting.

The mock-up in the Octatube factory clearly showed the inspiration derived from such a structure. As to material application: "there is not one ounce too much fat". The transparency of the structure underlines one of the evaluation criteria of its product development: minimal visual disturbances. It also underlines the over-value aimed at in this process: giving structures a certain inspiring form.

Professor Ludwig van Wilder of the TU Delft Architecture Department gave another point of view when he commented on the structure as an example of "exchanging material for brains". And clearly expressed that the development of glass structures stems from an intellectual challenge. Due to retardement in the overall building scheme of the shop, the curtain wall will be installed in February 1990.
12. GLASS MUSIC HALL IN THE BERLAGE EXCHANGE, AMSTERDAM

The Cool Cat wall is the predecessor of the glass walls around the Glass Music Hall built in the former Option Exchange Hall in the famous Exchange built by Dr. Hendrik P. Berlage between 1896 and 1903, one of the first modernist buildings in the Netherlands. In this hall a smaller volume was built to function as the acoustical rehearsal room for the Dutch Philharmonical Orchestra. See fig 20. This hall has been made of glass because of the desired dominance of the interior of the old building and because the glass box acts only as an acoustical envelope, while around it other activities still can take place without mutual disturbances. The size of the Glass Music Hall is 9 m high, 9/13/10.8 m wide (belly form) and 21.6 m long (see fig 21). The 4 walls are composed of glass panels sized 1.8 x 1.8 m. The walls are suspended from a table-formed rigid double layered space frame structure supported by 6 slender steel columns.

20.
Isometric view of Berlage's exchange with the Glass Music Hall inside, as originally designed by architect Pieter Zaanen.
21.
Bird eye's view of the Glass Music Hall sized 9/13/10.8 m wide, 9 m high and 21.6 m long, all covered with glass panels 1.8 x 1.8 m, as designed by the architect and the author.
Cross section of the Glass Music Hall in Amsterdam, with space frame roof structure and suspended glass panels in both walls, stabilised by a guy truss system spanned between space frame and reinforced concrete substructure. The glass walls are suspended from the edges of the space frame, and stabilised by double (counter-)spanning guyrods on cross bars (fig 22). These stabilising systems are only present on the inside of the hall, so that the outside has a slick glass surface. The roof plane of the hall is covered with laminated glass 5.5.1, grey tinted. The 3 rectangular sides are 8 mm grey tinted heat-strenthened glass panels, while the belly-formed (out of acoustical reasons) long wall has clear glass 8 mm heat-strengthened glass panels. The original suggestion by the author was to have an all-transparant clear glass box, with only the curved wall as grey tinted, to emphasize the form-deviation. However the architect decided to use the clear/grey glass panels just reversed to enlange the surprise effect when entering the Glass Box. The grey tinted glass has a remarkable cameleon effect. Seen from the outside, with light outside the glass box is dark grey. However, when sitting inside, and when the spotlights in the larger Berlage space are lit on the walls, the glass all of a sudden seems almost clear: It leaves a very good picture of the Berlage walls. Darkening the room can hence be done by dimming the outside lights.

The structure was completed in december 1989. The design of this structure has been jointly made by project-architect Pieter Zaanen and the author as a technical designer cum producer. The form of the hall has been analysed and advised permanently during the design phase by the acoustical advisor Peutz, and proved to give the desired acousitcal values when measured after completion. Figure 23 to 26 give an overall view and some details.

Building the Glass Music Hall has taught us one very important lesson: the type of metal connection node does not permit large deviations in size. Not in the overall size of the glass panel and not in the seams in between. We have used glass panels very accurately cut in the Swiss Securit factory with a water laser jet,
23 to 26.
Different details of the Glass Music Hall
giving bolt hole accuracies of +0,00 / -0,5 mm. The panel size
tolerance was +0,00 / -1,0 mm, orthogonally and diagonally. This of
course meant, together with the rigid connectors, that the max 1,0 mm
tolerance was to be met in the bolt hole. We also found that the
accuracy of these computer-cut glass panels was so high, that in the
total assembly the size of our steel structure became the point that
required the most attention, and appeared to be the most critical.
During the installation of the 4 glass panel wall, we tested the
failure of one of the upper panels, which all carry the deadweight of
the lower panels. The structural design predicted a square chain,
action after collapse of an upper panel, so that the deadweight of
the 4 lower glass panels was carried over by panel nr. 4 to her
adjacent panels nr 4 and again up to panels nr 5. And this happened
indeed. There was no progressive collapse. But more of these
practical experiments will have to be made before these type of glass
structures will be accepted as being as safe as any other material.
It still will remain glass in all its properties, and its development
requires a lot of patience, trial and error and feeling for material
and structural behaviour from the side of the struggling designer.

13. FLOWER SHOP, HULST NL

The last step in this development report is the realisation of the
first outdoor glass roof in a guyed structure by the author. Not yet
in a form where the underspanning-and-glass cooperation is fully
actively structural, but in a way that the glass is passively
structural. The architect of the building is Walter Lockefeer, a
young Dutch architect working in the tradition of Dom van der Laan
(a school of Dutch architects where traditional materials and
proportions result in a very primary architecture). The design of
the glass roof has been made by the project-architect and the author
jointly, while the architect worked out the design very refined in
cooperation with the Octatube design team, making simultaneous
models, computer graphics, detail designs and statical analysis (fig
27).

27. Overall view of the glass roof between the brickwork cubes
28 and 29.
Two perspective views of the glass roof with the RHS perimeter profile, inside the brickwork walls.

The Flower shop is a pavilion building, composed of 2 brickwork cubes approximately 6 x 6 x 6 m, with an entrance bay of 2.23 m wide. The two cubes are entirely closed in the outer walls. The glass roof is at 5 m level and leaves a 1 m high parapet running around. The roof is completely composed of flat double glass panels (composed as reflective, heat-strengthened upper panels 8 mm, 12 mm air and 3.3.1 laminated clear glass as the lower panels). The total size of the roof is 6 x 15 m; the individual panels measure roughly 1,450 x 1,450 m. The panels are each supported at the 4 corners by support brackets designed in the same mode as the 'frog fingers' or glass brackets of the Glass Music Hall in Amsterdam. The glass brackets are glued to the laminated lower glass panels, with a detail that still enables turning of the support in vertical direction due to deflections. On the same spot the upper glass panel is also supported by a solid prop in the 12 mm air cavity, sealed between upper and lower panel. The steel support elements have been glued with a specially tested type of glue to the glass inner pane to give a sufficient adherence for a horizontal shear and vertical uplift. Each of the described glass brackets has been elongated with a vertical 20 mm thick pole that has been guyed by 8 mm steel rods in the node as drafted in fig 25. The whole structure is surrounded by a RHS profile, supporting a continuous edge gutter. Hence this system can still be regarded as halfway between a closed roof system and an open structure with a perimeter ringbeam that takes the compression forces, omitted in the upper glass panels. The slope of the glass panels is 1%. Completion of the structure in March 1990. Fig 28 and 29 give some details.
REFERENCES


A STUDY ON BEHAVIOR OF RIGID JOINT SPACE TRUSS UNDER VERTICAL AND LATERAL LOADING

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Summary

A large scale space truss for constructing an artificial ground above facilities, for example, a railroad track, a large river, existing buildings and so on, is required to have the capacity for supporting the heavy vertical load and also resisting the lateral load. In this paper, the mechanical behavior of the rigid joint space truss, which was developed to satisfy the required capacity, was experimentally and analytically investigated under vertical and lateral loading conditions.

1. Introduction

A space truss has been used as a roof structure, and its characteristics regarding strength have been well understood through the use of analysis and experiments[1]–[5]. However, when the large scale space truss is employed as the artificial ground, extremely heavy vertical loads act upon it, and a great influence is also exerted upon it by earthquakes.

As is illustrated in photo. 1-1, the system of the objective space truss is composed of lower chord members, upper chord members which intersect the lower ones in a horizontal plane at an angle of 45° and diagonal members existing in the vertical plane which include the lower chord members. Compared with other space trusses which are formed with another arrangements of members, the space truss has fewer members and joints. The members which constitute the space truss are square-shaped steel pipes which possess high strength against buckling and can compose a large grid. The lower chord members have a rhombus sectional shape which rotates to 45° around the axial line, and the upper chord members have a normal square sectional shape. The diagonal members also have a rhombus sectional shape which rotates to 45° around the axial line, like the lower chord members and closes both the lower and upper chord members at an angle of 45°. Therefore, each member’s intersectional line at joint connections is longer than the perimeter of the surface of a steel pipe which is cut at a right angle. Consequently, the joint connections can be joined by means of fillet welding. This paper aims at understanding the elasto-plastic behavior of a space truss which can support a heavy load, as well as resist horizontal loads through the rigid welding of joint connections. This was done by carrying out of vertical and lateral loading tests after which analytical investigations of the test results were made.

2. Strength test for joint connections of square-shaped steel pipes

2.1 Aim

Two test series are prepared. The test aims of first test are understanding the strength and stiffness of the rigid joint parts of the lower chord members and the upper chord members and obtaining basic data for the analytical solution of the strength and deformation behavior of the space truss. The test aims of second test are discussing the detail of stiffened connection.

2.2 Outline of the test plan

For the first test, four cruciform specimens were selected from each joint connection of the lower and the upper chord members. Two of them were selected for studying the restraint effectiveness of the diagonal members which are cross-linked at an angle of 45°. For the second test, four specimens with two type inner stiffeners plate are prepared. Each of these
specimens has a shape and a dimension just as illustrated in Fig. 2-1(a)-(b) and (e),(f). Table 2-1 shows the mechanical properties of the materials. Photo. 2-1 illustrates a view of the test. Furthermore, the members in the horizontal direction, seen in the picture, are through members.

The specimen was collapsed when the proportional compressive load was added in both the vertical and lateral direction but here the lateral load was fixed at a maximum load of 500 tons. The shrinkage of the full and local length of members, the concentrated strain distribution of the joint connections, and the load were all measured, using displacement transducers, strain gauges and oil gauges respectively.

2.3 Displacement behavior

Fig. 2-2 shows the relationship between the load and the axial shrinkage of the local length in the vertical direction. The straight line indicates the calculated value of the elastic stiffness of one member which has the same cross-section and length as the cruciform specimen. It is clarified that compared with the calculated value the elastic stiffness of the value obtained from the test is lower, and that the stiffness and strength of the specimens X1-1 and X1-2 with the lower chord joint connections are lower than those of the specimens X2-1 and X2-2 with the upper chord joint connections. The strength of the specimens X2-1, X2-2 decreases due to the plate buckling of the joint connection. However, the strength of the specimens X1-1 and X1-2 gradually rises with an increase of the axial shrinkage which is caused by the progressive bending deformation in which the height of the rhombus section of the through member becomes small. The strength of the specimens X1-2 and X2-2 with diagonal members is larger than those X1-1 and X2-1 without diagonal members due to the restraint effectiveness of the diagonal members to the joint connection panels. The yield strengths of specimen X1-2 and specimen X2-2 decrease to approximately 0.3 times and 0.47 times the yield strength of the member sections respectively. Fig.2-3 shows the load-displacement curves of specimen with inner stiffener plate. The rigidity and strength of these types increase comparing with non stiffener plate types. Especially the elastic rigidity of connection on LX type is the same as the axial rigidity of bracing and the maximum strength is P/Py=1.02 which is almost full strength on the section of bracing member.

3. Frame test plan of rigid joint space truss

3.1 Establishment of the specimen's scale

Scale of the supposed structure is 216m x 378m and scale of one unit of space truss is 18m x 18m. Due to the weldability of the joint connections and the limitation of the plate thickness of the members, the specimen was scaled down to 1/7.7 of the original. Fig. 3-1 shows the truss specimen. Sectional dimensions for the specimen are indicated in Table 3-1. The names of each member in this table correspond to those shown in Fig. 3-2. Table 3-2 explains the mechanical properties of the materials. The frame specimen which was composed of the four units of the space truss, each of which has three lower chord grids, was made as a model for the peripheral part of the column of the supposed structure. This was done under the consideration of stress distribution during lateral loading.

As a boundary condition for the truss, the upper and lower chord members were fixed at column positions. This was done by judging that generality is not lost in the specimen. This judgement was made through an understanding of the stress and the deformation behavior during lateral loading, which was done on the basis of the result obtained from the linear analysis of the supposed structure. In order to investigate the collapse mechanism of the space truss at the time of the frame test, the reduced scale ratio for the cross-section of the columns supporting the space truss was set at 1/5.

3.2 Plan for loading and measuring

A uniformly distributed vertical load was added to four intersectional points on the lower chord members of each unit using spreader beam. As shown in Fig. 3-3, the load was added to each unit to the extent of the design load in order and finally it was imposed equally. The lateral load was added to the lower end of the central column of the specimen concentrically.
in one direction which was parallel to the lower chord members and was added monotonously (see Fig. 3-1). Photo. 3-1 illustrates a view of the frame test. In this lateral loading method, load distribution, which is almost the same as the load distribution acting on the truss during an earthquake is recognized. Fig. 3-4-(a), (b) show the measuring points of the displacement (B: horizontal, C: vertical), and (c), (d) and (e) represent those of the strain. The load, displacement and the strain were measured employing oil gauges, displacement transducers, and strain gauges respectively.

4. Vertical behavior

The influence of the boundary condition and the joint connection stiffness on the deformation behavior and the mechanical behavior of the trussed frame were investigated by means of an analysis. The analysis was made by the employment of a structure analysis program for a wide use NASTRAN. Fig. 4-1 indicates the boundary condition. (a) illustrates that the truss is joined with a rigid column and (b) is a figure showing that the truss is connected with a column which has the same stiffness used for the specimen. The stiffness of the joint connection obtained from the test results in chapter 2 was substituted for the stiffness of the lower chord members excluding the through members and the whole diagonal members. The stiffness of the members were reduced 25% considering the length of the members.

Fig. 4-2-4-4 show the fluctuations of the vertical displacement of the lower chord members and the axial strain of both the upper and lower chord members on the function of the sum of vertical loads acting on each unit successively when the boundary conditions and the stiffness of the joint connections are considered as parameters. The solid line indicates boundary condition (a) and the other lines indicate boundary condition (b) where the dotted line represent the reduced stiffness of the joint connections. When the vertical load is given to the unit its displacement and axial strain become larger. The vertical displacement of the intersecting points of the lower chord members around the central column, in Fig. 4-2, hardly fluctuates even when loaded with other units. Therefore, the units really don't influence each other very much. In the case of the frame specimen including the columns (boundary condition (b)), the displacement further increases and it is also found that the reduction of the joint connection stiffness exerts an influence on the displacement. Fig. 4-3 shows a fluctuation of the axial strain of the upper chord's central portion in unit 1 at the location illustrated in Fig. 3-4-(e). In boundary condition (a), the degree of the axial strain of the four members is almost equal. On the other hand, in condition (b), differences occur in the axial strain in accordance with the direction of each member and the strain concentrates on the highly rigid through members G3-23 and G3-26. The fluctuation of the strain when the load is given to unit 2, 3 and 4 is small and the influence exerted between each unit is low. The fluctuation of the axial strain of the lower chord members close to the central column is shown in Fig. 4-4. Under boundary condition (b), when a load acts on a subjective unit, first strains occur, and then these strains are reduced to almost zero when the load is extended to the other units. In this case the influence between each unit is recognized. In the case of condition (b), the polarization of the axial strains occur and the axial strains remain present.

Fig. 4-5 shows the lateral displacement of the outside column at the locations illustrated in Fig. 3-4-(a), (b). The solid line represents the test result and the broken line represents the analytical result. Boundary condition (b), was applied to the analysis in which reduced stiffness was used. Although the quantity of the lateral displacement is infinitesimal, the analytically obtained displacement of the part at which the upper and lower chord members were connected with the outside column corresponds well to the results of the test. Fig 4-6 explains the relationship between the load and vertical displacement which compares the results obtained by using this analytical condition with the results of the test. A large difference between the value obtained from the analysis and that from the test is not found, and these values well agree with each other. Accordingly it is recognized that the establishment of a boundary condition and the understanding of the stiffness of the joint connection are important for the analytical study of space truss behavior. In the following analysis the boundary condition and the reduced stiffness mentioned above are adopted.
5. Horizontal behavior

Fig. 5-1 shows the relationship between the load in a lateral direction, at the lower end of the central column and the lateral displacement at the same location. The solid line represents the value obtained from the test and the dotted line represents the value obtained from the analysis. The value of the test increases in the displacement range from the area where loading exceeds 90 tons but sharply decreases in the vicinity of 150 tons. This is caused by the fact that the lower chord member which is marked with a (1) in the frame figure in Fig. 5-2 which illustrate the vicinity of the central column, buckled. However, when the strength decreases to approximately 140 tons, the strength of the subject gradually becomes large again. Furthermore, even in the case that the displacement increases to six times the yield deformation which corresponds to the yield strength, a reduction of the strength can not be recognized.

In the analysis which was made in the boundary condition established in chapter 4, the strength of the joint connections (obtained from the strength test of the joint connections), was substituted for the strength of the regular members which is different from that of the through members. Concerning the yield strength of the diagonal members, it was fixed at 0.6 times their own primary strength. A bi-linear model was adopted as the non-linear characteristic of the materials and the strain hardening gradient was set at 1/100 of young's modulus E. The judgement of the yield strength was at four points on the cross-section at the nodal points of the beam finite element whose length is the same as the member's length. In order to make an equivalent evaluation of the buckling phenomena of the individual members, an analysis was made through the removal of the members which had a compressive yield at all points on the sections of both of the nodal points one after the other.

Fig. 5-2 shows the members having a compressive yield and the order of the strength of the space truss decreases due to the compressive yield of member (1). However, after that, the deformation increases without a decrease of strength when the compressive yield of members (2), (3), and (4) occurs. This phenomena well explains the mechanical behavior which was obtained from the test. Photo. 5-1 shows the flexural buckling deformation of member CD. From the fact mentioned above, it is clarified that even if the buckling of one member causes the loss of the capacity of the rigid joint space truss for supporting the load, the truss does not show a large decrease in strength and in fact can maintain its strength by a re-allocation of the stress of the individual members.

Fig. 5-3 explains the relationship between the load and the vertical displacement. The displacement of C1-11 and C1-15 increases equally to the extent of the maximum strength. After the buckling of member (1) occurs, the upward vertical displacement of the unit in the loading direction hardly changes but the displacement of the unit in the reverse direction increases further. Fig. 5-4 shows the axial strain of the continuous lower chord members on both sides of the central column in the loading direction. The fluctuations of displacement, both at the compressive side and the tensile side are symmetrical as long as the load is given at maximum strength. However, after the strength is lowered, the member which was subject to flexural buckling maintains the axial load without any brittle behavior. From this phenomenon, it is found that the axial load which has been supported by the compressive members is re-allocated to the tensile members and other members. Fig. 5-5 shows the fluctuation of the axial strain of the central part for the upper chord of the No.1 unit. The fluctuation of the strain is small and the influence exerted by the lateral load is hardly seen.

Fig. 5-6 shows the analytical results on the fluctuation of the axial forces of truss members when a few members had the compressive yield successively at all points on the section. The number of the compressive yield members corresponds to the number shown in Fig. 5-1. In this figure, only the change of the direction of axial forces is expressed. The bold lines represent the compressive members. There exists not only the members of which the axial forces increases in the same direction but also the members which change the direction to the initial direction. As the compressive members yield perfectly successively, compression members increase in the lower chords in case that tension members increase in the upper chords and the reverse phenomenon is also shown. It can be understood that the upper and lower chord members keep the balance well through the diagonal members.
6. Conclusion

In order to understand the elasto-plastic behavior of the rigid joint space truss which supports heavy loads, tests for both vertical and lateral load were performed and the results of these tests were analytically investigated. The results obtained from this study are summarized as follows:

1) When a vertical load is placed on a rigid joint space truss even if it is in a partially distributed condition, the influence exerted upon the deformation and the stress induced between each unit is small.

2) For the investigation of the mechanical behavior of a space truss through means of an analysis, it is of vital importance to model the boundary condition of the trussed frame based on the realities and to understand the strength and stiffness of the joint connections.

3) When a lateral load is given to the space truss in question, even if buckling occurs on an individual member, the space truss shows stable deformation behavior due to the reallocation of stress. Thus, it possesses sufficient ultimate strength and plastic deformation capacity.

References


Table 2-1 Mechanical properties of the materials

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\sigma_y$ (MPa)</th>
<th>$\sigma_u$ (MPa)</th>
<th>$E_t$ (%)</th>
<th>Rib (mm)</th>
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<td>X1-1</td>
<td>4.382</td>
<td>5.204</td>
<td>23.7</td>
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<td>X1-2</td>
<td>5.440</td>
<td>5.960</td>
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<tr>
<td>X2-1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>X2-2</td>
<td>-</td>
<td>-</td>
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<tr>
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<td>LX-2</td>
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<td>5.475</td>
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<td>X1-4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>28</td>
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Fig. 2-1 Shape and dimension of the connection specimen

Fig. 2-2 Vertical load-axial displacement curves

Fig. 2-3 Vertical load-axial displacement curves
Table 3-1 Dimensions of the truss specimen

<table>
<thead>
<tr>
<th>Assortment</th>
<th>Form of Section</th>
<th>4(cm²)</th>
<th>l (mm)</th>
<th>d/t</th>
<th>Assortment</th>
<th>Form of Section</th>
<th>4(cm²)</th>
<th>l (mm)</th>
<th>d/t</th>
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<td>Lower Chord</td>
<td>A</td>
<td>100° x 4.5</td>
<td>16.07</td>
<td>80.3</td>
<td>22.2</td>
<td>G</td>
<td>75° x 3.2</td>
<td>8.02</td>
<td>52.2</td>
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<td></td>
<td>B</td>
<td>150° x 8.0</td>
<td>33.63</td>
<td>233</td>
<td>25</td>
<td>H</td>
<td>100° x 4.5</td>
<td>16.67</td>
<td>38.3</td>
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<td></td>
<td>C</td>
<td>150° x 8.0</td>
<td>5.84</td>
<td>39.6</td>
<td>5.84</td>
<td>I</td>
<td>100° x 4.5</td>
<td>3.67</td>
<td>2.19</td>
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<td>Upper Chord</td>
<td>D</td>
<td>125° x 4.5</td>
<td>21.17</td>
<td>33.8</td>
<td>27.8</td>
<td>J</td>
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<td></td>
<td>E</td>
<td>200° x 8.0</td>
<td>45.63</td>
<td>22.4</td>
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<td>K</td>
<td>175° x 6.0</td>
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<td>F</td>
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<td>G</td>
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Table 3-2 Mechanical properties of the materials

<table>
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<tr>
<th>Base Metal</th>
<th>T (mm)</th>
<th>σy (t/cm²)</th>
<th>σu (t/cm²)</th>
<th>E1 (%)</th>
<th>σy (t/cm²)</th>
<th>σu (t/cm²)</th>
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<tr>
<td>□ - 100° x 100° x 4.5</td>
<td>4.23</td>
<td>3.822</td>
<td>4.576</td>
<td>34.28</td>
<td>3.77</td>
<td>4.07</td>
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<tr>
<td>□ - 150° x 150° x 6.0</td>
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<td>4.007</td>
<td>4.789</td>
<td>33.20</td>
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<td>4.06</td>
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Fig. 3-1 Shape and dimensions of the truss specimen

Photo. 3-1 View of the space truss test

180
Fig. 3-2 Name of each member

Fig. 3-3 Vertical loading plan

(a) Displacement transducers for the lower chord members

(b) Displacement transducers for the upper chord members

(c) Strain gauges for the lower chord members

(d) Strain gauges for the diagonal

(e) Strain gauges for the upper chord members

Fig. 3-4 Locations for the measurement of the displacement and strain
Fig. 4-1 Boundary condition

(a) Fixed support  
(b-1) Specimen's outside column support  
(b-2) Specimen's central column support

Fig. 4-2 Vertical displacement of the lower chord

Fig. 4-3 Axial strain of the upper chord

Fig. 4-4 Axial strain of the lower chord

Fig. 4-5 Lateral displacement of the outside column

Fig. 4-6 Vertical displacement of the lower chord

Fig. 5-1 Load-displacement curve of the central column
Fig. 5-2 Order of the compressive yield members

Photo. 5-1 Flexural buckling of member

Fig. 5-3 Load and vertical displacement curves of the lower chord

Fig. 5-4 Load and axial strain curves of the lower chord

Fig. 5-5 Load and axial strain curves of the upper chord

Fig. 5-6 Fluctuation of the axial forces of the truss members
TUBULAR STRUCTURES
ARCHITECTURAL CONCEPTS WITH STANDARDIZED SYSTEMS

A. Waberseck
Mero-Raumstruktur

Introduction

Characteristics
of different standard systems

- Ball Node
- Disc Node
- Cylinder Node
- Bowl Node

Summary
combinations of standard systems
Introduction

Although tubes constitute a high aesthetic construction element with minimum surface area and maximum resistance against buckling, there are still reservations to their use.

Both the connection of tubes among each other and the applications of roof and wall cladding are details which are still not well known to many architects and steel fabricators.

In respect to the construction details the easiest way is to weld the tubular sections in the factory, but you will often find arguments against that solution because of dimensions of sections becoming too big for

- transportation;
- galvanization and powder coating;
- erection.

Roof constructions designed as domes, pyramids, hypars or especially space frames are out of the afore-mentioned reasons often executed with standardized screwed connections.
Characteristics of different Standard Systems

The "system" is a standard solution for the connection between two or more members and this connection generally being made via a node. The length of members and angles between the members are not fixed so that they can be used for a lot of different designs.

The advantage of all standardized prefabricated node-member connections is that all members - no matter under what angle they have to be connected - can be cut rectangularly at their ends. The complexity of the system is concentrated in the shape and the exact manufacture of the nodes.

The Ball Node System

The spherical node is out of steel C 45. The standard diameters vary from 49,3 to 350 mm. The bolts are screwed into the node via a sleeve. Members out of circular hollow sections are designed for axial loads only. Therefore the loads have to be led into the construction via the nodes.

When working with textiles the ball node is the appropriate solution. The membrane can be fixed to the nodes easily. The combination of screwed structures and textiles is used especially for temporary wide span constructions.

Cladded either with sheets or with glass the ball node system needs additional purlins serving linear support. With the purlins fixed on stools in different height you will enable inclined roofs to be done above a horizontal structure. The structure can be designed

- single layer;
- double layer;
- or with multiple layers for a wide span.
MERO PLUS Systems

The top layer of the structure and the purlin system can be put in one profile, i.e. a rectangular hollow profile if the roof already is in shape of an inclination. The cladding can be fixed direct to the load bearing structure.

The Disc Node

Specialized for domes and other shapes with double curvature the disc node can only be used if the geometry of the load bearing structure consists of triangles. The structure is very light and economical. The triangular cladding, however, usually produced of rectangular panels has a lot of waste material.

The Cylinder Node

The cylinder node enables a bending resistant connection to be done with the members. Thus single layer constructions can be executed with curved or plane surface and with triangular or quadrangular grid. The rectangular or trapezoidal cladding elements are much cheaper than triangular panels.

In respect of economical solutions single layer structures have a limited span. According to snow and wind loads and the shape of the building you can achieve domes with a diameter of approx. 50 m.

The single layer cylinder node system can be stiffened and stabilized through the combination with a double layer system.
The Bowl Node

The bowl node may be used as connection between single layer and double layer constructions or as a uniform double layer system for wide span structures. The top chord consists of rectangular hollow section on which the cladding can be fixed direct by means of a sealing system. For diagonals and bottom chord circular hollow sections or even cables can be used.

Summary

The presentation of different structural elements and their application is the traditional way for an introduction.

Today's design work goes just the other way around. Beginning with the form of a building, followed by the cladding material, the support for the cladding, finally the load bearing structure has to be chosen, i.e. you will have to decide from the outside to the inside.

The restrictions in the dimension of the cladding - especially the glazing - lead to small grids in the surface area. For that reason the top grid of the node-member structure can be subdivided by one or several "secondary" members. They are connected without nodes direct to one another and serve to support the panels only and are not part of the main structure.

During the past 10 minutes I could present only the topics from the great variety of standardized connections. I wanted to prove that with the "Know-how" of using those systems highly aesthetic and efficient solutions for modern design are offered on the market.
STRESS DISTRIBUTIONS AT WELD TOES IN IMPROVED AND ORDINARY FILLET WELDS IN X-TYPE TUBULAR JOINTS

Kim S. Elliott
Department of Civil Engineering
University of Nottingham, Nottingham, UK.

Summary

Three-dimensional photoelastic analysis of tubular X-joints has been carried out to determine the effect of two different fillet weld profiles on surface stress concentrations. The joints studied had brace to chord wall angles of 60° and 90° and other typical geometric parameters common for this kind of work. The two different weld profiles were classed as "ordinary" and "improved"; the former conforming with the minimum requirements of the AWS code, and the latter using a small additional triangular fillet at the chord end of the weld to improve the geometry of the weld toe there. Reductions in SCFs were in the order of 20% at saddle positions and 3 to 10% at crown positions. The increased weld size shifted the stress distribution along the tube walls away from the crotch of the joint by a distance % of the increase in weld leg length. Stresses more than 3 wall thicknesses away from the crotch were not influenced by the different weld profile.

Notation

L, l, D, d, T, t, θ, ψ Tubular joint dimensions defined in Figure 3
H, h, a, α, η Weld profile dimensions defined in Figure 2
P Axial tensile load applied to brace tubes
K Stress concentration factor = σ_{max} / σ_{nom}
I Stress index = σ_{max} / σ_{nom}
V Shear force in chord tube
z Stress zone
σ Stress
σ_{nom} Mean axial stress in brace tube
σ_{max} Maximum stress
φ Angular position of σ_{max} in a fillet
γ D/2T
λ H/T

Subscripts

b: brace c: chord 0: Ordinary weld profile
s: shell n: notch I: Improved weld profile
A: axial stress B: bending stress SCF: Stress concentration factor

Abbreviations

Stress Analysis at Weld Toes

The big advantage in using frozen stress photoelasticity for three dimensional (3-d) stress analysis is that the analyst may choose an almost inexhaustible number of planes from which data may be collected. It is not always necessary to have prior knowledge of the positions of maximum stresses, or indeed of the principal plane in which the maxima lie. 3-d photoelastic analysis enables the stresses in the surfaces of models to be fully described, and the technique has been adopted for use in the analysis of multibraced 3-d tubular joints. The models, one of which is shown in Fig. 1, are scaled replicas of the type of fillet
welded joints found in an offshore steel jacket. The model shown in Fig. 1 is an X type joint comprising 3 tubes (one chord plus two braces). However, previous work by Little and Fessler (1), Marston, Ollerton and Fessler (2), Fessler and Edwards (3) and Elliott and Fessler (4) has included up to 6 brace tubes attached to one chord tube manufactured in a single homogeneous epoxy resin casting. The manufacturing procedure is well documented (eg 5) and need not be reproduced here.

In all these works continuous surface stress distributions have been obtained. These have been used to describe structural behaviour near to the intersections of the tubes. The most important stress is the maximum principal stress \( \sigma_{\text{max}} \). In dealing with a welded structure, this is even more important if the trajectory of \( \sigma_{\text{max}} \) happens to be perpendicular to the run of the weld toe. Despite their enormous size, offshore tubular joints are prone to fatigue failures on a microscopic scale. The fatigue crack initiation site coincides with the location of the stress concentration – at the toe of the weld.

Efforts have been made to improve the profile of the weld at the weld toes by grinding, TIG dressing etc. in an attempt to extend fatigue crack initiation life whilst simultaneously reducing the magnitude of the stress there. Other ideas have focussed on improving the shape of the final weld bead at the toe, or even employing skilled labour to lay down an additional fillet weld (using electrodes with favourable wetting characteristics) in an attempt to shift the critical hot-spot into a reduced stress field. Another reason for doing this is to reduce the likelihood of crack like defects occurring at the toe. These options are summarised in Fig. 2, which includes details of the “improved” weld profile which were first proposed by Marshall (6). This paper is concerned with the effects of the improved profile on stresses both in the surfaces of the tubes and at weld toes. The questions to be answered are: Is the improved profile effective in reducing stress concentrations, and what effect does the addition fillet have on the structural compatibility of the tubes?

The paper shows that both these items are significant only where the gradients of the surface stress distributions are large. Data from frozen stress photoelastic models are ideal for recording this type of information.

Shapes of Tubular Joints and Weld Profiles

The models chosen for analysis were X-joints with two different brace angles, \( \theta = 90^\circ \) and \( 60^\circ \) in Fig. 3, one value for the diameter to thickness ratio \( D/T = 25 \), one value for the diameter ratio \( \beta = D/d = 0.5 \), but two values for the wall ratio \( \tau = t/T = 0.5 \) and 0.3. In the outcome, inaccuracies in the manufacture of one of the models led to \( \tau = 0.25 \) and 0.35 in the saddle plane of the \( \theta = 90^\circ \) joint. The chord diameter was \( D = 200\)mm in both models. The models were loaded in axial tension applied to the brace walls by a single point load attached by wire and pulley to end plugs glued into the brace tubes at a distance of at least 3.8d (380mm) from the intersection. The chord tube was unrestrained and free to ovalise. The length of the chord tube, \( L = 30 \), was determined empirically using a technique where the diametrical deformation in the shortened tube was found to be equal to that of a very long (\( L/D = 14 \)) tube. The shapes and dimensions for the models are given in Table 1.

Two shapes of fillet welds were formed on the models on opposite sides of each plane of symmetry so that the effects of weld profile on stresses would not be confused with changes in tubular geometry. The shapes and dimensions of the welds are given in Fig. 2 and in Table 1. The “ordinary” profile conforms to the minimum requirements for weld leg
length projection onto the outside chord wall surface given in the AWS code of practice (7). The weld toe radii were made deliberately small, aiming for a ratio \( r/T = 0.03 \) to 0.06 at the chord end of the weld. The weld angles \( \alpha_C \) were made steep, i.e. between 45° to 72°. The length of the weld fillet along the outer brace tube was controlled by the requirement of making a brace wall weld preparation angle of \( \phi/2 \), where \( \phi \) is the local dihedral wall angle at each location in the joint. This led to smaller weld angles \( \alpha_C \equiv 18° \).

The "improved" profile conforms to the recommendations proposed by Marshall (6), which shows that an isosceles triangular fillet has been added to the ordinary profile at the chord end only. The brace end of the ordinary weld is unaltered. This additional fillet improves weld toe geometry in three ways: weld leg length is greater, weld angle \( \alpha_C \) is smaller, and larger toe radii may be formed. Although it is not possible to reproduce the shapes of individual weld beads on small scale photoelastic models, the weld toe geometries achieved here are realistic of some of the shapes used in other tubular joint work. Internal fillets at the root were formed using a single radii \( r_i \) to proportionate scale based on a root gap of 3 mm.

<table>
<thead>
<tr>
<th>Model Ref</th>
<th>Position in model</th>
<th>( \psi )</th>
<th>Weld Profile</th>
<th>( \tau )</th>
<th>Brace t</th>
<th>r/t</th>
<th>h/t</th>
<th>( \alpha_B )</th>
<th>Chord T</th>
<th>r/t</th>
<th>H/T</th>
<th>( \alpha_C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>X90</td>
<td>Crown</td>
<td>90° 0</td>
<td>0.49</td>
<td>3.85</td>
<td>0.41</td>
<td>1.20</td>
<td>12°</td>
<td>7.80</td>
<td>0.070</td>
<td>0.22</td>
<td>68°</td>
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<tr>
<td></td>
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<td>0.50</td>
<td>3.93</td>
<td>0.43</td>
<td>1.27</td>
<td>11°</td>
<td>7.82</td>
<td>0.065</td>
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<tr>
<td></td>
<td>Saddle</td>
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<td>0.65</td>
<td>2.18</td>
<td>20°</td>
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<td>0.18</td>
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<tr>
<td></td>
<td>Saddle</td>
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<td>0.46</td>
<td>2.08</td>
<td>22°</td>
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<td>40°</td>
<td></td>
</tr>
<tr>
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<td>27°</td>
<td>8.30</td>
<td>0.04</td>
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<td>60°</td>
<td></td>
</tr>
<tr>
<td></td>
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<td>0.96</td>
<td>0.93</td>
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<td>8.27</td>
<td>0.19</td>
<td>0.18</td>
<td>39°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Crown toe</td>
<td>120° 1</td>
<td>0.49</td>
<td>3.87</td>
<td>0.67</td>
<td>1.08</td>
<td>6°</td>
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</tr>
<tr>
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<td>Saddle</td>
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<td>1.16</td>
<td>7°</td>
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<td></td>
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<tr>
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<td>Saddle</td>
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<td>0.15</td>
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<td>12°</td>
<td>8.04</td>
<td>0.17</td>
<td>0.44</td>
<td>19°</td>
<td></td>
</tr>
</tbody>
</table>

In all models: \( D = 200 \pm 1.5 \text{mm}, d = 100 \pm 1 \text{mm}, L = 30 \text{ and } l = 3.8d \)

Table 1: Shape Parameters and Dimensions of Models
**Analysis**

**Stresses in Tubular Connections**

Axial forces cause bending moments in thin walled tubes. The magnitude of the moment, expressed in terms of the axial force, depends on the local flexibility of the joint at different positions in the intersection, i.e. crown and saddle. In single brace joints these are greatest at the saddle position, particularly where the $\beta$ ratio is between 0.3 and 0.8. The degree of bending at the crown is virtually independent of $\beta$. The proximity of other braces has been shown to have an important influence on bending in the tubes (8), but this has been eliminated in this work by using X-type joints.

The axial force in the brace tube causes diametrical deformation in the chord, i.e., the chord tube ovalises as shown in Fig. 4. The bending moment in the chord can be approximate using ring analogy, although this will over estimate the true value due to the longitudinal stiffness of the chord tube. These deformations cause bending in the brace which remains rigidly attached to the chord by the full penetration weld. The decay in bending moment in the brace at a distance $x$ from the intersection may be expressed in the form (neglecting small second order terms):

$$M = e^{-\mu x} (\cos b\mu x - \sin b\mu x)$$

where $a$ and $b$ are empirical constants which depend on the shape of the joint, and $\mu$ is called the characteristic length and is known to be a function of $\sqrt{RT}$.

Two points of contraflexure exist, points A and B in Fig. 4. The bending moments are insignificant beyond point B. The distance OB must be exceeded in model design (whether small or full scale) to ensure that boundary conditions do not inhibit the natural deformability of the joint. The magnitude of the axial force in the brace tube also varies with the circumferential position in the intersection. The chord is stiffer at the saddle position because the dihedral wall angle is usually larger than at the crown. Thus there is a maldistribution of axial loading in the brace which further complicates a simple ring analogy.

Stresses in the tube walls are the sum of axial ($\sigma_a = P/A$) and bending ($\sigma_b = M/IZ$) stresses. Outer and inner surface stresses are measured and the mean and semi-differences are computed to give $\sigma_a$ and $\sigma_b$, from which $P$ and $M$ are determined. These stresses are called "shell" stresses, and, despite the stiffening influence of the weld fillet, are a function only of tubular joint shape (i.e., X, T, Y, K etc) and geometry ($\theta$, $\gamma$, $\beta$, $\tau$ in X-joints). The surface stress is divided by the mean (or nominal) axial stress $\sigma_{nom}$ in the brace tube and is called the stress index $I$. The maximum stress index is the SCF, $K$. If only shell stresses are measured, the SCF is a shell factor $K_s$.

Notch stresses are found near to geometric discontinuities, such as at weld toes or at root gaps in internal fillets. The extent of notch stresses is small, about one wall thickness from the weld toe. Although notch stresses are additive to shell stresses they are expressed in multiples of the shell stress in order to generalise the results. Notch stresses are therefore a function of weld size and shape ($H, \alpha, r$) and position in the tubular intersection (crown and saddle).
Results

Axial Forces and Bending Moments in the Tube Walls of the X90 Joint

These may be computed from the outer and inner surface stresses which are presented in Fig. 5 for the brace wall, and in Fig. 6 for the chord wall. Results at the crown position are separated from those at the saddle position because the load paths differ.

Brace wall stresses in the crown plane (Fig. 5[a]) are nearly all due to wall bending. In all cases the axial stress index is less than \( \pm 0.4 \), and it is surprising to find compressive axial stresses in a brace loaded in tension. This is evidence of the considerable maldistribution of brace loading in the intersection. A first point of contraflexure occurs between 2.4\( t \) and 4.0\( t \) from the weld toes for \( \tau = 0.5 \) and 0.3 respectively. However, the distances to these points measured from the crotch of the joint are the same, ie 0.14\( d \) and 0.13\( d \), respectively. This shows that the decay in bending is independent of \( \tau \) and weld profile.

Brace wall stresses in the saddle plane are given in Fig. 5[b]. Results from joints having 3 different values for \( \tau \) are presented. The average axial stress index is about \( +2.0 \) (which is as expected because of the small axial stresses measured at the crown) and is not dependent on \( \tau \). Bending stresses are small (\( |\sigma\| \leq \pm 0.5 \)) beyond the first point of contraflexure, which occurs at \( y = 2.7t \) to 3.1\( t \) from weld toes. Between this point and the weld toe bending stress gradients are large and this affects the magnitude of the SCF at the toe. The degree of bending, which is expressed as \( \eta = (\sigma_B/(\sigma_A + \sigma_B)) \) increases with increasing \( \tau \). This implies that the differences in SCF in the ordinary and improved profiles is greater with increasing \( \tau \).

Chord wall stresses in the crown plane, Fig. 6[a], are caused by wall bending (due to deformations in the tube near to the brace wall), and axial forces (due to membrane action in the chord tube). Both types of stresses decay slowly with distance from the weld toe and therefore their effect on extrapolated shell SCFs is negligible. The stresses at the root fillet and in the chord plug are complex and difficult to interpret. They are also smaller in magnitude than at weld toes.

The most important stresses occur in the chord wall in the saddle plane, Fig. 6[b]. Results from joints having 3 different values for \( \tau \) are presented. Near to the weld toe the bending stresses are large and the degree of bending \( \eta \geq 5 \) is also large. Bending stress gradients are large and the bending stresses are zero at \( x = 3.0T \) to 3.2\( T \) from weld toes. This distance is independent of weld profile and \( \tau \).

Surface Stress Distributions at Weld Toes

Results are presented for the X-60° model only, because of the problems in taking measurements at weld toes in some photoelastically damaged areas of the X-90° model. Principal meridional stresses are shown in Fig. 7 for the two crown positions where the local wall angle is \( \psi = 60° \) and 120°. Hoop stresses (which are not principal values, but have trajectories perpendicular to the run of the weld toe and are therefore important) in the saddle planes are shown in Fig. 8. The important values are in the outside wall where I in excess of 10 were found at the saddle. Although inside wall (ie root gap) stresses are lower, the maximum value is -5.2, they are none the less important because they cannot be measured using surface strain gauges.

The experimental data points are plotted perpendicular to the surfaces of the weld profiles and therefore give a visual representation of the stresses there. It is therefore possible to measure surface stress gradients in the important positions near to weld toes, and to detect the positions at which notch effects begin. The distance from
the weld toe to the commencement of notch effected stresses is called the notch zone. Values for chord $Z_c$ and brace wall $Z_b$ notch zones are given in Table 2. The range of values is large, ie $0.2T \leq Z_c \leq 0.95T$ and $0.17t \leq Z_b \leq 0.5t$, but in general notch zones are smaller for the improved weld profile. 

Local peak stresses are found in both the brace and chord weld toes. In between these positions in the surface of the body of the weld, stresses are usually much smaller than peak values. The exception is in the improved weld profile where a geometric discontinuity, at the blend between the ordinary and the improved welds, creates a third peak stress. This third peak stress is equally as important as the stresses measured at weld toes because fatigue cracking has been observed in the second and third crevices between weld beads in some improved welds (9). The most important information carried in stress distributions is the disruption to the surface stresses in the chord wall caused by the additional weld fillet used to create the improved profile. The shift in the stress distribution is equal to approximately $\frac{\tau}{t}$ of the shift in the position of the weld toe. The shift is most pronounced at the saddle position where chord wall bending stresses are largest.

**Stress Concentration Factors**

Three SCFs are given although one, the notch factor $K_n$ is the derivative of the peak SCF, $K$ and the shell factor $K_S$. Shell factors are obtained from the linear extrapolation of stress indices beyond the end of the notch zone. Curves are fitted to the data points and the best straight line is used in the extrapolation method. This obviously is subject to graphical interpretation and error, and has been assessed to be $\pm 5\%$ (10). The peak SCF is a direct measurement and is susceptible to less error, typically $\pm 3\%$. Thus the notch factor may be quoted to within $\pm 8\%$.

Shell SCFs are given in Table 2 and Fig 9 for both the brace and chord weld positions. Positive values mean that the sense of the stress is the same as for the axial brace loading stresses. As expected, values in the saddle plane are greater in the ordinary profile weld toes (typically 5 to 10%) than in the improved because the former is closer to the crotch of the joint (which is known to be the most highly stressed region (10)). The effect of $\tau$ is taken care of by presenting values in terms of $K_S/\tau$ in Fig. 9, where the main variable is the weld leg length parameter $H/T$ or $h/t$.

Peak SCFs are presented in Table 2 for the $X-60^\circ$ model only. The SCF at the blend of the ordinary and improved profiles is, in some cases, the greatest value. The most important SCF is the maximum, $+10.2$ which was found in the saddle plane at the chord end of the ordinary profile.

Notch SCFs, $K_n = K/K_S$ are presented in Table 2 for the $X-60^\circ$ model only. With one exception, $K_n > 1.27$ and are greatest at the more acute weld profiles. Earlier and more extensive work has shown $K_n$ to be proportional to $\sin \alpha/2$ (10), and this is borne out in these limited results. Notch factors are nearly always greater in the ordinary profile welds.

The position of the peak SCF in the fillet is given by $\varphi$, see Fig. 2. The peak stress is always found in the weld toe fillet. The angular positions are given in Table 2. Values for $\varphi$ are greater (15$^\circ$ to 38$^\circ$) in the most acute welds, and smaller (8$^\circ$ to 26$^\circ$, but typically about 12$^\circ$) in the obtuse welds. Although these precise figures are subject to graphical error etc., they are probably not very important except for the fact that they show the peak SCF occurs well within the weld toe fillet, and not at the weld toe as has often been assumed.
Table 2: Magnitudes and Positions of Stress Concentration Factors and Notch Zones for all Models

Discussion

Interpretation of the Results

An "improved" weld profile may not always be beneficial. Only where the surface stress gradient in the tube wall immediately ahead of the weld toe is large is the improvement worthwhile. Two effects are responsible:

i) the position of the weld toe is shifted to a lower stress field and this disruption causes a shift in the surface stress distribution which is slightly less than would be expected.

ii) the improved weld profile is designed with the specific intention of reducing the weld toe angle and increasing the weld toe radius. Both these parameters are known to influence the notch stresses in welds. Although it has not been possible to carry out a parametric study on and , their effect on SCFs is clearly seen.

The approach taken in this paper in studying the influence of weld profile on surface stresses was first to consider the global effects of tubular interaction. These were given in Figs. 5 and 6 for four different cases. In each case stresses caused by axial forces and bending moments in the tube walls were presented. It was found that large surface stress gradients were associated with large bending effects, and that it was only in these positions that an improved profile was beneficial in reducing SCFs.

Correction Factors for SCFs in Different Profiles

Marshall proposed a modification factor to take into account the additional weld leg length in improved profiles (11). The assumption was that the surface stress gradient \( \frac{\partial \sigma}{\partial x} = 6V/T \), a fair assumption.
providing one knows what the shear force \( V \) in the brace wall is. The problem here is that \( V \) is dependent on just as many tubular joint parameters as the SCFs. The correction factor at the chord weld toes is given in ref. 11 by:

\[
\frac{\text{SCF(Improved)}}{\text{SCF(Ordinary)}} = \frac{0.3 \sqrt{\gamma} (\lambda - 0.5 \tau)}{0.3 \sqrt{\gamma}} < 1
\]

where the term \((\lambda - 0.5 \tau)\) represents the additional weld leg length \( H/T \) in the improved and ordinary profiles.

For the weld sizes and joint geometry used in this work, Marshall's correction factor is in the range of 0.74 to 0.82, with a mean value of 0.78. This is independent of \( \theta, \psi, \beta, \) and \( \tau \). The results from this work adjusted for different \( \tau \) ratios, are given in Table 3 as follows:

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<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>This work</td>
<td>Marshall</td>
<td>This work</td>
<td>Marshall</td>
</tr>
<tr>
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<td></td>
<td>0.95</td>
<td>0.84</td>
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</tbody>
</table>

Table 3: Shell SCF for Improved and Ordinary Weld Profiles. \( K_{S1} / K_{S0} \)

In most cases Marshall's equation over-estimates the reduction in SCF. There are two possible reasons for this. When the weld is increased in size the stiffness of the joint is locally greater. The result of this is that the shift in the stress distribution curve is less than expected and the reduction in SCF is not so great. The second reason is that not all the axial load in the brace wall is transferred and resolved in the outer part of the chord tube because axial stresses are known to be present in the inner part (called the 'plug') of the chord tube (10).

Conclusions

Surface stresses have been measured in the important planes of symmetry in X-type tubular joints. The brace to chord angle was 60° and 90° and other tubular parameters were typical for offshore structures. Values for the wall thickness ratio were varied from 0.25 to 0.50. The main focus of attention was on the effect of using two different weld profiles, an ordinary profile. Results have been given for surface stresses i) in the tube walls, from which axial forces and bending moments have been derived, ii) in the weld toe fillets from which the effects of the two different profiles are seen. The main conclusions are:

1. It is not worth forming an improved weld profile at the crown position. The stresses there are smaller than at the saddle and the gradients of stress are so small that it is not possible to shift the weld toe into a reduced stress field. The only benefit is in reducing notch effects, which could be more easily controlled by grinding or dressing.
2. Reductions in SCFs at saddle positions were up to 30% at the chord weld toe. The maximum shell SCFs were as follows:

<table>
<thead>
<tr>
<th></th>
<th>X90 Ordinary</th>
<th>Improved I+O</th>
<th>X60 Ordinary</th>
<th>Improved I+O</th>
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</thead>
<tbody>
<tr>
<td>Chord wall</td>
<td>7.65</td>
<td>6.55</td>
<td>0.85</td>
<td>5.55</td>
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<tr>
<td>Brace wall</td>
<td>8.15</td>
<td>8.00</td>
<td>0.98</td>
<td>4.65</td>
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</table>

3. There is a maldistribution of axial loading in the brace tubes. The average axial stress at the crown is 0.5 times the mean axial stress, whilst at the saddle it is 2.0 to 2.5 times the mean axial stress.

4. Tube wall bending moments are large in the saddle positions. Bending stresses are additive to axial stresses in the outer tube surface. The result is large concentrations of stress near to the chord weld toe. No feasible weld profile could reduce the shell SCF to less than 6 in the X90 joint, or less than 5 in the X60 joint, because of the maximum size of weld that could be reasonably placed in practice.

References


Fig. 1. 3-d Photoelastic Model of X90 Tubular Joint. Note permanent deformations after loading braces in axial tension.
Fig. 2. Definitions of Weld Profiles

Fig. 3. Definitions of Tubular Joint

Fig. 4. Deformations and Stresses in Tubular Joint
Fig. 5(a). Brace Wall Surface Stresses in Crown Plane for Model X90

Fig. 5(b). Brace Wall Surface Stresses in Saddle Plane for Model X90

Fig. 6(a). Chord Wall Surface Stresses in Crown Plane for Model X90

Fig. 6(b). Chord Wall Surface Stresses in Saddle Plane for Model X90

Fig. 9. Variation in Shell SCFs with Weld Leg Length
Fig. 7. Meridional Stress Distributions in Model X60 at Crown Heel (top) and Crown Toe
Fig. 8. Hoop Stress Distributions in Model X60 at Saddle Position
STRESS CONCENTRATIONS IN OVERLAPPED KT JOINTS

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ABSTRACT

Extensive research and development into stress concentrations in tubular joints have been done over the past two decades. Very little information does, however, exist about overlapped KT joints.

This paper describes the analysis of ten different overlapped KT joints. The results of the analysis have been summarized into parametric formulas for the stress concentration factors. The formulas are developed as modifiers to existing K joint formulas.

INTRODUCTION

Fatigue is one of the most common failure causes in North Sea steel platforms. The fatigue is to a large extent governed by the stress concentrations (hot spots) occurring in the welds between the braces and the chord of tubular joints.

The determination of these hot spot stresses is therefore very important when calculating or estimating the fatigue lives of steel platforms.

Parametric formulas for the determination of stress concentration factors (SCF) in various joint geometries have been developed. Little information exist, however, for overlapped KT joints.

This paper describes the results of a study into the behaviour of overlapped KT joints, and gives the background for a set of formulas developed for the prediction of SCF's in this type of joint.
OVERLAPPED KT JOINTS (KTO JOINTS)

Overlapped joints are by tradition unwanted in offshore jackets. The reasons for this are twofold:

- The overlapped joint is difficult and expensive to fabricate.
- Uncertainties exist about the static and fatigue strength of overlapped joints due to lack of design information.

Overlapped joints are, however, difficult to avoid when the chord diameter is relatively small compared to the brace diameters. Keeping such joints non-overlapped results in additional bending moments in the chord due to joint eccentricities.

Overlapped joints have in general terms better static and fatigue strength than simple joints for most load cases except for out-of-plane bending moments.

Overlaps were earlier catered for by using a very small value for the gaps between the braces. This was a very crude representation that did not take degree of overlap into account.

Efthymiou and Durkin (2) presented in 1985 equations for the SCF in, amongst others, overlapped K joints. These equations utilize a gap factor which assumes positive or negative values according to whether the joints are simple or overlapped.

While the axial load transfer between the braces in a simple joint is via the chord, the load transfer in an overlapped joint is partly via the chord and partly via the brace/brace welds. As the overlap increases, the amount of load transfer in the braces increases. This will at the same time reduce the load transfer, and hence the stresses in the chord.

ANALYSIS OF OVERLAPPED KTO JOINTS

Ten KTO joints have been analyzed in an internal Statoil project to investigate the behaviour of and stress concentrations in the KTO joints.

Only joints with the braces in one single plane have been analyzed.

The most common type of overlapping KT joints are joints where the K braces are not overlapping and the T brace are overlapping both K braces, as shown in figure 1.

Joints where the K braces are overlapping each other or overlapping the T brace are not common in offshore structures, and are therefore ignored.
There is a high number of variable geometric properties in KTO joints. The following have been investigated:

- K and T brace diameters
- K and T brace wall thicknesses
- gap between the K braces
- chord wall thickness

The following simplifications and assumptions were made in order to reduce the number of geometric variables.

- The diameter and thickness of the two K braces were always equal for a specific joint.
- The angles of the T and K braces to the chord were always 90° and 45° respectively.
- The T brace was always centric with respect to the two K braces and the chord.
- The length of the chord were always 5 times the chord diameter.
- The end fixities were unchanged during the test series.

The following KTO joints were analyzed to evaluate elastic behaviour and stress concentrations:

<table>
<thead>
<tr>
<th>Joint</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Basic joint</td>
</tr>
<tr>
<td>2, 1 &amp; 5</td>
<td>Variation of T brace wall thickness t₂</td>
</tr>
<tr>
<td>3, 1 &amp; 4</td>
<td>Variation of T brace diameter d₂</td>
</tr>
<tr>
<td>3 &amp; 6</td>
<td>Variation in diameter ratios d₁ and d₂</td>
</tr>
<tr>
<td>1, 7 &amp; 9</td>
<td>Variation in gap g between braces</td>
</tr>
<tr>
<td>1 &amp; 10</td>
<td>Variation of chord thickness</td>
</tr>
<tr>
<td>1 &amp; 11</td>
<td>Variation of K brace thickness</td>
</tr>
</tbody>
</table>

These variations are shown in figure 2 and the detailed geometries are shown in table 1.

<table>
<thead>
<tr>
<th>KTO</th>
<th>D</th>
<th>T</th>
<th>d₁</th>
<th>t₁</th>
<th>d₂</th>
<th>t₂</th>
<th>g</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1500</td>
<td>62.5</td>
<td>1000</td>
<td>40</td>
<td>800</td>
<td>30</td>
<td>200</td>
</tr>
<tr>
<td>2</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>40</td>
<td>*</td>
</tr>
<tr>
<td>3</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>4</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>900</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>5</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>20</td>
<td>*</td>
</tr>
<tr>
<td>6</td>
<td>*</td>
<td>*</td>
<td>800</td>
<td>*</td>
<td>600</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>7</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>120</td>
</tr>
<tr>
<td>9</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>60</td>
</tr>
<tr>
<td>10</td>
<td>*</td>
<td>55</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>11</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>30</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

Table 1 : Geometry of analyzed joint

NOTE : * means identical to joint 1
Two additional non-overlapped joints were analyzed for comparative purposes:

- K joint with all dimensions being identical to KTO joint 1 with the T brace removed.
- KT joint with gap between each brace being 200 mm. All other dimensions were equal to joint 1.

The purpose of the analysis of these two joints was to check the validity of existing K and KT equations for non-overlapped joints. It was also envisaged that any parametric formulas developed for overlapped KT joints would be based on existing parametric formulas for K or KT joints.

Six load cases were used in the analyses. These are shown in figure 3.

**FINITE ELEMENT PROGRAM TUJAP**

The finite element program TUJAP has been used in the joint analysis in this project.

The TUJAP program uses linear extrapolation of principal stresses from the Gaussian grid stress points to the weld toe at the element surface. The width of the first element row at the weld is automatically generated to give pre-determined locations of the extrapolation points.

The program was first calibrated with respect to element density. This was done analyzing simple T joints and comparing them to existing literature.

The program was then calibrated using two overlapped joints. The first had brace angles of 90° and 45° and was tested by Wimpey Laboratories (3). The second was a 60°/60° K joint tested by Veritec (4).

The K joint calibrations showed that the TUJAP program was able to give a good prediction of the stress concentrations.

The KTO joints analysis models were held fixed in one end of the chord, while the other end of the chord were free in axial direction and held in the two transverse directions.

**RESULTS FROM THE ANALYSIS**

In order to describe the overlapped KT joints in a parametric manner, it was necessary to introduce two new parameters. The new parameters describe the relationship between the diameters and wall thicknesses in the K and T braces.
The new parameters are defined as:

\[ \omega = \frac{d_2}{d_1} = \text{relationship between } T\text{ brace diameter } d_2 \text{ and } K\text{ brace diameter } d_1 \]

\[ \rho = \frac{t_2}{t_1} = \text{relationship between } T\text{ brace wall thickness } t_2 \text{ and } K\text{ brace wall thickness } t_1 \]

The presence of the T brace modify the stresses of the KTO joint compared to a similar K joint. The T brace does not, however, change the behaviour completely, and the similarity in behaviour to K joints are clear.

The effect of the T brace is to transfer forces between the K braces thereby reducing the force transfer in the chord. The stronger the T brace, the more force is transferred, and the stresses are thereby reduced.

The above will be further discussed in the next chapters.

The stresses in the chord on the inside of the K braces are always less than stresses in comparable locations on the chord on the outside of the K braces or in the K braces themselves. These stresses are therefore ignored in the following.

The same was true for the stresses in the areas on the K braces inside the T brace. The stresses in these areas are therefore also ignored.

**Balanced axial load in the K braces, load case 1**

The KTO joints subject to balanced axial load in the K-braces exhibit, in general, rather uniform and even stress concentrations. A typical example of this is joint no 1, where the difference between the maximum stress concentrations in the chord and braces were only some 12%.

There are few stress peaks in the joints. The stress concentrations are in general fairly uniformly distributed along the inter-member connections. A typical example of this is the braces to chord weld on the chord side of joint no 1, shown in fig.4.

The figure shows the principal stresses extrapolated to the weld toe. The uniform stress area extends some 30° along the chord/K-brace intersection.

The uniformity of stress concentrations are important for crack development and growth. It can be expected that a fatigue crack occurring in a uniform stress field will develop under a larger area and grow faster than for a peaked stress concentration of the same magnitude. The crack initiation time is, however, assumed to be dependant on the magnitude of the SCF only.

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Location of stress concentrations, load case 1.

The hot spot on the chord outside is, as shown in figure 4, not a clearly defined point. It covers some 30° of the braces at the saddle area.

The area on the chord outside the K braces inside the T brace is loaded due to the tension/compression action of the K braces. The highest stress concentrations occur at the crown points of the K braces.

The stresses on the outside of the chord are equal or in some cases even higher than the stresses in the chord outside.

There are also some equally high stresses on the inside of the chord in this area. The inside of the chord, however, should not have any circumferential welds with weld notch effects in this area, and the stresses inside the chord can therefore be ignored.

The hot spots for the K braces outside the T brace will generally be at the saddle to the chord. For some joints, however, the stresses at the crown with the T brace will be similar to the stresses at the saddle. This occurs for relatively thick T braces or T braces with relatively large diameter compared to the K braces.

The hot spots in the T brace are located in the saddle area of the weld between the K brace and the T brace. This shows that the K braces act as chord for the T brace. The saddle of the T brace to the chord does not have any significant stress since only the K braces are loaded in this load case.

The areas where the wall of the chord meets both the wall of the K brace and T brace are quite stiff since the tube walls stiffen each other. The stresses in this area due to brace loading are therefore small.

Stress concentration factors for KTO joints

As discussed earlier, the stresses in a KTO joint are similar to the stresses in a K joint, where the T brace modifies but does not change completely the stress pattern in the joint.

A KT joint on the other hand behaves quite differently due to the large separation of the K braces. If the T brace in this type of joint is ignored, the K braces will in reality act as separate Y braces.

The above was confirmed by the results of the K and KT joints described earlier that were analyzed for the purpose of the above comparison.
Due to the similarities between the KTO and K joints, all comparisons between the KTO joints and the development of parametric formulas for the stress concentrations in KTO joints are based on K joints with identical K brace and chord properties as the KTO joints.

The stresses obtained in the K joints from the FEM analysis have been compared to values obtained from the parametric equations by Efthymiou and Durkin (2), Kuang et al. (5) and Wordsworth (6).

The comparison showed that the Efthymiou equations gave the best results compared to the TUJAP analysis. Based on this and other comparisons between parametric equations, the Efthymiou equations were used as basis for the parametric equations for the KTO joints. The KTO equations are developed as modifications to the Efthymiou equations.

Stresses in the KTO joints for Balanced Axial Loading

The stress concentration factors found for the chord saddle in the analyzed KTO joints are between 9% and 24% smaller than values obtained from the Efthymiou equations for similar K joints. This reduction reflects the effect of the T brace.

The stress concentrations vary between 2.05 and 3.19 for the 10 analyzed KTO joints.

The stresses in the chord inside the T brace are in general less than for the chord outside the braces. Exceptions are when the T brace diameter is less than approximately 0.75 times the K brace diameter, i.e. when \( \rho < 0.75 \) or when the T brace is of relatively high diameter, i.e. when typically \( \omega > 0.8 \). The above transitions will be dependent on the overall geometry of the joint.

As with the chord inside stress values, the stresses in the K braces are in general less than for the chord outside. There are, however, exceptions to this. Small gaps and high stiffness in the K to T brace connection give higher K brace SCF's than in the chord outside.

The stresses in the T brace are, except for thin T brace wall thicknesses, less than the stress concentrations in the chord and in the K brace. To avoid the T brace being governing for the joint, it is recommended to use T brace thicknesses giving \( \rho > 0.7 \).

The T brace in this load case is unloaded. The stresses arise therefore from K-brace deflections and load transfer. A small \( \rho \) value means that the T brace is less able to withstand these loads.
Parametric equations for the stress components in the discussed areas are included in appendix 1.

BALANCED AXIAL LOADING IN ALL THREE BRACES

Load cases 2, 3 and 4 were run to evaluate the effect of varying load in the T brace. The loads were always kept balanced. Balanced in this context means that the sum of the components of the forces normal to the chord are zero.

The four load cases are shown in table 2.

<table>
<thead>
<tr>
<th>Load</th>
<th>Brace 1 (K)</th>
<th>Brace 2 (K)</th>
<th>Brace 3 (T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LS1</td>
<td>$P_1 = -1.00P_2$</td>
<td>$P_2 = \text{nominal}$</td>
<td>$P_3 = 0$</td>
</tr>
<tr>
<td>LS2</td>
<td>$P_1 = -0.88P_2$</td>
<td>$P_2 = \text{nominal}$</td>
<td>$P_3 = 0.10P_1$</td>
</tr>
<tr>
<td>LS3</td>
<td>$P_1 = -0.59P_2$</td>
<td>$P_2 = \text{nominal}$</td>
<td>$P_3 = 0.50P_1$</td>
</tr>
<tr>
<td>LS4</td>
<td>$P_1 = -0.42P_2$</td>
<td>$P_2 = \text{nominal}$</td>
<td>$P_3 = 1.00P_1$</td>
</tr>
</tbody>
</table>

Table 2: Load cases for balanced axial load

The KTO joints with balanced load in all three braces behave in principle in a fairly similar way to the case with load only in the K braces. Since three separate loads are applied, the stress field becomes more complicated. This has a tendency to make the stress concentrations more peaked.

The hot spot locations will be similar to the locations found for loading in the K-braces only.

The max SCF in the cord outside decreases as the loads in the braces are more evenly distributed. The decreases in SCF are shown in figure 5.

Similar decreases were obtained for the chord inside the T brace.

The T brace gets an increase in loading for the load cases with loads in all three braces. The stress variations in these load cases are therefore dependent on the geometry of the T brace itself. A small diameter T brace will actually see an increase in stress concentrations since the increase in brace stresses due to increased load is bigger than the reduction in the stresses due to reduced deflections.

Since the reduction in stresses for the joints appears to be relatively insensitive to joint geometry, modifiers to the parametric equations for loads in the K braces have been developed as shown in the appendix.

Provided that the T brace is kept relatively strong, the formulas for loading only in the K braces can be conservatively used if the load split between the braces are unknown.
UNBALANCED IN-PLANE BENDING MOMENTS

The unbalanced in-plane bending moments were applied to give equal stresses in all three braces. Unbalanced means that all moments acted in the same direction.

For non-overlapped joints, the behaviour and hence stresses are nearly independent on the number of braces and gap values. This is also true for the far crown areas of the KTO joints.

In the near crown area the overlapping T brace will stop the K-brace deflections. The near crown areas will therefore act as centres of rotation with relatively small deflections. The main stress concentrations will therefore be found in the far crown areas.

The hot spot locations for the chord outside and K braces are always in the far crown locations. For the chord inside the T brace the hot spot locations are located at the near crown of the K braces. The hot spots for the T brace are located some 20° away from the crown point to the K brace.

Hot spot locations for the unbalanced in-plane bending moments are independent of joint geometry variations.

The similarity of KTO joint behaviour to K and Y joint behaviour is also reflected in the stress concentration factors.

The Efthymiou, Wordsworth and Kuang parametric equations have also been evaluated for in-plane bending. The Efthymiou Y-joint equations were again found to give the best values when compared to the analyzed test joints. These equations have therefore been used as a basis in the following and also for developing specific modifiers for KTO joints for in-plane bending.

For stresses in the chord outside, there are some small variations with varying geometry of the T brace and the gap between the K braces. These variations are, however, so small that they can be ignored.

The obtained SCF values for the KTO joints also correspond well with the test K joint, which again correspond well with the Efthymiou Y-joint equations. These equations can therefore be used for determining the SCF of the chord outside in the KTO joints.

The stresses in the chord inside the T brace are small enough not to warrant a detailed investigation. Within the analyzed KTO joint series the SCF in this part would vary between 0.35 and 0.65 times the chord outside SCF. As a simplified guideline, the chord inside SCF can therefore be assumed to be 0.5 times the chord outside SCF.
The K brace SCF's obtained in the analyses correspond well with values obtained by the Efthymiou equation. There are some differences for variations in chord and K brace wall thickness, but the KTO values are less than or equal to the values from the Efthymiou equation. The Efthymiou Y-joint brace equation can hence also be used for KTO joints.

The stresses in the T brace for in-plane bending are considerably less than the stresses in the chord outside and the K braces.

On average the SCF in the T brace is 0.67 times the SCF in the K brace. This simplified expression gives an error of +12% to -9% for the KTO test joints.

### UNBALANCED OUT-OF-PLANE BENDING IN ALL BRACES

All three braces were loaded with moments giving equal stresses in the braces. The moments were unbalanced, i.e. acting in the same direction out of the joint plane.

The hot spots for this load case all lie in the saddle position of the joint K brace/chord or T brace/chord weld. The joint behaviour for the KTO joints are very similar to T joints for out-of-plane bending. The majority of the load resistance from the chord comes from the local chord wall bending over an approximately 30° sector of the brace footprint in the saddle areas. The stresses are, however, somewhat more peaked than for the axially loaded KTO joints.

The main difference between the KTO joints and the T joint is the amount of loading and relative width of load application from the braces to the chord. The KTO joints have, due to the overlap, a relatively short length of chord for load transfer compared to the T brace. This will result in higher stress concentration values. The KTO joints are in terms of joint behaviour more similar to K joints with added effects of the T brace than to non-overlapped KT joints. This is evident when the relative length of the braces along the chord is compared. Stress concentration formulas for KTO joints are therefore again based on K joint considerations.

Efthymiou and Wordsworth have both published equations for K joints for out of plane bending moments. Both equations assume that the stresses in the K joint can be found from single member SCF increased by carry-over factors due to the effect of the other brace.

As found previously, the Efthymiou equations gave also the closest conformity to the analyzed K joint for out-of-plane bending. These equations were therefore utilized as a basis for KTO parametric equations.
SUMMARY AND CONCLUSIONS

Ten KTO joints have been analyzed. The emphasis was put on the effect of variations in the T brace diameter and wall thickness. Variation in K brace diameter and thickness, gap and chord thickness were also investigated.

Due to the added geometric complexity of KTO joints, the following two new parameters were introduced in the developed parametric equations:

\[ \omega = \frac{T \text{ brace diameter}}{K \text{ brace diameter}} \]
\[ \rho = \frac{T \text{ brace thickness}}{K \text{ brace thickness}} \]

Parametric formulas have been developed for the KTO joints for axial load and in-plane and out-of-plane bending moments. These formulas are given as modifiers to the existing Efthymiou equations for K joints.

The following conclusions can be made regarding joint behaviour:

**Balanced axial loads:**
- The stress patterns for balanced axial loads are fairly even with few stress peaks.
- The stress in a KTO joint are always less than the stress in a comparable K joint.
- Significant reductions in SCF can be obtained by making a KT joint overlapped.
- High stress can occur in the T brace if \( \rho < 0.75 \).

**Unbalanced in-plane bending**
- The majority of the stress concentrations are found in the far crown areas.
- The SCF in the chord outside is independent of the T brace geometry and gap size.
- The effect of overlapping is in general small for unbalanced in-plane bending.

**Unbalanced out-of-plane bending**
- The largest stress concentrations occur around the saddle areas of the joints.
- The SCF's in the T brace is small, but the stresses in the chord and K braces are higher than for comparable K joints.

The number of test joints have been somewhat limited compared to the total number of geometric variables in KTO joints. Care should therefore be taken when using the developed equations for joint geometries largely different to the geometries analyzed in this work.
REFERENCES


Figure 1. Overlapped KT (KTO) joint
Figure 2 Variation of parameters in analyzed joints

<table>
<thead>
<tr>
<th>LOAD</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>LS 1</td>
<td>P1, P3 = NOM, P2 = 0</td>
</tr>
<tr>
<td>LS 2</td>
<td>P2 = NOM, P3 = 0.1P2</td>
</tr>
<tr>
<td>LS 3</td>
<td>P2 = NOM, P3 = 0.5P</td>
</tr>
<tr>
<td>LS 4</td>
<td>P2 = NOM, P1 = P3</td>
</tr>
<tr>
<td>LS 6</td>
<td>M1 = NOM, M2 = NOM, M3 = NOM</td>
</tr>
<tr>
<td>LS 8</td>
<td>M1 = NOM, M2 = NOM, M3 = NOM</td>
</tr>
</tbody>
</table>

Figure 3 Load cases
Figure 4. Stress concentrations along the chord/braces weld

Figure 5. Decrease in SCF according to brace load distribution for the saddle of the chord
APPENDIX - FORMULAS FOR SCF IN KTO JOINTS

Balanced axial load in 2 K-braces

Chord outside (BA2-Co):

\[
(\text{Eftymiou equation K1 })(1-0.3\omega)(1.4-0.63\rho+0.4\rho^{3.1})
\]
\[
(0.68-0.5\beta)(1-(1-\tau)^{5})
\]

Chord inside (BA2-Ci):

\[(BA2-Co)(0.42+0.72\omega)(1.19-0.25\rho)(0.8+1.5\zeta)\]

K brace (BA2-K):

\[
(\text{Eftymiou equation K2})(1.2-0.27\omega)(1+0.06\rho^{4.5})
\]

T brace (BA2-T):

\[
0.05(BA2-K)(0.78-(\text{ABS}(0.78-\omega))^{1.15})(1.54-\rho)
\]
\[
(0.74+\zeta^{0.09/\zeta})(0.67+\beta)(37-\gamma)
\]

Balanced load in all 3 braces \( P=P_1/P_2 \)

Chord outside (BA3-Co):

\[(BA2-Co)(1-0.18\rho)(1+(0.13-\zeta)2\rho)\]

Chord inside (BA3-Ci):

\[(BA2-Ci)(1-0.24\rho)(0.7-0.45\rho)\]

Unbalanced in-plane bending in all three braces

Chord outside (IPB-Co):

\[
\text{Eftymiou equation T8}
\]

Chord inside (IPB-Ci):

\[
0.5(\text{Eftymiou equation T8})
\]

K brace (IPB-K):

\[
(\text{Eftymiou equation K9})(1.3-0.6\tau)
\]

T brace (IPB-T):

\[
0.67(\text{IPB-K})
\]

Unbalanced out-of-plane bending in all 3 braces

Chord outside (OPB-Co):

\[
(\text{Eftymiou eqn. K14})(1+\omega^{3}\rho)(1+0.2\rho)
\]

Chord inside (OPB-Ci):

\[
0.43(\text{OPB-Co})
\]

K brace (OPB-K):

\[
0.9(\text{Eftymiou eqn. K15})(1+\omega^{2}(2.1-\omega)\rho(1.8-\rho))
\]

T brace (OPB-T):

\[
(\text{OPB-Co})\omega^{2}(1.65-\rho)(2.2-2.1\tau)
\]

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Assessment of parametric equations for stress concentration factors in welded tubular joints.

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University of Nottingham, England

Summary

Five parametric equations have been used to predict the hot spot stresses in unreinforced T, Y, X and K joints subjected to axial loading, in-plane bending and out-of-plane bending. The shapes and loading modes chosen were those for which sufficient experimental information was available to define their shape and the hot spot stresses by linear extrapolation of strain measurements on steel models according to the UKOSRP/ECSC definition. 165 such test records were identified.

The equations published by Kuang et al (called K) Gibstein, (G), Wordsworth and Smedley as modified by UEG (U), Efthymiou and Durkin (E) and Hellier, Connolly and Dover (H) all predict axial loading of T joints well. Axial loading predictions of Y joints from K, U, E, H and axial loading of X and K joints from U and E are also good.

The available experimental data only show reliable performance in in-plane bending of T and Y joints by U and E and in out-of-plane bending of T and Y joints by U, E and H and overprediction of out-of-plane bending of K joints by U and E.

1. Introduction

The assessment is carried out by comparing SCF values predicted by the different parametric equations with values obtained from strain gauge measurements of steel models of joints. The stress concentrations are due to the shell bending of the chord and braces. Local notch effects due to fillet welds etc. are excluded by using 'hot spot stresses', obtained by linear extrapolation along lines in the outer surfaces of chord and braces. The parametric equations are empirical equations which have been devised by several authors to express the SCF for a particular type of joint and particular loading mode in terms of the shape of the joint. Unreinforced T, Y, X and non-overlapped K joints are considered here under axial forces, in-plane or out-of-plane bending applied to the braces.

In addition to the empirical formulae in design codes, a number of empirical equations have been developed from the results of strain gauge measurements and finite element calculations. Those published up to 1984 are reviewed in the UEG book, Design of tubular joints for offshore structures (1). Since then equations have been published by Efthymiou and Durkin (2) and Hellier, Connolly and Dover (5).

2. Shapes and loading modes

The shapes and loading modes used in the assessment are controlled by the availability of equations and valid data for shapes within the applicability limits of the equations. Table 1 gives these, together with the identifications of the equations used. Fig. 1 shows
the usual definitions of dimensions.

3. The equations

All stress concentration factors are defined as the appropriate hot spot stress divided by the nominal stress in the brace. The latter is the axial force divided by the cross-sectional area or the maximum value of the outer surface stress when the brace is bent as a beam in in-plane or out-of-plane bending.

The hot spot stress, as defined in UKOSRP (6) is obtained by linear extrapolation to the weld toe of the surface stresses, measured at two points on the chord and brace in the saddle and crown planes as shown in Fig. 2. Because these extrapolations are along the surfaces, there are two possible hot spot values at each saddle and crown position and some of the parametric equations predict separate values for them as well as distinguishing between saddle and crown.

All the equations are complicated functions of the shape parameter listed in the headings of Table 2. The ratios $\beta$, $\gamma$ and $\tau$ are needed for all types of joint, whereas the brace inclination $\theta$ is only needed for Y and K joints, the chord length factor $\alpha$ is only of importance when there are significant components of force perpendicular to the chord axis (T, Y and single force loading of K joints) and the gap parameter $\xi$ only exists in K joints.

The equations referred to by the letters K, G, E and H (see Table 1) were obtained from finite element calculations, whereas 'U' were derived from strain gauge measurements of acrylic models. These equations were originally published by Wordsworth and Smedley (7, 8) but were extended and modified in Ref. 1; this modified version is used here.

The equations are not reproduced here because they would not help in understanding this paper. K, G and U are on pages C49, C50 and C86 respectively in Ref. 1 and the corrected version of E is presented in Ref. 9 in 35 separate equations to cover the various loading modes of the four simple joints studied here.

The ranges of shapes for which the equations are deemed applicable are shown in Table 2.

4. The data

In addition to numerous private enquiries, three separate international systems were used in 1988 for computerised searches from 1983 onwards. It was deemed that the compilers of Ref. 1 had collected all useful data up to 1983. All the data will be available in Ref. 11

The numbers of 'suitable' test results for the different types and loading modes are shown in Table 3. Suitable in this context means that sufficient information was given to calculate hot spot stresses as defined above (6). As shown in Table 3, four 'unusual' results were discarded. Unusual was defined as giving a ratio greater than 2 or less than $1/2$ of the average of all the relevant predictions divided by the experimental value. They were discarded because they seemed suspect.
Only results from steel model joints were used. In some cases several tests were carried out on the same model. There were seven pairs of joints which had effectively identical shapes and loading modes; these were six T joints, three in axial loading, one in in-plane and two in out-of-plane bending and one K joint in single brace axial loading.

An overall impression of the shapes of the suitable joints is obtained from Table 4, which gives the numbers of joints in each shape range for the different types.

5. Method of evaluation

Where a parametric equation predicts several hot spot stresses the maximum value has been taken and divided by the experimental value. The predicted maximum may be at a different position to the measured maximum.

6. Results

Table 5 shows the hot spot stress ratios for the type-load combinations for which more than five 'suitable' experimental results existed which had shapes within the applicability range of at least one equation. The minimum and maximum ratios of predicted/measured values are presented as well as the mean and standard deviation. The distributions of the mean values are shown in Table 6. Table 7 shows the maxima and minima of combinations for which there are no more than five results.

7. Discussion

Although up to eight measured hot spot stresses were available for a joint, only the highest value was used in the assessment because that value is assumed to lead to failure of the joint and parametric equations are used in the prediction of joint failure.

T and Y joints were combined because there are few suitable tests of Y joints (see Table 3). This seemed reasonable because it is generally accepted that the components of brace loads which cause direct forces in the chord produce much smaller effects than those which cause wall bending. This is taken into account by sin $\theta$ terms in the appropriate equations.

Table 1 shows that Gibstein only produced equations for T joints. As Table 2 shows that their applicability is not greater than that of other equations, their usefulness appears limited now.

Only modified Wordsworth and Efthymiou equations are available for all the joints considered.

Table 4 has been included to show the shapes of the suitable joints on which the comparison were carried out. If a particular joint requires detailed study, it may be helpful to know whether similar shapes of the particular type are likely to have been included in this assessment; this could assist in deciding how much reliance should be placed on predictions for that particular joint.
Tables 5 and 6 give the results of the comparison for the combinations of type and loading for which more than 5 comparisons could be made. All the mean values of the hot spot stress ratio are greater than 1, i.e., they are conservative, but in most cases subtracting one standard deviation gives ratios less than unity. Table 6 can be used to decide on a multiplying factor for predicted values for a particular type-loading mode combination to reduce the probability of an underestimate to an acceptable level. The differences between the numbers of tests quoted in Table 3 and Table 5 or 7 for a particular type-loading mode combination give the number of joints whose shapes are outside the applicability range of the equation considered.

The standard deviations of hot spot stress ratios shown in Table 5 vary from 0.16 to 0.27, if the very high values 0.41 and 0.33 for predictions from 'U' equations for out-of-plane bending are excluded. These high values are associated with small values of $\beta$ which are excluded from the range of the 'E' equations (see Table 2). The large variations shown by the other values of $\sigma$ (and the min. and max. values) are attributed to unavoidable errors in the values of measured hot spot stress which have been demonstrated (10). Some of the strain at the position further from the welds are small and the strains nearer to the weld depend critically on the positioning and working length of the strain gauge because they are in regions of high strain gradient. For many of the steel models from which results were used, the distance $a \approx 5$ mm. This distance is measured from the toe of (usually undressed) fillet welds. In spite of Irvine's investigation (6), the possibility of this strain being influenced by (inevitably highly variable) notch effects cannot be excluded.

Table 7 is included for completeness but the results can only give an indication because of the very small numbers of valid tests.

When comparing the predictions 'U' and 'E' in Table 5 it is seen that there is no significant difference between the mean values of the ratios and the standard deviations. Table 2 shows that their applicability ranges are also very similar; 'U' applies for smaller brace diameter ratios and chord length ratios, and thinner chords whereas 'E' permits thicker chords, thinner braces and smaller brace angles. There are no limits on gap length in 'U'.

8. Conclusions

The mean values in Table 5 suggest that the modified Wordsworth - Smedley equations (U) and the Efthymiou - Durkin equations (E) give good predictions for all joints under axial loading and for T and Y joints in both types of bending; they overpredict the hot spot stress ratios of K joints in out-of-plane bending. The other equations are also satisfactory for T joints in axial loading.

Acknowledgements

This investigation was part of the 1987-89 cohesive fatigue programme organised by University College, London.
References

1. UEG 'Design of tubular joints for offshore structures.' Vol 2, 1985, Section C.


Table 1 Availability of Equations

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Abbreviations:  
K = Kuang et al  
G = Gibstein  
U = Wordsworth and Smedley as modified by UEG  
E = Efthymiou and Durkin  
H = Hellier, Connolly and Dover

Table 2 Applicability ranges of equations

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Notes:  
na = not applicable,  
ns = not stated

Table 3 Numbers of suitable tests

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Notes:  
1. Numbers in parenthesis are numbers available after unusual results have been discarded.
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$L$ is between restraints or points of contraflexure

$\beta = d/D$, $\gamma = D/2T$, $\tau = t/T$, $\alpha = 2L/D$, $\varsigma = g/D$

Fig. 1 Notation

225
Table 5. Hot spot stress ratios

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Note: 'single' and 'both' refer to the braces which were loaded
σ refers to standard deviation
### Table 6. Distribution of hot spot stress ratios

Numbers of ratios are listed below the minimum value of the stated intervals.

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**Note:** * 1 ratio above 2.0
Table 7. Small sample hot spot stress ratios

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<td>3</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td>G</td>
<td>3</td>
<td>0.91</td>
</tr>
<tr>
<td>X</td>
<td>IPB</td>
<td>U</td>
<td>3</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E</td>
<td>5</td>
<td>0.92</td>
</tr>
<tr>
<td>X</td>
<td>OOPB</td>
<td>U</td>
<td>3</td>
<td>0.83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E</td>
<td>2</td>
<td>0.70</td>
</tr>
</tbody>
</table>

---

**Fig. 2** Positions of strain measurement to determine hot spot stresses (from Ref 1)

\[ a = 0.2(r/\ell)^{1/2}, \text{ but not smaller than } 4 \text{ mm} \]
SUMMARY

The fatigue performance of steel tubular joints of offshore structures is of primary importance in deciding the integrity of these structures. The fatigue analysis of these structures requires evaluation of stress concentration factors (SCFs) and fatigue strength of the tubular joints. The radial flexibility of the chord is the main parameter affecting the stress concentration around the intersection of chord and brace tubes. Among the different methods available to reduce stress concentration at the intersection, provision of internal ring stiffeners is often preferred for its advantages. Tests were conducted by the authors on stiffened and unstiffened tubular Y joints to investigate the effect of internal ring stiffeners on SCF. The experimental values of SCF are compared with the parametric formulae recommended by the Underwater Engineering Group (UEG) for stiffened and unstiffened tubular joints. The authors have also conducted fatigue tests on stiffened tubular joints under axial and moment loads. The experimental fatigue test results are compared with the recommendations made by the various coding authorities viz., DnV, API, UEG, and NPD. Results of the experimental investigations have been presented in this paper. The effect of ring stiffeners on the static and fatigue behaviour of tubular welded Y joints under axial and moment loads has been examined.

KEY WORDS

Fatigue; stress concentration; stress concentration factor; internal ring stiffeners; tubular joints; offshore structures

1. INTRODUCTION

Steel jacket type platforms used for the exploration of offshore oil and gas are composed of tubular members which are interconnected by welded joints. The complex intersections give rise to severe stress concentrations. Because of the inherent defects in welded connections and because of the high values of SCF at the intersections the severe fluctuations in the wave loading make the jacket platforms vulnerable to fatigue failure. In order to design the structure against fatigue failure, it is of prime
importance to predict the stress distribution around the intersection of chord and brace tubes, where the hot spots are located. Cracks are likely to initiate from these hot spots and propagate. In order to design the offshore structures for a longer fatigue life, it is essential to determine accurately the magnitude of stress concentration and also reduce it to a reasonable level. Stress concentrations can be reduced by providing internal ring stiffeners. Hence the fatigue life can be improved. Detailed investigations have been taken up by the authors on the static and fatigue behaviour of ring-stiffened steel tubular joints of offshore structures. In comparison to external gusset plates and external ring stiffeners, internal ring stiffeners do not attract additional wave forces and are less prone to corrosion.

Number of stiffeners, their location and dimensions are the three important parameters to be considered in the design of internally ring-stiffened joints.

Based on the earlier investigations conducted in Italy and reported by Pozzolini [4], three internal ring stiffeners are found to be the optimum number in the chord member, one at the centre of the chord, and the other two at the crown points of the intersection (brace faces). Based on the studies conducted by the authors, \( B = 0.2 \, D \) and \( t_s = 0.75 \, T \) would be the optimum values for the width and thickness of the internal rings [6]. The stiffened joints tested by the authors were provided with three internal rings. Figure 1 gives details of the test specimens. The dimensions of the specimens are also mentioned in Table 1.

2. Static tests

Fabrication of specimens

All the test specimens, for both static and fatigue tests, were fabricated by M/s. Mazagon Dock Ltd., Bombay, who are the leading fabricators of the offshore platforms in India. The specimens were fabricated adopting the same welding procedures followed for the fabrication of offshore platforms. The steel used conformed to API 5L grade B steel with a design yield strength of 240 MPa. Radiographic testing, ultrasonic pulse testing, and magnetic particle inspection were carried out on the welds.

Strain gauge instrumentation

In order to measure the hot spot stresses, the specimens were extensively instrumented with electrical resistance foil type strain gauges near the intersection of the chord and brace. Maximum hot spot stress in a tubular joint occurs at the weld toe of the intersection either in the chord or in the brace, depending on the geometry of the joint and loading condition. Recommendations of the European Coal and Steel Community (ECSC) were adopted for deciding the location of strain gauges. Each specimen was
instrumented with 80 rosettes of 2 mm gauge length near the intersection. The stiffened and unstiffened joints were subjected to all the three cases of loading conditions - axial loading of brace, in-plane bending of brace (IPB) and out-of-plane bending of brace (OPB). For determining the SCF, load was applied in stages and the corresponding strains were recorded. From the measured values of strains, principal stresses were determined. The values of hot spot stresses were obtained by extrapolation.

**Prediction of SCF for stiffened joints**

The experimental values of SCF for stiffened joints are given in Table 4. The experimental results were compared with the recommendations of UEG and the parametric formulae developed by the authors [6,8]. UEG has recommended the Wordsworth formulae with suitable modifications to allow for the increased stiffness of the chord in case of internally ring-stiffened joints. It recommends that the $\gamma$ ratio in the parametric formulae be replaced by a modified value as follows:

- **Axial**: $D/2T_e \times 0.70$ $0.30$
- **IPB**: $D/2T \times 0.15$ $0.85$
- **OPB**: $D/2T \times 0.30$

where $T_e$ is the effective chord wall thickness which would give the same moment of inertia as that of the chord wall plus stiffener. $T_e$ should be limited to a maximum of $2T$.

**Discussion of static test results**

As can be seen from Table 1, the stiffened and unstiffened joints had the same tubular dimensions. The stiffened joints were provided with three internal rings. In order to evaluate the effect of ring-stiffening and loading condition on stress concentration, one stiffened joint and one unstiffened joint were tested under each loading condition - axial loading of brace, IPB and OPB.

The experimental results of SCF for stiffened and unstiffened joints are given in Table 2. The percentage reduction in SCF due to stiffening is also indicated in the table. As can be seen from the table, a reduction of 45% was observed in the value of SCF under axial loading condition. It was also observed that the stress distribution around the intersection of chord and brace tubes at the weld toe was more uniform in the case of stiffened joints.

Under OPB, significant reduction of 67% in SCF was observed due to stiffening. Under IPB, a marginal reduction of 13% in SCF was observed. Thus, in the case of IPB, the reduction seems to be less significant, when compared with the other two loading conditions. Hence, it can be inferred that internal ring-stiffening is highly beneficial in the
case of axial loading of brace and OPB, and the same shows only marginal improvement in the case of IPB. This observation is in agreement with the earlier investigations carried out by Dharmavasan et al [2].

Table 3 gives the comparison of experimental values of SCF with the values of SCF calculated by using the parametric formulae recommended by Kuang, Gibstein and UEG.

In the case of axial loading of brace, the value of SCF based on the formula given by Gibstein is very close to the experimental value. In all the other cases, the analytical values are generally on the conservative side. In the loading case of OPB, the values of SCF based on the formulae given by Gibstein and UEG are very much on the higher side in comparison with the experimental value, showing a variation of 92% to 120% respectively.

Table 4 gives the comparison of experimental values of SCF with the values of SCF calculated by using the parametric formulae recommended by the authors and UEG for stiffened joints [6,8]. In comparison with the experimental values, the values based on the formulae given by UEG are on the conservative side for all the three cases of loading and a variation of 270% is found in the case of OPB. In the case of axial loading of brace and IPB, the values of SCF based on the formulae given by the authors are lower than the experimental values.

In the case of unstiffened joints, the lowest value of SCF was in the case of IPB. In the case of stiffened joints, the lowest value of SCF was in the case of OPB.

3. Fatigue tests

Fatigue life analysis

Two basic approaches are available for fatigue life assessment of structural components, namely, S-N curve approach and fracture mechanics approach. The first method which is currently in more general use is based on empirically derived relationship between applied stress ranges and fatigue life. The alternating stress, is taken as the nominal alternating stress multiplied by a stress concentration factor to account for local effects at the joints. The second method is based on linear elastic fracture mechanics which pre-supposes that a fatigue crack exists as a flaw in the structure and this crack grows with changes in stress. The crack growth is a function of the stress intensity factor, K, and is a material property that can be mathematically described. Current practice is to use the S-N approach for design and general assessment and the fracture mechanics approach for appraisal of existing joints with known defects and investigations on critical joints in a structure at the design stage.
The fatigue tests were conducted by using a highly sophisticated electro-hydraulic servo-controlled testing system. The fatigue loads were applied through an actuator of ±500 kN capacity and the loading was controlled by INSTRON 2180 desk top control console which is interfaced with HP 9000-310 computer system.

Three fatigue tests were conducted on stiffened steel tubular Y joints. These three stiffened joints were tested under different loading conditions, namely, axial loading of brace, IPB and OPB. All these fatigue tests were conducted in air under constant amplitude sinusoidal wave loading at a frequency of 2 Hz. The stress ratio R was kept at -1 for all the fatigue tests. In order to evaluate the effect of loading condition on fatigue life, all the three fatigue tests were conducted at a hot spot stress range of 204 MPa.

Crack depths were monitored during the fatigue tests at regular intervals by using a crack microgauge. Steel probes of 1 mm diameter were fixed at the weld toe of the intersection. These probes were connected to the ACM 3 multi-channel switching unit which in turn was connected to the U8 crack microgauge. The crack microgauge works under ACPD (Alternate Current Potential Drop) technique. Crack growths were monitored till the failure of the specimen.

In general, development of fatigue cracks may be classified into three stages: initiation, propagation, and failure.

\[ N_i : \text{Number of cycles to first visible cracking as detected by visual examination or NDT technique; considered as crack initiation life.} \]

\[ N_c : \text{Number of cycles at which full thickness crack occurs; considered as crack propagation life.} \]

\[ N_f : \text{Number of cycles to complete failure of the specimen.} \]

Discussion of fatigue test results

The crack initiation life \( (N_i) \) and the crack propagation life \( (N_c) \) were monitored for all the three specimens tested in fatigue for different loading conditions. In general, fatigue cracks initiate at the hot spots or at the defects, if any exist. The locations of initial fatigue cracks for all the three specimens were at the locations of maximum hot spot stresses at the weld toe. As anticipated, the fatigue cracks initiated in the chord member at the hot spots for the specimens tested under axial loading and OPB, whereas in the case of IPB, the first crack initiated in the weld toe of the brace. The crack propagation life, which is given by \( N_c \), is considered to denote the fatigue life for tubular joints. This is based on the assumption that the joint loses its stiffness considerably when the crack depth reaches the full thickness of the tubular member.
Table 5 gives the crack initiation and propagation lives for all the stiffened joints. For the joints tested, crack initiation period varied from 10% to 60% of the crack propagation period.

In the case of axial loading of brace and IPB, load was applied beyond the crack propagation life till complete failure to study the residual life. The final failure lives ($N_f$) for these two joints are also given in Table 5. The joints withstood 0.261 million and 0.820 million additional numbers of cycles respectively, even after the crack propagated through the thickness of the member. It can be seen from the table that the additional number of cycles taken to reach $N_f$ was quite appreciable. For the joint tested under OPB, the propagation life alone was 2.0 million cycles and it was observed that the crack growth rate was very slow. Therefore the test was terminated at $N_f$.

During the crack propagation phase, the internal rings acted as crack arrestors. Consequently, cracks did not continue to propagate along the weld toe, but propagated perpendicularly, to the areas of low local stresses.

For the stiffened Y joint tested under axial loading of brace, the crack initiation period was equal to 67% of the crack propagation period. This is in agreement with the tests carried out by Kerr et al [3].

The experimental fatigue lives were compared with the S-N curves recommended by the various coding authorities, namely, API, DnV, UEe and NPD. These S-N curves are mainly for unstiffened joints. It can be seen from Fig. 2 that the fatigue lives of the stiffened tubular joints tested lie above these S-N curves. Hence, these curves can be used conservatively for fatigue design of internally ring stiffened steel tubular joints also. Further, it can be observed from the figure, that for any given stress range, the stiffened joint takes considerable additional number of cycles in comparison to the corresponding stress range on any of the standard S-N curves.

4. Conclusions

* The static tests conducted on stiffened and unstiffened tubular Y joints have proved the efficiency of ring-stiffening in reducing SCF.

* Provision of internal ring stiffeners in the chord member at the intersection of chord and brace tubes results in more uniform stress distribution at the weld toe of the intersection.

* Ring-stiffening in the chord member is highly beneficial in the case of axial loading and out-of-plane bending loading conditions, whereas the improvement in in-plane bending loading condition is marginal.
Stiffened tubular joints have less stress concentration compared to unstiffened joints. Hence, for a given fatigue life, higher stress and therefore higher load, can be permitted in stiffened joints. Alternatively, for the same design load and hot spot stress, higher fatigue lives can be expected for the stiffened tubular joints.

Compared to axial and IPB loading conditions, crack growth rate for the joint tested under OPB seems to be slow during its propagation period.

The internal rings provided in the chord members act as crack arrestors during the propagation period and consequently, the cracks do not continue to propagate along the weld toe, but propagate perpendicularly arriving in an area of low local stress.

5. Acknowledgement

The authors thank Mr. N.V. Raman, Director, Structural Engineering Research Centre (SERC), Madras for his valuable support throughout the course of this investigation. This paper is published with the permission of the Director, SERC, Madras, India.

6. Symbols

- $B$: width of stiffener
- $d$: diameter of brace
- $D$: diameter of chord
- $N$: fatigue life
- $N_i$: number of cycles to crack initiation
- $N_f$: number of cycles to full thickness crack
- $N_r$: number of cycles to failure
- $S$: stress ratio
- $s$: stress range
- $t$: thickness of brace
- $t_e$: thickness of chord
- $T_e$: effective thickness of chord wall
- $T_s$: thickness of stiffener
- $\beta$: $d/D$; called as diameter ratio
- $\theta$: $D/2T$; indicates chord stiffness
- $\tau$: $t/T$; called as thickness ratio

7. References


<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test frequency</th>
<th>Loading</th>
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</thead>
<tbody>
<tr>
<td>S-Yu-1</td>
<td>Static</td>
<td>Axial</td>
</tr>
<tr>
<td>S-Yu-2</td>
<td>Static</td>
<td>IPB</td>
</tr>
<tr>
<td>S-Yu-3</td>
<td>Static</td>
<td>OPB</td>
</tr>
<tr>
<td>S-Ys-1</td>
<td>Static</td>
<td>Axial</td>
</tr>
<tr>
<td>S-Ys-2</td>
<td>Static</td>
<td>IPB</td>
</tr>
<tr>
<td>S-Ys-3</td>
<td>Static</td>
<td>OPB</td>
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<td>F-Ys-1</td>
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<tr>
<td>F-Ys-2</td>
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<td>IPB</td>
</tr>
<tr>
<td>F-Ys-3</td>
<td>2 Hz</td>
<td>OPB</td>
</tr>
</tbody>
</table>

D = 324 mm; T = 12 mm; d = 219 mm; t = 8.18 mm; \( \beta = 0.676 \); \( \gamma = 0.681 \); B = 75 mm; \( T_s = 12 \) mm; \( 8 = 60^\circ \); R = -1

TABLE 1. DETAILS OF STATIC AND FATIGUE TESTS
### TABLE 2. EXPERIMENTAL VALUES OF SCF FOR STIFFENED AND UNSTIFFENED JOINTS

<table>
<thead>
<tr>
<th>Type of loading</th>
<th>Unstiffened SCF</th>
<th>Stiffened SCF</th>
<th>Reduction in SCF due to stiffening</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Axial</td>
<td>S-Yu-1 6.83</td>
<td>S-Ys-1 3.73</td>
<td>45.38%</td>
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<tr>
<td>2. IPB</td>
<td>S-Yu-2 2.15</td>
<td>S-Ys-2 1.86</td>
<td>13.48%</td>
</tr>
<tr>
<td>3. OPB</td>
<td>S-Yu-3 3.15</td>
<td>S-Ys-3 1.04</td>
<td>66.98%</td>
</tr>
</tbody>
</table>

### TABLE 3. EXPERIMENTAL AND ANALYTICAL VALUES OF SCF FOR UNSTIFFENED JOINTS

<table>
<thead>
<tr>
<th>Type of loading</th>
<th>SCF</th>
<th>Percentage variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial</td>
<td>6.83</td>
<td>7.73</td>
</tr>
<tr>
<td>IPB</td>
<td>2.15</td>
<td>2.30</td>
</tr>
<tr>
<td>OPB</td>
<td>3.15</td>
<td>4.94</td>
</tr>
</tbody>
</table>

### TABLE 4. EXPERIMENTAL AND ANALYTICAL VALUES OF SCF FOR STIFFENED JOINTS

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Hot spot stress range (MPa)</th>
<th>No. of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. F-Ys-1</td>
<td>1.067 x 10</td>
<td>1.516 x 10</td>
</tr>
<tr>
<td>2. F-Ys-2</td>
<td>0.152 x 10</td>
<td>0.600 x 10</td>
</tr>
<tr>
<td>3. F-Ys-3</td>
<td>0.220 x 10</td>
<td>2.000 x 10</td>
</tr>
</tbody>
</table>

### TABLE 5. RESULTS OF FATIGUE TESTS ON STIFFENED JOINTS
FIG. 1 DETAILS OF TEST SPECIMENS

FIG. 2 COMPARISON OF EXPERIMENTAL RESULTS WITH FAMILY OF S-N CURVES
The project "Stress Concentration Factors for Tubular Complex Joints" was a two phase industry multi-sponsored project, completed in 1988 and now outside the restrictions of the confidentiality period.

The primary objective of this project was to obtain improved methods of dealing with fatigue aspects of complex joints and loadings, with the main emphasis being on single plane ring-stiffened tubular joints. The test programme consisted of 262 acrylic ring-stiffened T/Y, X and K joint specimens, with verification of sample measured stress concentration factors (SCFs) from independent finite element (FE) analyses and from steel joints tested in the UKOSRP II project.

SCFs were measured around the brace/chord intersection using both linear and non-linear extrapolation techniques and on the ring-stiffener itself. These measured SCF values resulted in the first comprehensive set of parametric equations to determine SCFs in ring-stiffened tubular joints. These equations, in association with guidance for their application, are presented as an appendix to this paper.

1. Introduction

The determination of reliable SCFs is fundamental to the hot-spot stress S-N approach to nodal joint fatigue life estimation. Consequently, over the last 25 years, considerable work has been performed to improve the reliability of SCFs for simple unstiffened, single plane, tubular joints. The prohibitive cost of full scale steel nodes has led to the development of acrylic model, photoelastic model, and FE analysis techniques to replicate results from steel joints, with parametric equations derived using these experimental SCF results.

In 1983, LR reviewed SCFs and loadings in complex tubular joints on behalf of the UK Department of Energy (DEn) [1]. This study emphasised the shortage of SCF data for ring-stiffened joints, overlapping joints, multi-planar joints and grouted joints. For ring-stiffened joints, it was proposed that the chord thickness could be modified to account for the increase in the moment of inertia due to ring-stiffening of the chord. The unstiffened parametric equations were then used with the $y$ parameter modified accordingly. However, it was noted that no account was made of ring spacing or stresses in the ring-stiffeners, and furthermore initial tests performed by LR showed that in addition to the ring inertia, the thickness of the ring was also a significant factor in determining SCFs.

As a result of the findings in this report, LR initiated an industry multi-sponsored project covering SCFs in tubular complex joints with the main emphasis being upon ring-stiffened single plane tubular joints. The results of this project were summarised by Smedley and Fisher [2].
2. Experimental techniques

In this project, T/Y, X, and K ring-stiffened joint acrylic models were tested across a comprehensive range of $\tau$, $\beta$, $\gamma$ parameters and stiffener configurations, see figure 1. Strain gauges were located around the brace/chord intersection in accordance with DEn recommended practice [3] with both linear and non-linear extrapolation techniques employed. Generally, strain gauge rosettes were placed at the saddle, crown, and at three interim locations, with one set of strain gauge rosettes corresponding to the brace/ring intersection (BRI). Strain gauges were also placed on the stiffener inside edge (typically at 10° intervals), and for some joints strain gauges were located on the ring web beneath the BRI. Details of the experimental procedure and the potential degree of experimental error have been described by Smedley and Fisher [2].

To assess the degree of correlation between SCFs measured using acrylic models and other experimental methods, independent FE analyses were performed by Koninklijke/Shell Exploratie en Produktie Laboratorium (KSEPL) using 3D shell elements and Wimpey Offshore using quadrilateral semi-loof thin shell finite elements. In addition, four steel joints tested in the UKOSRP II project were modelled in both acrylic and FE to verify these techniques with SCFs from steel complex tubular joints.

It was shown [2] that with the exception of the BRI region, there was a high degree of correlation between measured SCFs irrespective of the experimental technique employed. Investigation of the SCF distribution around the brace/chord intersection indicated that for the flanged ring configuration in particular, there is a very rapid change in stress around the BRI, see figure 2.

3. Summary of measured SCF results

Sixty five unstiffened joints and 258 ring-stiffened joints were tested, with both plate and flanged stiffeners investigated. Ring-stiffeners were typically located beneath the saddle, the crown, and separated by 80% of the brace footprint. An example of the measured unstiffened and ring-stiffened SCFs from a T joint specimen with mid-range parametric values is given in table 1. Furthermore, 4 T and K joint specimens incorporating longitudinal stiffeners in addition to ring-stiffeners were tested.

It can be seen, in table 1, that there is considerable relief in SCF at the saddle location due to the addition of ring-stiffeners. Only $\beta=1$ X and K joints showed little or no benefit from the inclusion of ring-stiffeners at the saddle. Under axial load and OPB, a centrally placed single ring-stiffener will give maximum relief at the chord saddle but will have a smaller effect on the brace saddle SCF. Consequently, the brace saddle SCF is likely to become critical with such a ring configuration present, whereas with two symmetrically placed ring-stiffeners the chord saddle SCF will tend to remain critical.

Ring-stiffeners reduce stress in the circumferential direction but have little influence in the longitudinal direction, consequently, at the chord crown location under axial loading, the SCF is almost unaltered by ring-stiffening the chord. It was noted that for $\beta \geq 0.5$ joints with heavy stiffeners, some load was attracted to the crown region and the crown SCF increased accordingly, particularly on the braceside. Therefore, the addition of ring-stiffeners to a K or KT joint where axial load is the
dominant loadcase will not necessarily lead to a significant reduction in hot-spot stress, since the unstiffened saddle and crown SCFs are generally of a similar magnitude. Under IPB small consistent reductions of the chord crown SCF occur when stiffeners are placed in proximity to the crown location, while at the brace crown there is generally less effect on the measured SCF. The addition of longitudinal stiffeners reduced the SCF at the crown by around 20% under axial load, and by around 30% under IPB.

On the chordside at the BRI, where fatigue failures occurred in the UKOSRP II flanged ring-stiffened joints [4], most joints tested show a local decrease in stress. However, it was found that around 50% of joints tested have the maximum braceside stress on the brace at the BRI, where a local increase in stress occurs, see figure 2. Analysis of the joints where the BRI is the location of maximum stress in the joint did not show any single geometric factor as the consistent cause. Regarding the ring web beneath the BRI where the ring is attached to the chord inner wall, none of the acrylic ring-stiffened joints tested and strain gauged in this region showed the ring web to be the location of maximum stress.

Ring stresses were measured at various arc positions around one quadrant of the ring inner edge. The maximum stress level recorded was often on the ring, especially when no flange was present. For axial load and IPB the maximum compressive or tensile stresses on the ring inner edge occurred around the 0° and near to the 90° positions from TDC, whereas for OPB the maximum stress occurred around the BRI position, see figure 3. In offshore structures, restrictions on inspecting the ring-stiffeners make it undesirable to have the minimum joint life in the ring-stiffener.

4. The derivation of parametric SCF equations for ring-stiffened joints

Measured SCF results from the 262 stiffened joints in the complex joints project were combined with results from 66 ring-stiffened joints tested by LR in 1983 and processed using a multivariable least squares curve fitting package developed for this project. The nominal brace stress used for the bending modes was the extreme fibre nominal bending stress.

The effects of differences in linear/non-linear extrapolation, lack of a weld fillet, and experimental errors around the brace/chord intersection, were considerably reduced by analysing the alleviation in SCF due to ring-stiffening rather than the SCF itself. (ie. If the absence of a weld fillet increases the chordside SCF on both an unstiffened joint and a stiffened joint by 5%, then the chordside SCF ratio is unaffected by the lack of a weld fillet). Thus, the measured stiffened joint SCF was divided by the equivalent measured unstiffened joint SCF, to give an 'SCF ratio', ie the stiffened joint SCF = unstiffened joint SCF x SCF ratio. For the stiffener itself, the maximum stress measured from strain gauges placed on the ring inner edge was divided by the nominal brace stress to give an SCF.

Initial examination of the SCF data showed that the SCF ratio at the saddle locations was generally inversely proportional to the ring thickness (Rtau), the ring inertia (K2), and to some function of the number of rings and their separation (Nre).

Thus, the equation initially had the form : \[ \text{SCF ratio} = \frac{f(t, R, \theta)}{Nre} \times \left[ \frac{1}{Rtau} + \frac{1}{K_2} \right] \]
The least squares curve fitting program was then employed to optimise the coefficients and exponents of the given parameters and wherever possible these numeric values and the form of the equations were simplified. Furthermore, consideration was given to keeping a consistent type of equation between a T/Y joint, a single brace loaded K joint and a K joint with both braces loaded. This philosophy led to an increase in the scatter between the measured data and the predicted results for individual equations. However, this increase in scatter was insignificant relative to the potential experimental error in the measured SCF data itself.

Factors of safety applied to these mean fit equations have been based on the standard deviation of the data. These factors give an expectation of underprediction of 10% for the SCF ratios at the saddle and 2.5% for the ring inner edge. The SCF ratios need be less conservative in design, since unstiffened joint SCF equations to be used in conjunction with the SCF ratios are themselves conservative to some degree. For the crown locations, characteristic expressions underpredicting around 5% of joints have been derived to cover the crown and interim BRI region.

These parametric equations are based on acrylic models with relatively long chord lengths. Unstiffened and ring-stiffened steel joint tests usually have short chord lengths and heavy chord end diaphragms. Therefore, unstiffened specimens restrained at the chord ends are quasi-stiffened joints, while for ring-stiffened joints chord end diaphragms have little additional influence. When deriving SCF ratios from test specimens, short chord effects on the unstiffened joint as described by Efthymiou [5] should be removed from the calculation. When deriving stiffened SCFs in design the unstiffened parametric equation should exclude any short chord correction factors.

Initial comparisons between the LR ring-stiffened SCF equations used in conjunction with the Efthymiou [5] unstiffened SCF equations, and steel models and FE analyses, show good correlation on the chordside and for the ring-stiffener inner edge while the predicted SCFs are often too conservative relative to the measured SCFs on the braceside.

5. Fatigue performance of ring-stiffened joints

In producing SCF equations for ring-stiffened joints, consideration has also been given to S-N curves. The limited number of ring-stiffened joint fatigue test results available consist of configurations with thin chord walls (16mm-20mm) and exhibited fatigue failures either on the brace/chord intersection away from the BRI location; on the brace/chord intersection at the BRI location; or in the ring-stiffeners themselves. As might be expected, fatigue failures on the brace/chord intersection away from the BRI are generally covered by the DEn mean 'T' curve, which is a 'hot-spot' curve based on simple tubular joint tests. For the three failures on the brace/chord intersection at the BRI [4] which all occurred in the chord, the fatigue lives are significantly lower than would be obtained from the mean 'T' curve. Given the cruciform type feature at the BRI an 'F' class curve may be more appropriate than the 'T' curve. For the ring-stiffeners themselves, an appropriate S-N curve would be dependent on the edge condition (eg flame cut, butt joint). If the S-N curve is derived using predicted SCFs rather than measured SCFs to determine the stress range, then all the 'in air' ring-stiffened joint
test results to date lie above the mean 'T' curve, and furthermore there appears to be less scatter in the test data.

The TWI with assistance from NEL and LR have initiated an industry multi-sponsored project to study fatigue behaviour of 10 ring-stiffened T and K joints, with particular emphasis on the BRI region. All the failures at the BRI to date have occurred on joints with $\gamma = 28.6$, significantly higher than the $\gamma$ values generally found on offshore platforms. Consequently, in the TWI project more realistic $\gamma$ values have been chosen ($T = 32\text{mm}, \gamma = 14.3$).

6. Conclusions

- SCFs have been measured by LR on 328 acrylic ring-stiffened T/Y, X and K joints tested between 1982 and 1988.
- SCFs values measured on the acrylic specimens gave good correlation with steel joints tested in the UKOSRP II project and FE analyses performed by Shell/KSEPL and Wimpey offshore, except at the brace/ring intersection where rapid changes in stress occur.
- It has been found that ring-stiffeners give considerable relief to the SCF at the saddle location ($\beta = 0.8$), while the SCF may increase at the crown or at the brace/ring intersection. The SCF on the ring-stiffener itself may be critical, particularly if no flange is fitted.
- Parametric equations for SCFs in ring-stiffened joints derived by LR, and presented in the appendix, enable SCFs in ring-stiffened joints to be estimated more accurately and with increased confidence.
- Fatigue failures that occurred at the brace/ring intersection in the UKOSRP II project fell below the mean 'T' S-N curve. TWI with assistance from NEL and LR have initiated a comprehensive multi-sponsored project to study fatigue behaviour of ring-stiffened joints.

Acknowledgements

The authors wish to thank their colleagues at Lloyd's Register and the sponsoring organisations for their valuable contributions during this project. The opinions expressed in this paper are those of the authors.

References

Figure 1. Ring-stiffened tubular joint - Nomenclature.

\[ \tau = \frac{t}{T} \]
\[ \beta = \frac{d}{D} \]
\[ \gamma = \frac{D}{2T} \]
\[ \alpha = \frac{2L}{D} \]
\[ \theta = \text{Brace to chord inclination} \]
\[ \zeta = \text{Brace toe separation} \frac{g}{D} \]

(K and KT joints)

**Figure 2.** Brace/chord intersection SCF distribution under axial load.

- **n** - Number of rings under the brace
- **p** - Average ring separation

\[ \text{SCF} \]

\[ 5.0 \]

\[ 4.0 \]

\[ 3.0 \]

\[ 2.0 \]

\[ 1.0 \]

\[ 0.0 \]

\[ 0^\circ \]

\[ 10^\circ \]

\[ 20^\circ \]

\[ 30^\circ \]

\[ 40^\circ \]

\[ 50^\circ \]

\[ 60^\circ \]

\[ 70^\circ \]

\[ 80^\circ \]

\[ 90^\circ \]

Saddle

Crown

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Figure 3. SCF distribution around the ring inner edge, plate stiffener.

Table 1. Measured SCFs for T joint (Ref. 14, T/5)
\((t=0.40, \beta=0.50, \gamma=24, \alpha=10, \theta=90^\circ)\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Location</th>
<th>Unstiff.</th>
<th>Stiffened Plate</th>
<th>Stiffened Flanged</th>
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<td></td>
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<td>Axial</td>
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<td></td>
<td>Chord Saddle</td>
<td>9.18</td>
<td>2.20</td>
<td>2.10</td>
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<td></td>
<td>Chord Crown</td>
<td>2.20</td>
<td>2.21</td>
<td>2.27</td>
</tr>
<tr>
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<td>1.54</td>
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<tr>
<td></td>
<td>Brace BRI</td>
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<td>1.12</td>
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<td></td>
<td>Brace BRI</td>
<td>1.99</td>
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<tr>
<td></td>
<td>Inner Ring</td>
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<td></td>
<td>Chord Crown</td>
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<td>1.40</td>
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<tr>
<td></td>
<td>Inner Ring</td>
<td>0.70</td>
<td>0.58</td>
<td></td>
</tr>
</tbody>
</table>

Plate Stiffener \(n=2, p/d'=0.8, \text{R}_{\text{tau}}=1.92, K_2=5.15\)
Flanged Stiffener \(n=2, p/d'=0.8, \text{R}_{\text{tau}}=1.28, K_2=7.05\)
(see appendix A for full nomenclature)
Appendix A: SCFs for ring-stiffened tubular joints

SCF ratio = Stiffened joint SCF / Unstiffened joint SCF

CS - Chord saddle  BS - Brace saddle  IR - Ring inner edge
CC - Chord crown  BC - Brace crown

d' - Brace footprint length = d/sinθ
p - Average ring separation (for 1 ring = 2 × distance of ring to saddle)
n - Number of rings under the brace footprint
lw - Length of ring web  tw - Thickness of ring web
lf - Length of ring flange  tf - Thickness of ring flange
be - Effective length of chord wall = MIN(1.56TVγ, p) (=1.56TVγ, 1 ring)

Chord Parameters:

\[ R_{tau} = \sum_{i=1}^{n} \frac{t_{w,i}}{T} \quad \text{and} \quad K_2 = \frac{(12I_{OC}/d')^{1/2}}{T} \]

Where: \[ I_{OC} = \frac{d'T^3}{12} + \sum_{i=1}^{n} \left( \frac{l_{w,i}t_{w,i}(l_{w,i}+T)^2}{4} + \frac{l_{w,i}t_{i}(l_{w,i}+T/2)^2}{2} + \frac{l_{w,i}^3}{12} + \frac{l_{f,i}^3}{12} \right) \]

\[ A_c = d'T + \sum_{i=1}^{n} \left( l_{w,i}t_{w,i} + l_{f,i}t_{i} \right) \quad \text{and} \quad \gamma_{cgr} = \sum_{i=1}^{n} \left( \frac{l_{w,i}t_{w,i}(l_{w,i}+T)}{2} + \frac{l_{f,i}t_{i}}{2} \right) / A_c \]

K_2 and Rtau' are the K_2 and Rtau parameters calculated for the pair of rings nearest to the crown. (i.e. if n<2 then set n=2 to find K_2 and Rtau')

Ring Parameters:

\[ K_1 = \frac{A_r}{D.T} \quad \text{and} \quad I_{mod} = \frac{(I_{OR}/\gamma_{cgr})}{(T^2/D/δ)} \]

Where: \[ I_{OR} = \frac{b_eT^3}{12} + l_{w}tw(l_{w}/2 + tf)^2 + beT(lw + tf + T/2)^2 + \frac{l_{w}^3}{12} + \frac{l_{f}^3}{3} \]

\[ A_r = b_eT + lw + tf + \text{be}T(lw + tf + T/2 + l_{w}t + tf/2) / A_r \]

For stresses occurring along the brace/chord intersection

The ring-stiffened joint SCF = SCF ratio × unstiffened joint SCF

(When using these equations on joints with short chords (i.e. a<12), the unstiffened joint SCF should exclude any short chord effects.)

For the ring-stiffeners themselves, the formulae predict the SCF on the ring inner edge where SCF = Ring inner edge stress/Nominal brace stress (nb. The SCF equations neither identify the angle around the ring-stiffeners nor the ring-stiffener which has this maximum SCF value).

The ring-stiffened brace/chord intersection SCF is the maximum of the

Unstiffened joint SCFSaddle × SCFRatioSaddle and Unstiffened joint SCFCrown × SCFRatioCrown

nb. On the chordside and ring inner edge under axial load, on the chordside and ring inner edge under OPB and for all locations under IPB a minimum SCF=1.5 is adopted. On the braceside under axial load and OPB a minimum SCF=2.5 is applied to cover the local increase in SCF at the BRI location that has been noted in some configurations.
A1. T/Y Joints

The SCF ratio at the chord and brace crown under axial load need only be applied to the unstiffened local shell deformation stress, while the chord overall stress is as for the unstiffened case.

Axial load

SCF ratio \( CS \) = \( \frac{0.1}{\tau \sqrt{\nu N_{re}^2}} \left\{ \frac{5.5}{K_2} + \frac{1}{R_{tau}} \right\} \) (T1) \( \sigma = 0.05 \)

SCF ratio \( CC \) = \( 1.5 \beta^{0.5} \) (T2) Char.

SCF ratio \( BS \) = \( \frac{0.12}{\tau \sqrt{N_{re}^2}} \left\{ \frac{10}{K_2} + \frac{1}{R_{tau}} \right\} \) (T3) \( \sigma = 0.05 \)

SCF ratio \( BC \) = \( 0.8 \sqrt{\nu (4\beta - 3\beta^2 - 0.5)} \) (T4) Char.

SCF ratio \( \text{IR} \) = \( \frac{0.3 \tau Y \sin \theta}{N_{re}^3} \left\{ \frac{0.15 \beta^{0.5}}{1 + \frac{1}{K_1}} \right\} \) (T5) \( \sigma = 14.0\% \)

Out-of-plane bending

SCF ratio \( CS \) = \( \frac{1}{8 \sqrt{\nu N_{re}^4 (\nu Y)}} \left\{ \frac{6}{K_2} + 1 \right\} \) (T6) \( \sigma = 0.05 \)

SCF ratio \( BS \) = \( \frac{0.36}{\tau \sqrt{N_{re}^5}} \left\{ \frac{3}{K_2} + \frac{1}{R_{tau}^{1.1-(n-1).p(d')}} \right\} \) (T7) \( \sigma = 0.06 \)

SCF ratio \( \text{IR} \) = \( \frac{0.15 \tau Y \sqrt{\nu \sin \theta}}{N_{re}^6} \left\{ \frac{0.5}{1 + \frac{1}{K_1}} \right\} \) (T8) \( \sigma = 18.6\% \)

In-plane bending

SCF ratio \( CC \) = \( \frac{0.1 \tau Y}{\sqrt{N_{re}^2}} \left\{ \frac{0.2 (3.7 + \frac{1}{R_{tau} K_2}}{0.35} \right\} \) (T9) \( \sigma = 0.07 \)

SCF ratio \( BC \) = \( 0.8 + (N_{re} Y)/(20) \) (T10) Char.

SCF ratio \( \text{IR} \) = \( 0.73 \tau Y \sin \theta \) (T11) \( \sigma = 15.5\% \)

A2. X Joints

Balanced axial load

SCF ratio \( CS \) = \( \frac{0.85 \sin \theta}{\sqrt{\nu N_{re} K_2}} \) (X1) \( \sigma = 0.04 \)

SCF ratio \( CC \) = \( 1.5 \beta^{0.5} \) (X2) Char.

SCF ratio \( BS \) = \( \frac{0.3 \sin 2\theta}{\tau \sqrt{N_{re}^2}} \left\{ \frac{5}{K_2} + \frac{(3\beta^2 - 3\beta + 1)}{R_{tau}} \right\} \) (X3) \( \sigma = 0.05 \)

SCF ratio \( BC \) = \( 0.8 \sqrt{\nu (4\beta - 3\beta^2 - 0.5)} \) (X4) Char.

SCF ratio \( \text{IR} \) = \( \frac{0.65 \tau Y \sqrt{\nu \sin \theta}}{N_{re}^3} \left\{ \frac{8}{1 + \frac{1}{K_1}} \right\} \) (X5) \( \sigma = 16.9\% \)
Balanced out-of-plane bending

\[
SCF_{\text{ratio CS}} = 0.26 \sin^2 \theta \left( \frac{4}{K_2} + \frac{1}{N_{re4}} \right) \quad (X6) \quad \sigma = 0.06
\]

\[
SCF_{\text{ratio BS}} = 0.25 \sin^2 \theta \left( \frac{1}{K_2} + \frac{(2.4\beta^2 - 2.7\beta + 1)}{N_{re5} [1.1 - (n-1)pR]} \right) \quad (X7) \quad \sigma = 0.06
\]

\[
SCF_{IR} = \frac{\tau \sin^2 \theta}{N_{re6}} \left( \frac{10.5 \gamma (1-\beta)}{Imod} + \frac{\gamma^2}{1000K_1} \right) \quad (X8) \quad \sigma = 0.06 \%
\]

Balanced in-plane bending

\[
SCF_{\text{ratio CC}} = \left\{ \begin{array}{c}
0.07 (\beta Y)^{0.2} \left( \frac{8}{K''_2} + \frac{1}{0.5 \text{ Rtau}} \right) \quad \theta = 0^\circ \\
1.0 \quad \theta = 90^\circ
\end{array} \right. \quad (X9a) \quad \sigma = 0.09
\]

\[
SCF_{\text{ratio BC}} = 0.8 + \left( \frac{N_{re7} \beta Y}{20} \right) \quad (X9b) \quad \text{Char.}
\]

\[
SCF_{IR} = \frac{1.75 \beta \gamma \sin^2 \theta}{K_1} \quad (X10) \quad \sigma = 24.6 \%
\]

A3. K Joints

Single axial load (one brace loaded)

\[
SCF_{\text{ratio CS}} = \text{as equation (T1)} \quad (K1) \quad \sigma = 0.05
\]

\[
SCF_{\text{ratio CC}} = 1.2 \beta^{0.3} \quad (K2) \quad \text{Char.}
\]

\[
SCF_{\text{ratio BS}} = 0.24 \left( \frac{\beta}{\gamma Y} \right)^{0.25} \left\{ \frac{7}{K''_2} + \frac{1}{0.5 \text{ Rtau}} \right\} \quad (K3) \quad \sigma = 0.07
\]

\[
SCF_{\text{ratio BC}} = 1.6 \beta^2 - 0.48 + 0.8 \quad (K4) \quad \text{Char.}
\]

\[
SCF_{IR} = \frac{0.3 \gamma \sin^2 \theta}{N_{re3}} \left\{ \frac{16 \beta^{0.15} \gamma^{0.5}}{Imod} + \frac{\beta^{0.5}}{K_1} \right\} \quad (K5) \quad \sigma = 25.38
\]

where brace A is the loaded brace

Single out-of-plane bending (one brace loaded)

\[
SCF_{\text{ratio CS}} = \text{as equation (T6)} \quad (K6) \quad \sigma = 0.06
\]

\[
SCF_{\text{ratio BS}} = \text{as equation (T7) x 1.1} \quad (K7) \quad \sigma = 0.07
\]

\[
SCF_{IR} = \frac{0.15 \beta^{0.75} \gamma (\sin^2 \theta)}{N_{re6}} \left\{ \frac{8 \beta^{0.5}}{Imod} + \frac{\beta^{0.5}}{K_1} \right\} \quad (K8) \quad \sigma = 21.8\%
\]

where brace A is the loaded brace

Single in-plane bending (one brace loaded)

\[
SCF_{\text{ratio CC}} = \text{as equation (T9) x 1.2} \quad (K9) \quad \sigma = 0.09
\]

\[
SCF_{\text{ratio BC}} = \text{as equation (T10)} \quad (K10) \quad \text{Char.}
\]

\[
SCF_{IR} = \text{as equation (T11)} \quad (K11) \quad \sigma = 21.4\%
\]

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Balanced axial load

\[ \text{SCF ratio}_{CS} = \frac{0.05}{2} \left( \frac{1}{N_{rel,1}} \right) \left( \frac{10}{R_{tau}} \right)^{0.35} \]  

(K12) \( \alpha = 0.07 \)

\[ \text{SCF ratio}_{CC} = 12 \beta^{0.3} \]  

(K13) Char.

\[ \text{SCF ratio}_{BS} = \text{as equation (K3)} \times 1.4 \]  

(K14) \( \alpha = 0.10 \)

\[ \text{SCF ratio}_{BC} = 0.58^2 + 0.8 \]  

(K15) Char.

\[ \text{SCF IR} = \text{as equation (T5)} \times \frac{2 \tau^{0.3} N_{rel,3}}{Y^{0.3}} \]  

(K16) \( \alpha = 23.3\% \)

Unbalanced out-of-plane bending

\[ \text{SCF ratio}_{CS} = \text{as equation (T6)} \times 0.8 \left( \frac{\sin \theta_A}{\sin \theta_B} \right)^{0.5} \]  

(K17) \( \alpha = 0.05 \)

where brace A is the brace under analysis

\[ \text{SCF ratio}_{BS} = \text{as equation (T7)} \times \left( \frac{0.75}{\beta^{0.2}} \right) \]  

(K18) \( \alpha = 0.06 \)

\[ \text{SCF IR} = \text{as equation (T8)} \times 0.8 \]  

(K19) \( \alpha = 24.6\% \)

Balanced in-plane bending

\[ \text{SCF ratio}_{CC} = \left\{ \begin{array}{l} \text{as equation (T9)} \times 1.16 \left( \frac{r}{\sin \theta} \right)^{0.6} \text{max} \\ \text{as equation (T9)} \times 1.2 \left( \frac{r}{\sin \theta} \right)^{0.6} \text{min} \end{array} \right. \]  

(K20a) \( \alpha = 0.09 \)

(K20b) \( \alpha = 0.09 \)

\[ \text{SCF ratio}_{BC} = \text{as equation (T10)} \]  

(K21) Char.

\[ \text{SCF IR} = \text{as equation (T11)} \]  

(K22) \( \alpha = 21.4\% \)

A4. Equivalent number of rings (\( N_{rel} \) values)

Axial load chord saddle :

\( N_{rel,1} = \exp(-0.3(p/d')^{0.2}) \)  

(n=1)

\( N_{rel,1} = 2.\exp(-0.2(p/d')^{0.2}) \)  

(n=2)

\( N_{rel,1} = 1+2.\exp(-2.5(p/d')^{0.2}) \)  

(n=3)

\( N_{rel,1} = 2.\exp(-0.2(p/d')^{0.2})+2.\exp(-5.0(p/d')^{0.2}) \)  

(n=4)

Axial load brace saddle :

\( N_{rel,2} = \exp(-0.2(p/d')^{0.2}) \)  

(n=1)

\( N_{rel,2} = 2.\exp(-0.5(p/d')^{0.2}) \)  

(n=2)

\( N_{rel,2} = 1+2.\exp(-3.0(p/d')^{0.2}) \)  

(n=3)

\( N_{rel,2} = 2.\exp(-0.5(p/d')^{0.2})+2.\exp(-6.0(p/d')^{0.2}) \)  

(n=4)

Axial load ring inner edge :

\( N_{rel,3} = \exp(-0.1(p/d')^{0.2}) \)  

(n=1)

\( N_{rel,3} = 2.\exp(-0.25(p/d')^{0.2}) \)  

(n=2)

\( N_{rel,3} = 1+2.\exp(-3.0(p/d')^{0.2}) \)  

(n=3)

\( N_{rel,3} = 2.\exp(-0.25(p/d')^{0.2})+2.\exp(-30.0(p/d')^{0.2}) \)  

(n=4)

OPC chord saddle :

\( N_{rel,4} = \exp(-1.4(p/d')^{0.5}) \)  

(n=1)

\( N_{rel,4} = 2.\exp(-1.4(p/d')^{0.5}) \)  

(n=2)

\( N_{rel,4} = 1+2.\exp(-3.0(p/d')^{0.5}) \)  

(n=3)

\( N_{rel,4} = 2.\exp(-1.4(p/d')^{0.5})+2.\exp(-6.0(p/d')^{0.5}) \)  

(n=4)

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OPB brace saddle:
\[ N_{re5} = \exp(-0.3(p/d')) \] (n=1)
\[ N_{re5} = 2.\exp(-(p/d')) \] (n=2)
\[ N_{re5} = 1+2.\exp(-9.0(p/d')) \] (n=3)
\[ N_{re5} = 2.\exp(-(p/d'))+2.\exp(-18.0(p/d')) \] (n=4)

OPB ring inner edge:
\[ N_{re6} = \exp(-0.2S(p/d,)^2) \] (n=1)
\[ N_{re6} = 2.\exp(-0.2S(p/d,)^2) \] (n=2)
\[ N_{re6} = 1+2.\exp(-4.0(p/d')^2) \] (n=3)
\[ N_{re6} = 2.\exp(-0.25(p/d')^2)+2.\exp(-30.0(p/d')^2) \] (n=4)

IPB chord crown / IPB brace crown / IPB ring inner edge:
\[ N_{re7} = p/d' \] (n=1)
\[ N_{re7} = (n-1)p/d' \] (n#1)
( nb. If \( N_{re7} \leq 0.65 \) then set \( N_{re7} = 0.65 \) )

A5. Range of applicability
The derived ring-stiffened joint equations for T/Y, X and K joints are generally valid for joint parameters within the following limits:

- \( 0.40 \leq \tau \leq 1.00 \)
- \( 0.26 \leq \beta \leq 0.80 \)
- \( 12.0 \leq \gamma \leq 33.3 \)
- \( 5.00 \leq \alpha \leq 13.3 \)
- \( 30^\circ \leq \theta \leq 90^\circ \)

\( \zeta = 0.067 \) (the \( \zeta \) term is not a very significant parameter)

Where joint parameters are not within the limits of the equations, a conservative approach should generally be employed (ie. adopt the larger SCF derived using the true value of a parameter and it's limit value).

LR do not recommend ring-stiffening joints with \( \beta > 0.8 \). However, if a joint with \( \beta > 0.8 \) is to be analysed the following approach is used:

i) Saddle - Apply the unstiffened saddle SCF.
ii) Crown - Apply the unstiffened crown SCF x crown SCF ratio.
iii) Ring - Apply the ring inner edge SCF.

A ring-stiffener between the brace toes of a K joint under unbalanced OPB may take significant loading. SCFs may be calculated by splitting the ring-stiffener between the braces (ie \( lw \times tw/2 \) under each brace toe).

A6. Design equation factors
The ring-stiffened joint SCF equations are mean fit equations. Beside each equation is the actual standard deviation of the data (for use with the SCF ratios) or the percentage standard deviation of the data (for use with the ring inner edge SCFs). At the chord and brace crown under axial load and at the brace crown under IPB a characteristic fit was applied to cover the BRI which could be the location of the maximum SCF.

The design factors used by LR are:

i) For the brace/chord intersection:
   \[ \text{SCF ratio}(\text{design}) = \text{SCF ratio}(\text{mean}) + 1.28\sigma_A \]
   where \( \sigma_A \) is the actual standard deviation of the equation.

ii) For the ring inner edge SCF:
   \[ \text{SCF}(\text{design}) = \text{SCF}(\text{mean}) \times (1+2\sigma_P/100) \]
   where \( \sigma_P \) is the percentage standard deviation of the equation.
PERFORMANCE OF FLATTENED TUBE CONNECTIONS

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Summary

Tests were conducted on a tubular frame containing four types of YT brace-to-chord joints - a gap joint, an overlap joint, a joint with the ends of the braces flattened, and one with flattened braces attached to a prestressed grouted sleeve. The conventional gap joint was the worst, and the prestressed grouted joint was the best, with regard to stress concentration factors, fatigue life and perhaps static strength. Subject to fabrication capabilities, the flattened tube joint is a competitive alternative to the conventional gap joint with profile cut braces.

1 Introduction

The use of flattened ends on tubular bracing members to facilitate connection to chords of trusses has the potential to simplify fabrication and reduce cost. Provided that the plane of the flat is normal to the plane of the truss out-of-plane rigid joints and stability of the truss are maintained, in plane the bracing members behave effectively as pin-ended struts or ties. The truss retains rotational stiffness at the joints through the continuity of the chord members. This type of flattening is considered in this paper. Flattening the tubes in the plane of the truss does not provide as good a structural performance, nor the economies of fabrication, as out-of-plane flattening.

Flattened ends are used in a number of lighter structures in buildings, and they facilitate erection in the Strarch system. If they perform well in those situations, they are likely to perform well in heavier structures as are found offshore, provided that the fabricator has the capacity to flatten larger tubes. Fabrication of connections should be simpler, replacing the profile cut with a plane intersection between the brace and the chord. Welding from both sides gives access to the root of the weld and the opportunity to achieve weld quality more easily.

It has been shown [1] that, provided that a finite length of flat within the range \( t \) to \( 1.5t \) is used, where \( t \) is the wall thickness, that flexural stresses transmitted through the welds to the chord as a result of lateral load on the brace, and the braces with flattened ends behave very much as pin-ended beams. Further, the use of flattened ends can eliminate the eccentricity of connection characteristic of gap YT joints made from profile cut tubes.

One reason for the structural efficiency of these connections lies in the fact that the flattened tube reaches further around the chord at the intersection than the circular cross-section, so that the axial force in the brace, which tends to concentrate in the stiff load path at the folds, is delivered tangentially rather normally into the wall of the chord. Peak stresses are reduced because the local forces are distributed by membrane action rather than flexural action in the chord.

This paper reports a pilot test program in which connections with flattened ends in the braces are compared with conventional profile cut connections subjected to the same loads.
Test Program

A study of comparative performance of four connection types was made in a frame, shown in Figure 1, in which the four connections were subjected to virtually the same loading. The connections were of the YT type in which the diagonal was in tension, balanced by a horizontal brace in compression. The four connection types were:

[A] Conventional gap joint (with eccentricity) with profile cut braces,

[B] Joint with flattened ends attached to a sleeve, which in turn was attached to the chord by prestressed grout (no shear keys),

[C] Conventional overlap joint with profile cut braces, and

[D] Joint with flattened ends to the braces (zero eccentricity).

A test with YT joints was chosen since the configuration is a realistic one, with load capacities which cannot be obtained by superposition of results from simpler Y and T joints.

Chemical prestressing of the grout in Joint B was achieved with a CSA additive [3, 4]. Hoop strains indicated an average prestress of 5.1 MPa at seven days. Since the gauges were disconnected for many months between casting and final testing the prestress during testing was unknown, but based upon experience with other tests it was estimated to be about 7 MPa. The protective varnish on the inner tube was not removed before casting, so that the coefficient of friction would have been about 0.7 [3, 4].

Coupon tests on samples taken from the tubes revealed similar strengths for tubes of all diameters, namely, 397 MPa yield stress and 464 MPa ultimate tensile strength, each with a range ± 3%.

The frame was designed so that an actuator provided the identical primary axial loads to all connections. To limit bending effects one diagonal passed through the other without connection. The chords and horizontal braces have a force in them nominally the same as the actuator load. The diagonals have forces nominally 1.414 times the actuator load.

A deficiency of the test rig is that the continuity of the vertical chord on the right hand side is lost through the need to insert the actuator. As a result the connections with flattened tubes did not have the rotational restraint that would normally be present in practice. Restraint was provided to the top connection by a pin-ended brace giving lateral support to the point of load application. A similar brace was not provided to the bottom connection for fear of providing excessive restraint, with significant consequences for the test, discussed later.

The test program had three phases:

(a) a static test to establish elastic strains and strain concentration factors (SNCF's),

(b) a fatigue test to determine relative sensitivities to fatigue damage, and

(c) an ultimate strength test to establish minimum load capacity of the connections.

At the end of the fatigue test, where failure occurred at Joint A, the frame was repaired and strengthened at the point of failure such that that node was eliminated from consideration in the ultimate strength test.
A partial picture of strains was obtained from 111 strain gauges attached in regions of strain concentration and also in regions of nominally uniform stress. Linear extrapolation from gauges at positions \(0.20(r^2)^{0.5}\) and \(0.65(r^2)^{0.5}\) from the weld toe were used to determine hot spot strains.

After a few cycles of load to 250 kN compression to achieve shakedown the SNCF's were obtained as the ratio of the extrapolated hot spot strain to the average measured strain in the bracing member. The results of these measurements are given in Figure 2. They are generally credible except for the SNCF's in the gap of Joint A, where the space was insufficient for proper use of strain gauges. The SNCF of 3.97 in the gap adjacent to the horizontal brace was the highest measured, and appears reasonable, but the corresponding SNCF of 0.33 for the diagonal brace cannot be accepted, especially since the fatigue failure in the subsequent test initiated at this point.

For the conventional joints the SNCF's are all well below those given by parametric equations for design [2]. This is not surprising since design equations are conservative to accommodate the scatter of experimental data with respect to predictions from semi-empirical formulae.

The main benefit of the test lies in the comparative values of SNCF for the different connections. The following observations can be made.

Ranked in order of maximum SNCF were Joint A (SNCF = 3.97), Joint D (3.17), Joint C (1.80) and Joint B (1.27). In the case of Joint B the SNCF was measured on the sleeve, not the tube grouted within.

The location of peak SNCF in the flattened tube connection, Joint D, was at the saddle, whereas the location in the conventional gap connection, Joint A, was at the crown. At the crown of Joint D the stresses are low, and even reversed with respect to the axial force in the brace.

The saddle SNCFs of Joints A and D were virtually the same (3.2). It is noted that, as a result of the flattening, the brace reaches further around the chord providing a longer run of weld for transfer of the brace force tangential to the wall of the chord.

The low SNCFs in the grouted connection are attributed to the prestress, which helped to ensure composite action so that the sleeve plus inner tube behaved as a single tube with much larger effective thickness.

4 Fatigue Strength

Ultrasonic examination of the welds prior to fatigue testing revealed no cracks. Using an Instron actuator the structure was loaded to \(\pm 75\) kN for 7,740 cycles followed by 6,320 cycles at \(\pm 237.5\) kN, at which point failure occurred. The latter load corresponded to an nominal average reversible stress of 30.9 MPa in the chords, 66.0 MPa in the horizontal braces, 93.3 MPa in the diagonal braces, and 1.11 MPa average shear/bond stress on the grouted sleeve.

The gap Joint A failed with a ductile fatigue fracture through the wall of the chord at the diagonal brace, extending from the crown toe in the gap most of the way around to the crown at the heel. It appeared to initiate at the toe where, as mentioned above, significant hot spot stress was not found using strain gauges. Nevertheless, the observed failure mode was expected.

The failed connection was repaired by gouging and rewelding, and strengthened by the addition on
two heavy gusset plates filling in the gap between horizontal and diagonal braces. It was intended to continue testing until a second connection failed. A further 350 cycles at $\pm 75$ kN and then 6,660 cycles at $\pm 237.5$ kN were applied before the test was abandoned without any detectable cracking in Joints B, C and D. Slip was not detected at the grouted Joint B. During this phase of the test cracks developed around the added gusset plates and around the 50 x 50 mm bars forming the yoke at the cross over of the diagonal braces. These were repaired and the welds were ground and peened several times in attempts to extend the life of the structure. Although the original mode of failure was eliminated the added stiffness of the gussets was increasing the in-plane bending moments being transmitted to the braces and the yoke, reducing rather than increasing the fatigue life of these elements.

This test revealed a fatigue strength of Joints B, C and D several times greater than Joint A, and possibly very much greater.

5 Ultimate Strength

The structure with the modifications to Joint A was subjected to a static test using a 1000 kN Mohr-Federhaff jack. The following observations are of note.

450 kN: First departure from linear in the load-deflection curve.

525 kN: Rotation of Joints B and D becomes noticeable. This would not have occurred if Joints B and D had been connected by a continuous member providing rotational stiffness.

645 kN: Cracks in the chord/diagonal brace saddle points at the weld toes were detected at Joint B (10 mm surface length, west side) and at Joint D (20 mm surface length, east side).

740 kN: Peak load prior to a sudden large rotation of Joint D. Local deformation of Joints A and D was clearly evident. The lack of rotational restraint of this joint contributed to this instability which made it impossible to increase the load to achieve joint failure.

At this peak load the nominal average stresses were 96.3 MPa in the chords, 205.6 MPa in the horizontal braces, and 290.7 MPa in the diagonal braces. However, strain measurements indicated that the stress distribution was far from uniform in the members. At 705 kN yielding was already evident in the diagonal BC away from the joints, but not evident in diagonal AD.

At the maximum load the average shear-bond stress on the chord at the grouted joint B was 3.3 MPa. There was no evidence of slip, and the shear displacement between the chord and the sleeve at the loaded end was 0.22 mm. The residual displacement after unloading was 0.05 mm.

Hoop strains in the plane of the horizontal members, excluding strain due to prestress, are plotted in Figure 3 for Joints B and D when the load was 705 kN. The two gauge points at the saddle point were used to determine the hot spot stress. The strains at the corresponding gauge points for the diagonal members were -1171 and 963 microstrain at Joint B, and -9998 and -3014 microstrain at Joint D (compression positive). These results indicate strains at Joint D two or more times those at the composite Joint B. Figure 3 also shows the local nature of the hot spot stress. However, the gauges for measuring hot spot stress at Joint B were positioned using the sleeve wall thickness rather than an effective thickness of the composite section.
Discussion and Conclusions

Of all the joints the grouted sleeve connection was the most stiff, and perhaps the strongest. With a prestress of about 7 MPa integral action for sandwich plate behaviour was readily achieved, without separation even for quite high tension in the braces. A sleeve length-to-diameter ration of 1.0 was more than enough to transmit the ultimate load of the braces into the chord.

The gap Joint A was evidently the least stiff, the weakest, and it had the lowest fatigue life. All these features were improved in Joint D with the flattened ends. These results have encouraged the development of a more detailed research program on both flattened and grouted sleeve connections, now in progress.

The tests described were carried out on a frame built to explore the possibilities of connections with flattened ends and/or grouted sleeves vis-a-vis conventional connections. The results indicate that there are potential benefits in reduction of stress concentration, stiffness, fatigue life and static strength. Reasons for the enhanced performance include the elimination of joint eccentricity otherwise imposed for gap requirements, improved landing of the brace on the chord, with a longer weld run where it is needed for transferring forces, and the virtual elimination of in-plane bending moments being transmitted from the brace to the joint.

Acknowledgement

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References


Figure 1. The test frame.
Figure 2. Experimentally determined values of SCNF on the test frame.
Figure 3. Hoop strains in horizontal plane of Joints B and D a 705 kN load.
APPLICATION OF THE FRACTURE MECHANICS APPROACH TO THE FATIGUE BEHAVIOUR OF WELDED TUBULAR STEEL STRUCTURES

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Summary

In this paper a brief description of the fracture mechanics (FM) approach for fatigue life and crack growth of welded tubular steel structures is given. Several assumptions made in the model are validated. To show the ability of this approach as a design tool, the effects of several parameters are determined by calculations and these are compared with experimental results. The paper concludes with some general remarks and recommendations.

1. Introduction

The fatigue life of a welded tubular steel structure can be determined either by the traditional S-N method or with a more sophisticated FM approach. The S-N method is based on experiments, resulting in graphs with the stress range ($\Delta \sigma$) on the vertical axis and the number of cycle to failure (N) on the horizontal axis. The FM approach on the other hand is based on a fatigue crack growth model. The material crack growth parameters of the model can be determined from standardized small specimens. The influence of the welded geometry is incorporated in the loading parameter ($\Delta K$).

With regard to the S-N method the FM approach has the advantage that it can be used as a design tool to study special effects such as weld toe grinding or scale effects and to simulate S-N curves for specific joints without testing. Another advantage is that the FM approach can be used as a maintenance tool to set up an inspection period and to calculate the remaining life of a discovered crack.

2. Background of the fracture mechanics approach

Fatigue crack growth models for welded structures, based on linear elastic fracture mechanics, have been described by several authors (Bell et al [1], Van Delft et al [2], Thorpe [3] and Dijkstra et al [4]). These models give a relation between the fatigue crack growth rate ($da/dN$) and the stress intensity factor range ($\Delta K$). In most cases the Paris-Erdogan relation [5] can be applied satisfactorily:

$$\frac{da}{dN} = C (\Delta K)^m$$  \hspace{1cm} (1)

The stress intensity factor range ($\Delta K$) is the difference between the maximum and minimum stress intensity factor ($K$). $K$ gives the magnitude of the elastic stress field near the crack tip:

$$K = \frac{\sigma J}{\pi a}$$  \hspace{1cm} (2)

Dijkstra et al [4] give specific information about $K$ for welded structures. The effect of the global geometry is incorporated in the
stress analysis, while the effect of the local geometry (weld shape, etc.) is incorporated in the determination of K.

The lifetime can be calculated by integrating the crack growth relation from an initial defect size ($a_i$) to a final or allowable defect size ($a_f$). Due to the complex relation between the crack size ($a$) and $\Delta K$, a numerical calculation procedure carried out by a computer must be used. Therefore TNO Building and Construction Research has developed the program FAFRAM (Fatigue Fracture Mechanics) (Dijkstra et al [6]).

3 Fracture mechanics model for a welded tubular steel structure

3.1 General

In order to verify the FM approach as a design tool crack growth calculation results are given in the next chapter. These calculations are carried out for fatigue tested tubular joints. Those tests were a part of ECSC-SMOZ (Dijkstra et al [7] and [8]) and MaTS-SMOZ programmes (Lourenssen et al [9]), carried out in the Netherlands. T- and X-joints with various dimensions were tested under several load conditions and environmental circumstances. The comparison in this study is focused on T-joints with diameter ratio ($\beta$) and wall thickness ratio ($r$) both equal to 0.5. The considered load condition is axial loading on the brace with a constant amplitude load and $R=0$. In the next sections the fatigue crack growth model used is worked out and validated with observed crack growth behaviour described in literature.

3.2 Modelling of the geometry

For an axially loaded T-joint with $\beta=0.5$ it is found that the crack always starts at the saddle point of the intersection in the chordwall at the weld toe. The crack has a more or less semi-elliptical shape and propagates along the weld toe. Only in the last part of the lifetime the crack branches away from the weld toe into the chord wall.

The fatigue crack growth model developed for these type of joints, assumes that the crack can be modelled as a semi-elliptical surface crack at the weld toe of a plate to which an attachment is welded (see figure 1). This plate can be seen as the chord wall of the T-joint and the attachment as the brace wall. In the model only the "chord" plate is loaded by membrane and bending stresses; the "brace" attachment is not loaded. Finite element calculations (Van Delft [10]) show that this modelling is in accordance with the stress situation of the T-joint. The stresses at the vicinity of the weld toe are mainly determined by forces in the chord wall and is hardly affected by forces in the brace wall.

In the model the weld geometry is characterized by the weld toe angle ($\theta$), the weld width (L) and the weld toe radius ($\rho$) (see figure 1). From the weld profiles determined from the test specimens it is assumed that $\theta=45^\circ$ and L=1.2T. Otegui et al [11] carried out measurements of toe radii along the weld in manual welded T-plates. They found an average value of $\rho=0.5$ mm; this value is used in the calculations.

3.3 Stress state

The stress range ($\Delta \sigma$) used in the model, must represent the effect of the global geometry without the raising effect of the weld geometry. For tubular joints $\sigma$ is equal to the hot-spot stress range ($\Delta \sigma_{HS}$) in the chord as defined and accepted by the Working Group III - Tubular Joints - of the ECCS (see also Dijkstra et al [8]). From the tests the hot-spot stress range ($\Delta \sigma_{HS}$) can be derived from the hot-spot strain ranges ($\Delta \varepsilon_{HS}$ and $\Delta \varepsilon_{HS}$) determined from strain gauge measurements with use of the
The extrapolated strain ($\varepsilon_{HS}$) parallel to the weld toe was determined for a limited number of tests. With use of eq. 3 and the results of those tests for which $\varepsilon_{HS}$ was determined, the bi-axial stress state for the considered joint is to be as follows:

$$\sigma_{HS} = 1.2 \varepsilon_{HS}$$  \hspace{1cm} (4)

The model divides the stress range ($\Delta \sigma$) into a membrane stress range ($\Delta \sigma_m$) part and a bending stress range ($\Delta \sigma_b$) part. For the considered joint finite element calculations and photo-elastic tests (Clayton [12]) indicate that the following relations can be estimated:

$$\Delta \sigma_m = 0.25 \Delta \sigma$$  \hspace{1cm} (5)

$$\Delta \sigma_b = 0.75 \Delta \sigma$$  \hspace{1cm} (6)

### 3.4 Stress intensity factor

$K$ of a semi-elliptical crack at a weld toe is expressed in the depth ($a$) and the width ($c$) direction (see Figure 1) as follows:

$$K_a = [M_{k,m,a} M_{k,b,a} M_{m,a} a_m + M_{k,b,a} M_{m,a} a_m] \sqrt{\pi a}$$  \hspace{1cm} (7)

$$K_c = [M_{k,m,c} M_{k,b,c} M_{m,c} c_m + M_{k,b,c} M_{m,c} c_m] \sqrt{\pi c}$$  \hspace{1cm} (8)

For the correction factors for the flat plate ($M_{m,a}, M_{b,a}, M_{m,c}$ and $M_{b,c}$) the solution of Newman-Raju (Newman et al [13]), which is based on finite element calculations, is taken. For the correction factors for the influence of the weld geometry ($M_{k,m,a}, M_{k,b,a}, M_{k,m,c}$ and $M_{k,b,c}$) the solution given in a proposal for a draft chapter fatigue of the new PD 6493 (Maddox et al [14]) with two modifications as described by Dijkstra et al [4], is used. The weld toe radius is taken into account and the $M_k$-values are reduced by using a reduction factor ($\omega$). For depth direction $\omega=0.9$ and for width direction $\omega=0.8$. With use of three-dimensional finite element calculations it is found that the values of the $M_k$-solution, which are based on two-dimensional calculations, are overconservative.

### 3.5 Crack growth relation

For the calculations presented in the next chapter the crack growth relation of eq. 1 is used. The crack growth constants are taken from the mean crack growth line of the IIW Recommendations [15]. For normal air the constants are equal to:

$$m = 3 \quad \text{and} \quad C = 1.832 \times 10^{-13} \text{ (units N and mm)}$$  \hspace{1cm} (9)

and for a marine environment these are equal to:

$$m = 3 \quad \text{and} \quad C = 1.404 \times 10^{-12} \text{ (units N and mm)}$$  \hspace{1cm} (10)
3.6 Initial crack dimensions

The initial crack dimensions are characterized by the initial crack depth ($a_i$) and the aspect ratio ($a_i/c_i$). For the initial crack depth $a_i=0.15$ mm is chosen, because this seems to be a realistic value for a starting crack at the weld toe.

Otegui et al [11] concluded that an aspect ratio of $a/c=1$, as defined in some models, is too high and therefore may not lead to realistic fatigue lifetimes. From detailed observations they found aspect ratios from $a/c=0.2$ up to 0.5, dependent from occurrence of coalescence. In figure 2 the evolution of aspect ratios for four situations are plotted for a T-joint with $T=32$ mm and $\Delta\sigma=400$ N/mm$^2$. Three different initial aspect ratios ($a/c=1.0$, 0.5 and 0.2) are considered; also the effect of three initial cracks (with $a/c=0.5$) which are separated 1.2 mm from each other, is considered. Also the lifetimes are given in figure 2. From these results and the observations of Otegui et al [11] it can be concluded that $a/c=0.2$ is a realistic estimation.

3.7 Validation crack development in tubular T-joints

For one of the T-joints tested for the ECSC-SMOZ programme crack growth data were obtained from benchmarks, which were introduced during testing (Noordhoek et al [16]). The weld toe was ground and therefore in the calculation $p=4$ mm is chosen. From strain gauge measurements and eq. 4 it follows that $\Delta\sigma=148$ N/mm$^2$.

The calculations presented in this section start with the crack dimensions and number of cycles which were determined for the first benchmark. This is done, because the initial crack of the model might not be representative for the specimen which was retested at a higher load level. In figure 3 and 4 the crack growth in depth and width direction, are compared with measured data. The crack growth is calculated with the $w$-reduction factors as proposed in section 3.4 and without these factors. From these figures it can be concluded that the $w$-reduction factors determined with finite element calculations, are in agreement with the experimental results.

4 The fracture mechanics approach as a design tool

4.1 General

In this chapter results of crack growth calculations are given for axial loaded T-joints. With these calculations S-N curves are simulated and compared with test results. If not stated otherwise, the following values are used: $a_i=0.15$ mm, $a_i/c_i=0.2$, $\theta=45^\circ$, $p=0.5$ mm, $\sigma_m^{\sigma_{b}}=1/3$, $\omega_a=0.9$, $\omega_c=0.8$, $m=3$ and $C=1.832.10^{-13}$.

4.2 Design curve

Thorpe et al [17] had defined a basic design S-N curve for 32 mm thick joints on basis of 21 test results. In figure 5 the design curve (mean - 2 standard deviations) and the upper limit of the test results (mean + 2 standard deviations) are compared with the simulated S-N curve for an as welded T-joint. It can be seen that the simulated curve, which is based on a mean crack growth rate, falls well between the two curves.
4.3 Scale effect

In figure 6 results of calculations for three thicknesses (T=6.3, 16 and 32 mm) are compared with test results. It can be concluded that the simulated S-N curves for T=16 and 32 mm are in agreement with the test results. For T=6.3 mm the agreement is not so good. This is probably due to the fact that for small thicknesses initiation, which is not taken into account in the model, forms an essential part of the total lifetime.

4.4 Weld toe radius

Most of the tests for T=32 mm were carried out for the as welded situation. In the calculations \( \rho=0.5 \) mm is assumed as an average value. For a limited number of tests the weld toe was ground. In figure 7 the results of the calculations for the as welded and the ground (\( \rho=4 \) mm) situation are compared with the test results. For the experiments an increase up to 150% was found (Dijkstra et al [8]) and for the calculation the increase is approximate 200%.

4.5 Environmental effects

To investigate the environmental effects, S-N curves for T=32 mm are simulated with the crack growth constants equal to eq. 9 and 10. In comparison with the test results the calculations for sea water seem to be conservative.

5. General remarks

The FM approach for fatigue life and crack growth of welded tubular steel structures is described. Several of the assumptions made in the fatigue crack growth model are validated with observed crack growth behaviour described in literature. The advantage of the FM approach is that it can be used as a design tool to study special effects and to simulated S-N curves. An other advantage is that it can be used as a maintenance tool. Special attention is given to the use of the model as a design tool. S-N curves are simulated and compared with a design curve and experimental results. With these calculations the influence of the scale of the joint, the radius of the weld toe (as welded and ground) and the environment are predicted quite well.

6. Symbols

a  crack depth  
c  half crack width
C,m  material crack growth parameters
K  stress intensity factor
\( \Delta K \)  stress intensity factor range
L  weld width
M  correction factor for the flat plate
\( M_k \)  correction factor for the weld geometry
N  number of cycles
R  stress ratio
T  plate thickness
Y  correction factor depending on geometry and loading conditions
\( \beta \)  diameter ratio
\( \varepsilon_{HS} \)  hot-spot strain (perpendicular to the weld toe)
\( \varepsilon_{HS} \)  extrapolated strain (parallel to the weld toe)
\( \theta \)  
- weld toe angle  
\( \rho \)  
- toe radius  
\( \sigma \)  
- applied stress  
\( \Delta \sigma \)  
- stress range  
\( \sigma_{HS} \)  
- hot-spot stress  
\( r \)  
- thickness ratio  
\( \omega \)  
- 3D reduction factor for 2D \( m \), \( k \)-values  
\( \Phi \)  
- elliptical integral of the second kind  
\( a \)  
- index for depth direction  
\( b \)  
- index for bending  
\( c \)  
- index for width direction  
\( f \)  
- index for final  
\( i \)  
- index for initial  
\( m \)  
- index for membrane

7. References


8. Figures

Figure 1. Semi-elliptical crack at weld toe.

Figure 2. Aspect ratios during crack growth for four different initial cracks.
Figure 3. Comparison calculated crack growth in depth direction for different $\omega$-reduction factors and test results.

Figure 4. Comparison calculated crack growth in width direction for different $\omega$-reduction factors and test results.
Figure 5. Comparison experimental scatter band and simulated mean S-N curve for as welded 32 mm T-joints.

Figure 6. Comparison simulated S-N curve and test results: scale effects.
Figure 7. Comparison simulated S-N curve and test results: toe radius effect.

Figure 8. Comparison simulated S-N curve and test results: environmental effect.
FATIGUE DAMAGE ACCUMULATION IN OFFSHORE TUBULAR STRUCTURES UNDER STOCHASTIC LOADING

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Summary

The fatigue life of offshore tubular structures under stochastic loading is studied in the present investigation. Fatigue test series with various types of stochastic loading that are realistic in relation to offshore structures have been carried through. The experimental investigation comprises both test series on full-scale tubular joints and test series on smaller welded test specimens.

The test series that have been carried through until now show a significant difference between constant amplitude and variable amplitude fatigue test results. For the welded plate test specimens, approx. 100 fatigue tests have been carried out, and the values of the Miner sum that were obtained in the variable amplitude test series, generally vary in the range 1/3 to 2/3. For the full-scale tubular joints, the number of test results is at present too limited to draw final conclusions. However, the results obtained until now indicate a value of the Miner sum of \( M \approx 0.8 \) for the variable amplitude tests.

1. Introduction

One of the problems in the design of offshore tubular steel structures that has attracted increased attention over the last few years is the problem of fatigue damage accumulation. Codes and specifications normally give simple rules, using a Miner summation and based on the results of constant amplitude fatigue tests, [3,4,5]. However, the actual load situations for offshore structures deviate very much from a constant amplitude loading, and the need for a better understanding of the fatigue behaviour under more realistic fatigue loading conditions is obvious.

On this background, an investigation of the fatigue life of offshore steel structures has been initiated at the Dept. of Structural Engineering of the Technical University of Denmark. The primary purpose of this project is to study the fatigue life of offshore steel structures under various types of stochastic loading that are realistic in relation to offshore structures.

The experimental investigation comprises both test series on full-scale tubular joints and test series on smaller welded test specimens. In the fabrication of the tubular joints, it has been emphasized that fabrication procedures, dimensions, materials, welding, quality control etc. correspond as precisely as possible to actual structures, constructed recently in the North Sea.

One of the special topics that is dealt with is the fatigue life of tubular joints that have been repaired. During in-service inspection of offshore structures, fatigue cracks are often found, and subsequently repaired. The fatigue life of such repair-welded tubular joints is studied, and compared with the original strength of the joint.

In the load simulation in the fatigue tests of the present investigation, a one-step Markov model is used. Both narrow-banded and broad-banded spectra, with irregularity factors ranging from 0.70 to 1.00, are investigated.

The materials that have been used until now in the fabrication of the test specimens have been ordinary offshore structure steels. However, the majority of the test specimens – both tubular joints and welded plate specimens – to be tested in the remaining parts of the investigation, will be in high strength steel with a yield strength of 700–900 MPa and with high weldability and toughness properties.
2. Experimental investigation

Test specimens

With the dimensions that have been chosen for the tubular joints, these correspond to a large number of the joints in the platforms of the Tyra Field. This field is, with 9 fixed platforms, one of the biggest Danish oil and gas fields in the North Sea. Compared to the largest joints in these platforms, the actual test joints are approximately half size.

The test specimens are carried out as double T-joints. This gives for each test specimen four critical areas with respect to fatigue cracks, since these may be expected to develop at the edge of the weld between the main tube and the secondary tube, in or near the symmetry plane of the test specimen. The test joint is loaded in bending, as may be seen in Fig. 1, [2].

In the fabrication of the tubular joint test specimens, it has been emphasized that fabrication procedures, dimensions, materials, welding, quality control etc. correspond as precisely as possible to actual structures, constructed recently. This means that the tubular joints tested can be considered to be representative for a large number of the platforms built recently in the North Sea in moderate water depths. The test specimens have been fabricated from seamless tubes. The material quality of both main tube and secondary tube is API 5LX GR.X-52N with a yield stress of $f_y = 363 - 381$ MPa and an ultimate tensile strength of $f_u = 506 - 548$ MPa.

The test series on smaller welded joints are carried out on test specimens that consist of a 40 or 90 mm wide main plate with two transverse secondary plates welded to the main plate by means of full penetration butt welds. This also gives four critical areas with respect to fatigue for these test specimens, since the fatigue crack will initiate at the edge of the weld and propagate into the main plate. Two different dimensions are used for these test specimens, to study the size effect. The applied loading in these tests is a central normal force in the main plate. The test specimens are shown in Fig. 2, [2].

The material used in these test series is St. 52-3, (Fe 510 C), according to DIN 17100, with a yield stress, $f_y = 400-489$ MPa, and an ultimate tensile strength, $f_u = 537-575$ MPa.

Test equipment and test procedure

In the test equipment used in the investigation on the tubular joints, the test specimen has a fixed support in the central plane, whereas the rest of the test specimen is free to move. The test joint is loaded in in-plane bending, using a 125 kN servo-controlled hydraulic actuator between the two secondary tubes. Strain gages are used to determine the stresses in the test specimens. This includes determination of the hot spot stresses in both main tube and secondary tube, as well as the stress distribution along the main tube in the symmetry plane and stresses in a number of points near the joint.

Furthermore, the stresses in the most critical areas with respect to fatigue are determined by use of thermoeelastic SPATE-equipment, and from finite element analysis. Fatigue crack propagation during the test is determined by use of AC-potential drop technique.

In the test series on the smaller welded joints, the tests are carried out in two fixed test frames, one with a capacity of ± 100 kN and the other with a capacity of ± 500 kN. The equipment — actuators, computers, valves, etc. — have been chosen so that a high frequency is possible in the tests of these series. With the smaller welded joints, tests may be run with frequencies of up to 100 cycles per second in tests with stochastic loading.

Small eccentricities due to the welding of the test specimens are inevitable in these
test series. This results in additional secondary bending stresses at the joint. Strain gages are used on all test specimens in these series to determine the resulting stresses from normal force and eccentricity moment.

3. Variable amplitude loading

The variable amplitude loading is generated by a computer programme "Tantalus", developed at the Dept. of Structural Engineering of the Technical University of Denmark, [1]. The programme simulates a stationary Gaussian stochastic process in real time, and it makes it possible to carry out the fatigue tests at high frequencies.

The load history used in the tests represents a stationary process, which does not directly correspond to a long term wave distribution. However, for each of the load spectra investigated, the different load levels, i.e. different equivalent stress ranges, will correspond to different sea states. Thus, the fatigue tests in each test series will give information about the fractional fatigue damage accumulation in the various sea states that form the total long term wave distribution.

Only the extremes of the process are needed, since the load course between consecutive extremes is considered unimportant. This enables use of a fast simulation algorithm, based on a one-step Markov model. In this load model, the next extreme to be generated $a^\ell$, will depend only on the present extreme, and not on the preceding load history, i.e. it has a one-step memory.

The total load range is divided into a discrete number of load levels $n$ (typically 64), which corresponds to the dimension of a two-dimensional Markov matrix. Each column and each row in the matrix contains cumulated transition probabilities, e.g. element $p_{ij}$ is the probability that the next load level is less than level $i$, given that the present load level is level $j$, or:

$$p_{ij} = P\{a^\ell < a_i | a^{\ell-1} = a_j\}$$

When the process is symmetric with respect to zero, only a triangular matrix is necessary. In this case, only maxima are generated, and minima are obtained by changing sign on every other maximum. The matrix element to be chosen, given a certain load level, is determined by use of a random number generator, as shown in Fig. 3.

The elements in the matrix have been determined numerically from the spectral density function of the wave elevation spectrum, [7, 8]. Three different matrices denoted BROAD64, PMMOD64, and NARROW64 have been used. BROAD64 was evaluated from a truncated white noise spectral density function, NARROW64 from a band limited block spectrum, and PMMOD64 from a modified Pierson-Moscowitz wave elevation spectrum. The main characteristics of these three spectra are given in Table 1. RMS- and RMC-values correspond to a maximum load level equal to the number of load levels, i.e. 64. The irregularity factor, $I$, is defined as the number of positive-going zero-crossings divided by the number of maxima.

Furthermore, initial test series were carried out using a spectrum, PM32 - a non-gaussian distribution - with similar characteristics as PMMOD64, but with more large cycles and a lower irregularity factor ($I = 0.75$) than PMMOD64.

The matrices have been determined using an upper truncation level at 4 times the standard deviation of the process. This corresponds to neglecting the highest 0.03% of the total probability. No lower truncation to avoid cycles below the fatigue threshold level has been introduced in the algorithm. This lower truncation is introduced with the rainflow count on the sequence of measured stresses. The lower truncation level $\Delta \sigma_{th}$ has been determined for each type of test specimen, by use of linear elastic fracture mechanics, on the basis of a choice of the threshold value of the stress intensity factor range, $\Delta K_{th} = 2.5 \text{ MPa} \sqrt{\text{m}}$, [9, 10].

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4. Fatigue test results

In the following are given the results that have been obtained until now in the test series on the small and the large plate specimens and on the tubular joints. For all three types of test specimens, initial test series with constant amplitude loading were carried out as a reference, and also to obtain the actual value of the exponent $m$ for calculation of the equivalent stress ranges of the tests with variable amplitude loading, cf. eq. 2.

For the plate specimens, test series with variable amplitude loading using the four different load spectra have been carried out. The results obtained in these test series are shown in the $S-N$ diagrams in Figs. 6–12. In the results from the variable amplitude tests, the stress parameter used is the equivalent constant amplitude stress range, $\Delta \sigma_e$, defined as

$$\Delta \sigma_e = \left[ \frac{\sum (n_i \cdot \Delta \sigma_i^m)}{\sum n_i} \right]^{1/m}$$

(2)

where

$\Delta \sigma_i$ is the number of cycles of stress range $\Delta \sigma_i$, obtained by rainflow counting

$m$ is the slope of the corresponding constant amplitude $S-N$ regression line.

The number of tests on the full-scale tubular joints that has been carried out until now is insufficient to make possible a statistical treatment of the test data. However, these results are plotted in the $S-N$ diagram shown in Fig. 13.

5. Fatigue test observations

When comparing the results obtained, it appears that there is a significant difference between constant amplitude and variable amplitude fatigue test results. For the test series on both small and large plate specimens, and with the different load spectra, there is a clear indication, that the fatigue life is shorter at variable amplitude loading than at constant amplitude loading for the same stress level. The difference in fatigue life is in the following quantified by the Miner sum, $M$, determined as the number of cycles to failure at variable amplitude loading, $N_{va}$, divided by the number of cycles to failure at constant amplitude loading, $N_{ca}$, at the same stress level. When the slope of the linear regression $S-N$ lines from variable amplitude and constant amplitude tests are not identical, $M$ will be a function of the stress level. In Table 2, values of the Miner sum calculated at different stress levels from the regression $S-N$ lines, are given for the test series on plate specimens with the spectra BROAD64, PM32, and PMMOD64.

As may be seen from Table 2, the Miner sum corresponding to failure in the variable amplitude test series, varies in the range $1/3$ to $2/3$ in all but one case. However, it should be emphasized that the values given in Table 2 are based on a limited number of fatigue tests, approx. 100.

Due to the scatter in the test results and the number of the tests and due to the limited number of test series carried out, it is not possible to draw final conclusions concerning the differences in variable amplitude fatigue life with the different load spectra. However, there seems to be a tendency that the Miner sum decreases with the irregularity factor of the spectrum.

With respect to the tubular joints, the number of test results is at present too limited to make possible any significant statistical analysis. If best fit lines are determined, assuming a slope of $m = 3$ for both constant amplitude and variable amplitude tests,
A Miner sum of $M \sim 0.8$ is found for the variable amplitude tests. A comparison of the results obtained in the test series on the small and the large plate specimens gives some information about fatigue size effects for these types of structural elements. For both constant amplitude tests, and variable amplitude tests with the spectrum BROAD64, it was found that the small plate specimens have a fatigue life that is 20–50% longer than that of the large plate specimens, depending on the stress level. However, for the variable amplitude tests with the spectrum PM32, the opposite behaviour was observed. In this case the fatigue life of the large specimens was found to be 10–30% longer than that of the small specimens.

Geometry correction factors, $F_G$, accounting for the effect of stress concentration due to geometrical discontinuity, for the two types of specimens, have been calculated by Yamada & Agerskov, [9, 10]. Depending on the crack size, the values of $F_G$ for the small specimens are 10–15% lower than the corresponding values for the large specimens. This difference corresponds to approx. 30–50% longer fatigue life for the small specimens, which is in good agreement with the observations in the test series with constant amplitude loading and with variable amplitude loading, using the spectrum BROAD64. However, these analytical results do not correspond to the test results obtained from the test series with variable amplitude loading, using the spectrum PM32. The question of the size effects will be further evaluated in the remaining test series of the investigation.

The cycle counting method that has been chosen for the analysis of the stress history, "rainflow counting", is usually recommended for broad band loading, [6]. Compared to simple range counting, rainflow counting will give more large cycles when used on a broad band spectrum, whereas the two methods with narrow band loading will yield practically the same result. The difference, which is illustrated in the damage diagram in Fig. 14, means that the Miner sum in broad band loading will be even lower by using simple range counting, compared to rainflow counting.

6. Conclusions

The fatigue test series that have been carried through until now in the present investigation show a significant difference between constant amplitude and variable amplitude fatigue test results. For the variable amplitude tests, the stress parameter used is the equivalent constant amplitude stress range, $\Delta \sigma_e$, and the difference between the constant amplitude and variable amplitude test results is quantified by the Miner sum, $M$, determined as the number of cycles to failure at variable amplitude loading divided by the number of cycles to failure at constant amplitude loading, at the same stress level. For the welded plate specimens, approx. 100 fatigue tests have been carried out. The values of the Miner sum that were obtained in the variable amplitude test series, generally vary in the range 1/3 to 2/3. For the tubular joints, the number of test results is at present too limited to draw final conclusions. However, the results obtained until now indicate a value of the Miner sum of $M \sim 0.8$ for the variable amplitude tests.

A comparison of the results obtained in the various test series on plate specimens with variable amplitude loading seems to indicate that the Miner sum decreases with the irregularity factor of the spectrum.

In the remaining parts of the investigation, further variable amplitude tests on the welded plate specimens will be carried out to produce a more comprehensive basis for the conclusions. For the tubular joints that have been tested, the fatigue cracks have been repair-welded according to specifications used presently in the North Sea, and the fatigue tests will be repeated to study the fatigue life of repair-welded tubular joints. Finally, test series are planned on both tubular joints and welded plate specimens in high strength steel with a yield strength of 700–900 MPa and with high weldability and toughness properties.
7. Acknowledgements

The present investigation is a part of a larger research program on the fatigue life of offshore structures under various types of spectrum loading and under various corrosion conditions. The funding for this program is provided by the Danish Technical Research Council, the Danish Council of Technology, the Nordic Fund for Technology and Industrial Development, and the Technical University of Denmark, who are gratefully acknowledged.

8. References

Table 1. Characteristics of load spectra used in the investigation.

<table>
<thead>
<tr>
<th>Spectrum</th>
<th>BROAD64</th>
<th>PMMOD64</th>
<th>NARROW64</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of load levels</td>
<td>64</td>
<td>64</td>
<td>64</td>
</tr>
<tr>
<td>Irregularity factor, I</td>
<td>0.745</td>
<td>0.817</td>
<td>0.987</td>
</tr>
<tr>
<td>Minimum load range</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Maximum load range</td>
<td>63</td>
<td>63</td>
<td>63</td>
</tr>
<tr>
<td>RMS (rainflow count)</td>
<td>18.6</td>
<td>19.8</td>
<td>22.9</td>
</tr>
<tr>
<td>RMC (rainflow count)</td>
<td>21.5</td>
<td>22.6</td>
<td>25.2</td>
</tr>
</tbody>
</table>

Table 2. Values of Miner sum, M, at different stress levels, $\Delta \sigma$.

<table>
<thead>
<tr>
<th>Spectrum</th>
<th>Plate Specimen</th>
<th>$\Delta \sigma = 100$ MPa</th>
<th>$\Delta \sigma = 200$ MPa</th>
<th>$\Delta \sigma = 300$ MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>BROAD64</td>
<td>Small</td>
<td>0.52</td>
<td>0.42</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>Large</td>
<td>0.37</td>
<td>0.42</td>
<td>0.46</td>
</tr>
<tr>
<td>PM32</td>
<td>Small</td>
<td>0.35</td>
<td>0.43</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>Large</td>
<td>0.61</td>
<td>0.66</td>
<td>0.69</td>
</tr>
<tr>
<td>PMMOD64</td>
<td>Small</td>
<td>0.92</td>
<td>0.62</td>
<td>0.49</td>
</tr>
</tbody>
</table>

Fig. 1. Test specimen from investigation on full-scale, double T-joint.
Fig. 2. Test specimens from investigation on smaller welded joints.

Fig. 3. Principle in variable amplitude load simulation.

Fig. 4. Density of maxima for the three spectra used.
150 Peaks simulated with BROAD64

Fig. 5. Example of load history. 150 extremes generated by use of BROAD64.

SN-curve for small test specimens. Constant amplitude loading.
Best fit: \( \log(N) = 12.476 - 2.851 \log(\Delta \sigma) \)

Fig. 6. Results obtained from constant amplitude tests on small plate specimens.

SN-curve for large test specimens. Constant amplitude loading.
Best fit: \( \log(N) = 12.625 - 3.063 \log(\Delta \sigma) \)

Fig. 7. Results obtained from constant amplitude tests on large plate specimens.
Fig. 8. Results obtained from variable amplitude tests with PM32 spectrum on small plate specimens.

SN-curve for small test specimens. Stochastic loading with PM32. Best fit: \( \log(N) = 11.496 - 2.586 \log(\Delta \sigma_e) \)

Fig. 9. Results obtained from variable amplitude tests with PM32 spectrum on large plate specimens.

SN-curve for large test specimens. Stochastic loading with PM32. Best fit: \( \log(N) = 12.374 - 2.945 \log(\Delta \sigma_e) \)
SN-curve for small test specimens. Stochastic loading with BROAD64.
Best fit : \( \log(N) = 12.790 - 3.149 \log(\Delta \sigma) \)

Fig. 10. Results obtained from variable amplitude tests with BROAD64 spectrum on small plate specimens.

SN-curve for large test specimens. Stochastic loading with BROAD64.
Best fit : \( \log(N) = 11.999 - 2.867 \log(\Delta \sigma) \)

Fig. 11. Results obtained from variable amplitude tests with BROAD64 spectrum on large plate specimens.
Fig. 12. Results obtained from variable amplitude tests with PMMOD64 spectrum on small plate specimens.

Results obtained from variable amplitude tests with PMMOD64 spectrum on small plate specimens.

Fig. 13. Results obtained in tests on tubular joints. PMMOD32 spectrum has been used in variable amplitude tests.

Results obtained in tests on tubular joints. PMMOD32 spectrum has been used in variable amplitude tests.

Fig. 14. Comparison of damage densities by rainflow counting and by simple range counting.
INFLUENCE OF CORNER RADIUS AND WELD DIMENSIONS ON THE STRESS CONCENTRATION FACTORS OF SHS T- AND X-JOINTS

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Summary

This paper deals with the influence of the corner radii and weld dimensions on the stress concentration factor (SCF) of a joint. The hot spot stress ranges, determined by the nominal stress ranges multiplied by the SCF, governs the number of cycles to failure. The SCF can be determined by applying parametric formulae, which describe the relationship between the non-dimensional parameters ($\beta$, $2\gamma$ and $\tau$) and the stress concentration factors. However, these formulae are determined for certain corner radii and weld sizes only. As different manufacturers supply tubes with different corner radii and as different welding standards and weld types demand different weld dimensions, it is important that the influence of variation of corner radii and weld dimensions on the SCFs as determined by the parametric formulae is known.

1. Introduction

As part of an extensive ECSC-CIDECT investigation into the fatigue behaviour of rectangular hollow section joints, T- and X-joints have been studied. See Wingerde et al [5] and Puthli et al [6]. As part of CIDECT programme 7K additional studies are being carried out. The partners participating in this CIDECT programme are: Mannesmannröhren-Werke A.G., Düsseldorf, Verenigde Buizenfabrieken, Oosterhout, TNO Building and Construction Research, Rijswijk and the Delft University of Technology.

A design method based on the hot spot stress is currently proposed. The hot spot stress is determined by multiplying the nominal stress range by a so-called stress concentration factor. The stress concentration factor SCF is defined as the hot spot stress range $S_r$, divided by the corresponding nominal stress range $\sigma_r$: $SCF = S_r/\sigma_r$. When the SCFs for various types of load (axial load or in-plane bending) are known, the hot spot stress range for any arbitrary combination of loads can be determined as follows:

$$S_r = \sigma_{r_{m1}} * SCF_{m1} + \sigma_{r_{a1}} * SCF_{a1} + \sigma_{r_{m0}} * SCF_{m0} + \sigma_{r_{a0}} * SCF_{a0}$$ (1)
The nominal stress range \( \sigma_r \) is the stress range in the member determined from beam theory, without taking the stress discontinuity due to the presence of the joint into account. The stress concentration factor SCF depends on the global joint geometry. SCF formulae in terms of the geometrical joint parameters \( \beta, 2\gamma \) and \( r \), are determined from the results of numerical analyses on T- and X-joints loaded by in-plane bending moments as well as by axial forces. See Wingerde et al [1]. The number of cycles to failure can now be determined from appropriate \( S_r - N_r \) lines which relate the hot spot stress to the number of cycles to failure. The experiments served to calibrate the numerical work and relate the calculated or experimentally determined hot spot stresses to the number of cycles. See Wingerde et al [2].

However, the parametric study which forms the basis of the proposed design method concentrates entirely on butt welded joints with specific weld dimensions \( (w_1-t+2, w_0-t/2) \) and corner radii \( (r=t \text{ to } 2t, \text{ depending on the width of the member, see Table 1}). The need therefore arises to determine the effect of different corner radii and welds with different dimensions on the stress concentration factor, as determined by the parametric formulae.

As part of the CIDECT research programme 7K "Fatigue Behaviour of Uniplanar Joints" (Wingerde et al [4]) the influence of corner radii and weld sizes is determined. This is done by studying a large number of geometries with varying weld dimensions and corner radii, as well as fillet welded geometries. As a result, the range of application of the established parametric formulae can be extended.

2. Approach for the determination of the hot spot stress

2.1 Positions where stress concentration factors are determined

In order to allow superposition of load cases, consisting of forces and in-plane bending moments on both members, it is necessary to establish fixed positions where the SCFs are determined. The stresses are considered along five lines A to E on the chord and brace (see Figure 1), based upon previous investigations (Wingerde et al [5]). Only stresses along these lines were considered. A consequence of this approach is that the hot spot stresses found in this way may underestimate the 'true' hot spot stress if the direction of the principal stresses deviates from these lines. This is especially the case if the stress concentration is less pronounced. Here the stresses at other positions or in other directions or at the inside of the members may be higher, particularly for fillet welds and joints with \( \beta=1.0 \). Therefore, a minimum SCF of 2.0 is specified for \( SCF_{m1} \) and \( SCF_{m1} \). Because fixed positions are selected, the determination of the hot spot stress according to eq. 1. becomes possible.

2.2 Extrapolation carried out to the weld toe

The actual hot spot stress is determined by geometrical influences, the shape of the weld and the condition of the weld toe. However, the last two mentioned effects are difficult to determine. Moreover, these effects depend, to a large extent, on the welder. A geometrical hot spot stress definition based upon linear extrapolation was accepted in Working Group III of the ECSC Offshore Programme for joints in circular hollow sections. It was shown later that the stress distribution is non-linear for particular joints. Consequently, the IIW recommendations (IIW [10]) no longer specify any method of extrapolation (linear or quadratic).
Therefore, in the analyses, both a linear and a quadratic extrapolation method are used, from within limits as described in Figure 2. The region of influence of notch stress to be ignored is taken as 0.4 t (with a minimum of 4 mm), following the procedure for circular hollow sections. Although both methods have been applied to evaluate the FE results of the research programme, this paper will concentrate on the quadratic extrapolation method, because it can follow the nonlinear geometrical stress better and gives a smaller scatter in the $S_t-N_t$ lines based on the experiments. See Wingerde et al [3]. However, the difference between linear and quadratic extrapolation is small in most cases, typically 10 to 20% and the quadratic extrapolation tends to be more sensitive to small changes in measured or calculated data.

3. Numerical work

In order to obtain enough data to study the influence of the corner radii of brace and chord, as well as the weld dimensions without an excessive number of FE analyses, 7 basic geometries have been selected for the parametric study. These geometries, which are a subset of the geometries analysed for the determination of the parametric formulae, are listed in Table 1. Four geometries study the effect of the $p$ ratio for $2\gamma=16$ and $\tau=0.5$. For $\beta=0.7$ and $\tau=0.5$, two additional values of $2\gamma$ are considered, namely $2\gamma=12.5$ and $2\gamma=25$, in order to get an impression of the influence of $2\gamma$. The influence of $\tau$ is studied by including a joint with $\tau=1.0$ for $2\gamma=16$ and $\beta=0.7$. As a consequence of this choice of parameters, some sections are outside the normal manufacturing ranges. These cases, which all have $r_0/b_0>0.25$ or $r_1/b_1>0.25$ are shown as dashed lines in Figures 5 and 6 and are commented upon wherever necessary. Because of the definition of lines A to E, the position shifts with $r_1$, so that lines A and E coincide for $r_1=3.41*t_1$ and for larger brace corner radii even cross each other, thus influencing the SCF. For $r_1=3.41*t_1$, $SCF_A=SCF_E$.

All basic geometries are butt welded, axially loaded T-joints. For these geometries, the following exercises are carried out:

1. In order to investigate the influence of the weld, all basic geometries are investigated 3 times:
   - fillet welds ($w_0=w_1=t_1*/2$)
   - fillet welds with full penetration ($w_0=w_1=t_1*/2$)
   - butt welds ($w_0=t_1/2$, $w_1=t_1+2$), used in the principal investigation, which forms the basis for the parametric formulae.

   See Figure 3 for an overview of the 3 types of welds investigated. In this way, both the influence of the weld penetration (between fillet weld and fillet weld with full penetration) and of a change in weld dimensions (between butt weld and fillet weld with full penetration) could be investigated.

2. In order to study the influence of the brace corner radius, 4 FE analyses per basic geometry have been carried out with $r_1/t_1 = 1, 2, 3$ and 4 respectively.

3. In order to study the influence of the chord corner radius, 4 FE analyses per basic geometry have been carried out with $r_0/t_0 = 1, 2, 3$ and 4 respectively.

The parametric formulae for the brace (lines A and E) have been combined by always choosing the higher SCF of the two. See Wingerde et al [1]. For all cases considered in this paper, line A has higher SCFs than line E. Therefore, line E is not considered in this paper.
4. Results of the numerical investigation
4.1 Influence of the weld on the SCF
4.1.1 Influence of the weld penetration on the SCF
The first comparison, shown by the shaded symbols in Figure 4, is between a normal fillet weld \((w_0 = w_1 = t_1/2)\) and a fillet weld with the same external weld dimensions, but with full penetration. As can be seen in Figure 4, the influence of the full penetration on the SCF is negligible in all cases investigated. The fillet welded joints with full penetration and the fillet welds with the same weld dimensions and joint geometry but no penetration have virtually the same SCF.

4.1.2 Influence of the weld dimensions on the SCF
The influence of the size of the weld is more important (the unshaded symbols). Two cases were analysed, namely \(w_0 = t_1/2, w_1 = t_1 + 2\) (a typical butt weld size) and \(w_0 = w_1 = t_1 = 2\) (a typical fillet weld size, but with full weld penetration). For smaller values of \(t_1, t_1 + 2 = t_1/2\) (about 10% difference), so that the only change occurs for \(w_0\). For \(r = 1.0, w_1\) changes about 20%, from 14.5 to 17.68 mm. As can be seen in Figure 4 and Table 2, the difference in weld dimensions has an important influence on the SCF.

- Results
Line A (brace) The SCFs of the fillet weld dimensions were about 33% higher than those of butt welded joints with the same geometry. The recommended factor of 1.4 for the determination of the SCF of fillet welds for line A (Puthli et al [6]) is confirmed by the results.

Line B, C, D (chord) For \(\beta < 0.7\) the SCFs of fillet welded joints were generally slightly lower than for butt welded joints (15% on average). But for \(\beta = 0.7\) the SCFs were 35% lower. For \(r = 1.0\), the difference was much larger (50%). The SCFs in the chord were generally lower for fillet welded joints, but the differences with butt welded joints varied from 0 to 50%, depending upon the joint geometry.

Therefore, based upon the limited number of geometries investigated, no general correction factor can be given for the chord. As a result, the existing SCF formulae and correction factors for the SCF of 1.4 for the brace and 1.0 for the chord are recommended for fillet welds.

4.2 Influence of the corner radii on the SCF
For the corner radii, the ratio of SCF/SCF\(_{\text{formula}}\) is plotted against the radius divided by the radius used for the determination of the formulae, as presented in Table 1. This presentation in Figures 5 and 6 shows directly the influence of the corner radius compared to the accuracy of the formulae, as for \(r = r_{\text{formula}}\), the SCF obtained should be SCF\(_{\text{formula}}\). It should be noted that the formulae can sometimes give a deviation of up to 20%, compared to the FE analyses on which they are based. See Wingerde et al [1]. After exclusion of the unrealistic geometries \((r/b > 0.25)\), shown as dashed lines in Figures 5 and 6, the general tendencies can be given for the influence of \(r_1\) and \(r_0\) on the SCFs of line A, B, C and D. These tendencies are summarised in Table 3, by giving the relative change in SCF per relative change in corner radius, i.e. the slope of the lines in Figures 5 and 6. The slope is the change in SCF (in %) when \(r\) is changed by \(r_{\text{formula}}\) (for instance from 1 to 2 times \(r_{\text{formula}}\)).
4.2.1 Influence of the corner radius of the brace on the SCF

- Observations
Of all brace corner radii investigated, only a few cases differ substantially more than 20% from the formulae. This is only the case for \( r_i / t_i = 4 \) and as the SCFs are lower than the parametric formulae, the formulae are safe. The only exception is line D, \( 2\gamma = 25 \beta = 0.7 \). This is mainly caused by the parametric formulae, which slightly underestimate the SCF in this case.

- Results
Per line, a general tendency can be found by taking the average for the geometries investigated. This is tabulated in Table 3.

Line A (brace) A clear influence can be found, independent of the geometry. The influence is linear in \( r_i \) for all geometries investigated, allowing a correction factor to be established.

Line B (chord) The influence is smaller than for the brace and slightly less linear in \( r_i \).

Line C (chord) The influence of the corner radius of the brace on the SCF along line C is rather small for all geometries investigated.

Line D (chord) The average influence of the brace corner radius for line D is small, but varying for different geometries. Especially for larger values of \( \beta \) and \( 2\gamma \), there appears to be an increase of SCF for larger brace corner radii. No uniform correction factor can be given or seems necessary.

4.2.2 Influence of the corner radius of the chord on the SCF

- Observations
The comparison of the FE analyses with the parametric formulae shows aspects which are very similar to the observations for the influence of the brace corner radii. Only a few cases differ substantially more than 20% from the formulae. This is only the case for \( r_0 / t_0 = 4 \) and as the SCFs are lower than the parametric formulae, the formulae are safe. The only underestimation is found for line D, \( 2\gamma = 25 \beta = 0.7 \) (as was the case for the influence of the brace corner radius).

- Results
Line A (brace) An influence can be found, which is dependent on the geometry. The influence is not linear in \( r_0 \). A correction factor can be established, but in order to be accurate, \( \beta \), \( 2\gamma \) and \( r \) may have to be incorporated.

Line B (chord) The influence is strongly dependent on the geometry of the joint and not linear in \( r_0 \).

Line C (chord) The influence of the chord corner radius on the SCF along line C can be found. Especially for different values of \( 2\gamma \) and \( r \) (see Figure 6) the effect is much smaller and not linear.

Line D (chord) A clear, linear influence is found, very suitable for the establishment of a correction factor.

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5. Experimental data on the influence of the corner radii

No experiments have been carried out to verify the numerically determined influences of corner radii in this research programme. However, some experimental evidence is provided by tests carried out on K-joints. Some of the joints were made of hot finished sections, other joints were made from cold finished sections (the latter usually having a larger corner radius). In this case the cold formed sections had a slightly better fatigue behaviour. This is in agreement with the numerical investigations which generally predict lower SCFs for larger corner radii. See Figure 7 (Noordhoek et al [7], Mang [8]). For other geometries, the hot finished sections may have the advantage of a slightly larger wall thickness in the corners, which is known to decrease the SCFs. See Wingerde et al [9].

6 Discussion of results

The influence of the corner radii is usually within the scatter band of the parametric formulae, as shown in Wingerde et al [1]. For some lines, the correction factor would be dependent on the joint geometry (requiring extensive additional investigations to cover the complete range of validity of the parametric formulae) and be non-linear in r. Incorporating all the influences would complicate the formulae considerably. Furthermore, differences in manufacturing and fabrication of the joints tend to cause much larger differences (see Wingerde et al [2], [9]). The designer would have to specify the corner radius, which would limit the choice of the supplier.

For the influence of the weld, the existing correction factor for the brace is adequate. To profit from the favourable influence of the weld dimensions on the SCFs in the chord, additional research would be necessary.

7. Conclusions

- The weld penetration (ie full penetration fillet or normal fillet weld) does not have much influence on the SCFs.
- The influence of the fillet weld size (w_o - w_i - t/2) for the brace can be accounted for by multiplying the SCF found from the parametric formula (obtained for w_o - t/2, w_i - t/2 + 2) by 1.4. For the chord, the SCFs of fillet welded joints are generally slightly lower, but this is dependent on the geometry of the joint, so that a correction factor of 1.0 is currently recommended.
- The range of validity of the formulae can be extended for r/t = 1 to 4, without correction factors being necessary. This is confirmed by the experimental results.

8. Acknowledgements

The financial support by CIDECT, Mannesmannröhren-Werke A.G. and VBF Buizen B.V. is appreciated, as well as the permission to publish this paper by CIDECT.
9. Symbols

CIDECT Comité International pour le Développement et l'Étude de la Construction Tubulaire
ECSC European Community of Steel and Coal
FE Finite Element
IIW International Institute of Welding

- $N_r$: Number of cycles to failure
- $SCF$: Stress concentration factor
- $S_r$: Hot spot stress range $= \sigma_r \times SCF$
- $\sigma_r$: Weld throat thickness
- $b$: External width of member considered
- $r$: Outer corner radius of member considered
- $t$: Wall thickness of member considered
- $w$: Weld dimension parallel to member considered

- $\beta$: Brace to chord width ratio
- $2\gamma$: Width to wall thickness ratio of the chord
- $\sigma_r$: Nominal stress range (max. stress range according to beam theory)

Brace to chord wall thickness ratio

Indices

- $o$: chord
- $b$: brace
- $a$: axial stress
- $m$: in-plane bending stress

10. References


CIDECT report 7K-91\3
TNO-IBBG report BI-91-017
Stevin report 25.6-91-04 (to be published)

TNO-IBBG report BI-89-064
Stevin report 25.6-89-23
CECA convention 7210-SA/111 (confidential)

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Stevin report 25.6-89-37
CECA convention 7210-SA/111 (confidential)

Stevin report 6-80-4
ECSC convention 6210 KD-1-103


Stevin report 6-87-11
CECA convention 7210-SA/111


11. Tables and Figures

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<tr>
<th>CHORD</th>
<th>BRACE</th>
<th>Non-dimensional parameters</th>
<th>Weld Size (external)</th>
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<tr>
<td>b₀</td>
<td>t₀</td>
<td>r₀</td>
<td>b₁</td>
</tr>
<tr>
<td>200</td>
<td>12.5</td>
<td>25.0</td>
<td>50</td>
</tr>
<tr>
<td>200</td>
<td>12.5</td>
<td>25.0</td>
<td>80</td>
</tr>
<tr>
<td>200</td>
<td>12.5</td>
<td>25.0</td>
<td>110</td>
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<td>16.0</td>
<td>140</td>
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<tr>
<td>200</td>
<td>16.0</td>
<td>32.0</td>
<td>140</td>
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<tr>
<td>200</td>
<td>12.5</td>
<td>25.0</td>
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Table 1. Summary of the basic geometries for the numerical work
### Table 2. Summary of the influence of the weld

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<tr>
<th></th>
<th>LINE A</th>
<th>LINE B</th>
<th>LINE C</th>
<th>LINE D</th>
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<tr>
<td>Minimum</td>
<td>1.32 (1.22)</td>
<td>0.66(^1) (0.45)</td>
<td>0.73(^2) (0.63)</td>
<td>0.79 (0.62)</td>
</tr>
<tr>
<td>Maximum</td>
<td>1.37</td>
<td>0.99</td>
<td>0.93</td>
<td>0.88</td>
</tr>
<tr>
<td>Average</td>
<td>1.33</td>
<td>0.84</td>
<td>0.85</td>
<td>0.83</td>
</tr>
</tbody>
</table>

Note:  
\( r=1.0 \) in brackets and not used to determine to average.  
\(^1\) \( \beta = 0.7, \ 2\gamma = 12.5 \) or 16.0  
\(^2\) \( \beta = 0.7, \ 2\gamma = 16.0 \)

### Table 3. Summary of the influence of the corner radius

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<th>LINE C</th>
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<td>0.75</td>
<td>1.07</td>
<td>1.04</td>
<td>1.03</td>
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</tr>
<tr>
<td>2.86</td>
<td>0.90</td>
<td>0.91</td>
<td>0.90</td>
<td>0.92</td>
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<td>( \Delta SCF(%) ) ( \Delta r ) ( (\text{SLOPE}) )</td>
<td>-12</td>
<td>-9</td>
<td>-9</td>
<td>-14</td>
</tr>
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<td>| | | | |</td>
<td></td>
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<table>
<thead>
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<th>( r/r_{\text{formula}} )</th>
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<td>2.00</td>
<td>0.78</td>
<td>0.84</td>
<td>0.88</td>
<td>1.15</td>
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<tr>
<td>( \Delta SCF(%) ) ( \Delta r ) ( (\text{SLOPE}) )</td>
<td>-14</td>
<td>-9</td>
<td>-4</td>
<td>+2</td>
</tr>
</tbody>
</table>
FIGURE 1. MEASURING LINES

FIGURE 2. EXTRAPOLATION

FIGURE 3. TYPES OF WELD INVESTIGATED

FIGURE 4. INFLUENCE OF THE WELD PENETRATION AND WELD DIMENSION ON THE SCF
Figure 5. Influence of the corner radii on the SCF for various values of $\beta$
FIGURE 6. INFLUENCE OF THE CORNER RADII ON THE SCF FOR VARIOUS VALUES OF $2\gamma$ AND $\tau$
FIGURE 7. TEST RESULTS ON K-JOINTS, TAKEN FROM [7], [8]
CHORD: 100x100x4 mm, BRACE: 60x60x4 mm
R=-1, Fe 37, FILLET WELDS
RECENT DEVELOPMENTS IN THE FIRE ENGINEERING DESIGN OF CONCRETE FILLED HOLLOW SECTION COLUMNS

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SUMMARY

Composite structures are increasingly used in building constructions [16]. They ideally combine the advantages of both their components: Reduced overall dimensions, easy prefabrication and connecting for steel and reduced thermal diffusivity, easy shaping of cross-sections, reduced U/A-ratio and great stiffness. Since 1974 a lot of fire-tests on all kinds of composite concrete/steel-elements, especially on columns, have been carried out in France, Belgium, UK, Canada, Finland and Germany [1 + 6]. The results were published and are therefore available. The aim of these activities was not only to determine the fire-resistance. It was also intended to find out the basis of calculation methods and to develop the technology.

Finalizing one of these projects, in December 1985 a fire-test was executed in a building situated in Stuttgart (FRG) [7]. Different types of composite slabs, beams and columns were subjected to a real ‘wooden crib’ fire.

1. FIRE-TESTS

1.1 Tests on single elements

The fire-resistance of concrete-filled tubes is primarily governed by the fire-resistance of the concrete core. Due to the restraining of the higher thermal elongation of steel, which is quicker heated-up than concrete, the tube takes more of the column force at the beginning of the fire-action. This may cause deloading or even decompression of the concrete. After some 15 minutes of ISO-fire-action the tube yields and nearly all of the load must be taken by the concrete core. This load redistribution can be calculated [8] and is shown schematically in Fig. 1.

Local buckling of the tube may happen. If the concrete core is reinforced, this effects the deflections only slightly but it does not affect the stability of the composite column. With increasing temperatures the column’s stiffness decreases. Finally the column fails due to the loss of stiffness, which causes a stability failure with remarkable deflections.

For fire-design it is preferable to have a low load-bearing capacity of the tube compared to that of the concrete core.
A minimum amount of longitudinal reinforcement is necessary to control the crack-width during the temporary decompression. Otherwise a single widely opened crack may develop, which then leads to a plastic hinge, when the tube yields. In this case the deflection of the column becomes so high that this immediately causes a rapid failure. The recommended reinforcement of the core may consist of bars, steel fibres or steel profiles. Typical sections are shown in Fig. 2.

In order to attain a fire-resistance of at least 60 minutes load-reductions are necessary. In some countries the bearing capacity is therefore only determined for the tube without taking into account any capacity of the concrete. In both cases the admissible load of the composite column depends on the required fire-resistance. The temperature in the steel is not a criterion for the ultimate limit state. The load-bearing capacity of the steel tube may disappear completely during a fire. The composite column does not fail because the inner core acts like a reinforced concrete member. Compared to a concrete column, the core of a concrete filled hollow-section is heated up a bit slower because of a shield-effect against radiation of the outer steel sheet and the higher water content of the enclosed concrete.

Openings in the tubes are important because they avoid excessive steam pressure inside the tube. Experience shows that re-drilling of the holes after the hardening of the concrete is very reliable.

1.2 Columns Built in Structures

Columns in frames which are prevented from swaying are generally centrically loaded or at least they are assumed to be, in order to simplify the safety check. While an isolated fire in only one storey takes place the last rotations of the columns are significantly restrained. This is due to the fact that the stiffness of the hot column decreases whereas the stiffness of the connected elements beneath and also partly of the ones above remains unchanged. Only the ceiling is strongly heated—up whereas the floor is protected by the cool, fresh air needed for combustion, ashes or other materials. Tests on columns were carried out by controlling the last rotations of the columns with the result that equivalent imposed or restraining actions took place.

At the beginning of the fire-action a nearly constant rate of dilatation of the heated ceiling causes imposed bending-moments, which are released when the outer steel yields or the concrete loses strength and stiffness. After some 30 minutes the rate of displacement decreases due to increasing deflections of the ceiling. Therefore and because of the on-going loss of the column’s stiffness the restraint decreases. It disappears completely when a certain flexural deformation of the column is attained. Then the increasing deflection of the column is controlled by the connected beams or similar elements. There were no differences
observed between stiff connections which join flanges and webs of the beams, and soft connections which join only the webs with bolted shear plates. The measured fire-resistance corresponded very well with testing the columns with fixed ends and without displacements. For edge-columns of a multi-bay frame also node rotations due to beam deflection have to be considered. The tests showed that the fire-resistance for this case, corresponded to that of columns tested with one end fixed and the other one pin-jointed.

The results of these investigations lead to an improved and more economical fire-design of columns in the EUROCODES No.2 (concrete structures), No.3 (steel structures) and No.4 (composite structures).

1.3 Connections between Columns and Beams

Composite beams and columns may be fire-resistant without any further protection like cladding or spray. It therefore was logical to develop connections which also do not need protection after having mounted the elements. Typical connections are shown in Fig. 2.

The connecting element, shear plate or corbel, has to be connected in both cases to the core of the concrete-filled hollow section column, because only this part of the cross-section remains strong under fire-attack, whereas the tube looses strength almost completely. The loads are transferred from the beams to the columns by vertical plates (Fig. 3a) or corbels (Fig. 3b). The corbel in Fig. 3b has enough volume so that the heating is reduced.

1.4 Results of a Real Fire-Test

A laboratory in Stuttgart served to demonstrate the fire-resistance of composite structures [7]. It was also intended to study which kinds of destructions occur and by which means the structure can be repaired. All the structural elements were designed for 90 minutes fire-resistance. Wood-cribs were arranged in one half of the top-storey. They represented a fire-load of 30 kg/m² in one room and 45 kg/m² in an other. This was nearly twice the value of the real fire-load in laboratory-rooms. Another fire in a room with 15 kg/m² wood cribs was made to study fire-propagation along the facade. The measured temperatures were up to 1100 °C. Complete burning-out was allowed. The slab was loaded with barrels containing water, to have the structure nearly under service load conditions.

The structure reacted even better than expected. Cracking was observed at columns and at beams, as well as falling-down of concrete. The steel sheet of the slab in some types of applied construction moved completely away from the concrete and could not be used anymore. The concrete of the slab showed some spalling. All deformations to be seen during the fire, had disappeared afterwards. Columns and beams needed only little repair, like closing of cracks and
arrangement of new concrete where it had fallen down. Hollow sections had only to be cleaned. Additional reinforcement or substitution of outer steel was not necessary.

2. ASSESSMENT BY CALCULATION

For generalization of test results and for a real fire-design of load-bearing components, computer programmes have been developed. For an example the computer programme STABA-F is described in the following. It was developed to support the afore mentioned investigations theoretically and numerically. Both material and geometric nonlinearities were considered.

The heat-transfer from the fire to the structural element depends on

- material and the surface of the member,
- colour of the flames,
- geometry and material properties of the furnace walls,
- ventilation conditions.

Heat conduction is described by the well-known Fourier-equation, valid for homogeneous and isotropic materials. Applied to composite structures some simplifications are necessary:

- water vaporizes once it has reached its boiling-point,
- movement of the steam is put together with other effects,
- consumption of energy for vapourizing the water, and other similar peculiarities are taken into account in a simplified way by suitable design values for the specific heat-capacity of concrete with up to 200 °C,
- concrete is taken into account in a simplified way as a homogeneous material, the heterogeneous structure as well as capillary pores and internal cracks are lumped together.

A finite element method in connection with a time-step integration is used to calculate the temperature distribution in the section. The time-steps have to be chosen quite small, because the characteristic values of the thermal conductivity, specific density and specific heat-capacity very much depend on the temperature. To determine the temperature distribution, a rectangular network is preferred with a maximum width of less than 20 mm. In the region of the structural steel it is advantageous to reduce the width of the network to the thickness of the structural steel-profile. The elements of the cross-sectional discretization have corresponding thermal materials of steel or concrete.

Knowing the temperature distribution and with regard to the following simplifications

- the Bernoulli-Navier hypothesis,
- only uniaxial stresses are taken into account, shear stresses are neglected,
- there is no slip between concrete and steel,
- the stress-strain relationships are nonlinear elastic.
the non-linear moment/curvature-relationship (stiffness) is calculated.

Due to the element temperatures, the thermal strains for the cross-section elements, are derived by using the temperature dependent thermal strain for concrete and steel. In case of fire the material normally is subjected to a transient process with varying temperatures and stresses. To get material data of direct relevance for fire transient creep-tests are carried out. During the test the specimen is subjected to a certain constant load and a constant heating rate. From these tests uniaxial stress-strain characteristics are obtained which include the temperature-dependent elastic strains and the comparatively large transient creep strains. Stress-strain relationships at elevated temperatures for reinforcing steel are shown in Fig. 4.

An accurate evaluation of the load-bearing behaviour has taken into account the influence of mechanical (nonlinear moment/curvature relationship) and geometrical (2nd order theory) nonlinear interaction between loads and deformations. To determine bending moments, shear force, slope of the bar and deflection, the method of transferring these values from one division to the next is used. The applied computer programme STABA-F gave the results of Figure 5, comparing calculated and tested fire-resistance times.

3. FIRE-DESIGN - AIDS AND RULES

Derived from the test-results and from the accompanying calculations design-charts and rules for the practical fire-design have been built-up during the last few years. These tools lead to an increasing application of composite structures in most European countries; exceptions are for instance UK and Sweden. CIDECT for instance financed two projects [13] and [15], where admissable loads for certain concrete filled hollow-section columns were given depending on buckling length and required fire-resistance. An example of those charts is introduced in Fig. 6. The Studiengesellschaft für Anwendungstechnik von Eisen und Stahl e.V., Düsseldorf, supported a similar project. All these charts are only applicable on centric loading, which is normally uncommon in building practice.

In Germany there were built-up official design guides, which will soon become part of the national building code for fire-design DIN 4102. Basis of these results are the tests, which were carried out in the past, and the theoretical work, which completed the knowledge of the fire-behaviour. These fire-design tools give minimum section sizes, maximum load factors and constructional details, which have to be kept, if a structure can be put in fire classes. These tabulated data are adopted to the draft of EUROCODE No. 4 (Design of Composite Structures), Part 10 (Structural Fire-Design). The design tables are valid for centric and
eccentric loading. The load reduction for eccentric loading is the same for 'cold' and 'hot' states:

$$\frac{N_{cr,e}(\theta)}{N_{cr}(\theta)} = \frac{N_{cr,e}}{N_{cr}}.$$  \hspace{1cm} (1)

The fire-design of structures according to tables is the lowest level of accuracy and effort in the EUROCODES. Therefore the results are normally on the safe side. More advanced methods, so-called 'simple calculation methods', where equations, which normally have no physical background, lead to results for the load-bearing capacity of structures after a given time of ISO 834-fire exposure. Examples for these methods in EUROCODE No. 4 were built-up for composite beams, composite slabs and for composite columns, providing at this stage for concrete filled hollow-sections columns in braced frames the afore mentioned design charts [13]. The appliance of those charts was extended to eccentric loading by formula (1).

The afore mentioned simple solutions are only based on the ISO 834-temperature/time-curve. General calculation models [8 + 11] are the most sophisticated solutions at the highest level. They allow a determination of the development and distribution of the temperature inside the structural elements and the mechanical behaviour of the structure or a part of it. These models can be used for single elements (columns, beams, slabs), sub-assemblies or entire structures. At this stage only calculations under ISO 834-fire give reliable results. The actual research work is now focussed on the behaviour of structures under 'natural fires'.

REFERENCES


Steel tube QR 260x7.1
Concrete
Reinforcement

\[ f = 240 \text{ MPa} \]
\[ f_c = 30 \text{ MPa} \]
\[ f_y = 420 \text{ MPa} \]

Fig. 1 Calculated stress distributions for a reinforced hollow-section after different times of ISO-fire-action.

Fig. 2 Typical cross-sections of concrete filled hollow section columns

Fig. 3 Beam-to-column connections
Fig. 4 Stress-strain relationships at elevated temperatures for structural steel [8]

<table>
<thead>
<tr>
<th>Type of cross section</th>
<th>Number</th>
<th>$t_{\text{cal}}/t_{\text{test}}$</th>
<th>(t_{\text{mean}})</th>
<th>(t_{\text{max}})</th>
<th>Stand Devia.</th>
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<tr>
<td>Reinforced concrete</td>
<td>47</td>
<td>0.573</td>
<td>0.966</td>
<td>1.959</td>
<td>0.207</td>
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<tr>
<td>I-profiles, concreted</td>
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<td>0.831</td>
<td>1.055</td>
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<td>I-sections, embedded</td>
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<td>0.068</td>
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<td>1.042</td>
<td>1.509</td>
<td>0.210</td>
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<tr>
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<td>0.861</td>
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<td>1.503</td>
<td>0.126</td>
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<tr>
<td>All tests</td>
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<td>0.573</td>
<td>1.016</td>
<td>1.959</td>
<td>0.193</td>
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</table>

Figure 5 Comparison between calculation and test results of columns under ISO 834-fire

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Axial Load Diagram for Column with circular Hollow Section 219.5x4.5

Fig. 6 Design chart for a concrete filled hollow-section [16]

Fig. 7 Design table for concrete filled hollow-sections according to EUROCODE 4 Part 10 [17]
AN OVERVIEW OF CIDECT RESEARCH
ON TUBULAR STRUCTURAL JOINTS:

N.F. Yeomans

British Steel - General Steels - Welded Tubes.

Summary

CIDECT (Comite International pour le Developpement et l’Etude de Construction Tubulaire) was formed in 1962 by a number of major tube manufacturers interested in the structural application of tubes. The objective of this organization was, and still is, to initiate and co-ordinate research and to obtain technical data on the behaviour of structural hollow section members, joints in structural hollow sections, and also information on fabrication and construction with these types of structural members.

To this end CIDECT has formed over the years a number of working groups to deal with specific topics of interest. One such group is the Joints Behaviour Group which deals with all aspects of jointing between structural hollow sections including static and fatigue behaviour and capacity and both welded and bolted joints. The other main working group is the Fire and Stability Group, which, in addition to research related to fire and stability problems, deals with all other areas of interest such as manipulation (bending), wind loading, corrosion protection etc.

This paper gives an overview of the work of CIDECT’s Joints Behaviour Group; but it does not give any great detail of specific projects as these are dealt with elsewhere or in other papers being presented at this symposium.

1. Introduction

Over the years since its formation CIDECT has created a number of working groups with responsibility for particular areas of research. The Joints Behaviour Group was set up in 1988 by the amalgamation of two previous groups Welded Joints and Fatigue Behaviour into one working group.

This group is responsible for all aspects of CIDECT research in the area of joints between structural hollow sections and between structural hollow sections and other types of structural section. This covers static and fatigue loading, welding and bolting and various aspects of fabrication and manipulation associated with joint capacity and joint design.

CIDECT’s other main working group is the Fire and Stability Group which, as well as fire and stability problems, also covers research projects on hollow section bending, wind loading and corrosion protection.
2. Static Loading

2.1 Welded Joints

Nearly all of the research carried out in the last twenty years or so on the static capacity and behaviour of structural hollow section joints has been carried out by or has been to some extent been controlled by CIDECT, the main exception to this being work on circular hollow section joints made in Japan by Kurobane and others [1] [2]. As well as work on plane frame connections between structural hollow section members (figure 1), which has been published by various organizations (e.g. IIW [3] [4]), CIDECT has also been working in the areas of multi-planar joints [5] [6] [7] [8]; joints were the ends of bracing members have been modified by part flattening [9] [10] (figure 2) or multiple flat cutting [11] (figure 3) to simplify the preparation required when joining bracings to a circular hollow section chord member.

Other research has examined double chord joints using rectangular hollow sections [12] (figure 4); flattened end bracings with gusset plates [13]; vierendeel joints [14] [15]; various types of connections to hollow section columns [16] [17] [18].

Work has also just been completed, in Canada, on research into weld size requirements for fillet welds in structural hollow lattice construction [19].

2.2 Bolted Joints

A number of investigations have also been made on bolted joints of various types. These include beam (structural hollow or conventional open sections) to structural hollow section columns [16] [17] [18] [20] and end-to-end connections [21] [22] [23] [24] which are used mainly for on-site bolting, were this is preferred to site welding.

3. Fatigue Loading

Virtually all research work carried out on the fatigue capacity and general fatigue behaviour of rectangular hollow section joints has been made by or under the auspices of CIDECT. However, the vast majority of work on circular hollow section joints has been made outside CIDECT by various branches of the off-shore oil industry, whose 'model' test specimens are of a size normally associated with onshore tubular structures, and are therefore directly applicable. As a result CIDECT has been able to concentrated it's efforts in the rectangular hollow section area.

The main areas of investigation have been on plane frame lattice girder type joints [25] [26], low cycle fatigue [27] (figure 5), and on ways of reinforcing joints [28] [29] if they either develop a crack or require strengthening for some other reason. A large investigation is currently nearing completion on multi-planar joints including some full size girders [29].

4. CIDECT Design Guides

One of CIDECT's main objectives is to increase the awareness of structural engineers to the use and design of structural hollow section in buildings and constructions of all types. To this end CIDECT has in
the past produced monographs on various aspects of the design of tubular structures [30] [31] [32] [33] [34]. These tended to contain a great deal of background information on the research work that had been carried out in a particular area and any design guidance they contained tended to become lost or difficult to find; as a result they tended to be referred to by other researchers rather than designers. CIDECT has, as a result, determined to produce design guides on specific topics which will, it is hoped, be much more useful and 'user friendly' to designers than were the old monographs. The first of these design guides entitled "Design Guide for Circular Hollow Section (CHS) Joints Under Predominantly Static Loading"[35] was published in April of this year. It deals with all aspects of the design of circular hollow section joints were definite design recommendations are available and suggests methods of design in a conservative way for areas where definitive design recommendations are not possible.

Work has now started on further design guides, which will cover the following areas of interest:

- Circular and rectangular hollow section joints under fatigue loading
- Rectangular hollow section joints under predominantly static loading
- Structural stability of hollow sections
- Fire protection of structural hollow section columns

The first two above are being produced under the guidance of the Joints Behaviour group, whilst the latter two are the responsibility of the Fire and Stability group.

5. Codes of Practice and Standards

CIDECT has over the years become an acknowledge leader in the design of structural hollow section constructions and as a result has been able to exercise some influence on the contents of various national and international standards. Recently CIDECT has been able to discuss the contents of the parts of Eurocode No.3 "Design of Steel Structures" which are relevant to the design of structural hollow sections with members of the editorial group of EC3. This has been especially true concerning the fatigue of tubular structures (Section 9) and hollow section lattice girder connections (Annex K).

6. References

Reports on CIDECT sponsored research programmes, CIDECT Monographs and CIDECT Design Guides are available from the CIDECT Technical Secretariat, c/o Mannesmannrohren-Werke AG, Mannesmannufer 3, D-4000 Dusseldorf 1, Federal Republic of Germany; or, in some instances, from the CIDECT member companies.

6.1 General References


6.2 CIDECT Research Reports

[5] Redwood, R.G. and Harris, P.J. "Welded joints in triangular trusses". Dept. of civil engineering, McGill University, Monreal, Canada, 1981 and subsequent reports. CIDECT report nos. 5W-81/1, 5W2-84/3 and 5W2-86/1


[9] Thiensiripipat, N. and Morris, G.A. "Static behaviour of cropped end connections in tubular trusses". Department of civil engineering, Manitoba university, Winnipeg, Canada, 1975 and subsequent reports. CIDECT report nos. 5K-75/9, 5K-80/12 and 5K-81/9


[14] Tabuchi, M. Kanatani, H. Kamba, T. "The local strength of welded RHS T-joints subject to bending moment". Faculty of engineering, Kobe university, Kobe, Japan, 1984. CIDECT report no. 5AF-84/5.


[18] Dunberry, E. LeBlanc, D. Redwood, R.G. "Simple beam connections for concrete filled steel columns". Department of civil engineering, McGill university, Montreal, Canada, 1985. CIDECT report no. 5AL-85/4


[20] Petit, L. Plumier, A. Rondal, J. "Tests on T-Type bolted joints in hollow sections intended to transmit a moment". Faculty of applied science, Liege university, Liege, Belgium, 1986. CIDECT report no. 6B-86/1


[22] Mang, F. "Investigation of bolted flange joints in rectangular and circular hollow sections". Karlsruhe university, Germany, 1977. CIDECT report no. 8A.

[23] Kato, B. Mukai, A. "Bolted tension flanges joining square hollow section members". Faculty of engineering, Tokyo university, Tokyo, Japan, 1982. CIDECT report no. 8B


6.3 CIDECT Monographs

[32] Monograph No.5: Concrete-filled Hollow Section Columns, 1977
[33] Monograph No.6: The Strength and Behaviour of Statically Loaded Welded connections in Structural Hollow Sections, 1986

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Figure 1. Typical tubular joints and nomenclature.
Figure 2. Partially flattened bracing ends.

Figure 3. Multiple plane cut bracing ends.
Figure 4. Double chord joint types.
Figure 5. Low cycle/high stress fatigue behaviour.

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ON CONVERGENCE OF YIELD LINE THEORY AND NONLINEAR FEM RESULTS IN PLATE STRUCTURES

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Summary

The plastic yield load capacity of RHS joints has been calculated using yield line theory and materially nonlinear FEM calculations. A plane stress state was assumed in the yielded condition, i.e. the bending stress was $2/\sqrt{3}$ times the uniaxial yield strength of the ideal plastic material, corresponding to a plastic state Poisson's ratio of 0.5 instead of the elastic 0.3. The yield line theory was further modified by incorporating a shear strength reduction, which was verified by FEM using plane strain models. The total capacity of some rectangular hollow section T- and K-joints was verified by nonlinear FEM, and predicted by the theory presented as the sum of the semi-infinite transverse heel side capacity and the capacities of other straight yield line couples in transversal and longitudinal directions, having hinge lengths equalling the dimensions of the brace of the FEM model.

1. Introduction

The load capacity of RHS joints can be determined by experimental tests, using nonlinear finite element calculations, or from yield line theory. Experimental tests give the true ultimate capacity of individual test specimens. Due to the scatter in the material properties and the dimensions, especially in the weld leg lengths, some scatter in test results is also observed. Nonlinear FEM calculations are a flexible, if rather laborious way to analyse and to understand the behaviour of joints. However, the most commonly used element type, the thin shell element, does not take into account the effect of through-thickness shear stresses. Moreover, the results can be sensitive to the choice of the element size. The third method, yield line theory, is a valuable tool in practical engineering work. Design rules for RHS joints have been developed, mainly based on experimental tests, but also backed up with the yield line theory and nonlinear FEM calculations. However, such design formulae are valid in a specified range only. A well verified yield line theory would be a valuable method for fast calculation of cases outside the validity range.

The accuracy of nonlinear FEM calculations is verified by simple models where the exact plastic limit loads are easily calculated. The accuracy of the limit load of more complicated details can be estimated using
the accuracy of the loads of individual plastic hinge couples.

2. Materials and methods

2.1 Structural details studied

The details studied by the finite element method were, Figure 1:

1. Long cantilever plates simulating one half of a unit length plastic hinge couple in bending.
2. Plate and shell models for simulating the chord face plate behaviour of RHS-joints.
3. Plane strain models for taking into account shear stress reduction in plastic moment capacity, RHS corner radius and stress triaxiality effects.

The accuracy of FEM to predict the true plastic limit load has been calculated and the error rate reported for the basic cases of 1. The theory for separating the shear forces carried by individual plastic hinges in the biaxial stress state is presented and verified by nonlinear FEM-calculations, case 2. The theory in the yield line method to take into account shear strength reduction in the biaxial stress state is also presented and verified by plane strain nonlinear FEM calculations.

2.2 Material properties

The material used in the calculations was steel, Young's modulus 207 GPa, the elastic Poisson's ratio 0.3. The nonlinear material behaviour was ideal elastic-plastic, yield stress 250 MPa in most of the cases considered.

2.3 Finite element models and calculations

A cantilever model shown in Figure 1a was used to simulate the half gap of a plastic hinge couple. Abaqus thin shell S4R5 linear and S8R5 parabolic elements were used in nonlinear runs to calculate the effect of the length of the element next to the plastic hinge nodes, Ref.[5].

Abaqus S4R thin shell linear elements were also used in eight nonlinear runs to calculate the strength of an RHS face subjected to a transverse line load, Figure 1b. The nodes in the line of loading were constrained to displace equally corresponding to so-called "Knife Edge Loading". The total width B was divided to 16 elements of equal width. The ratio \( \beta \) between the "knife edge" width and the chord width was varied initially to give the following values: 0.125, 0.375, 0.625, 0.87. The ratio B/T had a constant value of 12.5. Loading at \( \beta = 0.875 \) did not give convergent results because the gap between the "knife edge" and the side wall was modelled by only one element. Further calculations with slightly modified meshes with \( \beta \) equal to 0.6, 0.6, 0.66 and a finer mesh with \( \beta = 0.8 \) were also performed.

The superposition of plastic load capacities of submodels was compared with the capacity of the whole model, Figures 1c, 1d and 1e. Four K-joints were also analysed by i) materially, and ii) materially and geometrically nonlinear analysis using Abaqus thin shell S8R5 elements. The dimensions of the K-joints are listed in Table 3. Boundary
conditions corresponded to the test rig conditions of Koskimäki & Niemi [3]. The chord was in all cases pre-tensioned by 36% of the yield strength 467 MPa, ultimate stress being 527 MPa. The loads corresponding to the onset of yielding and the results of earlier FEM runs of Ref.[3] were compared with the results of the yield line theory presented later in this paper.

The effect of shear strength reduction was calculated by FEM in three different ways. A cantilever model simulating a half gap was calculated using Abaqus CPE4 plane strain elements, Figure 1f. One quarter of the gap was modelled using antimony. Five models were analysed with gap to thickness ratios G/T being 12, 2, 1.2, 0.6 and 0.2. The element half thickness was divided into 10 elements and the half gap consisted of 20 elements; all of the 200 elements of one run were normally equal in size. The built-in end was allowed to contract by applying shear force loading at both ends. The shear force distribution at the built-in end was adjusted to correspond to the yield state shear stress distribution to avoid any strength increase due to stress triaxiality. Three models of a full straight gap loaded by a shear force couple were also calculated for comparison. G/T ratios analysed were 1, 0.5 and 0.1. The effect of stress triaxiality due to the RHS corner radius and due to the notch effect of the corner weld toe were analysed for two G1/T ratios of 2 and 1.1, Figure 1g.

2.4 Stress biaxiality modification of the yield line theory

One of the basic assumptions of the classic yield line theory is the use of uniaxial yield strength as the stress criteria for failure. In beam structures the uniaxial stress assumption is valid. In the straight yield lines of RHS joints, however, the bending stress state is biaxial because the transverse strains of straight plastic hinges are constrained. Therefore the ratio of the transverse stress and the bending stress is equal to the Poisson's ratio. In elastic and plastic ranges of steel the Poisson's ratios are 0.3 and 0.5, respectively.

The Von Mises yield criterion in the triaxial stress state is:

$$f^2 = \frac{1}{2} \cdot \left[ (\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 \right] + 3 \cdot (\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2)$$

(1)

Assuming no shear, a longitudinal bending stress equal to \(\sigma_x\), and a transverse stress, \(\sigma_y = 0.5 \cdot \sigma_x\), then the yield criterion gives:

$$f^2 = \frac{1}{2} \cdot \sigma_x^2 \cdot (0.5^2 + 0.5^2 + 1)$$

(2)

Thus

$$\sigma_x = \frac{2}{\sqrt{3}} \cdot f_y \quad \text{or} \quad \sigma_x = 2 \cdot \tau_{\text{yield}}$$

(3)

The plastic moment of a unit width plate of thickness T using the stress value from eq. 3 is, as also shown for circular hollow sections by Grzebedieta [4]:

$$m_p = \frac{2 \cdot f_y \cdot T^2}{\sqrt{3} \cdot 4}$$

(4)

The shear strength of a unit width plate of thickness T is not affected
by any restraint. Thus the pure shear capacity is:

\[ q_{po} = \frac{1}{\sqrt{3}} f_y \cdot T \]  

(5)

The shear force capacity of a gap of width \( G \) with plastic hinges at each end, determined by the flexural failure is:

\[ q_p = m_p \cdot \frac{2}{G} \]  

(6)

Using eqns. 4 and 5, eq. 6 can be rewritten as:

\[ q_p = \frac{T}{G} \cdot q_{po} \]  

(7)

Thus, the shear load capacity \( q_p \) in a wide gap is the pure shear capacity of the plate \( q_{po} \), multiplied by a bending reduction factor, \( T/G \). The total load carried by the straight hinge mechanism \( Q_s \) is the sum of \( Q_1 \) as shown by equation 8, \( G_1 \gg T \).

\[ Q_s = \sum Q_i = \sum L_i \cdot q_i = \sum L_i \cdot q_{po} \cdot \frac{T}{G_i} \]  

(8)

The "Knife" force transmitted through the semi-infinite transverse heel end of a T- or a K-joint can be estimated using a transverse hinge distance \( x \) as the gap. The assumption of straight yield lines is not true for the remote yield line, because a saddle surface can form, but this effect has been neglected. The conventional solution for the distance, \( x \) (see also Figure 2), solved for a simple mechanism consisting of straight hinges, as recommended e.g. by Eurocode 3, Reusnik & Wardenier [1] is:

\[ x = 0.5 \cdot B \cdot \sqrt{(1-\beta)} = G_1 \sqrt{(1-\beta)} \]  

(9)

Using a circle mechanism as the failure mechanism, the capacity of a point loaded plate can be derived as

\[ Q_c = \pi \cdot f_y \cdot T^2 \]  

(10)

The shear force carried by one corner as a point load is proposed, conservatively in this paper, to be as one quarter of \( Q_c \). The total load capacity of a T- or K-joint is the sum of the corner point load capacity \( Q_c \), and the sum of the capacities of the straight hinges around the brace \( Q_s \).

2.5 Shear stress correction of the yield line theory

The presence of the shear forces normal to the plate reduces the plastic moment capacity available in the hinges. The shear strength reduction factor can be approximated by assuming the biaxial limit state bending stress, see eq. 2, and a shear stress evenly distributed across the thickness. In the limit state the stresses are as follows

\[ \tau_{zx} = q_p / T \]  

(11)
Using the von Mises yield criterion, eq. 1, the effective axial stress ratio in bending can be written as

\[
\sigma_y / f_y = \frac{2}{\sqrt{3}} \cdot \frac{1}{\sqrt{1 + (T/G)^2}}
\]

Finally, the shear load capacity, see eq. 7, taking into account the shear stress correction, using eqns. 14,13 and 5, can be expressed as:

\[
q_p = q_{po} \cdot \frac{1}{\sqrt{1 + (G/T)^2}}
\]

Equation 15 is valid both in pure shear, G=0, and in pure bending, G >> T. The effect of the shear strength reduction is shown in Figure 4.

2.6 Transverse knife edge loading on one face of an RHS

The load capacity of a plate, loaded by a transverse "knife edge" load, can be calculated as the sum of the point load capacity, Q_1, and the transverse line load capacity, Q_t. The shear stress reduction is taken into account in the corners only, where the average distance between yield lines, G_1/2, is used as the gap size. The distance x between the straight transverse yield lines is assumed always to exceed 2·T, such that no reduction is required, see Figures 2 and 3.

\[
Q_k = Q_1 + Q_t
\]

\[
Q_1 = 4 \cdot q_{po} \cdot \frac{T}{G_1} \cdot \frac{1}{\sqrt{1 + (2 \cdot T/G_1)^2}}
\]

\[
Q_t = 4 \cdot q_{po} \cdot \frac{T}{x} \cdot \frac{B}{2}
\]

Minimizing the sum of Q_1 and Q_t with respect to x, the unknown distance x of the transverse plastic hinge can be derived as

\[
x = T \cdot \sqrt{(0.5 \cdot B/T) \cdot [(G_1^2/T^2) + 4]}^{1/4}
\]

In the two extremes of G_1, either 0 or B/2, the unknown x is either \(\sqrt{B \cdot T}\) or B/2, corresponding to the x of the shear mechanism, and the x of the straight line mechanism. The total "knife edge" load Q_k(G_1) calculated using eqns. 16,17,18 and 19 is

\[
Q_k(0) = 4 \cdot q_{po} \cdot T \cdot \sqrt{B/T}
\]
\[ Q_k (B/2) = 4 \cdot q_{po} \cdot T \cdot \frac{1.47}{2} = n \cdot f_y \cdot T^2 \cdot 1.47 = Q_c \cdot 1.47 \] (21)

From eq. 21 it is seen that the straight line mechanism of Knife loading in the biaxial stress state overestimates the bending capacity of the point load case \( Q_c \) by 1.47, but estimates the accurate shear capacity of eq. 20. The formula of \( Q_k \) has been modified by the ratio \( \pi/4 \) to also predict the accurate \( Q_c \), being \( Q_{km} \):

\[ Q_{km} = \pi \cdot f_y \cdot T^2 \cdot \sqrt{\frac{1}{(2 \cdot G_1/T^2 + 3.43)^{1/4}}} \] (22)

In a full width joint, yielding of the brace can reduce the shear capacity further.

### 3. Results

The load capacities of long gaps, solved by the thin shell cantilever FE models, corresponded to the capacities solved by the modified yield line theory, eq. 15, within 0.4\% in all cases. However, in order to obtain such consistent results, the gap used in the yield line method was reduced by the factor \( 1-1/G \), where \( l \) is the length of the linear, quadrilateral element next to the plastic hinge.

**Thin shell T-and K-joint models**

In Figure 4 the capacity of "Knife edge" loading calculated is shown, according to EC3, Corner+Straight side (eqs.8,9,10) \( (Q_s + Q_c)/Q_c \) and according to the modified method (eq.22) \( Q_{km}/Q_c \), of three B/T ratios 40,20,10, respectively.

In Figure 5 the results of thin shell element analyses of the "knife edge" loading are shown together with the curve of the modified circle mechanism theory (eq.22), presented in this paper.

The results of the submodel superposition and the results of the complete model calculation are shown in Table 1, together with the theoretical yield line model results, and modified by the FEM element size to the gap size ratio.

In Koskimäki & Niemi [3] various boundary conditions were analysed. In Table 2 FEM results and the results of the yield line theory, presented here, are compared for all the cases analysed in [3].

Brace forces of the four different K-joint FEM models of ZA,ZB,ZC,ZD and brace forces calculated using the Yield Line method, multiplied by the element length correction factors (EL) 1.07, 1.13, 1.06, 1.20, and the pre-tension correction factors (PF) 0.95, 0.95, 0.93 and 0.89, respectively, are presented in Table 3.

**Plane strain models**

In Figure 6 relative shear capacities of plane strain FEM models, compared with the capacities according to the yield line theory, are shown relative to the shear capacity \( q_{po} \).
Plane strain cantilever models with a gap to thickness ratio $G/T = 12$ and varying element biasing (half or full model), gave from 1.00 to 1.08 times higher capacities than predicted by the biaxial stress state yield line model (or the thin shell model with the span length correction). In all cases the equilibrium force tolerance $PTOL$ was set to $1/10000$ of one element axial yield force. Axial and shear stress distributions of the plastic hinge are shown in Figure 7 for $G/T$ ratios 12, 1.2 and 0.6.

4. Discussion

The results of the fine mesh plane strain cantilever models in almost pure bending, i.e. $G/T=12$, were too sensitive to the choice of the element size and biasing. Inaccuracy problems are to be expected for much coarser meshes in more complicated solid element models.

When nonlinear FE analysis is used, the straight plastic hinges form at the integration points next to the weld toe, not at the weld toe node line. The centre point of the actual yield line will also be formed somewhat outside the weld toe, as discussed below.

The results of the new yield line method are promising. The calculated yield line results matched well with the results of nonlinear FE analysis of K-joints, Tables 1, 2 and 3. Difficulties will still arise in determining the effective gap when the gap approaches zero. Some scatter in the FEM results is observed due to variations in element size and biasing (Table 1, Fig. 4) and mesh type (Fig. 5).

Stress triaxiality at the weld toe increases the strength such that the plastic hinge in the triaxial stress state (Eq. 4) forms at a distance of 20% of the thickness, $T$, when the gap size exceeds twice the thickness. Thus the results of the FE analysis and the yield line theory should converge and agree with experimental test results if the plastic hinges are forced to form at that distance from the weld toe, using correct element sizing and element thickness modification techniques. Correspondingly, in the yield line model, the yield lines should be placed 0.2·$T$ outside the actual weld toe, in order to achieve results comparable with experimental tests.

In the rounded corner area of hollow section chords the longitudinal plastic hinges form at the intersection of the web plate of the chord and the corner rounding.

The results from plane strain models indicated that the core yields by plastic shear while the bending moment is carried by the surface material. The shear strength reductions correlating with the FEM results are either linear interpolation from pure bending capacity to shear capacity - the biaxial stress reduction method, as proposed here, or one similar to the membrane stress reduction, not the reduction in the uniaxial yield stress, which gives excessively shear strength values with zero gaps.
5. Conclusions

The bending stress in ideal plastic material of straight plastic hinges is $2/\sqrt{3}$ times the uniaxial yield stress.

Formulae have been presented for taking into account the multiaxial stress state including the shear stress correction. They extend the validity range of the yield line theory down to zero gap sizes.

The capacities of the chord plate of RHS joints and similar structures can be summed from the separate capacities of straight plastic hinge couples.

The thin shell elements of Abaqus do not take into account the transverse shear reduction, giving excessively high load capacities with small gaps.

6. Acknowledgements

Petri Tarjavuori and Zhang Zhiliang performed most of the FEM-calculations, and Professor Erkki Niemi has advised in development of the theory. All work at Lappeenranta University of Technology, and their support in preparing this report is gratefully acknowledged.

7. References


### Table 1. Results of submodel superposition

<table>
<thead>
<tr>
<th>Case</th>
<th>FEM [3]</th>
<th>Y.L./FEM</th>
<th>·EL</th>
<th>·PF</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-G-T-P</td>
<td>650 kN</td>
<td>0.96</td>
<td>1.09</td>
<td>0.99</td>
</tr>
<tr>
<td>A-G-Y-P</td>
<td>650 kN</td>
<td>0.96</td>
<td>1.09</td>
<td>0.99</td>
</tr>
<tr>
<td>B-G-Y-0</td>
<td>650 kN</td>
<td>0.96</td>
<td>1.09</td>
<td>1.09</td>
</tr>
<tr>
<td>D-G-Y-P</td>
<td>620 kN</td>
<td>0.92</td>
<td>1.04</td>
<td>0.95</td>
</tr>
<tr>
<td>E-M-Y-P</td>
<td>650 kN</td>
<td>0.87</td>
<td>1.0</td>
<td>0.91</td>
</tr>
<tr>
<td>F-M-Y-0</td>
<td>660 kN</td>
<td>0.86</td>
<td>0.98</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Explanations: Cases H/A/B/D/E/F of Ref.[3]; - Geometric or - Material nonlinear; - Test or - Yield line restraints; - Pre-tension or - O-tension; ·EL = multiplied by Element Length correction factor (Constant value 1.14); ·PF = multiplied by Pre-tension correction Factor (Constant value 0.91)

### Table 2. Brace forces of the K-joint of Ref.[3]

<table>
<thead>
<tr>
<th>Case</th>
<th>Brace Angle</th>
<th>Transv. Gap</th>
<th>Brace Forces</th>
<th>·EL</th>
<th>·PF</th>
</tr>
</thead>
<tbody>
<tr>
<td>ZA-T-P</td>
<td>100·100·6</td>
<td>45</td>
<td>32</td>
<td>560 kN</td>
<td>0.94</td>
</tr>
<tr>
<td>ZB-T-P</td>
<td>100·100·6</td>
<td>45</td>
<td>21</td>
<td>660 kN</td>
<td>0.91</td>
</tr>
<tr>
<td>ZC-T-P</td>
<td>70·70·5</td>
<td>60</td>
<td>38</td>
<td>240 kN</td>
<td>1.02</td>
</tr>
<tr>
<td>ZD-T-P</td>
<td>70·70·5</td>
<td>60</td>
<td>11.5</td>
<td>344 kN</td>
<td>0.94</td>
</tr>
</tbody>
</table>

Explanations: Cases H/A/B/D/E/F of Ref.[3]; - Geometric or - Material nonlinear; - Test or - Yield line restraints; - Pre-tension or - O-tension; ·EL = multiplied by Element Length correction factor (Constant value 1.14); ·PF = multiplied by Pre-tension correction Factor (Constant value 0.91)

### Table 3. Brace forces of K-joints analysed, Chord 150·150·8
Figure 1  
a) Thin shell cantilever model  
b) "Knife Edge Loading" model  
c) Symmetric quarter of a T-joint  
d) Antisymmetric part of a K-joint  
e) Superposition of Symmetric and Antisymmetric models  
f) Plane strain cantilever model  
g) Cross section of an RHS-chord, plane strain model

Figure 2  
Yield lines of the heel end "Knife Edge loading"

Figure 3  
Shear strength usage factors

Figure 4  
Capacity of Knife Edge Loading
Figure 5 Capacity of Knife Edge Loading, FEM vs. Y.L

Figure 6 Relative shear capacity of plane strain FEM models compared with the capacity according to the yield line theory.

Figure 7 Stress distribution in cross section of a half cantilever.
THREE DIMENSIONAL CROSS JOINTS UNDER COMBINED AXIAL BRANCH LOADING

by Gwynne Davies¹ and Koji Morita²

Nottingham¹ and Chiba² Universities

Summary

Despite the value of using finite element techniques, plastic and yield line approaches continue to give valuable understanding and insight into the behaviour of three dimensional cross joints. The simple "ring" approach for circle or rectangle, indicates that a significant increase in strength of multi planar circular hollow section joints can be anticipated, when branch forces are either both tensile or compressive. A similar approach for a rectangular joint indicates little increase in strength above the planar cross joint. The former is confirmed by the finite element approach, and the latter by yield line analysis. Some consideration is also given to membrane action for two and three dimensional joints.

1. Introduction

A considerable amount of effort is being expended at present on the determination of the strength and stiffness of three dimensional joints in both welded Circular (CHS) and Rectangular Hollow Sections (RHS). In CHS construction the purpose has been to investigate the fatigue behaviour of joints used in offshore structures. Both experimental and analytical techniques have concentrated on determining the concentration of stress adjacent to weld toes. This has involved large scale tests, photo elastic techniques, and more recently analytical methods frequently involving the Finite Element method (FE)(1). While the former tend to be expensive and also imperfect, the FE method has developed to the stage that by using substructuring it is possible to analyse not only a joint but also whole structures, so that the full effect of non rigid connections and weld shape details can be readily incorporated into the model. This means that where dedicated computers are available, they can be significantly used to examine the effects of the various joint and member parameters.

The use of FE for determining collapse behaviour is a more recent development. Several suites of programs are now available which allow for non linear material properties, and the effect of large deflections or change of geometry, which frequently occur in these joints towards collapse. However the storage and computer time involved for non symmetric joint situations can be quite prohibitive. It is likely that such analyses will for the present time be only used for major structural situations, or more commonly by organisations involved in producing design recommendations. The current strategy is to carry out a parametric study.
using the FE method, supported by others, and validated by large scale testing of "as built" joints for benchmarking at selected parameter points.

Detailed FE elastic analysis for CHS joints was first developed because of the fatigue needs of offshore structures, while interest in the use of RHS for onshore structures encouraged the use of non linear FE (2). With the increasing role of RHS in topside structures the interest is now also reversed (3). The non linear approach is particularly useful as the effects of material non linearity or large deflection, and their influence on instability can all be considered together, or separately if certain facilities are suppressed in order to identify and understand how the joint behaves.

The purpose of this paper is to indicate that the methods of plastic or yield line analysis still has an important part to play and may identify basic behaviour at very low cost, and in particular aid to the understanding of joint behaviour, at a level an ordinary engineer can comprehend. Consideration is first given to double T joints in both RHS and CHS joints, but based on very short chord lengths so that the basics can be idealised as a planar or ring problem.

2. Rigid plastic circular and rectangular "ring" models

Fig. 1 & 2 show the simple plastic hinge collapse models for RHS and CHS cross joints of finite length L respectively, and the collapse loads are given by

$$P_Y = \frac{8m_p L}{1 - \beta (b_o)} = \frac{2\sigma_{\text{eo}} t_o^2 L}{1 - \beta b_o} \quad (1)$$

for the RHS joint, and by eq. 2 for CHS joints

$$P_Y = \frac{2\sigma_{\text{eo}} t_o^2}{1 - \beta} \frac{L}{d_o} \quad (2)$$

It is worth noting that where the chord is the same width \((b_o = d_o)\), then for small deflections the RHS joint has the same strength as the CHS joint, for the same branch/chord width ratio \(\beta = b_l/b_o\) or \(d_l/d_o\) and thickness \(t_o\). The square of the thickness is usual where failure of the joint is by plastic flexure of the tube wall. The same strength would have been obtained if plastic hinges occurred at any position on the chord side walls, as shown in fig. 1(b). There is only one possible position however for the plastic hinges in the CHS case, as shown in Fig. 2(a). It is helpful to note the line of action required of the applied force \(P_Y/2\) to satisfy equilibrium, and the two plastic moments as shown in fig. 2(b).

Fig. 3 considers the case of joints with a square chord with equal width branches. The mode of failure of the RHS joint is indicated, for compressive branch forces and represents two independent mechanisms, both shown in fig. 3(a) by
Where the chord is rectangular then collapse would be given by the lower value of $P_{yy}$ or $P_{yx}$, associated with the respective value of $\beta_y$ or $\beta_x$. This indicates the greater variation possible with RHS than CHS, and the extra care needed in design. Where the branch forces are in the opposite sense, a single independent mechanism of failure occurs as shown in fig. 3(b), where the relation between $F_{yx}$ and $F_{yy}$ is given by

$$F_{yy}(1 - \beta_y) + F_{yx} \frac{h_0}{b_0} (1 - \beta_x) = \frac{8m}{b_0} \frac{L}{b_0} = 2\sigma_{e0}^2 \frac{L}{b_0}$$  \hspace{1cm} (4)

Where $\beta_y = \beta_x$, as for the square chord with equal branches or a rectangular chord with branch members of the appropriate ratio then

$$F_{yy} + F_{yx} \frac{h_0}{b_0} = \frac{8mp}{1 - \beta} \left( \frac{L}{b_0} \right) = 2\sigma_{e0}^2 \left( \frac{L}{b_0} \right)$$  \hspace{1cm} (5)

or for square chord

$$F_{yy} + F_{yx} = \frac{8mp}{1 - \beta} \left( \frac{L}{b_0} \right) = 2\sigma_{e0}^2 \left( \frac{L}{b_0} \right)$$  \hspace{1cm} (6)

This can be non-dimensionalised by introducing the values determined from eq. 3.

$$\frac{F_{yy}}{P_{yy}} + \frac{F_{yx}}{P_{yx}} \cdot \frac{h_0}{b_0} = 1$$  \hspace{1cm} (7)

and the plastic collapse interaction diagram is shown in fig. 4. It is most important to notice that the basic plastic collapse mechanism for the "ring type" RHS connection indicates clearly that there is no increase in strength of the joint when both forces act in the same sense i.e. either both tensile or both compression. Thus any increase in strength over that given by eq. 1 will have to depend on material strain hardening or effects associated with change of geometry.

It is now worthwhile to consider the basic behaviour of the two dimensional circular chord with four branches as shown in fig. 5(a). The mechanism shown can be used to determine the interaction when the branch forces are in the opposite sense. For gap type joints it should be noted that if opposite branches are of the same diameter then the upper limit on $\beta$ is given by

$$\arcsin \beta_x + \arcsin \beta_y \leq \frac{\pi}{2}$$  \hspace{1cm} (8)

Where all the branches are the same size then $\arcsin \beta \leq \pi/4$ i.e. $\beta \leq 0.707$. 

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The interaction for rigid plastic failure is given directly by

\[
F_{yy} \left( \sqrt{1 - \beta_x^2} - \beta_y \right) + F_{yx} \left( \sqrt{1 - \beta_y^2} - \beta_x \right) = 2ae_{e0}^2 \frac{L}{d_0}
\]  \hspace{1cm} (9)

or for the same size branches

\[
F_{yy} + F_{yx} = \frac{2ae_{e0}^2 \frac{L}{d_0}}{\sqrt{1 - \beta^2} - \beta}
\]  \hspace{1cm} (10)

If \( F_{yx} = 0 \)

\[
F_{yy} = \frac{2ae_{e0}^2 \frac{L}{d_0}}{\sqrt{1 - \beta^2} - \beta}
\]  \hspace{1cm} (11)

This should be compared with eq. 2 to indicate the difference of strength purely associated with "welding in" the horizontal tube

\[
\frac{F_{yy(eq,11)}}{F_{yy(eq,2)}} = \frac{1 - \beta}{\sqrt{1 - \beta^2} - \beta}
\]  \hspace{1cm} (12)

The increase varies from 1.03 to 2.00 as \( \beta \) increases from 0.2 to 0.6. In the case of the square ring there is no increase just due to "welding in" the cross branches.

The case of the collapse of the ring under compression forces in the branches is generally analysed in terms of plastic hinges forming at the mid points of the gap as shown in fig. 5(b). It can however be shown that this is a special loading case. Collapse generally occurs with a two hinge model, as shown in fig. 6(a), which is a modified form of that shown in fig. 5(a), with one hinge occurring nearer the more lower loaded branch. For these models it is assumed that during deformation the ratio of the branch forces \( k = F_x/F_y \) is maintained. This means that the slope \( \theta \) of the resultant reaction \( R = 0.5\sqrt{(F_{yx}^2 + F_{yy}^2)} \) must be the same at both the vertical and horizontal edges of the branches. To ensure equilibrium, these two reactions must have coinciding lines of action as shown in fig. 6(b) for various ratios of \( 0 < k < 1 \). If the wall plastic moment of resistance is the same at all points along the length of the arc, then the eccentricity of the thrust line must be equi-distant from the plastic hinges formed for each value of \( k \) as shown. Here it is clearly seen that the intermediate hinge must form away from the mid point of the arc in general, except for the particular case where two independent mechanisms can form for the case \( k = 1 \), i.e. where \( F_x = F_y \). Where all the branches are the same size, and \( k < 1 \), then one hinge will always form at the edge of the vertical branch, and the other on the arc next to the horizontal branch and vice versa. The case where the two vertical branches are different to the horizontal deserves special consideration. The equilibrium of a typical arc is shown in fig. 6(b). The edges of the branches are defined by \( \sin \alpha_y = \beta_y \) and
\[ \sin \alpha_x = \beta_x. \] The maximum arc moment is assumed to occur at an inclination \( \theta. \)
The equilibrium of arc AB requires
\[ \frac{F_{Yx}}{2} \cdot y - \frac{F_{Yy}}{2} \cdot x + 2m_pL = 0 \] (13)
since \( y = \frac{d_0}{2} (\cos \alpha_y - \sin \theta) \) and \( x = \frac{d_0}{2} (\cos \theta - \sin \alpha_y) \) then eq.13 can be written as
\[ F_{Yy} (\cos \theta - \sin \alpha_y) - F_{Yx} (\cos \alpha_y - \sin \theta) - 2\sigma_{eo}t_0 = 0 \] (14)
This can be re-written in the form
\[ F_{Yy} = 2\sigma_{eo}t_0 \left( \frac{L}{d_0} \right) (\sin \theta + \cos \theta - k\cos \alpha_y - \sin \alpha_y)^{-1} \] (15)
and optimised for the minimum value of \( F_{Yy} \) when \( \tan \theta = k. \) Back substituting gives a solution satisfying the uniqueness theorem, viz
\[ P_{Yy} = 2\sigma_{eo}t_0 \left( \frac{L}{d_0} \right) (\sqrt{1-k^2} - k\cos \alpha_y - \sin \alpha_y)^{-1} \] (16)
It should be noted that this solution is only valid when the hinge forms along the free arc, i.e. if \( \alpha_x \leq \theta \leq \frac{\pi}{2} - \alpha_y \) or \( \sin^{-1} \beta_x \leq \tan^{-1} \frac{\sigma_{Yx}}{\sigma_{Yy}} \leq \cos^{-1} \beta_y. \)

Eq. 10 and 16 are plotted as an interaction diagram in fig. 7. The plastic strengths asymptote to infinity at \( \beta = 0.707 \) rather than 1.00 as for a single pair of branch cross joint in eq. 2, but can be higher for unequal branch joints. The equations assume that the plastic moment is unaffected by the magnitude of the branch axial force \( R. \) If this taken into account approximately according to the relation
\[ m_p' = 0.25 \sigma_{eo}t_0 \left( 1 - \left( \frac{R}{t_0\sigma_{eo}} \right)^2 \right) \] (17)
then there is a significant reduction in the positive interaction as indicated in fig. 7, for the case of \( \beta = 0.6 \) and \( d_0/t_0 = 40. \)

3. Planar cross joints

The Structural Hollow Section yield line mechanism for a planar RHS cross joint is well documented (4), and shown in fig. 8. The minimum plastic collapse load is given by eq. 18 for the optimised straight yield line pattern confined to the chord connecting faces. The use of fan yield lines gives a slightly lower value. Yield lines involving the chord side faces generally give higher upper bound values, and are therefore invalid for small deflections.
\[ P_Y = \frac{2\sigma_{eo}t_0^2}{(1 - \beta_y)} \left( \gamma_y \beta_y + 2\sqrt{1 - \beta_y} \right) \] (18)
The authors (5) have considered the effect of membrane stress in the chord wall giving rise to increased strength but decreasing stiffness due to the extending yield lines with increased load. An example is shown in fig. 9, where two mechanisms of failure involving membrane effects are considered - the first (a) allowing for a mechanism confined to the connecting walls, while (b) allows for associated sidewall deformation also. A comparison with a Tee joint tested is given. It is clear that the small deflection yield line prediction gives a good indication of the strength of the joint, but that the gain in strength is more closely associated with the mechanism which combines side and cross wall yield lines with connecting wall membrane action. There is of course an upper limit on the increasing strength, provided by the limited axial capacity of the remainder of the crosswall in opposing active membrane action.

Kurobane (6) has shown that a good fit to full CHS cross joint experimental results can be obtained for the whole range of $\beta$ by modifying the circular ring model estimate given in eq. 2 on replacing $(1 - 13)$ by $(1-0.8113)$ giving a characteristic strength

$$P_k = \sigma_{eq}b_o^2 \frac{5.7}{(1-0.81\beta)}$$

More recently this approach has been modified in the IIW recommendations to include for the effect of chord slenderness. Some work has also been carried out by Makino et al(7) to apply yield line techniques to CHS joints, but this is not yet at an advanced stage.

4. Three dimensional cross joints

The transformation to allow for the shape of the joint along the length of the chord can be carried out analytically for RHS joints using the yield line approach as shown in fig. 10. When $F_y$ and $F_x$ act in opposite sense, an appropriate independent mechanism is formed, where the optimised extent of the yield line pattern on each chord face is not governed by $\tan \lambda_y = \sqrt{(1 - \beta_y)}$ and $\tan \lambda_x = \sqrt{(1 - \beta_x)}$. The analysis takes into account both rectangularity and orientation of all the members without difficulty giving rise to the plastic collapse interaction relation when $h_x = h_y$;

$$F_{yx} + F_{yx} \left( \frac{1 - \beta_x}{1 - \beta_y} \right) \frac{h_o}{b_o} = \frac{2\sigma_{eq}b_o^2}{1 - \beta_y} \left( r_y \beta_y + 2\sqrt{(1 - \beta_y)} + (1 - \beta_x) \left( \frac{h_o}{b_o} \right)^2 \right)$$

For a square chord with equal square branches acting in opposition, then $\tan \lambda_y = 0.707 \sqrt{(1 - \beta)}$ compared with $\sqrt{(1 - \beta)}$ for the two dimensional joint, i.e. the yield lines spread further along the chord than for the equivalent two dimensional joint. For branch members acting in the same sense i.e. both compression or tension, two independent mechanisms can form identical with those obtained for the two dimensional cross joint given by eq. 18, ensuring no interaction for the plastic yield line capacity - as indeed was observed with the case of the square ring. The question remains as to whether membrane action will increase the three dimensional strength, above that for a planar RHS cross joint. It is likely that the
behaviour will be nearer to that indicated by curve (a) fig. 9, but since there will be longitudinal membrane action on all four walls, axial yielding of the chord remainder will occur at a lower limit. It is therefore likely that tests to be under taken shortly will only achieve a marginal improvement of capacity above that of the two dimensional cross joints, and be associated with the large deflection membrane action.

Based on their non linear FE approach to three dimensional X joints Paul et al (3), have comparison the FE non dimensional joint strength, with the recommendations of API and AWI, and these are indicated in fig. 11 with a factor of safety of 1.7 applied to the FE results. Mäkeläinen (8) has also modified simple plastic ring expressions to match the FE results. There is however a dearth of test results for such joints. However these approaches show that there is an appreciable increase in joint capacity for 3D CHS X joints when the members are in compression.

5. Concluding remarks

The paper has described various circular and rectangular ring models, yield line models for RHS joints, and non linear FE models for multi-planar CHS X joints. It is clear that for CHS joints a significant increase in strength is achieved while branches are loaded in the same sense. However it has been shown that the same is unlikely for RHS joints, where both rectangular ring models and yield line analysis predict no increase in strength. Any increase which does occur will be due to large deflection or corner (greater yield stress) effects. Experimental work to be carried out shortly will clarify this.

6. References

7. Notation

$b_0, b_x, b_y$ width of RHS chord and branches
$d_0, d_x, d_y$ dia of CHS chord and branches
$F_{Yx}, F_{Yy}$ co-existing branch forces producing plastic mechanism; $k = F_{Yy}/F_{Yx}$
$h_0, h_x, h_y$ depth of RHS chord and branches
$L$ effective length of chord; $l = 0.5b_0\sqrt{(1 - \beta_y) + (1 - \beta_x)(h_0/b_0)^2}$
$m_p, m_p'$ full and reduced chord wall plastic moment/unit length
$P_Y, P_{Yx}, P_{Yy}$ plastic strength for single planar joint
$R$ resultant force $= 0.5\sqrt{(F_{Yx}^2 + F_{Yy}^2)}$
$r, r_x, r_y$ rectangularity of RHS branch; $r = h/b$
to chord thickness
$\alpha_x, \alpha_y$ angle of inclination for CHS ring - fig. 6.
$\beta, \beta_x, \beta_y$ branch/chord width ratio; $\beta_x = b_x/b_0$ for RHS, and $\sin\alpha_x$ for CHS etc
$\lambda_x, \lambda_y$ inclination of yield lines in fig. 10
$\theta$ inclination of plastic hinge in CHS ring model - fig. 6
$\sigma_{eo}$ yield stress of chord

![Figure 1](image1.png) **Figure 1** Modes of failure for RHS cross joints

![Figure 2](image2.png) **Figure 2** Mode of failure for CHS cross joints

![Figure 3](image3.png) **Figure 3** Modes of failure of biaxially loaded RHS joints
Figure 4  Interaction diagram for RHS “ring type” joints

Figure 5  CHS ring with four branches

Figure 6  Mode of failure for CHS ring under compressive branch loads

Figure 7  Interaction diagrams for CHS ring
Figure 8 Yield line mechanism for a planar RHS cross joints

Figure 9 Comparison of various collapse mechanism including membrane action with test results (5)

Figure 10 Yield line mechanism for a three dimensional RHS cross joints

Figure 11 Comparison of interaction contours according to API, AWS and Finite element results (3)
COMPARISON OF THE THEORETICAL BEHAVIOUR OF TWO RECTANGULAR HOLLOW SECTION TRUSSES

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Summary

The results of the Finite Element Analysis of two Warren trusses T1 and T2 are compared with existing experimental results [1]. Truss T1 is made with noded gap joints with a large gap \( g = 40 \text{ mm} \), whereas T2 is made with 100% overlap joints with large eccentricity equal to half the depth of the chord. The effect of the weld on the joint flexibility and thus on the truss behaviour is highlighted. 'Semi loof' shell elements are used to model the chord and the branches, whereas 'solid' elements are used for the weld. Some recommendations are given for numerical modelling and for design.

1. Introduction

In recent years, the behaviour of structural connections has been the target of extensive research work. Trusses made of hollow sections have traditionally been designed on a pin-joint basis. In such analysis, it is assumed that there are no local distortions of the chord and, hence, no axial displacement of the branches relative to the chord axis can occur. Previous research work [2] has shown that the connections are neither rigid, nor pinned, but the joint flexibility lies intermediate between these two extreme cases.

The flexible joint analysis technique described in reference [3] has been used to analyse two Warren trusses, T1 having a large gap, and T2 having 100% overlap joints with large eccentricity. The joints, interconnected by 'beam elements', are incorporated in the truss analysis with their respective stiffness matrices (see Figure 1). The joints and member numbering is shown in Figure 2, with the boundary conditions and the loading. The bending moment distributions in the members for both T1 and T2 are presented and have been compared with existing experimental results. The analysis was carried out at an applied load of 50 kN (carefully chosen from the experimental data to lie well within the elastic range of overall truss behaviour). The PAFEC finite element package was used.

2. Analysis of trusses T1 and T2

For symmetry reasons, only half of each truss was analysed, and the boundary conditions were such that no out-of-plane deformations were allowed. For each of the trusses a pin-joint, a rigid-joint, and a flexible joint analysis was carried out. For truss T2, a further case was considered in which the filled weld between the branches and the chord was modelled by including suitable solid elements. Comparisons are made here with the test results.
The axial force distribution in the members for both trusses is not shown here, the results of the different analysis cases agreeing closely. The pin-joint analysis agreed with the measured values for both trusses to within 5%. (This excludes member (9) - (11) at the end, that carried a small load, -zero for pin-joints- and the associated strut (9) - (8) and tie (9) - (10). A tensile force in (9) - (11) leads to an increased compressive force in strut (9) - (8) and a decreased tensile force in tie (9) - (10), the difference from the pin-joint case being 17%. Such changes might be expected in any welded trusses for which certain members carry nominal axial forces only.)

The prediction of bending moment distribution based on the rigid and flexible joint analyses did show a considerable differences in some positions, and are compared with the measured values in Figures 3 and 4 for the chord members.

It can be seen that the flexible joint analysis gives considerably improved agreement for both trusses. For the branches, rigid joint analysis of T2 gave poor results for bending moment, showing errors of 100%, with sign reversed, in some cases. The results for T1 were similar. With a flexible joint analysis there was some improvement, with signs being correctly predicted in most cases. For the chords, the specific inclusion of the weld modeling (for T2) had little effect. This was not true, however, for the bracings, and this will be discussed further in section 4.

The flexible joint analysis gave a good prediction of the mid-span deflection compared with the measured value at 50 kN ram load (the error being +2% for T1 and +6% for T2) -see Table 1. The pin and rigid joint analyses underestimated the value for T1 by 17% and 19% respectively, the agreement for T2 being closer. The joint flexibility for T1 clearly contributed significantly to the overall truss deflection.

3. Finite element modelling of the weld

3.1 Introduction

A stress analysis of a RHS joint [4] has shown that at the branch corners the stress may reach well beyond the yield stress at a load level well below the working load. Such a joint (No 6 in Figure 2) was analysed by applying actual loads taken from the experimental data obtained by Philiastides [1]. In the same reference [4], it was shown that in the finite element (F.E) analysis the force distribution in the branches, was very sensitive to the mesh used. This was especially true so far as bending moment was concerned. For these reasons it was decided to highlight the effect that the weld would have (i) on the joint flexibility, (ii) and thus on the overall truss behaviour. For truss T2 only, the weld was modelled for the K-joints but not for the corner joints.

The application of a powerful numerical discretisation procedure like the F.E method has encouraged researchers to use many kinds of element to model welds, from flat triangular [5], to quadrilateral [6], to three dimensional isoparametric [7]. Buyan et al [8] used 8-noded brick elements to model the chord/brace surface and 6-noded prism elements to model the weld for a CHS joint. The method is very expensive and cannot be justified here as the through thickness stress in the plates is negligible.
3.2 The weld model

Before incorporating the weld, the joints had been modelled using 8-noded thin plate elements, each node having 6 degrees of freedom (d.o.f.). However, to model the weld required a solid element having only 3 d.o.f. per node. Thus the two types of element were incompatible. To overcome the problem it was necessary to re-model the complete joint using SEMILOOF shell elements having three d.o.f. per node (The semiloof shell element is the most complex of the elements currently used in F.E. analysis. A literature review of the element can be found in reference [9]). Figure 5 shows the weld in the joint environment while in Figure 6 the F.E mesh is shown.

4. Influence of the weld modelling

4.1 Introduction

Results obtained for a ram load of 50 kN for different cases of analysis without the weld modelling have been presented in Figures 3 and 4. The inclusion of the weld in the analysis was found to have very little effect on axial forces, chord moments and overall deflection. However, it was found that the inclusion of the weld had a considerable effect on the bending moments in the bracings.

4.2 Effect of the weld

(i) On the joint stiffness matrix

Table 2 shows a 5x5 block of the adjusted [10] stiffness matrices of the 100% overlap joint before and after inclusion of the weld. A comparison of the figures indicated that
-Overall, the inclusion of the weld made the joint stiffer. This is reflected by an increase of the pivots (compare the stiffness matrix coefficients in Table 1).
-The nature of the sign of the stiffness coefficients generally remained unchanged.
-The inclusion of the weld changed the bending stiffness coefficients by as much as 130% in some cases (see underlined coefficients in Table 2).
-The terms most affected by the presence of the weld were those related to the branches (see bold coefficients in the matrix in Table 2). This could be due to the fact that the eccentric loading of the branch walls by the fillet welds was now recognised in the idealisation.

(ii) On the truss behaviour

The bending moments in the branches, with and without the weld are compared with the test results in Figure 7. It can be seen from the diagrams that overall the inclusion of the weld, although limited to one set of joints, brought the computed values closer to the measured ones. Although the actual moment values are small compared to the corresponding values for the chords, they are not insignificant. The branch members are likewise small compared to the chord members and the bending stresses that result can be comparable. In applying to the branch members the design rules of BS 5950 [11] for axially loaded members with bending, the terms due to the moment in the overall buckling
check are some 70% of those due to axial load. The branch moments are therefore significant.

5. Conclusions

In the analysis of RHS trusses with either gap or overlap joints, the following points may be noted:
- Axial forces can generally be predicted with sufficient accuracy with pin-joint analysis. Some relatively large errors can be expected in members nominally carrying zero load, and other members connected to them.
- For chord moment prediction in a gap joint truss (similar to T1), a rigid-joint analysis can be misleading. A flexible-joint analysis gives a much better prediction. For an overlap joint truss (such as T2) a rigid-joint analysis is satisfactory.
- A rigid-joint assumption in analysis gives poor results for branch moments for both gap and overlap trusses. A flexible joint analysis, including modelling of the weld can give greatly improved accuracy.
- Moments in RHS trusses are significant in design, both for the chord and for the branches. This is true under both static and dynamic loading.
- Designers of dynamically loaded structures must be aware of the very high localised stress levels at the corners of the branches.
6. References


Mid-span Deflection (mm)

<table>
<thead>
<tr>
<th></th>
<th>Truss T1</th>
<th>Truss T2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rigid-joint</td>
<td>17.13</td>
<td>16.00</td>
</tr>
<tr>
<td>Flexible-joint (without weld)</td>
<td>21.69</td>
<td>18.03</td>
</tr>
<tr>
<td>Flexible-joint (with weld)</td>
<td>-</td>
<td>16.71</td>
</tr>
<tr>
<td>Experimental</td>
<td>21.23</td>
<td>17.00</td>
</tr>
</tbody>
</table>

Table 1. Mid-span deflection for different analysis cases

Table 2. 5x5 Block of the Stiffness Matrix of a 100% Overlap Joint

Table 2a  Without Weld

```
9641.53  29.77
-80.83   29.77
-8635.25 -39.74 63108.18
807.16   10.50 -1833.26 3193.05
51.94    -0.54  62.79 -745.15  282.75
```

Table 2b  With Weld

```
11449.35  37.06
-104.56   37.06
-9437.98  -47.05 77594.10
185.73    21.73 -2153.75 4588.00
157.89    -2.68  75.23 -1019.60 370.58
```
Figure 1  Warren girder "Blown-up" into 'Joint elements' and 'Beam elements'

Figure 2  Layout of Trusses T1 and T2 With Loading and Boundary Conditions.  
(all dimensions in mm)
Figure 3 Bending moment distribution in the chord (truss T1)

- Test results
- F.E analysis (flexible joints)
- Rigid joint analysis

Scale 1mm → 65 Nm

Figure 4 Bending moment distribution in the chord (truss T2)
Figure 5  Welds modelled at a joint

Figure 6  Local detail of the weld F.E mesh
Figure 7 Bending Moment Distribution in the Branch Members (Truss T2)

Figure 7a (Without Weld)

Figure 7b (With Weld)

- - - Members
- - - Experimental Results
- - - F.E Analysis (Flexible Joints)

Scale: 1 mm → 8 N.m
AN EXPERIMENTAL STUDY ON JOINTS OF NEW TRUSS SYSTEM USING RECTANGULAR HOLLOW SECTIONS

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Summary

A new truss joint system with rectangular hollow steel sections was developed to increase the efficiency of truss joints in rectangular hollow steel section truss structures. This paper describes the systematic experimental study of T-joints and K-joints in the new joint system, discusses the local ultimate strength, deformation and stress of the T-joints and K-joints, and demonstrates the features and validity of the new joint system. The experimental results are organized to derive equations for estimating the local ultimate strength of the new T-joints and K-joints. The values of ultimate strength estimated by the equations are found to closely agree with the observed values of ultimate strength.

1. Introduction

In the newly developed types of joints reported here, the cross sections of the chord and brace members composed of rectangular hollow steel sections are rotated $45^\circ$ about the member axis as compared with the conventional type of joint. In the new joint, a rectangular hollow steel section that serves as the brace and has a bill-shaped end is framed into another rectangular hollow steel section that serves as the chord. This joint type features the following advantages: i) Since the sides of the chord form an angle of $45^\circ$ with the brace axis, the joint has high stiffness and smoothly carries the axial force of the brace into the chord. ii) The brace serves as a stiffener plate for the chord and restrains the local deformation of the chord. iii) The weld length of the joint is longer than that of conventional type joints. These advantages provide the new joint system with higher joint efficiency than that of joints in conventional truss systems of circular or rectangular hollow steel sections. Design equations for the new joint system cannot be found in any written material, however. Basic experiments were conducted on T-joints and K-joints in preparation for the practical application of the new joint system.

2. Experiment on Local Ultimate Strength of T-Joints

2.1 Design of Experiment

Twenty-five T-joint specimens were prepared by carefully and systematically changing the $D/T$ ratio of the chord and the $d/D$ ratio of the brace to the chord, where $D$ and $T$ are the outside dimensions across the flats and wall thickness of the chord, respectively, and $d$ is the outside dimension across the flats of the brace. The geometry and dimensions of one T-joint specimen is shown in Fig. 1. The angle between the brace and chord, or the brace angle, is $90^\circ$ in all specimens. Five $D/T$ ratios with a high frequency of usage in actual structures were selected over the range of 16.7 to 41.7, and the basic section was set at $D=250^2 \times 9$. The prepared specimens are listed, together with their experimental results, in Table 1. Braces of large enough wall thickness were used to prevent their local failure. Axial compression force was applied to the brace of each specimen to determine the ultimate strength of the joint. For the four Type F specimens, the axial compression force $N$ was applied to the chord. A specimen with axial compression force applied to the chord is shown in Fig. 2. The mechanical properties of the specimen materials are given in Table 2.
2.2 Experimental Results
When each specimen was observed during loading, it was found that as the axial compression force of the brace increased, the brace collapsed at the end, and the chord expanded in some side portions. After the ultimate strength was observed, the end of the compression brace sank deep into the chord, and the chord sides locally deformed over some region, as shown in Fig. 3. No local deformation was observed at the end of the compression brace, and the ultimate strength of the joint was governed by the local failure of the chord in the vicinity of the joint in all specimens.
Figure 4 shows an example of axial strain distribution as measured by strain gauges attached around the end of the brace. The values of the strain $\varepsilon$ are shown as being made dimensionless by the average strain $\varepsilon_A$. Strain concentration is evident at the brace edge where the ridge line of the chord crosses the brace. This strain distribution is totally opposed to the shape of strain distribution observed with conventional T-joints in circular hollow steel sections in which the strain is minimum in the contact area of the chord and brace and is maximum at the outermost edge of the brace. This is one of the most important features of joints in the new truss system discussed here.
Figure 5 shows load-deformation curves for the Type B specimens. The axial compression force of the brace is plotted along the vertical axis. The vertical deformation of the cross section of the chord as measured by the displacement meter installed in the chord is plotted along the horizontal axis. An elastic linear relation is observed in the initial stage. The deformation increment gradually increases until the maximum ultimate strength is reached. A sudden load decrease is not observed after the maximum ultimate strength, and a plastic deformation capacity sufficient enough for local deformation is expected.
Figure 6 shows load-deformation curves when the axial compression force was applied to the chord. The load-deformation curves obtained when the axial compression force was not applied to the chord are included for the purpose of comparison. When the axial compression force $N$ is 0.3$N_y$ (where $N_y$ is the axial yield compression force of the cord), the maximum ultimate strength and stiffness are little different from those observed when the axial compression force was not applied to the chord. When the axial compression force applied to the chord is 0.6$N_y$, the loss of stiffness occurs when the load is still low, and the maximum ultimate strength declines to about 70% of that observed when the axial compressive force is not applied to the chord.
Figure 7 shows the maximum ultimate strength observed when the axial compression force was applied to the chord. The coordinate $\mu$ is the maximum ultimate strength made dimensionless by the maximum ultimate strength measured when the axial compression force was applied to the chord. Each curve in Fig. 7 indicates the influence term $\mu$ of axial compression force reported in the written material. When the axial compression force applied to the chord exceeds 0.3$N_y$, its effect on the maximum ultimate strength is considered to increase. The magnitude of the effect with the new joint system is shown to be the same as observed in existing tubular joints.

2.3 Maximum Ultimate Strength of T-Joints
Table 1 lists the observed values of local ultimate mate strength $P_{U}$ of the T-joints. Table 1 also includes the calculated values of local ultimate strength $P'_{U}$ of T-joints in circular hollow steel sections having the same cross sectional area and geometric moment of inertia as those of the T-joints specimens discussed here. The local ultimate strength $P_{U}$ of the new joints and the local ultimate strength $P'_{U}$ of the conventional joints change with the brace outside dimension and chord wall thickness. The local ultimate strength $P_{U}$ of the new joints exceeds the local ultimate strength $P'_{U}$ of the conventional joints for almost all test specimens. This tendency increases with decreasing chord wall thickness and brace outside dimension. In an extreme case, $P_{U}$ is nearly three times as large as $P'_{U}$. This feature verifies the validity of the new joint system.
When the observed local ultimate strength $P_U$ of the T-joints in the new joint system is arranged and statistically processed, the following equation is derived for estimating the local ultimate strength $P_O$ of the T-joints in the new joint system:

$$
\frac{P_O}{T\sigma_y} = 0.211 - 0.147\frac{d}{D} + \frac{1}{1.794 - 0.942\frac{d}{D}} D
$$

(1)

Figure 8 shows the relationship between the experimental values of $P_U$ and the values of $P_O$ estimated by Eq. (1). It is apparent that the local ultimate strength of the T-joints in the new joint system can be represented by $P_O$ in Eq. (1) with very high accuracy.

Figure 9 shows the relationship between the observed values of local ultimate strength $P_U$ and the calculated values of local ultimate strength $P_O$, together with the calculated values of local ultimate strength $P_O$ of T-joints in other circular hollow steel sections. The calculated values of local ultimate strength of T-joints in the new joint system, circular hollow steel sections, and conventional rectangular hollow steel sections are indicated by the solid, broken, dot-dash curves, respectively.

3. Experiment on Local Ultimate Strength of K-Joints

3.1 Design of Experiment

Many experimental results are reported for K-joints in circular hollow sections. The experimental results show that the ultimate strength of a K-joint is affected only a little by the tension brace and instead depends on the brace gap, brace angle, and offset (eccentricity) between the intersection of brace centerlines and the chord centerline.

In the present experiment, 16 test specimens were designed as shown in Table 3 with $D$, $d_c/D$, $d_t/D$, $g$, and $e$ taken as parameters (where $D$ is outside dimension across the flats of the chord; $T$ is the wall thickness of the chord; $d_c$ is the outside dimension across the flats of the compression brace; $d_t$ is the outside dimension across the flats of the tension brace; $g$ is the gap between the braces; and $e$ is the offset or eccentricity between the brace and chord centerlines). The geometry of one K-joint specimen is shown in Fig. 10. The brace angle $\theta$ is 45° for all specimens. The mechanical properties of the specimen materials are given in Table 4. The loading apparatus shown in Fig. 11 was used to apply axial compression force to one brace, axial tension force to the other brace, and axial compression force to the chord. When each test specimen was installed on the loading apparatus, due care was exercised so that the centerline connecting the upper and lower supports would coincide with the centerline of the compression brace, thereby preventing the generation of bending moment at the joint.

3.2 Experimental Results

As the load increased during the loading process, local deformation occurred in the chord near the end of the compression brace, except for the specimen KE-1. The deformation of chord cross sections is shown in Fig. 12. The deformation of the chord is large between the compression brace and tension brace. For the specimen KE-1 with large eccentricity $e$, the chord suffered local buckling in the rear of the contact area with the tension brace.

Fig. 13 shows load-deformation curves for the specimen KE-2. One curve shows the deformation of the chord in the contact area with the compression brace, while the other curve shows the deformation of the chord in the contact area with the tension brace. It is evident that the ultimate strength of the K-joint depends on the deformation of the chord in the contact area with the compression brace.

Fig. 14 shows load-deformation curves for the Type KG specimens of the same chord cross section but different brace gap. The load is made dimensionless by the estimated ultimate strength $P_O$ of T-joint as given by Eq.(1). When there is no brace gap, the local ultimate strength $P_U$ is high, but the load drops rapidly.
Fig. 15 shows load-deformation curves for Type KE with the same brace cross section and different eccentricity e. It is apparent that when the eccentricity is small, the ultimate strength is high, but the loss of load is large after the maximum ultimate strength. This fact is important and should be born in mind during the actual design of hollow steel section truss joints. If an attempt is made to design a truss joint without eccentricity, the brace gap changes with the brace outside dimension, and there is no brace gap when the brace dimension is large. As a result, the joint increases in local ultimate strength, but undergoes brittle failure.

Fig. 16 shows the relationship between the local ultimate strength $P_u$ and brace gap $g$. It is apparent that the chord wall thickness does not exert any great effect on the $P_u$-g relationship and that the effect of the gap on the local ultimate strength is not great, either. The brace gap is dependent on the brace outside dimension and eccentricity. When the ultimate strength is taken into account, there seems to be no need for distinguishing the brace gap dependent on the brace outside dimension from the brace gap dependent on the eccentricity.

The local ultimate strength $P_u$ of the specimens is listed in the Table 5. In the table, $P_0$ is the ultimate strength estimated by Eq.(1) for a T-joint of the same type as the K-joint in question; $cP_k$ is the ultimate strength of a K-joint in circular hollow steel sections having the same section properties as the K-joint in question; $rP_k$ is the ultimate strength of a K-joint in rectangular hollow steel sections made by the conventional method; and $P_k$ is the ultimate strength of the K-joint in question and calculated by an equation described later. Figure 17 compares the ultimate strength of K-joints made by the new method with that of K-joints in circular hollow steel sections with the same section properties. The ultimate strength of K-joints made by the new method is superior to that of K-joints in circular hollow steel sections with the same section properties. Like the T-joints, this tendency increases with decreasing $dc/D$ and increasing $D/T$. Figure 18 compares the ultimate strength of K-joints in rectangular hollow steel sections made by the new method with that of K-joints in rectangular hollow steel sections made by the conventional method. The validity of the new joint system is made clearer when observed in comparison with joints in circular hollow steel sections.

Figure 19 shows the principal strain distribution of the chord as determined by strain gauges attached to the chord portion between the braces. Large strain is produced in the chord sides to which the brace is jointed, and little strain is produced in the other chord sides. This means that in the vicinity of the joint, the force applied to the compression brace is not equally transmitted throughout the cross section of the chord, but is transmitted to the tension brace or the chord through a narrow region.

The strain distribution of the brace end shows that stress concentration is produced at the portion where the ridge line of the chord crosses the brace, as is the case of the brace in the T-joint. It may be safe to think that the force applied to the brace concentratedly acts on that portion.

### 3.3 Equation for Evaluating Maximum Ultimate Strength of K-Joints

It was judged from the brace and chord strain distributions that the force applied to the compression brace acts on the chord as two concentrated loads and spreads through a narrow region in the chord. Accordingly, an attempt was made to evaluate the ultimate strength of K-joints in the new truss system.

Assume that two concentrated forces applied at points 1 and 4 spread in the region 2-1-3 as shown in Fig. 20 and that the resultant stress is uniformly distributed in the region 2-1-3 as shown in Fig. 21. Also assume that when the combination of the normal stress $\sigma$ and shear stress $\tau$ in the section 4-3 reaches the yield stress $\sigma_y$, the ultimate strength of the section 4-3 reaches the limit and governs the local ultimate strength of the joint. The local ultimate strength of a K-joint in the new joint system under these conditions can be estimated by

$$\frac{P_k}{T\sigma_y} = \frac{4\alpha}{\sqrt{1 + 2\sin^2 \theta}} \left(\frac{D}{T}\right)$$

(2)
The effective section modulus $\alpha$ depends on $d_c/D$ and $D/T$ and assumes the values shown in Fig. 22.

The value of ultimate strength $P_k$ estimated by using $\alpha$ is related to the observed value of ultimate strength $P_u$ in Fig. 22. It is evident that the ultimate strength of K-joints estimated by Eq. (2) reproduces the observed ultimate strength of K-joints with considerable accuracy and on the safe side, although no allowance is made for the position and outside dimension across the flats of the tension brace and for the bending of the sides of the chord.

4. Conclusions

Experiments were conducted on the local ultimate strength of joints in rectangular hollow steel sections in a new truss system. The local deformation and stress conditions of the joints were determined as a result. The truss joints studied have a sufficient plastic deformation capacity, are dynamically superior to conventional joints in circular hollow steel sections, and are considered to pose no serious problems for practical application.

The experimental findings about the new truss system are as follows:
1) The distribution of local stress in the end of the compression brace in a new K-joint is totally different from that observed in conventional K-joints. The axial force applied to the compression brace acts at the ridge of the chord as if it were two concentrated loads.
2) The ultimate strength of new joints is higher than that of conventional joints in circular hollow steel sections. This tendency increases with decreasing chord wall thickness and brace outside dimension.
3) The ultimate strength of T-joints can be estimated by Eq. (1) with high enough accuracy.
4) The ultimate strength of K-joints can be estimated by Eq. (2) with considerable accuracy and on the safe side.

References

Table 1  Summary of Specimen Types and Experimental Results for T-Joints

<table>
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<tr>
<th>Specimen No.</th>
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<th>D/T</th>
<th>Bracing d (mm)</th>
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<th>N/Hy</th>
<th>Pu (tf)</th>
<th>Pu' (tf)</th>
<th>Pu/Pu'</th>
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Table 2  Mechanical Properties of Chord Materials for T-Joints

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<th>σu (tf/cm²)</th>
<th>E (%)</th>
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Fig. 1  Test Specimen for T-Joints

Fig. 2  Test Set-up for T-Joints under Axial Force

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Fig. 3 Profiles of Deformed Chord of T-Joint

Fig. 5 Load-Deformation Relationships (B-Type)

Fig. 6 Load-Deformation Relationships (F-Type)

Fig. 7 Influence Prestressing Chord on the Strength of T-Joints

Fig. 4 Strains Distributions in Bracing of T-Joint

Fig. 8 Data for T-Joints Compared with Eq.(1)

Fig. 9 Ultimate Strength of T-Joints
<table>
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<tr>
<th>Specimen No.</th>
<th>Nominal Size</th>
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<th>Bracing</th>
<th>Joint ( g / T )</th>
<th>( e / (D/2) )</th>
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| KE - 1      | -200\times 8   | 25.0      | 0.7     | 0.7            | 0.7            |
| 2           | -200\times 12  | 0.5       | 0.5     | 0.0            | 0.0            |
| 3           | -200\times 12  | 0.5       | 0.5     | 0.0            | 0.0            |
| 4           | -140\times 12  | 0.7       | 0.7     | 0.0            | 0.0            |
| 5           | -120\times 9   | 0.6       | 0.6     | 40             | 8.9            |
| 6           | -140\times 9   | 0.7       | 0.7     | 0.0            | 0.0            |
| 7           | -120\times 9   | 0.6       | 0.6     | 80             | 17.8           |

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<th>Steel Base Metal</th>
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<td>-0.02</td>
<td>71.5</td>
<td>56.3</td>
<td>1.27</td>
<td>1.69</td>
<td>3.68</td>
<td>56.5</td>
</tr>
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<td>5</td>
<td>26.3</td>
<td>0.699</td>
<td>0.700</td>
<td>0.0</td>
<td>0.08</td>
<td>108.2</td>
<td>78.5</td>
<td>1.39</td>
<td>0.88</td>
<td>2.36</td>
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</tr>
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<td>6</td>
<td>26.2</td>
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<td>0.297</td>
<td>8.9</td>
<td>0.00</td>
<td>68.4</td>
<td>51.2</td>
<td>1.34</td>
<td>1.58</td>
<td>2.30</td>
<td>51.4</td>
</tr>
<tr>
<td>7</td>
<td>26.2</td>
<td>0.402</td>
<td>0.596</td>
<td>8.5</td>
<td>0.02</td>
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<td>55.5</td>
<td>1.40</td>
<td>1.39</td>
<td>2.58</td>
<td>63.8</td>
</tr>
</tbody>
</table>

* : from actual measurement
Fig. 10 Test Specimen for K-Joints

Fig. 11 Test Set-up for K-Joints

Fig. 12 Profiles of Deformed Chord of K-Joint

Fig. 13 Load-Deforption Relationships (K-2)

Fig. 14 Load-Defomation Relationships (KG-Type)

Fig. 15 Load-Defomation Relationships (KG-Type and K-5)

Fig. 16 Relationships between Ultimate Strength and Gap "g" of K-Joints
THE DESIGN OF SHEAR TABS WELDED TO HSS COLUMNS

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Summary

The results of two research projects are presented in this paper. One was an experimental investigation of connection behavior for shear tabs welded to HSS columns. The specific objectives of the testing and evaluation were:

a) Evaluate the capacity of the tabs and identify the limit states.
b) Evaluate the strength of the tube wall.
c) Determine the affect of tube wall flexibility, beam stiffness and shear tab dimensions on the reaction eccentricity.
d) Determine what affect the bolt tightness has on the eccentricity and the ultimate capacity of the shear tab.

The purpose of a related project was to obtain an indication of whether the shear tab connection would reduce the HSS column strength. In this limited study the variables were a comparison of shear tabs and through-plates, and the tightness of the bolts.

1. Introduction

One of the simplest and most economical simple beam connections is the single plate framing connection, or shear tab. Presently, the shear tab is most extensively used with wide-flange columns or other supporting structures which provide essentially rigid faces at the weld line. When the shear tab is welded to an HSS (Hollow Structural Section) column, Fig. 1, the flexibility of the tube face precludes a rigid face assumption and there is uncertainty regarding the application of design procedures that are based on research using W shaped columns. There is also concern by some designers regarding a possible weakening of the column due to the local distortion of the column face at the shear tab. Through-plates are often specified to provide some reinforcement to the HSS walls. The slotting of the HSS and fitting the through-plate adds considerable cost to the connection. This investigation examines the behavior and evaluates the shear capacity of the shear tabs welded to HSS columns and presents some limited information on the effect of shear tabs on HSS column capacities.

2. Background

The shear tab is considered a simple connection, and as such, is normally proportioned for shear only. Sufficiently large moments, however, may develop in the connection which can affect the design of the fastening elements, i.e. bolts and welds. The amount of moment developed in a simple connection is $M = R \times e$, where "$R$" is the reaction shear and "$e$" is the reaction eccentricity. The reaction eccentricity is defined as the distance from some reference (weld line or bolt line).
to the point of zero moment. The reaction eccentricity depends on a number of factors, such as the number of bolts, the dimensions and material of the shear tab, the amount of end rotation in the beam and the relative rigidity or flexibility of the supporting structure, or column. When the shear tab is welded to a rigid support, the rotational restraint in the connection is released through shear deformation of the bolts, distortion in the bolt holes, out-of-plane bending of the plate and bolt slippage. When the shear tab is welded to a flexible support, such as a tube wall, the flexibility provides an additional path through which the rotational restraint can be released.

The tightness of the bolts used in shear tab connections may be divided into two categories, snug tight and fully torqued. Snug tight is defined as the tightness that exists when all plies in a joint are in firm contact. This condition is attained by tightening the bolts with a hand wrench and stopping when further bolt turning can only be accomplished by extra effort. Fully torqued bolts are tightened until the tension in the bolt reaches 70% of the specified minimum tensile strength of the bolt. Fully torqued bolts rely on friction to carry the working load shear and the rotation of the shear tabs would be the same as the end slope of the beam. With snug bolts or with fully torqued bolts after slippage occurs, there will be a slope change between the tab and the beam.

Research on the shear tab has been done by Richard [1] and Astaneh [2]. Each investigation studied shear tabs welded to wide-flange columns and produced a design procedure for this type of connection. The former study concentrated on reaction eccentricity and involved finite element modeling, cantilever testing to obtain moment-rotation characteristics and full scale testing. The study found that connection eccentricity increased with the number of bolts, the size of the bolts and the thickness of the plate. These tend to increase the rotational stiffness of the plate. An increase in the span-to-depth, L/d, ratio of the beam also increased the connection eccentricity.

The research in Astaneh's study concentrated on experimental determination of the ultimate strength of the connection. The failure modes identified from the testing were:

a) Shear failure of bolts
b) Yielding of gross area of plate
c) Fracture of net area of plate
d) Fracture of welds
e) Bearing failure of beam web or plate

Each component in the connection (the bolts, welds and plate) must be designed to resist either shear or a combination of shear and moment. The moment developed in the connection depends on the reaction eccentricity. Astaneh found that the eccentricity was proportional to the number of bolts in the connection.

The flexibility of the tube wall introduces an additional limit states into the design of shear tabs welded to HSS columns. The tube wall in this type of connection may experience either a bending failure or punching shear failure.
The limit state of bending of tube walls with members branching into
them is determined using yield line theory [3] and plastic analysis.
For particular configurations of tube walls and branching members, a
pattern of yield lines is developed that form the boundaries of
triangular and trapezoidal plates. Bending moments in the tube wall are
redistributed along these lines and become the locations for the
formation of plastic hinges. When enough hinges form, a mechanism
develops and a limit load is reached.

Punching shear failure is defined as the point at which the normal
bending stress in the tab exceeds the through-thickness shear resistance
of the tube wall around the perimeter of the branching member. Assuming
that the tube face must develop the yield strength of a unit length
of the shear tab, an expression for the minimum thickness of the tube
wall can be obtained.

\[ t_{tw} \geq \frac{[F_{ypl}]}{1.2[F_{utw}]} t_{p} \]  

(1)

Even when the connection is properly designed to carry the beam reaction
shear, it is important to know if the flexibility inherent and desired
in the connection might affect the column capacity. The moment
developed in the connection is small so that it would have no more
impact than the eccentricity of the reaction with a wide flange column.
The major concern is the local imperfection resulting from the
distortion of the tube face as the connection rotates. Some early tests
[4] indicated that there may be a reduction in the column strength of an
HSS column. However, these tests were conducted with cantilever beams
that had large eccentricities from the nature of the load and large end
rotations. With beams that are simply supported at both ends, the
rotation is self limiting to the end slope of the beam and the column is
braced at the point of the local distortion.

3. Connection Test Program

In establishing the details of the connections to be tested, standard
details used in United States fabricating practice were used. The
following parameters were fixed:

a) Tube material : ASTM A500 Grade B, (\(F_y = 317\)MPa)
b) Shear Tab material : ASTM A36, (\(F_y = 248\)MPa)
c) Beam material : ASTM A36, (\(F_y = 248\)MPa)
d) Bolt type, 19mm dia. : ASTM A325, (\(F_u = 838\)MPa), threads
   excluded from the shear plane
e) Weld material : E7018 electrodes, (\(F_u = 483\)MPa)
f) Distance from welds to bolts : 76mm

As indicated in Table 1, the variables in the test program were three
different tab sizes, five ratios of flat width/thickness (b/t) in the
HSS, the number and tightness of bolts and two beam sizes. Each test is
referred to by an alphanumerical code; the letter, "H" or "L", indicates
the rotation group, "high rotation" or "low rotation"; the number is the
b/t ratio of the tube wall; and the last letter indicates whether the
bolts are snug "S" or tight"T."
GROUP TEST SHEAR TAB DIMENSIONS

<table>
<thead>
<tr>
<th>GROUP</th>
<th>SHEAR TAB TUBE SIZE</th>
<th>BOLT TIGHTNESS</th>
<th>WELD SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td>HIGH</td>
<td>H5T 229 x 7.9</td>
<td>102 x 152 x 12.7</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>H5S 229 x 7.9</td>
<td>102 x 152 x 12.7</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>H10S 229 x 7.9</td>
<td>102 x 152 x 7.9</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>H16T 229 x 7.9</td>
<td>152 x 76 x 7.9</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>H16S 229 x 7.9</td>
<td>152 x 76 x 7.9</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>H40T 229 x 7.9</td>
<td>203 x 76 x 4.8</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>H40S 229 x 7.9</td>
<td>203 x 76 x 4.8</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>H45T 229 x 6.4</td>
<td>305 x 203 x 6.4</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>H45S 229 x 7.9</td>
<td>305 x 203 x 6.4</td>
<td>45</td>
</tr>
<tr>
<td>LOW</td>
<td>L5T 381 x 7.9</td>
<td>104 x 152 x 12.7</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>L5S 381 x 7.9</td>
<td>102 x 152 x 12.7</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>L16S 229 x 7.9</td>
<td>152 x 102 x 7.9</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>L45S 381 x 7.9</td>
<td>305 x 203 x 6.4</td>
<td>45</td>
</tr>
</tbody>
</table>

The width of all shear tabs was 114 mm. The "Group" designation indicates the relative end rotation of the test beam. The tube dimension listed first is the width of the tube wall to which the shear tab is welded. The length of the columns used in the tests was 1.52 m, with the connection at mid height.

Table 1 - Properties of the shear tab test specimens

The test beams with single concentrated loads were designed to transfer the same end reaction and rotation as a comparable uniformly loaded beam. It can shown that the length of a simulated uniformly loaded beam, \( L_u \), is related to the length of the test beam with a concentrated, \( L_p \), and the distance from the concentrated load to the reaction, \( \alpha \).

\[
L_u = \sqrt{2\alpha(2L_p - \alpha)}
\]  

(2)

Two uniformly loaded beams with different L/d ratios were chosen so that connection behavior as a function of end rotation could be studied. The end rotation is a function of the L/d ratio, \( \Theta = (2/3)(f/E)(L_u/d) \). One beam was designated the "high rotation" beam, which was used with the 3 bolt connections and had an \( L_u/d \) ratio of 23, which is typical of the span-to-depth ratio of beams used in building construction. The second beam was designated the "low rotation" beam and was used with both the 3 bolt connections and the 5 bolt connections. This beam had an \( L_u/d \) ratio of 9.75. The beams selected were a W310x129 and a W460x106. A summary of the beam properties is given in Table 2.

Table 2 - Properties of the test beams

<table>
<thead>
<tr>
<th>GROUP</th>
<th>BEAM</th>
<th>( L_p )</th>
<th>( \alpha )</th>
<th>( L_u )</th>
<th>( L_u/d )</th>
<th>( \Theta_{max} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>HIGH ROTATION</td>
<td>W310x129</td>
<td>7.62m</td>
<td>1.33m</td>
<td>6.10m</td>
<td>23</td>
<td>0.0195r</td>
</tr>
<tr>
<td>LOW ROTATION</td>
<td>W460x106</td>
<td>5.08m</td>
<td>0.77m</td>
<td>3.81m</td>
<td>10</td>
<td>0.0080r</td>
</tr>
</tbody>
</table>

Test Setup

The basic test setup is shown in Fig. 2. The sensors used to obtain data from the connection tests included load cells, strain gages and displacement transducers. The load cells were used to determine the applied force and the beam reaction at the far end. By subtracting the
the far end reaction from the loading force, the shear force in the connection was determined. The primary displacement transducer was used to measure the displacement at the shear tab.

Six gages were mounted on the beam between the connection and the point load, 3 on the top flange and 3 directly beneath on the bottom flange. To determine the reaction eccentricity, these strains were averaged, using the formula \( \epsilon_{\text{avg}} = \frac{\epsilon_b - (-\epsilon_t)}{2} \), and a strain gradient was established. The distance from the point of zero strain to the tube wall was defined as the reaction eccentricity. This method of analysis was very sensitive to small variations in strain, so linear regression was used to partially compensate for the sensitivity and all eccentricity values were qualitatively checked by examining the bolt bearing patterns. The connection moments were determined at the bolt line and the weld line by multiplying the reaction shear by the corresponding reaction eccentricity.

Test Results

The results from the tension tests of the shear tab coupons and from the tube material are given in Table 3. Two different plate materials in the 7.9mm thickness were provided by the fabricator. The material with the 7.7mm thickness had a rounded stress-strain curve, while the curves for the other two tab materials were sharp yielding. The stress-strain curves for the tube material were rounded, similar in shape to those for the 7.7mm shear tab material.

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Yield Stress Nominal (MPa)</th>
<th>Yield Stress Actual (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Elongation %</th>
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</thead>
<tbody>
<tr>
<td>TABS</td>
<td>7.9</td>
<td>7.70</td>
<td>248</td>
<td>479</td>
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<tr>
<td></td>
<td>7.9</td>
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<td></td>
<td>6.4</td>
<td>6.07</td>
<td>248</td>
<td>501</td>
</tr>
<tr>
<td>TUBES</td>
<td>4.8</td>
<td>4.39</td>
<td>317</td>
<td>491</td>
</tr>
<tr>
<td></td>
<td>6.4</td>
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</tr>
<tr>
<td></td>
<td>7.9</td>
<td>7.75</td>
<td>317</td>
<td>440</td>
</tr>
<tr>
<td></td>
<td>12.7</td>
<td>11.94</td>
<td>317</td>
<td>495</td>
</tr>
</tbody>
</table>

Table 3 - Material properties

The maximum shear resisted by each connection and the averaged reaction eccentricity relative to the weld line are summarized in Table 4. The corresponding theoretical capacities for five failure modes are also listed. Failure in each connection was defined as the point at which the shear versus tab displacement curve flattened. The load beams, which had to be reused for each test, were designed to remain elastic at the calculated shear yield of the tabs. The unexpectedly high yield stress of the shear tab material increased the shear capacity of the tabs 150%; at this load, the beams would become inelastic. Therefore, four tests were stopped prematurely to prevent the beam from yielding. The maximum loads normalized by the theoretical yield strengths of the tabs \((V_{\text{res}}/V_r)\) is plotted in Fig. 3.
Theoretical test tpl dpl cave Vtest kN Vt kn Vfrac kn Vbv kn Vbb kn Vweld kn

<table>
<thead>
<tr>
<th>TEST</th>
<th>tpl mm</th>
<th>dpl mm</th>
<th>cave mm</th>
<th>Vtest kN</th>
<th>Vt kn</th>
<th>Vfrac kn</th>
<th>Vbw kn</th>
<th>VWBD kn</th>
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<tr>
<td>H5T</td>
<td>8.31</td>
<td>229</td>
<td>135.6</td>
<td>333*</td>
<td>374</td>
<td>418</td>
<td>294</td>
<td>396</td>
</tr>
<tr>
<td>H5S</td>
<td>7.77</td>
<td>229</td>
<td>123.7</td>
<td>329*</td>
<td>409</td>
<td>374</td>
<td>325</td>
<td>387</td>
</tr>
<tr>
<td>H10S</td>
<td>8.31</td>
<td>229</td>
<td>103.1</td>
<td>326*</td>
<td>374</td>
<td>418</td>
<td>365</td>
<td>494</td>
</tr>
<tr>
<td>H16T</td>
<td>7.82</td>
<td>229</td>
<td>59.2</td>
<td>313</td>
<td>409</td>
<td>374</td>
<td>387</td>
<td>471</td>
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<td>H16S</td>
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<td>64.8</td>
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<td>418</td>
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<td>H40T</td>
<td>7.77</td>
<td>229</td>
<td>30.1</td>
<td>321*</td>
<td>409</td>
<td>374</td>
<td>329</td>
<td>396</td>
</tr>
<tr>
<td>H40S</td>
<td>7.77</td>
<td>229</td>
<td>30.0</td>
<td>310*</td>
<td>409</td>
<td>374</td>
<td>325</td>
<td>391</td>
</tr>
<tr>
<td>H45T</td>
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<td>229</td>
<td>24.9</td>
<td>266</td>
<td>289</td>
<td>311</td>
<td>316</td>
<td>316</td>
</tr>
<tr>
<td>H45S</td>
<td>7.82</td>
<td>229</td>
<td>33.8</td>
<td>313</td>
<td>409</td>
<td>374</td>
<td>334</td>
<td>405</td>
</tr>
<tr>
<td>L5T</td>
<td>7.80</td>
<td>381</td>
<td>71.9</td>
<td>636</td>
<td>681</td>
<td>140</td>
<td>698</td>
<td>845</td>
</tr>
<tr>
<td>L5S</td>
<td>7.80</td>
<td>381</td>
<td>50.0</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L16S</td>
<td>8.31</td>
<td>229</td>
<td>28.7</td>
<td>373</td>
<td>374</td>
<td>418</td>
<td>325</td>
<td>436</td>
</tr>
<tr>
<td>L45S</td>
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<td>381</td>
<td>8.1</td>
<td>587</td>
<td>685</td>
<td>623</td>
<td>574</td>
<td>698</td>
</tr>
</tbody>
</table>

Table 4 - Experimental and theoretical maximum shear capacities

* Tests H5T, H5S, and H10S began yielding in shear, but the tests were stopped because the beam had reached yield. Test L5S also began yielding in shear, but the test was stopped due to tipping of the test fixtures.

The capacities for the bolts in shear and bearing (Vwb and Vbb) and the welds (Vweld) take into account the eccentricity of load and are calculated by the ultimate strength method. The theoretical shear yield capacity is denoted Vt and the shear fracture of the tab by Vfrac.

The following failure modes were identified from the testing:

a) Yielding of the gross area of the tab  
b) Bearing failure of the tab  
c) Fracture and yielding of the welds  
d) Punching shear failure of the tube wall  
e) Surface tearing of tube wall material beneath weld  
f) Lateral buckling of the tab  
g) Shear yielding and fracture of the bolts

For those connections which failed, all experienced a shear yielding of the gross area of the tab. This was indicated by the flaking of the white wash and the obvious shear distortion in the tab. The shear tab, however, was able to sustain load and even experience an increase into the strain hardening range. Actual failure was precipitated by some other failure mechanism. In all cases, the maximum shear resisted by the tab was less than the nominal calculated gross area yield and as shown in Fig. 3, the reduction in load carrying capacity increased slightly as tube wall deflection increased. With the flexible tube walls (b/t > 16), the yielding seemed confined to the plate area between the top and bottom bolt holes, as indicated by white wash flaking. Therefore, less plate area was available for resistance. In contrast,
the shear tabs welded to the stiff tube walls (b/t ≤ 10) experienced yielding along the entire length of the plate and there was no noticeable distortion in the tube.

All of the connections exhibited possible multiple failure modes. Most of the failure modes are typical of simple connections, but two should be pointed out for discussion. One of these is the separation of the weld from the tube wall, which may be attributed to either lamellar tearing of the tube wall material or lack of penetration of the weld into the tube wall. The shear tabs used in the first two tests were welded to the tube wall with no preparation of the materials to take the weld. The subsequent failure in each of these two tests was a separation of the weld from the tube wall. For the remaining tests, the mill scale on both the tube wall and the tab, in the area of the weld, was removed and a preheat of approximately 100°C was applied to the connection before full welding. The separation problem was practically eliminated, with a few non-critical local exceptions. Therefore, it is recommended that careful preparation of the weld surfaces be undertaken to avoid this type of failure.

Another possible mode of failure is the warping of the tab due to lateral-torsional twisting of the beam end. The test beams were not restrained laterally during the testing (the point load provided for some lateral restraint). It was concluded that the shear tab offers little resistance to lateral-torsional distortion and it is recommended that if the shear tab is at the end of a long unbraced span, a brace point should be established near the shear tab.

The reaction eccentricity was most affected by the b/t ratio of the tube wall and the L/d of the beam. It decreased with b/t of the tube wall and L/d of the beam, as shown in Fig. 4. The moments developed in the connections never exceeded 20% of the fixed end moment, so the beam framing into the shear tab can be considered simply supported.

The method of bolt installation had no noticeable effect on the ultimate load (Fig. 3.) At working loads the connections with the fully torqued bolts were stiffer than the connections with the snug tight bolts. The deflection of the beam at the bolt line was less and the eccentricity was larger. As the connection load increased, the bolts were brought into bearing and the connections exhibited behavior similar to the behavior of the connections with snug tight bolts. The average eccentricities for the two different bolt tightness conditions showed very little difference for the various connections.

4. Column Test Program

This program consisted of four tests of the type shown in Fig. 5. A 5.08m long HSS column had W beams connected by tabs at midheight. The beams were loaded to produce 133kN reactions at the shear tabs and then the column was loaded at the top until failure occurred in the lower half. The variables in the tests were whether the connection was a tab or through-plate and whether the bolts were snug or fully tight. The b/t of the column face was 19. The connections were made with three 19.1mm bolts and 229x114x1.4mm plates.
The major conclusion of the study is shown in Fig. 6. These are the load vs. deflection curves, where the deflection is the lateral displacement at the middle of the lower half of the column and the load is that in the lower half. The ultimate loads in the lower half of the columns are also tabulated in Fig 6.

5. Conclusions

a) Although observations indicate that the tabs in the connections all yielded in shear, they ultimate loads were less than the calculate shear yield. However, shear yielding did not terminate the load carrying capacity. Failures in the tabs were by other limit states, particularly bolt bearing, weld failure, and net section fracture. Lateral-torsional distortion is an important consideration. Bolt tightening did not influence the tab limit states.

b) Punching shear and weld tearing are the important limit states for the HSS wall. Yield line mechanisms did not fully develop due to the self limiting rotations of the simply supported beam.

c) Eccentricity increases with L/d of the beam and the tab length and thickness. It decreases with b/t of the HSS. Except for very stiff HSS with small b/t or stiff beams with small L/d, the eccentricity is within the tab between weld and bolt lines.

d) Shear tabs did not reduce the HSS column capacity in comparison to through-plates and the bolt tightening had only slightly more impact on column strength.

6. Symbols

- a distance from column face to concentrated load on test beam
- b flat width of HSS column (full width - 3*thickness)
- d depth of the beam
- dp1 depth or length of the shear tab plate
- E modulus of elasticity
- e eccentricity; eb to bolt line; ew to weld line
- eave average eccentricity to weld line during a test
- Fy yield strength; Fypl of shear tab plate
- Fu ultimate strength; Futw of the tube wall
- f stress level in beam
- L beam length; Lu simulated uniformly loaded beam; Lp test beam
- M moment in the connection
- R beam reaction
- t thickness; tw tube wall; tpl shear tab plate
- V_TEST experimental shear capacity of connection
- V_Y theoretical shear yield capacity of shear tab
- V_FRAC theoretical net section shear fracture capacity of shear tab
- V_BY theoretical shear capacity of bolts including eccentricity, eb
- V_BB theoretical bearing capacity of bolts including eccentricity, eb
- V_WELD theoretical capacity of welds including eccentricity, ew
- ε measured strain in the beam; eb bottom; et top; eave average
- θ slope at the end of the beam

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7. References


![Figure 1 - Shear tab on HSS column](image1)

![Figure 2 - Setup for connection tests](image2)
Figure 3 - Eccentricities

Figure 4 - Shear capacities

Figure 5 - Column test

Figure 6 - Column load-deflection
BUCKLING AND POST-BUCKLING STRENGTHS OF COLUMNS WITH A SQUARE HOLLOW SECTION

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Summary

A tubular column with a cold-formed and welded square hollow section undertakes a plastic strain history due to roll forming and seaming by welding, and accordingly has complex residual stresses. Flexural buckling and post-buckling strengths of a tubular column are influenced by these residual stresses and plastic strain history such as the Bauschinger and work hardening effects. In this paper, buckling and post-buckling strengths of tubular columns with a square hollow section are examined by considering both bi-axial residual stresses and plastic strain history.

1. Introduction

A cold formed and welded square hollow tube undertakes a plastic strain history due to roll forming and seaming by welding, and accordingly has complex residual stresses. Flexural buckling and post-buckling strengths of a tubular column are influenced by these residual stresses and the Bauschinger and work hardening effects. Different from the hot rolled shapes, the cold formed and welded square tube contains not only longitudinal residual stresses but also circumferential residual stresses. In this paper, roll forming process of a square hollow section was analyzed first, to make clear the longitudinal and circumferential residual stress distributions and the Bauschinger and work hardening effects. And the buckling and post-buckling strengths of a square hollow section column were analyzed considering bi-axial stress condition.

2. Bi-axial stress-strain relationship

To analyze the manufacturing process and also the buckling behavior of a tubular column, stress-strain relationship under bi-axial stress condition is necessary. A two surface type model proposed by an author[1] is used here such as shown in Fig.1. The outer surface is a loading surface and the inner surface a control surface. Kinematic hardening occurs when the center of the loading surface falls within the control surface, while combined isotropic hardening or softening and kinematic hardening occurs when the center of the loading surface moves on the control surface. To obtain initially bi-linear and subsequently tri-linear type uniaxial stress-strain relationship such as shown in Fig.2, a series model is used. The stress-strain relationships for the element "a" and "b" are bi-linear type shown in Fig.3. Using von Mises's yield condition, the yield and control surfaces are defined as:

\[ f = (\sigma_x - \sigma_x^0)^2 + (\sigma_y - \sigma_y^0)^2 - (\sigma_x - \sigma_y)(\sigma_y - \sigma_y^0) - \delta^2 \]  
\[ g = \alpha_x^2 + \alpha_y^2 - \alpha_x \alpha_y - \delta^2 \]  

(1a)
(1b)
where \( \sigma_X \) and \( \sigma_0 \) are the longitudinal and circumferential normal stresses; \( \alpha_X \) and \( \alpha_0 \) are the coordinates of the center of the yield surface; and \( \bar{\sigma} \) and \( \bar{\alpha} \) are the radii of the yield and control surfaces. Using associated flow rule and linear strain hardening rule, the incremental stress-strain relationship in the plastic range is obtained as:

\[
\dot{\varepsilon}_x = s_{11}\dot{\sigma}_x + s_{12}\dot{\sigma}_0
\]
\[ (2a) \]

\[
\dot{\varepsilon}_0 = s_{21}\dot{\sigma}_x + s_{22}\dot{\sigma}_0
\]
\[ (2b) \]

where

\[
s_{11} = [1+(1-\mu)(\sigma_X-\alpha_X)-(\sigma_0-\alpha_0)/2]/\mu \bar{\sigma}^2 \]
\[
s_{12} = [-\nu+(1-\mu)(\sigma_X-\alpha_X)-(\sigma_0-\alpha_0)-(\sigma_X-\alpha_X)/2]/\nu \bar{\sigma}^2 \]
\[
s_{21} = s_{12}
\]
\[
s_{22} = [1+(1-\mu)(\sigma_0-\alpha_0)-(\sigma_X-\alpha_X)/2]/\mu \bar{\sigma}^2 \]

The isotropic and kinematic hardening proportioning parameter was conducted as a poly-linear function of the radius of the yield surface.

The isotropic and kinematic hardening proportioning parameter was conducted as a poly-linear function of the radius of the yield surface.

3. Analysis of manufacturing process

To quantify the residual stress distributions and the Bauschinger and work hardening effects, manufacturing process was analyzed. As Kato et al proposed [2], the process of a cold-formed and welded square hollow tube can be divided into following four steps.

Step 1: Uncoiling and leveling of a coil sheet
Step 2: Roll forming to a circular cross section and seaming by electric resistance welding
Step 3: Roll forming to a square hollow section and sizing
Step 4: Unloading from sizing

According to the following assumptions, the manufacturing process was analyzed:

(1) The wall element is divided into five thin plate elements as shown in Fig. 4.
(2) Only the longitudinal and circumferential normal stresses act on each element.

Fig. 5 shows an example of residual stress distributions in the longitudinal and circumferential direction analyzed. As a result of different strain history, the residual stress distribution of corner part is different from that of flat part. The manufacturing process is so complicated, the distribution of residual stresses is not linear along the thickness of the wall. These residual stresses can't be measured directly, and its magnitude should be estimated by measuring the released strain by sectioning method. Fig. 6 shows the released strain distributions obtained. The longitudinal released strain is generally larger than the circumferential strain. And the released strain at the flat part is larger than the strain at the corner part. The broken lines, the analytical results, coincide fairly well with the experimental ones. Fig. 7 shows radii of the yield surfaces, where element "a" shows the Bauschinger effect and "b" the work hardening effect. The effect of the plastic strain history is large at the outer and inner surfaces, and small at the middle plane of the wall.

![Fig. 4 Thin plate element](image)

![Fig. 5 Residual stress distribution](image)
4. Buckling and post-buckling strengths

4.1 Method of analysis

Buckling and post-buckling strengths of columns with a square hollow section were analyzed considering bi-axial residual stresses, the effect of plastic strain history and geometric imperfections. The assumptions used are as follows:

(1) A column is divided into sixteen elements along the longitudinal directions, twelve elements along the circumferential direction and five elements along the thickness of the wall such as shown in Fig. 6.

(2) Curvature and axial strain distribute linearly along the length of the elements.

(3) Initial deflection shape is a sinusoidal and the maximum deflection is equal to L/1000, where L is a length of a column.

Assuming the axial strain distributes uniformly along the thickness of the wall, the stress-strain relationship of each thin plate element \((i,j,k)\) is:

\[
\begin{align*}
\sigma_x(i,j,k) &= k_{11}(i,j) \epsilon_x(i,j) + k_{12}(i,j,k) \epsilon_y(i,j) \\
\sigma_y(i,j,k) &= k_{21}(i,j,k) \epsilon_x(i,j) + k_{22}(i,j,k) \epsilon_y(i,j)
\end{align*}
\]

In the circumferential direction, no axial resultant force exists:

\[
\sum_{k=1}^{5} \sigma_y(i,j,k) \Delta A = 0
\]

where \(\Delta A\) is the area of a thin plate element. Using above relation, Eq (4)
can be expressed as:

$$\delta x(i,j,k) = K(i,j,k)\varepsilon_x(i,j)$$  \hspace{1cm} (5)

Summing up these stresses, axial force and bending moment can be obtained as follows:

$$\dot{N} = \sum_{j=1}^{7} \sum_{k=1}^{5} \delta x(i,j,k) \Delta A$$  \hspace{1cm} (6a)

$$\dot{M} = \sum_{j=1}^{7} \sum_{k=1}^{5} \delta x(i,j,k) \eta_j \Delta A$$  \hspace{1cm} (6b)

where $\eta_j$ = the coordinate from the centroid. Assuming plane section remains plane during deformation, the longitudinal strain at any arbitrary element (i) of the cross section is described as:

$$\dot{\varepsilon}_x(i,j) = \dot{\varepsilon}_x(i) + \eta_j K_x(i)$$  \hspace{1cm} (7)

in which $\dot{\varepsilon}_x(i)$ = the axial strain at the centroid; and $K_x(i)$ = the curvature. Using the stress-strain relationship obtained above, the relation between the generalized stress $(N, M)$ and the generalized strain $(\dot{\varepsilon}_x, K_x)$ can be obtained. The equilibrium equation in the incremental form can be expressed as follows:

$$\dot{N}(U+\dot{U})+N\dot{U} = \dot{M}$$  \hspace{1cm} (8)

in which $U$ = the lateral deflection; and dot = the increment. The relation between the lateral deflection and curvature is as follows:

$$\dot{U}_i = \sum_{m=1}^{8} C_{im}\dot{\kappa}_m$$  \hspace{1cm} (9)

where $C_{im}$ = the influence factor of the curvature on the deflection. Using these equations, non-linear simultaneous equations can be obtained.

4.2 Buckling and post-buckling strengths

The buckling strength of cold-formed columns with a square hollow section is affected by residual stresses, plastic strain history and geometric imperfections. Analysis was made here for the two mild steel tubes mentioned above (Fig.6). Fig.9 shows the column curves obtained. The ordinate shows the buckling strength divided by the yield stress of the original steel plate. The dashed curve shows the strength of an annealed tube. Except for the short column, the buckling strengths of cold-formed tubes are smaller than that of an annealed tube, because of large residual stresses and the Bauschinger effect. For the short column, the buckling strengths of cold-formed tubes are larger than that of an annealed tube, because of the work hardening effect. These results coincide with the experimental evidence obtained by Kato et al [3]. Fig.10 shows the axial load-contraction relationship. The solid line shows the behavior of a cold-formed tube and the dashed line the behavior of an annealed tube. In the post-buckling range, cold-formed columns are larger in resistance than annealed columns because of the work hardening effect. Fig.11 shows the longitudinal and circumferential stress distributions at the center section of the column at the maximum strength and in the post-buckling range. At the maximum strength, the longitudinal stress distribution varies through the thickness of the wall due to the residual stresses, and large circumferential residual stresses still exist, except for the short column. In the post-buckling range, the longitudinal stresses distribute uniformly through the thick-
ness of the wall, and the circumferential residual stresses disappear, because of large plastic strain.

Fig. 8 Column with square hollow section

Fig. 9 Column curves

Fig. 10 Axial force-displacement relationships

5. Conclusions

Roll forming process of a square hollow section tube was analyzed to make clear the residual stress distribution and the effect of plastic strain history. Buckling and post-buckling strengths of the columns were analyzed considering bi-axial residual stresses and the Bauschinger and work hardening effects. And the following conclusions were obtained:

(1) The longitudinal and circumferential residual stresses can be estimated by analyzing the manufacturing process proposed here.

(2) The influence of the residual stresses and the Bauschinger and work hardening effects on the buckling strength are large for the columns with intermediate and relatively large slenderness ratios.

(3) For the columns with small slenderness ratio, the buckling strength of cold-formed tubes is greater than that of annealed tubes, because of the work hardening effect.

(4) The post-buckling strength of cold-formed columns exceeds the strength of annealed columns due to work hardening effect.
Fig. 11 Longitudinal and circumferential stress distributions

6. Symbols

- $\Delta A$: Area of a thin plate element
- $D$: Width of a tube
- $E$: Young's modulus
- $\Delta L$: Axial displacement
- $\Delta L_0$: Axial yield displacement
- $M$: Bending moment
- $N$: Axial force
- $N_0$: Axial yield force
- $t$: Thickness of a tube
- $\bar{a}$: Radius of a control surface
- $a_x$: Coordinate of the center of a control surface (longitudinal)
- $a_y$: Coordinate of the center of a control surface (circumferential)
- $\varepsilon_x$: Longitudinal strain
- $\varepsilon_y$: Circumferential strain
- $K$: Curvature
- $\mu$: Strain hardening coefficient
- $x_j$: Coordinate of the $j$th element
- $r$: Radius of a loading surface
- $\sigma_{cr}$: Buckling strength
- $\sigma_e$: Elastic buckling strength
- $\sigma_y$: Yield stress
- $\sigma_r$: Residual stress
- $\sigma_x$: Longitudinal normal stress
7. References


Summary

The results of 18 tests on slender composite columns consisting of rectangular hollow steel sections filled with concrete are presented. The columns had a length of 3 m and a cross-section of 120x120 mm. They were simply supported and the load was normally applied with an eccentricity of 20 mm. As a reference, the squash load was evaluated with tests on short columns (stub tests).

The aim was to evaluate the possible advantages of high strength concrete, confining effects of composite sections and the effect of the shear transfer at the interface.

Primary parameters which varied between the tests were: concrete compressive strength, steel yield stress and thickness of the steel tube. In additional tests the effect of load eccentricity, additional reinforcement in the column, debonded interface and the way of load application were examined.

For composite columns loaded eccentrically, the load bearing capacity as well as the ductility in the ultimate state increased with the concrete compressive strength.

1. Introduction

In the construction of buildings great effort is made to increase the structural flexibility. This has resulted in the need for columns with reduced cross-sections. To achieve high load bearing capacity with small cross-sectional areas it is worthwhile to study the possibility of utilizing high strength concrete and composite effects.

The properties of high strength concrete diverge from normal strength concrete and it is not possible to assume the same procedures for design as with normal strength concrete.
2. Test program

In this research project slender composite columns consisting of rectangular hollow steel sections filled with concrete were tested, Grauers et al [1]. The aim was to study the effect of composite action, high strength concrete and confinement from the steel tube. Simply supported columns with a length of 3 m and a cross-section of 120 x 120 mm were loaded to failure by an axial load. In most cases the load was applied with an eccentricity of 20 mm. This test series consisted of 18 composite columns, with 4 empty steel tubes as reference. The test setup for the long columns is shown in figure 1. For each long column, a companion short column with a length of 0.5 m and with the same material properties was loaded concentrically to evaluate the squash load (stub test). The test setup is shown in figure 2.

The primary parameters were concrete compressive strength, steel yield stress and thickness of the steel tube. The concrete compressive strength varied between 39 – 103 MPa (mean values), referring to tests on \( \Phi \)150 mm cylinders according to ISO-Standard. Two types of steel were used, normal strength and high strength with yield stress of about 300 MPa and 400 MPa respectively. The thickness of the steel tube was either 5 or 8 mm.

3. Test results

3.1 General behavior

The behavior of the long columns with high strength concrete and eccentric load is exemplified by typical relationships between the load and the deflection and between the moment at the mid-section and the deflection in figure 3. The deflection was defined at the mid-section of the column in relation to the position of the simple supports and therefore included the end eccentricity. However, with the specific support details, the end eccentricity varied with the end rotation of the column. The measured deflection was therefore adjusted with regard to the actual position of the support bearings. As appears from the relationship between moment and deflection, the stiffness changed drastically in two points. The first change occurred at the maximum load which coincided with yielding of the steel tube at the compressive edge of the column. Yielding was verified by the strain guage measurements at the mid-section. The maximum load bearing capacity for the actual long composite columns was always determined by the strength of the steel section. At this stage there was no visible buckling on the compressive edge of the column. The second change of stiffness was due to yielding of the steel tube also on the tensile edge of the column.
One important factor for the behavior of both the short and the long columns was the relative nominal capacities of the steel and concrete sections. For instance, the ratio between the load bearing capacity of the long columns and the ultimate capacity of the short columns decreased with increasing ratio $N_c/N_s$ for the eccentrically loaded columns.

To evaluate the contribution of confinement, steel strains in the horizontal direction at the column mid-section were plotted versus strains in the vertical direction for both the compressive and the tensile edges of the long columns. This is exemplified in figure 4.

The horizontal strain gauges on the tensile edge of the long columns were always under compression closely corresponding to the Poisson effect and no effect of confinement could be observed. Also at the compressive edge it was not possible with any degree of accuracy to prove any confinement effects. Tensile strain was observed but did not exceed what could be expected from the Poisson effect. However, when the maximum load was reached, in average only about 30 % of the nominal capacity of the concrete core was utilized.

Due to confining effects, the capacity of the short columns increased in average about 6 % compared with the total nominal capacity. As a rule this beneficial effect has not been proven in previous tests on rectangular steel-concrete columns, Kennedy [2]. An attempt to explain the behavior is presented by Cederwall [3].

### 3.2 Effect of concrete compressive strength

For the same type of steel tube, the load bearing capacity increased with concrete strength. More importantly, the concrete strength contributed to an increased ductility of the long columns in the ultimate state, see figure 5. At the maximum load, in average only about 30% of the nominal capacity of the concrete, $N_c$, was utilized in the long columns. When the column was exposed to excessive deflection in the ultimate state the high strength concrete section had a capacity in reserve which could compensate for the yielding of the steel section. This effect appears clearly from figure 5.

### 3.3 Effect of steel section capacity

The load-deflection curves for four columns with the same concrete strength, but with different steel yield stress and thickness of the steel tube are shown in figure 6. The long columns with thicker steel tube had a higher stiffness and load bearing capacity, but the column with a lower yield stress had a more ductile behavior.

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3.4 Effect of initial end eccentricity

One of the long columns was loaded with 10 mm eccentricity and one column concentrically. All other long columns were loaded with 20 mm eccentricity. The effect of initial end eccentricity is illustrated by load-deflection curves in figure 7. As can be seen in the figure, both the load bearing capacity and the column stiffness increased with decreasing eccentricity. On the other hand, the ductility decreased.

Based upon these tests, it seems clear that high concrete strength is favorable with regard to the load bearing capacity for columns loaded with no or small eccentricity. This also appeared from the tests on the short column where the contribution from the concrete core always exceeded the nominal capacity $N$. However, the positive effect of high concrete strength on the ductility was only observed in long columns with considerable end eccentricity where the concrete contribution to the load bearing capacity was small.

3.5 Effect of bond and load application

For four of the columns in the test series, the way of load application and bond properties at the steel-concrete interface varied. The actual relationships between the load and deflection are presented in figure 8.

On two of these columns, the load was applied on the steel section only. This was achieved by means of 10 mm deep recesses in the concrete core at both the ends of the column, see figure 9b.

On the other two, the load was applied on the concrete core only, by means of a 50 mm thick steel plate which fitted into the hollow steel section, see figure 9c. By this arrangement the total length of the column between the supports became 80 mm longer than for the other long columns. Of this reason the relationships in figure 8 are not quite comparable.

The steel-concrete interface was debonded in one of the columns of each type of load application. The steel tubes were treated inside with an elastic joint fill which was covered with a 0.2 mm plastic film before concreting. The debonding effect was verified on separate detail tests.

The highest load bearing capacity was obtained for the column where the load was applied on the concrete core only and the steel-concrete interface was debonded. In this case no axial load could be transferred to the steel tube. Thus the steel tube was only affected by curvature forces due to the eccentric loading on the concrete core. As a result, the yielding of the steel tube on the compressive edge of the column was delayed compared with the columns where the steel tube also carried an essential part of the axial load.
directly. No visible buckling occurred when the maximum load was reached.

For the case when the load was applied on the steel tube only and the interface was debonded the column behaved more or less like a hollow steel column without a concrete core. The concrete core probably cracked in flexure at an early stage of loading and thereafter could not contribute to the capacity of the column. The load-deflection relationship was almost identical with that of a comparable empty steel tube.

For the cases where the natural bond between steel and concrete was not prevented by artificial means, the columns had a remarkable good behavior irrespective of the way of load application. The behavior was similar to that of the comparable column where the load was applied on the total cross-section at the ends.

To study the crack pattern on the concrete core of the long columns, two edges of the steel tube were removed at mid-section of the columns. It can clearly be seen that the crack spacing and crack width was quite large for the columns with debonded interface and small for the columns with natural bond between the steel tube and the concrete core, see figure 10.

4. Conclusions

Confinement effects were found in the tests on the short columns with concentric load (stub tests). The ultimate load of the composite section exceeded the sum of the nominal capacities of the steel and concrete sections with about 6%. The load bearing capacity for the long columns was always determined by yielding at the compressive edge of the steel tube. For the columns with considerable end eccentricity, the contribution from the concrete core to the capacity was in average only about 30% of the nominal capacity of the concrete core.

For the long columns with small or no eccentricity the contribution to the capacity from the concrete core was important but those columns showed a less ductile behavior.

When the load was applied on the steel section only, or on the concrete core only, the column had the same behavior as a comparable one where the load was applied on the total end cross-section but only if the natural bond at the steel-concrete interface was undisturbed.

When the steel-concrete interface was debonded there was no ductile behavior. When, in this case, the load was applied on the steel section only, the concrete core did not contribute to the behavior and the column behaved as an empty steel tube. On the other hand, when the load was applied on the concrete core only and the interface was debonded, the load bearing capacity increased because the steel tube was not loaded axially, only by the curvature forces.
5. Acknowledgement

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6. Symbols

- $A_c$: Area of concrete section
- $A_s$: Area of steel section
- $e$: Eccentricity
- $f_{cc}$: Concrete compressive strength evaluated from compression tests on concrete cylinders with a diameter of 150 mm and a height of 300 mm according to ISO-standard
- $f_{sy}$: Yield strength of the steel tube evaluated from deformation controlled compression tests on 300 mm high steel tubes
- $N_c$: Nominal capacity of the concrete section: $N_c = f_{cc} \cdot A_c$
- $N_s$: Nominal capacity of the steel section: $N_s = f_{sy} \cdot A_s$
- $t$: Thickness of the steel tube

7. References


Figure 1. Test setup for the long columns

Figure 2. Test setup for the short columns (stub tests)

Figure 3. Comparison between load-deflection and moment-deflection relationships for long composite columns with high strength concrete
Figure 4. Typical relationship between vertical and horizontal strain at the mid-section for one of the long columns: a) tensile edge b) compressive edge

Figure 5. Effect of the concrete compressive strength on the load-deflection relationships for long composite columns with the same type of steel tube (yield stress 400 MPa, thickness 8 mm): a) $f_{cc} = 103$ MPa b) $f_{cc} = 47$ MPa c) $f_{cc} = 39$ MPa d) empty steel tube
Figure 6. Effect of steel section capacity on the load-deflection relationships for long composite columns with the same concrete compressive strength ($f_{cc} = 46$ MPa): a) $f_{sy} = 376$ MPa, $t = 8$ mm  
b) $f_{sy} = 300$ MPa, $t = 8$ mm  
c) $f_{sy} = 438$ MPa, $t = 5$ mm  
d) $f_{sy} = 304$ MPa, $t = 5$ mm

Figure 7. Effect of initial end eccentricity on the load-deflection relationships for long comparable composite columns: a) $e = 0$ ($f_{cc} = 80$ MPa)  
b) $e = 10$ mm ($f_{cc} = 80$ MPa)  
c) $e = 20$ mm ($f_{cc} = 103$ MPa)
Figure 8. Effect of load application and bond at the steel-concrete interface on the load-deflection relationships for comparable composite columns:
- a) and b) load applied on concrete section only
- c) and d) load applied on steel section only
- a) and c) interface with natural bond
- b) and d) debonded interface

Figure 9. Various ways of load application:
- a) on total cross-section
- b) on steel section
- c) on concrete section

Figure 10. Crack spacing and crack width at mid-section of the long columns:
- a) Debonded interface between the steel tube and concrete core
- b) Natural bond between steel tube and concrete core
CONCRETE-FILLED RECTANGULAR HOLLOW SECTION X and T CONNECTIONS

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Summary

The implications of filling the chord member of Rectangular Hollow Section (RHS) truss connections with concrete, as a means of obtaining improved strength and stiffness, are explored in this paper. X connections with the branch members loaded in axial compression were considered to likely benefit the most from concrete-filling, so 14 RHS specimens were subjected to transverse compression but with the load applied through bearing plates to avoid simple failure of the RHS branch members. The parametric tests, performed on a common size of rectangular member, varied in the amount of concrete filling, were loaded with different bearing areas, and had the tube oriented in different directions. After analyzing the data in various ways it was concluded that a conservative lower-bound method for estimating the strength of concrete-filled RHS under transverse compression was to ignore the contribution of the steel, (except for providing 2-dimensional confinement to the concrete), and determine the capacity by the bearing strength of the concrete. In calculating the dispersed bearing area only longitudinal load dispersion along the RHS need be considered, and in accordance with dispersion angles and limits already established by concrete design codes.

1. Introduction

Fabricators of structural steelwork are frequently placed in the position of performing "connection design" after the sizes of all structural members have been determined. With tubular construction the connection strength frequently controls the member selection, so connection reinforcement is often necessary if the primary hollow section members cannot be changed. One option available to fabricators is to add stiffening plates to the exterior of the hollow section, but another and less visible alternative for certain connection types is to fill the hollow section with concrete or grout. This paper hence describes an experimental investigation into the behaviour of concrete-filled Rectangular Hollow Section (RHS) X and T connections and discusses the implications for the design of these connections.

Filling the chord members of an RHS truss with concrete or grout, either along the full length of the chord or just in the vicinity of critical connections, has two main disadvantages: the concrete will increase the dead weight of the structure and it involves a secondary material with associated labour costs. On the other hand, the strength of certain connections is likely to increase, and if the members are completely filled there are the further benefits of enhanced member capacity and improved fire endurance.

Concrete-filled hollow section truss connections have already been used in Canada but there has been no way of quantifying the structural capacity. Concrete-filling has also been used in connections and pile sleeves in offshore structures. In this case the concrete provides additional strength for the connections and can mechanically connect piles to jacket legs for load transfer from the structure to the seabed. Some experimental work has been done in this area, mainly by companies and agencies under contract to oil companies. Four Circular Hollow Section (CHS) T connections that were tested by Wimpey Laboratories in compression showed the ultimate strength of the connections to be significantly improved when the connections were concrete-filled [1].
A punching shear failure of the chord face was observed in the two empty connections, but this mechanism was prevented by the concrete in the two filled connections. In the filled connections, the observed maximum loads were actually governed by squashing of the compression brace, as might be expected with such a rigid base for the branch member. However, such tests resulting in squashing of the compression brace do little to reveal the true ultimate capacity of the chord section, so this failure mode was prevented in the tests described herein by using bearing plates to transmit transverse compression loads to the hollow section chord member.

2. Experiments on RHS under Transverse Compression

Two lengths of RHS, each of nominal size 7" x 5" x 3/16", labelled L1 and L2, were used to make the test specimens. Seven pieces, each 750 mm in length, were cut from L1 and L2 to produce a total of 14 specimens as shown in Table 1. Three of the specimens were left empty as control specimens while the others were filled with varying lengths of concrete (Lc). In order to centre the concrete in the partially-filled specimens, styrofoam spacers were used and the RHS tubes were stood on end to be filled. The concrete in the full specimens was compacted with a vibrator, while in the partially full specimens the concrete was placed and rodded in layers. All of the specimens were "field cured" in the laboratory, with their ends covered with plastic to retain some moisture during curing.

Two loading plates for each specimen were cut from 16 mm, mild steel plate and welded to the specimen centred on top and bottom. Four specimens of Type A were made, (see Table 1), with 127 x 127 mm loading plates welded to the 127 mm wide face, (b/b0 = 1.0), and with concrete lengths of: zero, 125 mm, 300 mm and 750 mm. Four specimens of Type B were made, with the specimens oriented in the same fashion and with the same lengths of concrete, but with the loading plates being 64 mm wide by 127 mm long (Hence b/b0 = 0.5). The RHS was then oriented with the 178 mm wide face as the loaded face to make 4 specimens of Type C and 2 of Type D. Type C specimens had loading plates which were 64 mm wide by 127 mm long, (hence the same bearing area as Type B but with b/b0 = 0.357), and concrete lengths as for Types A and B. The type D specimens duplicated 2 of the Type C specimens except the bearing plates were oriented in the opposite direction (see Table 1). Thus, a series of parametric variations in the test specimens was designed, with the object being to evaluate the effect of one particular geometric parameter at a time while the others were maintained constant.

The hollow sections used were cold-formed, stress-relieved tubes (Class H) having a nominal yield stress of 350 MPa (Grade 350W). The measured material properties are given in Table 2 and it can be seen that L1 and L2 can be considered identical. Concrete properties were determined from 102 mm diameter x 203 mm long cylinders which were also "field cured". An average of 6 cylinders tested in compression, at an age of 33 days, gave a peak compressive stress (f′c) of 43.3 MPa. Split cylinder tests were performed on 2 more cylinders to indirectly determine the tensile strength (f′t) of 4.0 MPa. Although there is an international trend to the use of high strength concretes, relatively low strength concrete was deliberately used so that:
(a) it would duplicate the types of concrete likely supplied by a steel fabricator who was inexperienced in mix design, and
(b) failure modes would incorporate failure of the concrete.

A 300 mm long bare concrete block specimen, with cross-sectional dimensions approximately equal to the concrete blocks formed inside the RHS specimens, was also made by using one of the (eventually) unfilled tubes as a form. This block was also subsequently tested in compression along with the other 14 RHS specimens, by applying load via full-width bearing plates, to check the bearing capacity of the concrete filling without the benefit of confinement from the steel tube.
The specimens were loaded in transverse compression to failure, in a monotonic, quasi-static manner, using a 5000 kN capacity stiff-frame Universal Testing machine. A typical testing arrangement is shown in Fig.1. Vertical deformations of each specimen during testing were recorded using displacement transducers, as well as lateral deformations at mid-height of the tube wall (see Fig. 1).

3. Results of Transverse Compression Tests

For the Type A specimens (see Table 1), the empty specimen (No. 1) failed by web crippling of the chord walls, which was to be expected for a width ratio ($b_l/b_o$) of 1.0. The chord walls also bulged when the specimen with 125 mm of concrete was tested (No.5). As a result, large gaps between the chord walls and the concrete were clearly visible, vertical cracks formed in the concrete and eventually the concrete spalled off into the RHS. With a longer length of concrete (Nos.7 and 3) large bulges formed in the tube side walls, but only in the immediate vicinity of the loading plate. For specimen No. 3, (fully filled with concrete), the concrete extended beyond both ends of the specimen by several millimetres, after ultimate load. Fig. 2 shows how the ultimate strength progressively increased, with increasing length of concrete, as did the stiffness in the direction of the load. The principal test results are also recorded in Table 1.

With the Type B specimens, (identical to Type A except for having half the bearing width and bearing area), the empty test specimen (No. 2) failed by yielding of the chord face and the remaining concrete-filled specimens failed by crushing of the concrete. Compared to Type A specimens the Type B specimens with concrete exhibited much less lateral deformation of the chord side walls at ultimate load, even though the vertical deformations of the concrete-filled Types A and B were similar (see Table 1). Thus, without the direct compression loading of the chord side walls as in Type A, the concrete of the Type B specimens appeared to expand laterally with the steel at the same rate, with eventual bulging of the side walls at ultimate load.

The Type C specimens, (identical to Type B except for a change in chord orientation and hence $h_d/b_o$ and $b_l/b_o$ ratios), behaved in a similar manner to their Type B counterparts but the concrete-filled specimens displayed a significant increase in ultimate strength despite having the same bearing areas (see Table 1). Finally, two Type D specimens were tested, (identical to Type C but with the bearing plates rotated through 90°), and significantly lower ultimate strengths were achieved relative to their Type C counterparts.

The single concrete block was tested with the long wall vertical and a full width bearing plate, thereby making it analogous to the Type A specimens. It failed in a brittle manner at a peak compressive stress ($q$) of 36.5 MPa - somewhat lower than the cylinder compressive strength.

4. Analysis of Transverse Compression Tests

The influence of many parameters is difficult to assess, but the maximum specimen strength always increased as the length of concrete ($L_c$) increased, as shown in Fig. 3. However, as the graphs tend to level off with higher $L_c$ values it would appear that there is some limiting value of $L_c$, and hence dispersion area, beyond which no increase in strength will be observed. The ratio $q/f_c$, or observed bearing stress ($P_{max}/A_j$) to the crushing strength of the concrete, is tabulated in Table 1 and reaches a peak value of approximately 2.0 for Types A and B, 4.4 for Type C and 2.8 for Type D specimens. Specimen Types C and D were identical except for the orientation of the bearing plates, so the lower $q/f_c$ values in Type D would appear to be caused by the higher width ratio ($b_l/b_o$); i.e., more load is transferred into the side walls causing them to bulge and provide less confinement, so that less dispersion of stress can occur, resulting in a lower strength. However, this argument is negated by comparing Type B ($b_l/b_o = 0.5$) results with Type A ($b_l/b_o = 1.0$), as both have approximately the same $q/f_c$ values, for a particular $L_c$ (see Table 1). The
other parameter which changed from Type C to Type D was \( h_l \), the length of the loading plate along the longitudinal axis of the RHS, and the foregoing suggests that the bearing capacity is more likely to be a function of \( h_l \).

The influence of RHS orientation can be studied by comparing Types B and C. It appears that sidewall strength might play a significant role in the behaviour of concrete-filled RHS connections, as the Type C specimens with a much higher \( q/l_f' \) have a stockier chord side wall, (lower \( h_l/h \)), than their Type B counterparts. However, another contributing factor to the differences in strength between Types B and C is that a concrete block will have a lower crushing strength anyway with a large aspect ratio (height to width ratio), compared to a small aspect ratio.

The Canadian [3], American [4] and European [5] concrete design codes all propose similar methods for determining the bearing strength of concrete when the loaded area is surrounded by additional concrete. They each recommend that the ratio of the bearing stress to the crushing strength of concrete, \( q/l_f' \), should be taken as \( \sqrt{A_2/A_1} \), where \( A_1 \) is the loaded bearing area and \( A_2 \) is the dispersed bearing area, with the latter being concentric and geometrically similar to the loaded area. The bearing stress is assumed to disperse at a slope of 2:1 until the edge of the concrete is reached or until a limiting value of \( A_2 \) is reached (see Fig.4). The Canadian [3] and American [4] codes give an upper limit to \( \sqrt{A_2/A_1} \) of 2, while the European code [5] specifies an upper limit of 3.3. Studies of confined unreinforced concrete in bearing have been carried out by a number of researchers including Hawkins [6], who developed the following expression for maximum bearing stress:

\[
q/l_f' = 1 + (50/l_f') \left( \sqrt{A_2/A_1} - 1 \right)
\]

The above equation was developed for \( f_c' \) in psi; for the 43.3 MPa concrete used in this study the equation becomes:

\[
q/l_f' = 1 + 0.631 \left( \sqrt{A_2/A_1} - 1 \right)
\]

This fairly conservative proposal of Hawkins [6] is shown graphically on Fig. 5, along with the recommendations of the Canadian/American concrete codes [3,4] and European concrete code [5].

For the 11 concrete-filled specimens in this study of transverse compression, \( A_f' \) and \( f_c' \) are known quantities but the dispersed bearing area \( A_2 \) is open to interpretation and definition. Several different means of calculating \( A_2 \) were investigated as follows:

(i) **Equivalent Concrete Models.** It was initially thought that the confining steel could be replaced by a thick band of equivalent concrete confinement, to permit the bearing stress to disperse quite far. However, this method gave ratios of \( \sqrt{A_2/A_1} \) which were much larger than \( q/l_f' \), so it was abandoned as it would considerably overestimate the strength of concrete-filled RHS connections.

(ii) **Longitudinal Dispersion only.** This presumes that the presence of the steel is to provide some confinement only and has no effect on the dispersion width, but greatly enhances the dispersion length. The extent of longitudinal dispersion (at 2:1) depends on the length of concrete in the RHS and also depends on the RHS depth, since dispersion of stresses can not go beyond the member centreline when it is loaded on top and bottom. The resulting comparison with test results is shown in Fig.5.

(iii) **Longitudinal and Lateral Dispersion.** This is similar to above with lateral dispersion of stress also being permitted at a slope of 2:1, subject to the lateral dispersion not going beyond the tube side walls. A problem with this method is that the limited lateral dispersion available constrains the longitudinal dispersion resulting in consistently overconservative results. (see Fig. 5).
(iv) Longitudinal Dispersion + Lateral Dispersion beyond Chord Walls. This is similar to above with the lateral dispersion permitted to extend beyond the tube walls, supposing that the tube sidewalls are equivalent to a large amount of concrete. \( A_2 \) was therefore calculated by assuming that the degree of longitudinal dispersion limits the amount of overall dispersion. This was also found to be an imperfect behavioural model (Fig. 5).

None of the methods above matches closely with either the design codes or Hawkins’ method [6], as Fig. 5 reveals. In particular, none of the methods accounts for the fact that when the specimens are tested in orientation 2 (Table 1), the side walls of the chord are stiffer and able to provide greater confinement. However, the best method to use appears to be to calculate \( \sqrt{A_2/A_1} \) based on longitudinal dispersion alone and to use \( q/f'_c = \sqrt{A_2/A_1} \), with an upper limit on \( \sqrt{A_2/A_1} \) of 3.3 as suggested by CEB-FIP [5]. The correlation with test results using this method is shown in Fig. 6, and the bearing stress is predicted fairly closely for specimens of Types A, B and D. The predicted \( q/f'_c \) ratios for specimens of Type C are very conservative, but in practice it is not common to have connections with this geometry (i.e., a low \( b_i/b_o \) ratio or a high \( h_i/b_i \) ratio). Thus, in general, for most common RHS connections, this method should give a good to conservative prediction of the connection strength.

5. Conclusions

1. Concrete-filling of RHS greatly enhances their performance under transverse compression. The RHS provides confinement for the concrete which allows it to reach bearing capacities greater than its crushing strength as determined by cylinder compression tests.

2. For Limit States Design the factored resistance, (or design strength), of a concrete-filled RHS, compression-loaded, X connection (or even T or Y connections), can be taken as:

\[
N_1^* = \frac{\phi_c f'_c A_1}{\sin \theta_1} \sqrt{A_2/A_1},
\]

where \( \phi_c \) is the appropriate resistance factor, (approximately the inverse of a partial safety factor), for the applicable design specification. In Canada, \( \phi_c \) can be taken as 0.6. \( A_2 \) should be determined by dispersion of the bearing load at a slope of 2:1 longitudinally along the chord member, as shown in Fig. 6. The value of \( A_2 \) is limited by the length of concrete and \( \sqrt{A_2/A_1} \) cannot be taken to be greater than 3.3.

3. The following are also recommended for general design application of eq. 3:

\[
\begin{align*}
& h_i/b_o \leq 1.4 \\
& L_c \geq (h_i/Sin \theta_i) + 2h_o.
\end{align*}
\]

6. Symbols

- \( A_1 \): Bearing area over which the transverse load is applied.
- \( A_2 \): Dispersed bearing area.
- \( b_i \): External width of RHS member \( i \) (90° to plane of truss), \((i = 0, 1)\); width of bearing plate (Fig. 6)
- \( d_i \): Depth of stress dispersion in concrete (Fig. 6)
- \( f'_c \): Crushing strength of concrete by cylinder tests
- \( h_i \): External depth of RHS member \( i \) (90° to plane of truss), \((i = 0, 1)\); length of bearing plate (Fig. 6)
- \( i \): Subscript to denote member of connection; \( i = 0 \) refers to chord; \( i = 1 \) refers to RHS branch member or bearing plate.
$L_{c^*}$  Length of concrete in RHS chord member  

$N_{i}$  Connection factored resistance, expressed as an axial force in branch member/bearing plate.  

$P_{max}$  Maximum load recorded for a test specimen  

$q$  Maximum bearing stress recorded for a test specimen  

$t_i$  Thickness of RHS member $i$, ($i = 0,1$)  

$w_s$  Width of stress dispersion in concrete (Fig. 6)  

$\beta$  Width ratio between branch member (or bearing plate) and chord $= b_1/b_o$.  

$\phi_c$  Resistance factor for limit states design of concrete in bearing.  

$\theta_1$  Included angle between branch member and the chord  

7. References  


8. Acknowledgements  

Financial support for this project was provided by the Comité International pour le Développement et l'Etude de la Construction Tubulaire (CIDECT Program 5AV), the University Research Incentive Fund of the Government of Ontario (URIF Award TO 14-013), Ipso Inc., and the Natural Sciences and Engineering Research Council of Canada (NSERC). Advice on concrete properties and behaviour from Professors R.H. Mills, M.P. Collins and S.A. Sheikh is appreciated.
### Table 1: Description of RHS Specimens for Transverse Compression Tests, and Results.

<table>
<thead>
<tr>
<th>Specimen Type and $b_1/b_o$</th>
<th>RHS Dimensions</th>
<th>Bearing Area $A_1$ (mm²)</th>
<th>Peak Load $P_{max}$ (kN)</th>
<th>$q/f'_c$</th>
<th>Vertical Displacement at Peak Load $\Delta_y$ (mm)</th>
<th>Lateral Displacement at Peak Load $\Delta_l$ (mm)</th>
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<tbody>
<tr>
<td>A 1.0</td>
<td>L1 0</td>
<td>0</td>
<td>393</td>
<td>-</td>
<td>1.1</td>
<td>3.8</td>
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<tr>
<td></td>
<td>L1 125</td>
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<td>902</td>
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<td>4.8</td>
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<td></td>
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<td></td>
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<td></td>
<td>1421</td>
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<td>2.0</td>
<td>6.2</td>
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<tr>
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<td>18.4</td>
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<td></td>
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<td>991</td>
<td>2.83</td>
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<td>1.3</td>
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</table>

Notes: a. $f'_c = 43.3$ MPa

### Table 2: Geometric and Mechanical Properties of Rectangular Hollow Sections Tested.

<table>
<thead>
<tr>
<th>RHS</th>
<th>Nominal Dimensions (mm)</th>
<th>Measured Dimensions (mm)</th>
<th>Nominal Area (mm²)</th>
<th>Measured Area (mm²)</th>
<th>Measured Area relative to Nominal (%)</th>
<th>Measured Yield Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>177.8x127.0x4.78</td>
<td>177.8x127.5x4.75</td>
<td>2760</td>
<td>2706</td>
<td>-2.0</td>
<td>329.5</td>
</tr>
<tr>
<td>L2</td>
<td>177.8x127.0x4.78</td>
<td>177.8x127.8x4.75</td>
<td>2760</td>
<td>2683</td>
<td>-2.8</td>
<td>330</td>
</tr>
</tbody>
</table>

Notes: a. Determined by cutting a prescribed length of RHS, weighing it, then using a density of 7850 kg/m³ to calculate the cross-sectional area [2].

b. Determined by the average of 2 tensile coupons taken from the flat of the RHS.
Figure 1: Typical Testing Arrangement for RHS Transverse Compression Test (Specimen 13)

Figure 2: Load vs. Vertical Displacement for Type A Specimens under Transverse Compression
Figure 3: Maximum Load vs. Length of Concrete

Figure 4: $A_1$ and $A_2$, as defined by the American Concrete Code [4].
Figure 5: Evaluation of Bearing Models with Transverse Compression Tests.

Figure 6: Recommended Method for Determining Bearing Capacity of Concrete-Filled RHS Loaded in Transverse Compression.
STRUCTURAL DESIGN OF HIGH RISE BUILDING CONSISTS OF
CONCRETE FILLED SQUARE TUBULAR COLUMNS AND STEEL COMPOSITE BEAMS
AND ITS EXPERIMENTAL VERIFICATION

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Summary
The response behavior of a designated 31 story office building consists
of concrete filled square tubular columns and steel composite beams with
new type of inner diaphragms is investigated through analysis on global
structure and test on subassemblies. The results confirm sufficient seis­
mic strength of the building which justifies the structural design.

1. Introduction
Concrete filled tubular columns have better response behavior as well as
fire resisting capacity in comparison to hollow tubular columns due to
their composite form action. Therefore, this type of columns are used in
the fabrication such as high rise buildings. The structure using square
tubular columns (STC), which is stiffened by inner diaphragms in beam­
column joint part, has desirable finishing in all parts after proper de­
dsign. In this type of structure, the inner diaphragms should be capable
of good infiltration working of concrete and stress transfer in the part
of beam to concrete filled tubular column joint. The inner diaphragms
which have the above characteristics are employed. There are few experi­
mental data on this type of structure and the design procedure concerning
well defined stress transfer in the joint is not yet established [1]. A
31 story office building, consists of concrete filled square tubular col­
umns and steel composite beams with new type of inner diaphragms, is
investigated in this study. The investigation contains structural analy­
sis (static and dynamic) and experimental study by earthquake type load­
ing on inner beam-column subassemblies.

2. The building and its design
2.1 Outline of the building structure
A line diagram of elevation and a typical story plan of the building struc­
ture are illustrated in Figs.1-2. It is an office building with rise 31
stories and 4 basements. The height of 1st story is 8.5m and other from
2nd to 30th, is 3.8m. After a precise study on the structural frame sys­
tem and economical feasibility, the structure of rigid frame consists of
concrete filled square tubular column and composite beam using new type
of inner diaphragms is selected. This building is one of the tallest
buildings of this type of structure in Japan [2]. Both the square tubular
columns and H-shaped beams are built up members. The grades of steel
plates used are SM490A (t<40mm) and SM520B (t>40mm, thermo-mechanical
control process steel). H-beams, whose webs have comparatively large depth-thickness ratio, is connected with RC slab by headed studs. The columns and the beam flanges are butt welded at construction site and beam webs are fixed by high tension bolts. Fig.3 shows the new type of inner diaphragms. One is horizontal type with a circular hole (HT) and the other is vertical type with a plate crossing (VT). A normal weight concrete with very fine powder of blast furnace slag but very small bleeding is used with an aim to good infiltration. The specified design strength of normal weight concrete used in column is 29.4 to 41.2 N/mm² and that of light weight concrete used in slab is 17.6 N/mm².

2.2 Structural design

The building structure has been designed in such a way that it possesses adequate strength capacity and ductility in response to earthquake excitation and the collapse mechanism of the structure is due to the development of beam dissipation mechanism. To achieve the mechanism, the structure is designed so that the ultimate shear capacities of beams, columns and panels and ultimate flexural capacity of columns are larger in comparison to the ultimate flexural capacity of beams. Both static and dynamic designs are applied in the upper part of the structure. The static design consists of working-stress design (WSD) and ultimate-strength design (USD). The WSD is based on the criteria that the stresses of members are within elastic limit and story drift is within 1/200 rad. The USD is due to the confirmation of strength capacities, deflection modes, and collapse mechanism. The two seismic criteria in Table 1 are checked by dynamic design.

2.2.1 Static design

The WSD has the scope of design for gravity load, wind load and seismic load of moderate earthquake. Fig.4 shows the wind load and seismic load of WSD and also the capacities of elastic limit (Qₑ), global yielding (Qᵧ) and ultimate (Qᵤ). The stresses of members due to the wind load and the seismic load are within the elastic limit. Also the story drift is within the limit of 1/200 rad. (Fig.7). In the USD, inelastic frame analysis has been done by incremental lateral loading method, considering the same seismic loading distribution as of WSD. The following assumptions are made to carry out the analysis. (a) The bottom end of the column is fixed. (b) Column and beam members are modeled by linear elements with yield hinge at their ends. (c) The yield point of steel is 1.1 times of the specified value. (d) The joint panel is assumed to be elastic. To ensure the formation of beam dissipation mechanism, the ultimate strength of members are defined as follows.

(1) Ultimate strength of beams

Flexural capacity: $$b_M = B_f \cdot b_f \cdot t_f \cdot d_{bf} \cdot B_y \cdot b_w \cdot d \cdot b_y / 4$$

Shear strength: $$b_Q = t_w \cdot b_w \cdot (Q_g + (M_P \cdot M_N \cdot b_w) / l)$$

where, Qₜ is shear strength due to gravity load; Mₚ, Mₙ are positive and negative ultimate flexural strength of composite beam [3] and l is the length of clear span of beams.
(2) Ultimate strength of columns

Flexural capacity:

$$c_u^M U + c_u^M L \geq 1.3(M_p + M_N)$$  \hfill (3)

Shear strength:

$$c_u^Q \geq 1.2(b_u + b_M U)/h_o$$  \hfill (4)

where, $M_u^U$, $M_L^L$ are ultimate moments of columns at upper and lower faces of joints and $h_o$ is the clear span length of columns.

(3) Panel moment

Panel moment capacity:

$$j_u^M U + b_u^M P + b_M N$$  \hfill (5)

$M_u^U$, $M_L^L$, $j_u^M U$, $Q$ are calculated according to Architectural Institute of Japan (AIJ)[4].

The analytical results obtained in $Y$ axis, is taken as an example to investigate the strength properties. Fig.5 illustrates the formation of plastic hinge at $Q_u$. From the $Q$-$Q$ relation of each story, the story shear force at the first plastic moment is determined as $Q$. The minimum ratio of $Q$ to the seismic load used in WSD ($Q_d$) are 1.54 in the case of 2nd to 14th stories. $Q$ are determined when any one (or more) of the stories is deflected about 1/50 rad. When the stories from 3rd to 14th are deflected about 1/50 rad., the minimum ratio of $Q$ to $Q_d$ becomes 1.94. In this stage, plastic hinges are formed at both ends of 2nd to 28th story beams and bottom end of a column as assumed in the design criteria. $Q$ are determined when each story is deflected to 1/50 rad. In the upper stories, $Q_u$ become larger in comparison to $Q_y$.

2.2.2 Dynamic design

The time history inelastic response analysis is carried out and the factor of safety against seismic loading is determined comparing with the criteria in Table 1. To carry out the analysis, multidegree of freedom system with a horizontal deformation and a rotation for a story are used. This model reflects the axial deformation with flexure and it can be regarded as a shear flexure model. The restoring force characteristics, obtained from the Shear-Deflection relation established by load incremental method in static inelastic analysis, are separated in flexural and shear component, which are regarded as linear and inelastic respectively. The Normal Trilinear Hysteresis Rule is used to the shear component (Fig.6). The employed earthquake waves with maximum input accelerations are shown in Table 2. The response results are shown only in the Y-axis. The EL CENTRO and HACHINOHE earthquake waves are resulted in larger response values. Fig.7 shows shear coefficient, story shear, story drift and ductility factor in the case of those earthquakes for level 1 and 2. The story shear of earthquake level 1 are within $Q_d$ and well below $Q$. Also the story drifts are well below the permissible limit of 1/200 rad. The story shear of earthquake level 2 are less than $Q$. The ductility factors and the story drifts are well below their permissible limits of 2 and 1/100 rad. respectively. Therefore, the seismic design criteria tabulated in Table 1 are highly satisfied by the earthquake waves.

3. Experimental verification of structural responses

To ensure the safety of the building structure, interior beam-column sub-assemblages are tested. Based on the test results and analytical studies, the procedure for the estimation of strength capacity, rigidity and ductility of the members are developed for the justification of structural
design.

3.1 Test program

Five 1/2 scale model specimens were prepared and tested. Their cross sectional properties and the geometry are listed in Table 3 and shown in Fig.8. Table 4 lists the mechanical properties of the materials. Inner diaphragms equivalent to designated ones are used. The main variables considered in the experiment are (1) width-thickness ratio of STC, (2) type of inner diaphragms (HT, VT), and (3) horizontal stiffness of beam webs. In the preliminary study of planning for specimens, it was predicted that the diaphragms of specimen No.2 whose width-thickness ratio of STC is larger and No.5 whose diaphragm is of VT, have been yielded before yielding of beam flange [1,5]. Except No.4 which is supplied by horizontal stiffner, the webs of all other specimens buckled due to shear at the maximum loading. Static reversal loadings are applied at both ends of the beams (Q_h) while constant axial loads (N) were under application on columns (Table 3). Fig.9 shows details of diaphragms.

3.2 Outline of test results

Fig.10 shows the relation between shear strength of columns (Q_c) and story drift (R_t). It includes calculated values for Q_c by Eq.(1), which is used for USD. All the specimens show the - 3 spindle type hysteresis response of excellent ductility up to R_t = 25 x 10^-3 rad. The failure modes for maximum load are distinguished in three types such as the development in the shear buckling of beam webs (No.1, No.3, No.5), development in the displacement of joint panel and column (No.2), breaking of beam flange at the beam ends (No.4). At the maximum loading, crack appeared at the end of scallop of beam flange in the case of No.1 and No.5. Excluding No.3 whose STC is thicker, diaphragms of all other specimens appear to be yielded. The diaphragms of No.2 and No.5 yielded before yielding of beam flanges, and therefore, the yield or maximum strength of beams of these specimens were smaller than that of the other specimens.

3.3 Response behavior of structural members

The synthetic curves of structural members are shown in Fig.11. The response behaviors of structural members are interpreted below.

3.3.1 Response behavior of beams

(1) Yield strength of diaphragms

As the diaphragms of No.2 and No.5 were yielded before the yielding of beam flanges, their yield strengths can not be computed from flexural analysis of cross section. The joint yield strength is analysed by using the limit analysis, proposed by Morita et al [1]. Fig.12 and Fig.13 illustrate the deformation pattern of diaphragms at the end of experiment and the synthetic strain distribution of beam ends respectively. The yield line mechanism, assumed based on Figs.12-13, is shown in Fig.14. Using the notations as in the figure, the following equations can be derived for yield strength.

Bearing capacity of column flange (\( c_{P_y} \))

\[
 c_{P_y} = 4 m \left\{ (2X + b_f) / B + b / X \right\}
\] (6)
where, \( c'_p = c'_y c_t t^2/4 \); for HT \( X = \sqrt{B_c B_t - b/2} \).

\[
\text{for VT } X = \sqrt{4 b \cdot M_y / (8 c_p s d_y + \cdot t d_q )}.
\]

Bearing capacity of inner diaphragm \( d_{P_y} \)

HT Diaphragm:
\[
d_{P_y} = \sqrt{2 h_d \cdot t_d d_y}
\]

VT Diaphragm:
\[
d_{P_y} = d_y t_f + X
\]

Based on the results obtained from the above equations, the yield strength of beams is estimated as follows (Fig. 15).

\[
c_{b_d y} = (c_{P_y} + d_{P_y}) h/l_b = c_{M_b d y}/l_b
\]

where, \( h \) is moment arm (Fig. 8) and \( l_b \) is shown in Fig. 8. Table 5 lists the measured and calculated results which show good agreement.

(2) Yield strength of beam

The yield strength of beam of specimens No.2 and No.5 are calculated by Eq.(8) and that of the others by e-function method (the relation of stress-strain of concrete is outlined by e-function and steel is considered to be perfectly elastoplastic assuming that plane section remains plane after bending) [6]. This calculated results \( (c_{b_d y}) \) are compared with the measured values \( (c_{b_y}) \) and presented in Table 6 and Fig. 11(a). As shown, they are mutually agreed with average ratio of 0.99.

(3) Maximum strength

The ratio of maximum strengths, measured in experiment \( (c_{b_y}) \) to computed by specified equation of design [3] and e-function \( (c_{b_u d}, c_{b_u e}) \), are 0.85 to 1.05, 0.89 to 1.09 in the case of positive moment and 1.17 to 1.30, 1.25 to 1.38 in the case of negative moment. The measured values, in the case of negative moment, became much higher in comparison to the calculated values due to the fact that the effect of strain hardening in steel is not considered. Additionally, in the case of specimens No.2 and No.5, the joint ultimate strengths are calculated using \( c_{b_u e} \) in place of \( c_{b_d y} \) in Eqs.(6)-(8). The ratio of the measured to calculated values \( (c_{b_d u}) \) show good agreement with average ratio of 1.0. In the case of all other specimens, the flange breaking tensile forces \( (P_u = P_s + P_t + c_{b_f}) \) are calculated and using them in Eq.(8), the ultimate strengths \( (c_{b_f u}) \) are computed. The ratio of the measured to calculated values show good agreement with average ratio of 0.97.

(4) Initial stiffness and deflection behavior

The deflection angles at the maximum load were \( R = 319.0 \times 10^{-3} \) rad. (synthetic values were \( R = 21.0 \) to 37.2 \( \times 10^{-3} \) rad.) and showed higher ductility. The comparison between the calculated elastic stiffness and measured initial stiffness are presented in Fig.11(a). They show good agreement.

3.3.2 Response behavior of joint panels

In the case of the specimens whose joint panels are yielded, the yield
strengths \( (Q_{py}) \) [4] are compared with the assured values \( (Q_{py}) \), presented in Table 7 and Fig.11(b). The measured and calculated values agree well with average ratio of 0.98. As shown in Fig.16, the tubular panels approach to the shear yield strain. As shown in Fig.11(b), the calculated elastic shear stiffness agree with the measured initial stiffness except in the case of No.3.

3.3.3 Response behavior of columns

As shown in Fig.11(c), the calculated elastic stiffness show good agreement with the measured initial stiffness.

4. Conclusion

Seismic design method is established on a 31 story office building consists of concrete filled square tubular columns and steel composite beams by the prediction of structural response behavior. It is shown through the investigation that the building possesses sufficient strength against seismic load.

5. Acknowledgements

The authors would like to express their thanks to Mr. Kinya ANDO, manager, Messrs. Kiyoto EBISUGI and Hirofumi ORIMO, chief engineers, of Design Section and to all members of Structural Engineering Laboratory, of Fujita Corporation for their cooperation.

6. References

Fig.1 Line Diagram of Elevation

Fig.2 Typical Story Plan

Fig.3 Details of Inner Diaphragms

Fig.4 Story Load and Shear Capacity Distributions

Fig.5 Plastic Hinge Formation

Fig.6 Normal Trilinear Hysteresis Rule
Fig. 7 Maximum Response Behaviors

Fig. 8 Geometrical Configuration of Specimens and Loading System

Fig. 9 Details of Diaphragms

Fig. 10 Shear Force of Column-Story Drift Hysteresis Loops
Fig. 11 Synthetic Curves of Structural Members

Fig. 12 Deformation Pattern of Diaphragms at the End of Experiment

Fig. 13 Synthetic Strain Distributions at Beam End

Fig. 14 Yield Line Mechanisms

Fig. 15 Moment Arm Configuration

Fig. 16 Shear Strain Synthetic Curves of Joint Panels
Table 1 Earthquake Level and Dynamic Design Criteria

<table>
<thead>
<tr>
<th>Earthquake Levels</th>
<th>Seismic Responses</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Members do not yield</td>
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<tr>
<td>(25cm/sec)</td>
<td>Story drift.</td>
</tr>
<tr>
<td>2</td>
<td>Structure is not collapsed</td>
</tr>
<tr>
<td>(50cm/sec)</td>
<td>Story ductility factor, M.I.0</td>
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Table 2 Earthquake Waves with Input Maximum Acceleration

<table>
<thead>
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<th>Maximum Acceleration</th>
<th>Acceleration at Level 1</th>
<th>Duration of Time (sec)</th>
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<tr>
<td>EL CENTO 1940 NS</td>
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<td>255</td>
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<td>TAFT 1952 EW</td>
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Note: unit in cm/sec²

Table 3 Cross-sectional Properties of Members and Axial Load on Columns

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Columns</th>
<th>Beams</th>
<th>Diaphragms</th>
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<th>D/No (ratio)</th>
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<td>□ 300x100x16</td>
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<td>-</td>
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<tr>
<td>NO. 4</td>
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<td>Sliffer only in</td>
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</tr>
<tr>
<td>NO. 5</td>
<td>□ 300x100x16</td>
<td>Web of No.4</td>
<td>-</td>
<td>250</td>
<td>0.20</td>
</tr>
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</table>

Note: No. - Spec No

Table 4 Mechanical Properties of Materials Used in Specimens

| (a) STEEL |
|-----------|-----------|-----------|
| Plates   | σy = 1.04  | σu = 1.11 |
| (mm)      | (N/mm²)   | (N/mm²)   |
| PE-02  | 346.4      | 401.4     |
| PE-10  | 346.4      | 401.4     |
| PE-16  | 346.4      | 401.4     |
| PE-22  | 346.4      | 401.4     |

Note: Normal concrete used in hollow tubular column.
Light weight concrete used in slab (ρ = 1.12).
Ex measured at 1/4 B.

Table 5 Comparison between Measured and Calculated Yield Strength of Diaphragms

<table>
<thead>
<tr>
<th>Specimens</th>
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<th>Yield Strength of Diaphragm</th>
<th>Measured (Exp.)</th>
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</table>

Note: unit in kN

Table 6 Comparison between Measured and Calculated Yield and Maximum Strength of Beams

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<th>Specimens</th>
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Note: unit in kN

Table 7 Comparison between Measured and Calculated Yield Strength of Joint Panels

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Note: unit in kN

401
PUSH-OUT TESTS ON CONCRETE-FILLED STEEL HOLLOW SECTIONS

H Shakir-Khalil

Civil Engineering Department
University of Manchester, England, UK

Summary

Push-out tests have been carried out on 24 concrete-filled steel hollow section tubes. The steel-concrete interface was 200, 400, or 600mm long, and the interface of half the specimens was covered with oil. Moreover, 28 concrete-filled steel hollow section tubes of similar and larger sections were also tested. They were 450mm long, and the steel-concrete interface about 400mm long. The tested specimens consisted of an equal number of steel square and circular hollow section tubes.

The load was applied at the top of the specimens on the concrete core, and was resisted either by resting the steel section on its base, or through the use of brackets welded to the sides of the steel specimens. Specimens were tested both with and without the use of 'Hilti' nails as mechanical shear connectors.

The tests showed clearly that the resistance of the specimens to the push-out load is a function of the shape of the steel hollow section used, the condition of the steel-concrete interface and also the way in which the load is applied to the steel section.

1. Introduction

The current work in the Civil Engineering Department, University of Manchester, UK, was made possible by a 3-year cooperative grant to the author from the Science and Engineering Research Council (SERC) in order to investigate beam-column connections in composite structures. British Steel Technical, Corby, UK, who are fully involved in the project, provided all the steel and manufactured test specimens required for the experimental programme. The work presented here forms only part of the preliminary investigations which have been carried out in the project with a view to studying the bond strength between the concrete core and the surrounding steel hollow section. The study also investigates the push-out shear resistance provided by the mechanical shear connectors used in order to improve the beam load transfer at the beam-column connection of composite structures.

The results of the experimental work on 36 specimens, series 'X', 'A' and 'B', have already been published [1]. The work investigated the push-out resistance of the specimens due to the steel-concrete bond, and also the gain achieved from the use of mechanical shear connectors. The connectors consisted of either Hilti nails [2] or grade 4.6 black bolts. The latter were fixed to the steel tube either by the use of tapped holes or by the flow drilling manufacturing process [3]. All specimens were tested by resting them on the steel tube at the base and applying the load on the concrete core at the top. The work clearly showed the advantage of using the Hilti nails in increasing the load carrying capacity of the specimens, and also the ease of fixing them by a relatively inexperienced operator.

In the work reported here, it was decided to investigate the effect of the interface length on the carrying capacity of the specimens, and also to study the adverse effect of oiling the concrete-steel interface. It was also decided to apply the load to the steel section through the use of steel brackets in order to simulate practical simple beam-column connections in multi-storey buildings. Only Hilti nails were used as mechanical shear connectors in this work. The nails used were DN drive Hilti nails, 3.7mm in diameter and 62mm long.
2. Test specimens

The steel used was mild steel grade 43, and the concrete mix was 1:1.9:2.4/0.55. For all specimens, the length of the steel–concrete interface was maintained at approximately 50mm less than the specimen length. When Hilti nails were used, they were fixed to the steel section before the casting of the concrete core.

The specimens were prepared by being clamped to the vibrating table and the concrete was then cast to about 50mm below the top end of the specimen. Five 100mm concrete cubes and two solid 150x300mm cylinders were cast with each set of test specimens.

2.1 Test series 'Y'

The steel hollow sections used for this series were 150x150x5RHS and 168.3x5CHS. Four specimens 250, 450 and 650mm long were prepared from each size, i.e. 24 specimens in all from both sizes. All specimens of the same size of steel section were cut from the same length of steel tubing. Only two of the 650mm long specimens from each size were provided with strain gauges. The gauges were fixed on opposite sides of the specimens with a view to providing information on the strain distribution along the steel tubes.

2.2 Test series 'C' & 'D'

Six specimens of 150x150x5RHS were tested in series 'C', and two of 168.3x5CHS in series 'D'. Each specimen was provided with 200mm high brackets through which the load applied to the concrete core was transferred to the steel section. Two of the square specimens had the brackets connected at the middle of the specimens' side walls, and the other four of these specimens had their brackets connected to the corners. The latter specimens were tested with a view to investigating such an unorthodox method of beam–column connections, and to studying whether such connections might be an improvement on the standard beam–column connections. Only two of the square specimens with brackets connected to their corners were provided with 12 nails. These test series consisted of four pairs of identical specimens. The specimen sizes of these series were the same as those used in Series 'A' and 'B' tested previously [1]. They were tested in order to compare the load carrying capacity of identical specimens when loaded both directly at the base of the steel section, and also through brackets welded to the steel section.

2.3 Test series 'E' and 'F'

Ten specimens were tested in each series, and the steel hollow sections used were 200x200x6.3RHS and 219.1x6.3CHS respectively. All specimens were 450mm long, and were provided with 200mm high brackets welded to the steel tubes on opposite sides. Each series consisted of five identical pairs of specimens, one pair of which was not provided with any means of mechanical shear connectors. The other specimens were provided with either 4, 8 or 12 Hilti nails.

3. Test results

The lower ends of the test specimens were machined. The specimens which were not provided with brackets, series 'Y', were tested by being seated on a machined thick plate and the load applied at the top to the concrete core through the use of a ball and socket loading system, Fig.1a.

The specimens with the brackets were on the other hand supported on a large square hollow section on which the brackets were seated at a distance of about 30mm from the face of the steel section, Fig.1b. The support arrangement was such that it allowed the brackets rotational freedom. This system was not always successful as a
result of the actual failure load being, in most cases, far in excess of the predicted load to which the bracket details had been designed. This caused very high frictional forces to develop between the brackets and supports, and the mode of failure was sometimes by failure of the bracket instead of by excessive steel–concrete slip. In view of the high failure loads of the bracketed specimens, and as a result of the inadequacy of the bracket design, some of the specimens provided with brackets were tested by being seated on the lower end of the steel tube.

The predicted failure loads reported below are obtained on the basis of British Standards BS 5400 [4], BS 5950 [5] and BS 8110 [6]. The steel–concrete bond strength given in the latter is $0.4\text{N/mm}^2$. The load carrying capacity of the shear connectors was taken as the least of the bearing capacity of the wall of the steel section, the bearing capacity of the concrete, and the shear capacity of the shear connector. This resulted in a predicted carrying capacity of $4.03\text{kN}$ per nail as based on the shear capacity of the nails if a shear strength of $375\text{N/mm}^2$ is assumed. It should however be stated that this predicted shear strength capacity of the Hilti nails is a great deal less than the $14.53\text{kN}$ which was obtained experimentally from the average of four tests carried out on nails in double shear. The nail carrying capacity of $4.03\text{kN}$ was nevertheless used as the basis of the predicted failure loads of the specimens. This is as a result of the fact that the experimental value of $14.5\text{kN}$ is far in excess of both the bearing capacity of the concrete and the steel wall. The predicted failure loads of the specimens which were provided with Hilti nails were taken as the sum of the steel–concrete bond strength and the load carrying capacity of the shear connectors.

3.1 Test series 'Y'

The properties and test results of this series are shown in Table 1. The fourth digit '0' of a specimen's designation indicates that the steel–concrete interface had been oiled prior to casting the concrete core. The steel sections used were $150\times150\times5\text{RHS}$ for $Y_1$–$Y_3$, and $168.3\times5\text{CHS}$ for $Y_4$–$Y_6$. The tested specimens in each group had concrete–steel interface lengths of approximately 200, 400 and 600mm, i.e. in the ratio of 1:2:3. However, as can be seen from the table, the failure loads of each group of specimens, whether 'dry' or 'oiled', are not in a similar ratio. More tests seem to be required in order to establish the relationship between the failure load and the interface length.

The average bond strength result for each group clearly indicates that the bond strength of the circular specimens is on average about 82% and 64% higher than that of the rectangular specimens for the 'dry' and 'oiled' conditions respectively. Furthermore, the 'dry' specimens give average bond strengths which are about twice those of the 'oiled' specimens for both the rectangular and circular specimens. The relatively wide range of bond strength obtained within each group of specimens seems to indicate that the shear resistance of concrete–filled steel hollow sections is rather sensitive to the conditions of the concrete–steel interface, and also to the irregularities in the internal dimensions of the steel hollow section. These factors respectively affect the micro and macro resistances of the section to the push–out force.

Figs.2a and 2b show the load–strain relationships for the 650mm long RHS and CHS specimens respectively. Similar to earlier results published elsewhere [1], they show the gradual strain increase in the steel tube wall from the top of the specimens to their base. This is clearly due to the load transfer from the concrete core to the steel hollow section through the bond acting along the concrete–steel interface.

Figs.3 and 4 show the load–slip relationships for the rectangular and circular specimens respectively, with Figs.3b and 4b showing these relationships for the specimens in which the concrete–steel interface had been oiled prior to the casting of the concrete core. In the linear part of the curves, the slip is seen to increase at a relatively higher rate in the case of the 'oiled' specimens. Furthermore, the 'oiled' specimens exhibit a longer transitional curve between the linear part and the point at which the maximum load is reached. Apart from the above, there seem to be few other differences in the characteristics of the load–slip curves of the 'dry' and 'oiled' specimens.
3.2 Test series 'C'

Fig. 1b shows the test arrangements of the specimens tested in this series, and Figs. 5a&b give the load–slip relationships of all six specimens in this series. Table 2 gives the test results of all eight RHS and CHS specimens of both series 'C' and 'D'.

As can be seen from Table 2, the control specimens C1a and C1b, in which the brackets were welded halfway between the section corners, gave an average failure load of 199.3kN compared to the value of 45.6kN of similar specimens tested previously, A1a and A1b, in which the specimens were supported on their base [1]. Although the strains were recorded along the steel section as shown in Fig. 1b, the load–strain relationships are not given here. This is a result of the fact that the relationships were rather erratic, partly due to the fact that the brackets were not perfectly seated on the supports, and this caused the specimens to rock on their supports. However, the general trend of the strains showed that compared to the specimens of series 'A' and 'B' tested previously [1], the recorded strains of these specimens were much lower, and also that the lower ends of the specimens were in tension. The latter resulted from the way the specimens were supported which caused the steel–concrete bond at the lower end of the specimens to cause longitudinal tensile stresses in the steel section in the level below the bracket. This caused circumferential contraction of the steel tube, and consequently increased the resistance to the concrete slip. This is the opposite of what took place in the previous work in which the lower end of the steel tube underwent relatively high longitudinal compressive strains. This caused circumferential expansion of the section, and helped ease the passage of the concrete under the effect of the push-out load.

The control specimens with brackets connected at the corners, C2a and C2b, gave an average failure load of 232.7kN which is higher than the value of 199.3kN of specimens C1a and C1b in which the brackets were connected to the steel wall halfway between the corners. However, specimens C2a and C2b were seen after failure to have suffered distortions as the compressive forces at the top of the brackets were taken by bearing on a small area of the concrete core. Consequently, the end sections were seen to have deformed to a diamond shape. The ends of the specimens were also observed to rock on flat surfaces due to out of plane deformation of the end sections of the steel tubes. The failure loads of all four control specimens of this series are seen to be on average over four times those tested previously [1], and in which similar specimens were tested when seated on the lower end of the steel tube.

Similar to the control specimens, the specimens provided with 12 nails also gave failure loads which were much higher than their counterparts in the previous series [1]. Depending on one's definition of a failure load as obtained from the load–slip relationships shown in Figs. 5a&b, the failure loads of specimen C3a and C3b are seen to be 717/856kN and 528/800kN respectively. These values are more than twice the average failure load of 280kN of similar specimens, A3a and A3b, of the previous work [1] in which the specimens were tested by resting them on their base, Fig. 1a.

Fig. 5a shows that the plain specimens failed suddenly, slipped by about 3mm, and that the applied load dropped considerably before it could be further increased. On the other hand, Fig. 5b shows that the specimens with nails exhibited no drop in the applied load. It seems that after the break of steel–concrete bond, most of the load was transferred to the nails which by bending provided additional resistance to further slip.

3.3 Test series 'D'

Only two specimens, D1a and D1b, were tested in this series; they were identical to each other, and neither was provided with shear connectors. They were tested in order to compare their results with the control specimens B1a and B1b tested previously by resting them on their base [1]. The load–slip relationships of specimen D1a is shown in Fig. 6. Both specimens carried much higher loads than predicted as shown in Table 2. Specimen D1b which was tested first had to be unloaded and then reloaded as its carrying capacity was beyond the load range of 500kN to which the test machine was originally set. Specimen D1a also had to be unloaded and then reloaded as a result of
gross distortions of the supporting base on which the brackets were seated. Both specimens supported maximum loads in excess of 800kN, indicating a bond strength value of over 4N/mm². Their 'failure' was due to the failure of the brackets which were not designed to support such high loads. The loads supported by these specimens are a great deal higher than the 91.2kN average failure load of similar specimens, BIa and BIb, tested previously by being seated on their base [1]. Even at loads of 800kN, very little slip had taken place, and the tests were only terminated due to excessive deformation and tearing of the brackets which had not been designed to support such high failure load.

3.4 Test series 'E'

Table 3 shows the properties and test results of the ten specimens tested in this series which consisted of five identical pairs of specimens. Specimens EIa and EIb were the control specimens, whereas the others were provided with 4, 8 or 12 nails. The load–slip relationships of specimens EIa–ESa are shown in Fig.7. Specimens EIb–ESb behaved similarly. With the exception of specimen EIb, none of the other specimens exhibit a definite failure load, and the 'failure' loads in Table 2 are therefore given for several values of slip. The failure loads of specimens EIa&b are seen to be less than the predicted loads, indicating a bond strength less than 0.4N/mm². All the other specimens which were provided with nails, failed at loads much higher than the predicted values, and the tests were only terminated when the slip exceeded 25mm. It is noteworthy that specimens E4a&b, in which the nails were fixed to the same steel wall as the brackets, indicated higher failure loads and better load–slip characteristics than specimens E5a&b in which the nails were fixed to the other steel walls of the section.

It should be stated that, as reported earlier [1], the nails do not normally fail under the effect of the push-out load. They bend, are flattened against the steel wall, take a hook-like form and do not release their grip on the concrete core. This goes some way towards explaining the shape of the load–slip curves which exhibit no sudden failure. They also show the ever increasing slip under a gradually increasing load. The 62mm long nails only release their grip after a large slip has taken place, or when, as happened in some cases, their heads shear off.

3.5 Test series 'F'

The properties and test results of this series are summarized in Table 4. Although all specimens were provided with brackets, only specimens F1a&b were tested making use of the brackets as supports, Fig.1.b. When testing specimens F1a&b, their brackets failed at 947kN and 971kN respectively. The specimens were unloaded, seated on their base and retested. The load–slip relationships of both specimens are shown in Fig.8 for both loading cycles. However, the results of the second loading cycle are not included in Table 4 due to their unreliability as a result of bracket failure and the resulting distortion of the section wall which affected both bond and slip.

As a result of the bracket failure of specimens F1a&b, specimens F2–5 were tested by seating them on their base, Fig.1.a. The test results of these specimens are given in Table 4, and the load–slip relationships of specimens F2a–F5a are shown in Fig.9. Specimens F2b–F5b showed similar characteristics. The specimens are seen to have exhibited no definite failure, and Table 4 gives the 'failure' loads at various slip values. The table shows that at 2 and 4mm slip, the specimens supported loads of about twice and three times the predicted failure loads respectively. The maximum loads when the tests were terminated, were in excess of four times the predicted failure loads.

4. Conclusions

The results of the tests show that for the specimens tested in series Y, oiling the steel–concrete interface results in halving the bond resistance of the concrete-filled steel hollow sections. Changing the interface length does not seem, however, to have a direct
and proportional effect on the load carrying capacity of the specimens. This could be due to the fact that the bond strength, as based on the push-out failure load, is rather sensitive to the local imperfections of the steel-concrete interface, and also to the overall longitudinal variations in the dimensions of the steel hollow section.

The Hilti nails used in this work as mechanical shear connectors, have proved to be very effective in transferring the load between the concrete core and the steel hollow section. Their resistance has been found to be several fold their predicted capacity. They also have the advantage of suffering a large slip without actually failing.

The tests clearly show that circular sections are much more effective than rectangular sections in resisting push-out forces. This is probably due to the fact that the resistance of the circular section to a push-out force is greatly enhanced as a result of any longitudinal variation in the internal dimensions of the steel tube.

Loading specimens through brackets has been shown to increase the load resistance to push-out forces as compared to seating the specimens on their base. This is probably as a result of the 'pinching' effect of the moment applied by the bracket. However, an added beneficial effect is due to the contraction of the lower end of the steel tube which is subjected to longitudinal tensile stresses when the specimen is loaded through brackets welded to the steel tube.

5. Notation

- \( A_s, A_c \): Areas of steel and concrete respectively
- \( f_b \): Steel-concrete bond strength
- \( f_{cu}, f_{cy} \): Characteristic 28-day cube and cylinder strength of concrete
- \( L_i \): Length of steel-concrete interface
- \( N_e, N_p \): Experimental and predicted failure loads

6. Acknowledgements

The work reported here is part of an experimental programme financed by a 3-year research grant from the Science and Engineering Research Council (SERC). All the steel and the manufactured test specimens were provided by British Steel Technical (BS-T), Corby, England, in accordance with the terms of the cooperative SERC grant. The author wishes to acknowledge the assistance received from, and technical discussions carried out with, BS-T personnel during the course of the work. He also acknowledges the help received from Mr M A Mahmoud, the project’s Research Assistant, in conducting the experimental work and also in the plotting of the experimental results.

The test specimens in series Y were tested by Dr Y G Pan, with the assistance of Mr M A Mahmoud, during his work with the author in a collaborative research programme between Harbin Architectural and Civil Engineering Institute, Harbin, China and the University of Manchester, U.K.

7. References

2. Hilti (Gt. Britain) Ltd., 35 Washway Road, Sale, Cheshire, England, UK.
3. Flowdrill bv, FLOWDRILLING, Populierenlaan 18, 3735 LH Bosh en Duin, The Netherlands.
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* 150x150x5RHS  ** 168.3x5CHS

Table 1: Properties and test results of Series 'Y'

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* 150x150x5RHS  ** 168.3x5CHS

Table 2: Properties and test results of Series 'C' & 'D'
Table 3: Properties and test results of Series 'E': 200x200x6.3RHS

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Table 4: Properties and test results of Series 'F': 219.1x6.3CHS

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Fig. 1: Test arrangements for specimens
Fig. 2: Load-strain relationships; Series 'Y'

3(a) 'Dry' Specimens

Fig. 3: Load-slip relationships; RHS, Series 'Y'

4(a) 'Dry' Specimens

Fig. 4: Load-slip relationships; CHS, Series 'Y'

4(b) 'Oiled' Specimens
5(a) Specimens C1 & C2
Fig. 5: Load-slip relationship; Series 'C'

5(b) Specimens C3a & C3b

Fig. 6: Load-slip; Specimen D1a

Fig. 7: Load-slip; Ela - E5a

Fig. 8: Load-slip; Fla & Flb

Fig. 9: Load-slip; F2a - F5a
CAPACITY OF CHS T-JOINTS UNDER COMBINED OPB AND AXIAL LOADS
AND ITS INTERACTIONS WITH FRAME BEHAVIOR

Y. Kurobane, Y. Makino, K. Ogawa and T. Maruyama
Kumamoto University

Summary
Circular tubular T-joints were tested under combined OPB and axial loads. Based on the test results, 3 new ultimate capacity formulas for the joint under a OPB load and for the joint under combined loads are proposed. Emphasis is laid on that bending moments acting at joints in trussed structures are governed by frame behavior. The behavior and design of K-joints under combined OPB and axial loads in a complete truss are discussed and compared with those of T-joints.

1. Introduction
Tubular joints in truss structures come under combined bending and axial loads, owing to secondary effects, eccentricities in the joints and lateral loads acting directly on the members. Also, the joints are under combined loads after one or more of the members have buckled. An extensive test on T-joints provided much information on the capacity of T-joints under combined bending and compressive loads [1]. Nevertheless, AWS Code proposes a simple interaction formula for combined load effects basing on a limited data [2]. IIW Recommendations specify no design equations for joints under bending, although the effects of bending moments due to eccentricities in joints are automatically incorporated in the design equations [3]. Bending moments in members depend on the stiffness of both members and joints and vary nonlinearly with the applied load. Designing tubular joints against combined loads is not a simple question because the behavior of joints under combined loads interacts with the behavior of trusses. In view of the complexity of the subject, study on the behavior of tubular joints under combined loads still continues to be in demand. This stands in contrast to joints under axial loads, for which design criteria and data available are fairly complete. An extensive series of tests on tubular T-joints under combined out-of-plane bending (OPB) and axial loads was planned. The T-joint was selected simply because it is one of typical tube-to-tube joints studied in the past. Although the tests have not been completed yet, this paper summarizes tentative conclusions drawn until now. The first phase of these tests was reported elsewhere [4]. Further, a test of a complete truss is described as an illustration to show how the joint behavior interacts with failure of the whole truss. The K-joints in this truss came under combined OPB and axial loads after lateral buckling of the chords. The capacity of the K-joints is compared with that of T-joints.

2. Tests of T-joints under combined OPB and axial loads
All the T-joint specimens prepared for this test program are listed in Table 1. Of these 52 specimens, 10 specimens have not yet been tested.
The specimen designations and nominal dimensions of the chords and braces are shown in this table. Measured dimensions and mechanical properties of tubes used for the tests are shown in Table 2. Each of the specimen designations consists of 3 groups of letters hyphenated together: the 1st group indicates the type of the chord; the 2nd group, the outside diameter of the brace; the 3rd group, the type of loading (C=compression, T=tension, M=bending, CM=combined compression and bending, TM=combined tension and bending). The chords for all the specimens have a nominal outside diameter of 216.3 mm, with the chord thickness varying from 4.5 to 8.2 mm. The brace diameter to chord diameter ratio $d/D$ is varied. The material for the chords is also varied widely (See Table 2).

The axial load $N$ and/or the out-of-plane shear load $Q$ were applied at the brace end, which induced bending and torsional moments in the chord. The loading scheme is illustrated in Fig.1. All the specimens have stiff end plates at the both ends of the chords, which are pinned to a reaction frame. Two hydraulic rams, each with an independent pump, are used for bending and axial loading. In most of the combined load tests, an axial load was first applied and then a lateral load was applied.

Axial strains were measured at two sections on the brace surface and at two sections on the chord surface using 4 one-way strain gages at each section. Out-of-plane deflections of the brace were measured at two points on the brace. Axial movements of the brace were measured at the loading end of the brace with a pair of displacement transducers. All these transducers were mounted on supporting frames that were bolted to the end plates at the chord ends from both sides of the specimen. Horizontal movements of the pin installed at the heads of the two rams were also monitored by a displacement transducer. Specimen configuration and the measuring system are shown in Fig.1.

Since the shear load required to obtain a predetermined bending moment in the brace is fairly small compared with the axial load in combined load tests, the effect of friction on the magnitude of bending moment cannot be ignored. The values of bending moment in combined load tests are those corrected for the effect of frictional force in a way as shown below.

The moment $M_f$ calculated from the shear load and P-delta moment is larger than the actual moment $M$ in the brace as much as the moment $M_{slip}$ due to frictional force as shown in Fig.2. Since actual moments in braces can be evaluated from strain gage readings, which are reliable at least when the braces are in an elastic region, $M_f$ is determined by

$$M_f = M - M_{slip} \quad (1)$$

The value of $M_f$ can be inferred also from

$$M_f = \frac{k}{K+k} M_{1,slip} \quad (2)$$

where $M_{1,slip}$ denotes the value of $M_1$ at the instant when the pins start to slip and rotate, while $K$ and $k$ denote the elastic stiffnesses of the specimen and loading system respectively. Although the frictional coefficient was found to be of about 0.15, the values of bending moment are obtained by

$$M = M_1 - M_f \quad (3)$$

assuming that frictional force is constant throughout the test after the pins have started to rotate for each combined load test.
The ultimate capacity of joints is defined as the first peak load in a load vs. deformation curve, following the exactly the same rule as that used in the authors' previous study [5]. The ultimate capacities thus determined from the tests are shown in Table 1. The moment used here and hereafter is the moment in the brace at the chord surface. Examples of moment vs. rotation curves for the joints under combined bending and tensile loads are shown in Fig.3 (a), (b), (c). The rotation represents a rotation of the brace measured by the two displacement transducers described earlier but a rotation due to elastic deflection of the brace and elastic torsion of the chord is subtracted from it. The ultimate capacities mentioned above are shown by open circles in these figures.

Modes of failure observed during test are listed in Table 1. When specimens failed in a mixed failure mode, a more governing failure mode is shown earlier. Test was interrupted because of excessive deformations or a failure of an end plate in a few combined bending and tensile load tests. When, however, these joints sustained significant failures before test was terminated, the maximum load is considered close to the ultimate capacity, which is identified by the symbol LCU in Table 1.

3. Ultimate capacity of joints under axial or bending load

The ultimate capacities observed in compressive load tests are compared with the following prediction equation [5] as shown in Table 1.

\[
N_{u,o} = 4.83[1 + 4.94(\frac{d}{D})^2_1 \frac{D}{T} + 0.233 \frac{L}{D} - 0.75 \frac{L}{T} \sigma_y]
\]  

(4)

The test results generally agree with the predictions except for Specimens L-A-C and M-C-C, which show greater capacities than the predictions. High axial tensile stress in the chord due to end restraints as well as low yield stress versus ultimate tensile strength ratio of the chord material may account for these discrepancies. However, the existing data base for T-joints under compression is still insufficient to carry out a reanalysis and improve the above equation.

In the same way tensile load test results are compared with the following prediction equation [5] as shown in Table 1.

\[
N_{u,o} = 1.61[1 + 4.94(\frac{d}{D})^2_1 \frac{D}{T} + 0.765 \frac{L}{D} - 0.75 \frac{L}{T} \sigma_y]
\]  

(5)

Again the test results generally agree with the predictions, except for Specimens L-A-T and M-A-C-T, in which bending deflections of the chord and the brace, respectively, governed the ultimate loads.

A regression equation for the ultimate bending capacity is derived from the OPB load test results and test results obtained at TNO [11]. The equation is shown below, which is compared with test results in Fig.4 as well as in Table 1.

\[
M_{u,o} = \frac{0.991d}{1-0.785d/D} \frac{D}{T} \frac{0.289}{\sigma_y/\sigma_u}^{-1.123} \frac{L^2}{T \sigma_y}
\]  

(6)

where the number of tests=18 and COV=0.115

It is worth noting that the OPB capacity is well fitted to a mathematical model identical to the ultimate capacity equation for the X-joint.
4. Ultimate capacity of joints under combined loads

The observed ultimate capacities \( (M_u, N_u) \) for joints under combined loads are shown in Table 1. The ultimate capacities are normalized in the form \( M_u / M_{u,o} \) and \( N_u / N_{u,o} \), and are plotted in Figs. 5 and 6, where \( M_{u,o} \) and \( N_{u,o} \) are the predicted ultimate capacities of the same joints under pure OPB and axial loads respectively.

For the joints under combined OPB and compressive loads, the ultimate capacities fall on the line

\[
\frac{M_u}{M_{u,o}} + \frac{N_u}{N_{u,o}} = 1
\]

(7)

On further straining of the joint after the ultimate load, the load path moves along the above line. This suggests that each joint is sustaining plastic deformation under a load varying along the failure envelop and that the load path has no significant effect on the joint capacity.

The strength interactions observed here is similar to that for an idealized wide-flange section with no web under strong-axis bending. This seems reasonable because the brace wall of the T-joint is mainly supported at the hot-spots on the saddle points when the joint is subjected to OPB and compressive loads.

On the contrary, T-joints show a greater bending capacity under a combined tensile load. From the strength interactions in Fig. 6 the ultimate capacities can be represented by the two straight lines,

\[
\frac{M_u}{M_{u,o}} - 0.22 \frac{N_u}{N_{u,o}} = 1
\]

(8)

\[
0.24 \frac{M_u}{M_{u,o}} + \frac{N_u}{N_{u,o}} = 1
\]

(9)

where the number of tests=9 and COV=0.094. The test results for Specimens L-A-TM, H-A-TM and MH-C-TM, which are shown by triangles in Fig. 6, were omitted in the regression analysis because failure was governed by bending of the chord for the first 2 specimens and by bending of the brace for the last specimen. In this analysis rays going through the origin of the coordinate axes and data points were considered and the error (distance between the data point and the fitted lines on the ray) sum of squares was made minimum.

As seen in Fig. 6, the bending capacity reaches the highest value of 1.16 times the capacity under pure bending when a tensile load increases to 72.2% of the capacity of the joint under tension, and then drops quickly with an increase of the tensile load.

The predicted ultimate capacities shown in Table 1 give the points of intersection of the fitted lines with the rays.

5. Interactions between joint and frame behaviors

A series of tests of 17 complete trusses with circular hollow sections have been carried out at Kumamoto University. All the specimens were Warren-type trusses and were tested under reversed loading so that the trusses would reach a complete failure after 2 to 4 cycles of loading.
Tentative findings drawn from the tests concerning interactions between the joint and truss behaviors can be summarized as follows:

1. Deflection of trusses increases with flexibility of joints. However, the elastic compliance of joints has only limited effects on deflection of the trusses. When the joints sustain plastic deformation, effects of joint deformation on truss deflection are not ignorable; when the joints are ductile, the truss can show a ductile behavior even when the members remain in an elastic region.

2. Since the in-plane stiffness of tube-to-tube joints is greater than the out-of-plane stiffness of the same joints, most of the braces sustain out-of-plane buckling. However, in-plane deflections were invariably observed preceding occurrences of out-of-plane buckling owing to secondary moments existing in the braces. When an eccentricity in the joints is large, the braces sustain in-plane buckling. After a chord or brace has buckled, the effective length factor decreases until it reaches a value of about 0.5 when surrounding members are still in an elastic or nearly elastic region. Therefore, the joint stiffness gives no significant effect on the post-buckling behavior of members.

3. Shell bending or local buckling failures of joints were observed at loads close to the predicted joint capacities according to the formulas developed for the K-joint under axial brace loading [5,6]. However, after one of the members has buckled, the K-joints come under combined axial and bending loads and fail at a load lower than the joint capacities predicted by the formulas mentioned above.

4. Cracks were frequently found along the toes of the welds between the brace and the chord after a few cycles of loading. These cracks were found only after the joints sustained significant shell bending deflections in the chord and brace walls but extended rapidly as the cycling progressed leading to a complete separation of the brace from the chord. These cracks are due to brittle failure of material. Material at the weld toes sustains large plastic strain when shell bending deflection occurs. As repeated shell bending deflection is applied, material becomes very brittle and cracks initiate and propagate rapidly. The shell bending or local buckling capacity of joints is a useful criteria to prevent these cracks, which frequently invite an integral failure of trusses.

The 3rd point mentioned above on joints under combined loads is the subject for later discussion. The behavior of K-joints after buckling of braces has already been discussed rather fully elsewhere [7,8]. This paper describes the behavior of K-joints after lateral buckling of chords.

6. K-joints under combined axial and OPB loads

A test of a Warren truss with overlapping K-joints is outlined first. The truss is 1250 mm deep and 4330 mm long as shown in Fig.7. Both the upper and lower chords are of 139.8x3.6 mm and all the braces are of 76.3x2.8 mm, respectively, in nominal cross-sectional dimension. A double-acting ram was used to apply an alternating deflection to the cantilevered truss. The position of load application is 1380 mm away from the truss end because universal joints and a loading frame was mounted between the truss and the ram. Lateral displacements of the chord were prevented at those positions denoted by open circles in Fig.7, which correspond to the center of out-of-plane rotation of the universal joints. Hysteresis loops for the truss observed in the test are shown by solid lines in Fig.8, where dashed lines show results of a non-linear
frame analysis. A point-hinge analysis was used, in which deformation of joints was modeled on an ideal elastic plastic load-deformation curve. Details of the joint model are described elsewhere [9]. Deterioration of the bending stiffness of the members due to local buckling is ignored in this analysis, which is a main cause for discrepancies between the test and analysis results in this figure.

Major events relevant to failure of the truss observed during test are shown in Fig.8. In this test failure was mainly governed by lateral buckling of the chords. The observed lateral deflections are depicted in Fig.7. The chord walls sustained local buckling when lateral deflections became large, which significantly deteriorated overall stiffness of the truss.

After the chords have buckled laterally, the K-joints come under combined axial and out-of-plane bending loads. Two examples of load paths of the axial force and the OPB moment acting on one K-joint, determined from the stresses in the two braces framing into it, are illustrated in Figs.9,(a) and (b). The stresses in the braces were measured at 5 cross sections on each brace using 4 strain gages at each section. The load paths do not start from the origin of coordinate axes because of dead weight of the loading system. The chained lines in the figures show assumed strength interactions between axial compressive and bending loads, where the moment capacity is calculated by the prediction formula for the Y-joint (There is no such a formula for the K-joint.),

$$M_{u,o} = \frac{T_{u,o}}{\sin \phi}$$ (10)

In the above equation, the subscripts Y and T denote the joint types, while $\phi$ signifies the angle between the chord and the brace. The axial load capacity was calculated by the prediction formula for the K-joint (Note that the axial capacity of the K-joint is determined by the ultimate load on the compression brace). The axial force decreases and the bending moment increases after lateral buckling of the chord as seen in these figures. The load paths for the tension braces lead to the outside of the interaction curves for the two reasons: (1) restraining effects given by the compression brace which is under much smaller bending moment; (2) a greater moment capacity of the joint under combined tensile and bending loads as already discussed in the previous Section. Shell bending deflections of the chord walls under the compression braces were detected by investigators at the points denoted by triangles, which suggests that shell bending occurred at stages where the load paths of the compression braces approached the interaction surfaces. In other words the K-joints sustained shell bending failure under combined axial and OPB loads at which the axial load is much lower than the capacity of the K-joint under axial brace loading. After shell bending failure the OPB moments in the compression braces increase in the reversed direction. An 80 mm long crack was found at Joint 6 along the toe of the welds with Brace 2 (The joint and brace numbers are indicated in Fig.7) after test was terminated.

7. Conclusions

As a result of extensive series of tests, an ultimate capacity polygon for T-joints under combined OPB and axial loads is proposed. As shown in Fig.5, the ultimate capacity of T-joints under combined OPB and compressive loads is represented by a straight line linking between the points
of pure bending and pure axial capacities. When the OPB load is combined with a tensile load, the moment capacity increases with an increase in tensile load until it reaches the maximum capacity at the tensile load equal to 72% of the tensile capacity. With a further increase in tensile load the moment capacity drops quickly (see Fig. 6). These rather complex strength interactions are reflected in behavior of K-joints in a truss.

Joints in a truss are subjected to combined axial and bending loads after members have buckled. Then, the joints fail owing to shell bending deflection under combined axial and bending loads at loads lower than the capacity of the joints under an axial load only. When the truss is subjected to cyclic loads, shell bending failure leads to a rapid extension of cracks along the weld toes. Two approaches are possible to safeguard against failures of joints due to combined load effects. The first approach is to design trusses to have strength sufficient to resist the life-time maximum loads so that the trusses remain nearly elastic. In this case joints can be designed assuming that they are under axial loads. The second approach is to design trusses to have sufficient ductility so that they will not collapse under the most unusual external excitations. In the latter case joints should be designed against combined loads. The latter design approach is frequently preferred to the first one in the earthquake resistant design of building and offshore structures, because to design structures for great earthquakes by requiring that the structures remain nearly elastic would be grossly uneconomical and would represent the payment of too great a cost to provide for the probability of such an occurrence. Current AIJ Recommendations tentatively propose that the joint should be designed to resist loads of 20% greater the ultimate capacity of members framing into the joint [10]. The truss illustrated herein is designed following this rule. The joints have axial capacities of about 20% greater than the buckling capacities of the chord (see Figs. 9). Although the joints demonstrated shell bending, local buckling and cracking failures under cyclic loading, failure of the truss was principally controlled by lateral buckling of the chords.

8. References


<table>
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<tr>
<th>Specimen Designation</th>
<th>Chord</th>
<th>Brace</th>
<th>Test Results</th>
<th>Predictions</th>
<th>Modes of failure</th>
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Table 1. Summary of specimen types and test results for T-joints.

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Note: STK41: Japanese Industrial Standard structural steel tube with specified minimum tensile strength of 41 kgf/mm²
HW60, HW80: Welding Engineering Standard weldable high-strength steel with specified minimum tensile strength of 60 and 80 kgf/mm² respectively
ERW: cold-formed welded tube
SL: hot-rolled seamless tube

Table 2. Mechanical properties of materials for T-joints.
Fig. 1. Test set-up for T-joints.

Fig. 2. Effect of friction in combined load test.

Fig. 3. Moment vs. rotation curves.
(combined OPB and tensile load test)

Fig. 4. Ultimate capacities of T-joint under OPB moment compared with predictions.
Fig. 5. Ultimate capacities and load paths for T-joints under combined OPB and compressive loads.

Fig. 6. Ultimate capacities for T-joints under combined OPB and tensile loads. (▲=premature failure)

Fig. 7. Warren truss and its failure modes.
Fig. 8. Load-deflection hysteresis of Warren truss.
(1) lateral buckling of upper chord, (2) shell bending deflection of chord wall in Joints 2 and 6, (3) Local buckling in upper chord, (4) lateral buckling of lower chord and additional shell bending deflection of chord wall in Joint 6, (5) local buckling in lower chord)

Fig. 9. Load paths of OPB and axial loads acting on one K-joint.
ULTIMATE CAPACITY OF GUSSET PLATE-TO-TUBE JOINTS UNDER AXIAL AND IN-PLANE BENDING LOADS

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Kurokami 2-39-1, Kumamoto 860, Japan

Summary

This study deals with XP and TP-joints with gusset plates welded to the surface of a CHS member. The types of load are compression, tension and in-plane bending applied to the gusset plates. Past researches carried out by Kurobane et al.[1], Akiyama et al.[2] and Wardenier[3] were insufficient to make the design formulae as accurate as those for tube-to-tube joints.

This report attempts to improve the ultimate and yield strength equations using an FE analysis and additional test. The FE analysis takes into account geometrical and material non-linear behavior of the joint models. Both the analysis and additional test were found to be useful to fill the gap of the existing data.

1. Introduction

Unstiffened tubular X, T and K-joints that are basic forms of CHS joints were tested extensively and established accurate design formulae based on reliability criteria. However, insufficient data for gusset plate-to-tube joints are available to derive design formulae that are as accurate as those for tube-to-tube joints. This study on simple gusset plate-to-tube joints is not only the basis for the derivation of design formulae for these particular joints, but also forms the basis for the deviation of the design formulae for other types of more sophisticated joints e.g. tube-to-tube joints reinforced with gusset plates.

2. Types of gusset plate-to-tube joints

The gusset plate-to-tube joint is formed of gusset plates and a tubular member. It consists of an X or T-joint with welded gussets instead of braces and is called a gusset plate-to-tube joint. The joint corresponds to the tube-to-tube X or T-joint and are named XP and TP-joints respectively. The types dealt with here are shown in Table 1. The loading conditions are axial compression, axial tension and in-plane bending applied to the gusset plate.

3. Comparison of FE analyses with tests

In this study, a non-linear FE analysis is used to fill the gap of exiting data and to re-evaluate the ultimate strength formulae. The FE analysis program used in this report is called COSMOS/M.

In order to verify the FE program, FE analysis results were compared with tests of gusset plate-to-tube joints[4]. The test specimens are shown in Table 2. The tensile coupon test...
results of the tube are shown in Fig. 1 together with the stress–strain diagrams used for the FE analysis. The load–deformation curves of the XP1 joints are shown in Fig. 2 together with the FE analysis results.

In the FE analysis material and geometrical nonlinear options were used. The elements used in the analysis are quadrilateral thick shells having four nodes. A sketch for element meshes for the specimen is shown in Fig. 3. The number of elements is 104 for 1/4 chord of the XP1 joint. Two cases of the stress–strain relationship for the chord materials are chosen in this analysis. One is the tri–linear relationship which is estimated by the predicted equation for cold formed tubes[5]. The other is the multi–linear relationship that is fitted with the tensile coupon test results(see Fig. 1). Stress–strain diagrams obtained by tensile coupon tests exclude the residual stress, because the stress is released by cutting off the tube to make a coupon. Fig. 2 shows the FE analysis results and the test results for XP1 joints. When a multi–linear relationship is used, the FE analysis results correspond closely to the test results until the large deformation takes place. Therefore, the FE analysis with the options used is considered to be a reliable tool to investigate the ultimate behavior of simple gusset plate–to–tube joints.

4. Ultimate strength under compression

The available investigations for gusset plate–to–tube joints are offered by Kurobane et al.[1], Akiyama et al.[2] and Wardenier[3]. All of these studies proposed the ultimate strength for gusset plate–to–tube joints. Kurobane et al. derived formulae for gusset plate–to–tube joints using the ultimate strength formulae for unstiffened tubular joints. Wardenier's equations were based on Kurobane et al's analyses, but only considered influences of important geometric variables on the ultimate strength. The Akiyama's equations gave yield strength of the joints and were based on a series of tests with relatively thin–walled chord sections. The yield strength was defined as the load at which a sudden decreases of stiffness was observed and is represented by a kink in the load–deformation relationship if plotted in their logarithmic scale.

Table 3 and Table 4 show all of XP and TP–joints test data collected by the authors. The Akiyama's formulae underestimate the ultimate strength. The Kurobane's formulae predict the ultimate strength well except for the Akiyama's data which had distinctive features that the diameter to thickness ratio was large(D/T>70). When D/T is large, the Kurobane's equations are on the unsafe side both for compression and tension. Wardenier's formulae predict the ultimate strength well for a wide range of geometrical variables.

Wardenier[6] argues as follows: "As the tubular joint tests mainly had β ratios 0.25<β<1.0, Kurobane's formulae may not be accurate for very low β-values. For example the original formulae for the XP2 joints give a higher strength than for the TP2 joints, which is not logical (see Fig. 4 for β=0)." β is defined as d/D for the tube–to–tube joints, so herein β is replaced by C/D. The FE analyses were performed on XP1 and TP1–joints to fill the gap of data for low β-values, in which the following values were chosen: β is 0.09, 0.30 and 0.45. The value for D/T are 20.0 and 48.1. The ultimate strength was usually defined by the maximum value of brace axial force. As seen in Fig 5(b) (D/T=48.07 and C/D=0.09), the joint has a load–deformation curve in which an unstable phenomenon does not occur until the joint reaches failure. As deformation enlarges, stiffness increases again. Since stiffness increases due to change in shape of the joint, the axial force of brace before the stiffness increases again is defined as the ultimate strength.

The ultimate strength according to the FE analysis and Kurobane's equation are shown in Fig. 6. The FE analysis results are lower than the predictions according to Kurobane's equation and an increase in difference is observed with the decrease of β-values. The fact that
Kurobane's formula is based on a data base of joints with $0.2 < \beta < 1.0$ is reflected in the decrease in accuracy for $\beta$-values smaller than 0.2. On the other hand, the FE analysis may be advised to include the weld size into $\beta$-value. The weld size effect becomes larger as $\beta$-value becomes smaller. The brace depth $C$ is replaced by $C+2T$ when the weld size equal to the chord thickness, although this effect is small as for the ultimate strength. It is necessary to re-evaluate the prediction formula for joints having small $\beta$-value.

Both of the ultimate strengths for XP1 and TP1-joints are nearly equal when $\beta$-value is small (see Fig. 7), since the failure pattern/yield pattern of the joint are similar to each other. An analysis by yield line theory[7] indicated the same results.

TP-joints are weakened by beam bending stress in the chord, while XP-joints are not. This effect is included in Kurobane's equation with an $L/D$ term. But uncertainties lie in that the $L/D$ term includes both of the effects of the chord bending and stiffening the shell walls.

5. Ultimate and yield strengths under tension

Kurobane et al.[1] proposed the ultimate strength equations for joints under tension. These are derived in the same way as for joints under compression. Test data for the XP and TP-joints are given in Table 5 and Table 6 respectively. Note that some of the ultimate strength data are differently defined from the definition by the authors for the XP2 and TP2-joints (see Table 5 and Table 6).

The load–deformation curves for joints under tension are given by curve A and B in Fig. 8. The ultimate strengths according to the definition used for compression test is denoted by open dots. The failure mode of tension loaded joints is characterized by crack initiation and breaking of welds at the chord to gusset plate intersection. Cracks occur before the ultimate load which is subjected to a wide fluctuations due to scatter in fabrication. Therefore the definition for the ultimate strength for compression can not be applied to tension and the yield strength is used to describe the ultimate state for tension. The yield strength is defined as the load at which a kink in the load–deformation curve occurs if plotted on log–log scales (see ref.1)). This definition, first used for tube–to–tube joints, is also adopted for the gusset plate–to–tube joint. The yield strength of specimen XP2–T–1 having clear load–deformation curves are shown by open dots in Fig. 2. Extent of yielded area at a yield strength as defined previously according to an FE analysis are shown in Fig. 9. This stage shows just beginning of the yielded region extension. It is reasonable to define the yield strength by the authors' rule. The yield strength formulae are derived in the same way as the ultimate strength formulae for tension test.

6. Ultimate strength under in–plane bending

The ultimate strength equations for joints under in–plane bending were proposed by Akiyama et al.[2], Wardenier[3] and Makino[8]. In the Makino's equations for the XP2 and TP2–joints $M_u$ is replaced with $B\cdot N_u/2$ (see Fig. 10(a)), in which $N_u$ is the ultimate strength in compression and $B$ is the gusset width. Although these equations underestimate the ultimate strength, they form lower bound predictions and can be used until more sophisticated design formula are developed. The XP4 and TP4–joints are derived by taking no account of the web and replacing the flange capacities with the ultimate strength of XP1 or TP1–joints in compression (see Fig. 10(b)). The Wardenier's formulae are based on the same concept. The Akiyama's formula for TP2–joints is based on a series of the test results. For XP2 and TP2–joints the Wardenier's formula adopts Akiyama model.

Table 7 shows all of the test data collected by the authors. It is insufficient to examine the ultimate formulae over a wide range of the geometrical variables. Additional tests were
performed to fill the gap of existing data and for the comparison with the FE analysis results. These specimens dimensions and the test set-up are shown in Table 8 and Fig. 11. The material properties and the stress–strain diagrams of the chord material are shown in Fig. 12. The test results and FE analysis results are shown in Fig. 13. Agreements between the test and analysis results are not as good as those for joints under tensile or compressive loads. FE analysis results underestimate the test results for XP2 joints and overestimate those for XP4 joints.

7. Conclusion

All the available literature and the ultimate strength formulae for the gusset plate–to–tube joints were investigated. Furthermore, the FE analysis was used to examine the behavior of the joints. The following conclusions were drawn.

(1) Wardenier's formulae are simple and predicts well joint strengths in compression.
(2) In the case of small $\beta$–ratio, the ultimate strength equations of the XP2 and TP2–joints should be re–analyzed.
(3) The design formulae for joints under tension should be based on the yield strength of joints.
(4) In the case of in–plane bending load, the ultimate strength formulae are insufficient to derive the design formulae.
(5) As the test data is sufficient to derive the design formulae applicable the various joints, the additional tests or FE analysis should be performed effectively.

Reference

(9) K.Washio, Y.Kurobane, T.Togo, Y.Mitsui and N.Nagao,"Experimental Study on Ultimate Strength of Tubular Joints with Gusset Plate (Pt.1)," Summary Papers, Annual Conference of AIJ (1970.9) in Japanese
(10) unpublished test results obtained by authors in Kumamoto Univ. from 1976 till 1977
Table 1 Type of Gusset Plate-to-Tube Joints.

Table 2 Test Specimens.

428
Table 3 Test Data of XP-Joints under Compression.

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Table 4 Test Data of TP-Joints under Compression.

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### Table 5 Test Data of XP-Joints under Tension.

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* Indicates the load at fracture (maximum load)

### Table 6 Test Data of TP-Joints under Tension.

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<th>C (cm)</th>
<th>L (cm)</th>
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<th>$R_{u,e}(t)$</th>
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* Indicates the load at fracture (maximum load)

* Indicates the load at crack detection
Table 7 Test Data of XP and TP-Joints under In-Plane Bending.

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Table 8 Dimensions of Specimens.

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Fig. 1 Stress-Strain Diagram.

Fig. 2 Load-Deformation Curve for XP1 Joints.
Fig. 3 Element Mesh for Specimen.

Fig. 4 Comparison with XP1 and TP1 Equations.

Fig. 5(a) FE Analysis Results for XP1 and TP1-Joints.

Fig. 5(b) FE Analysis Results for XP1 and TP1-Joints.

Fig. 6 Ultimate Strength Equations and FE Analysis Results.
Fig. 7 Von Mises' Stress at Ultimate Stage for XP and TP-Joints.

Fig. 8 Definition of Ultimate Strength under Tension.

Fig. 9 Von Mises' Stress at Yield Load.

Fig. 10 Statics Model for In-Plane Bending.

Fig. 11 Specimens and Test Set-Up.

Fig. 12 Stress-Strain Diagram.
Fig. 13 Tests and FE Analysis Results.
THE INFLUENCE OF THE CHORD AND CAN LENGTH ON THE STATIC STRENGTH OF UNIPLANAR TUBULAR STEEL X-JOINTS

G.J. v.d. Vegte *
R.S. Puthli */**
J. Wardenier *

* Delft University of Technology
** TNO Building and Construction Research

Summary

In this paper the results of 20 finite element analyses are presented in order to investigate the influence of the chord and can length on the static strength of compression loaded uniplanar X-joints. The influence on the static strength of the geometrical parameter $\alpha$ as well as the can length is shown. Furthermore, the ultimate load values obtained from the numerical analyses are compared with the recommended formulae of the IIW, API and the AWS. It appears that for uniplanar X-joints, the API gives minimum required can lengths which are too small, resulting in unsafe design strengths. For the $\beta$-value investigated (0.48), the AWS gives a good approximation for the reduction in joint strength with reduction in can length.

1. Introduction

In the past, many questions arose regarding the influence of the chord and can length on joint strength. Both the API [1] and the AWS [2] give recommendations for the can length. However, the minimum required can length recommended by API is significantly less than that required by the AWS. For the API, this may result in unsafe designs. In order to investigate the influence of the chord and can length on the static strength of uniplanar X-joints, non-linear finite element analyses on tubular X-joints, subjected to compression loading, have been performed. The general purpose finite element computer program MARC has been used for the numerical work. Four values of the chord length to chord radius parameter $\alpha$ have been considered for four $\beta$-values and five can lengths have been analysed for one $\beta$-value. Additional studies are planned to determine the influence of the can length for other $\beta$-values. Finally, the numerically determined ultimate loads are compared with the formulae recommended by API [1], AWS [2], IIW [3] and Kurobane [4,5].

2. Research programme

The research programme is summarized in table 1. The configuration of the X-joints is shown in figure 1. The study consists of a total of 20 finite element analyses on tubular X-joints subjected to compressive brace loading. Four $\alpha$-values ($\alpha = 3.0, 6.0, 11.5$ and $18.0$) have been considered for four $\beta$-values ($\beta = 0.25, 0.48, 0.73, 1.0$). Five can lengths have been analysed for one $\beta$-value (0.48). For the joints with $\beta = 0.25$ and 0.48 the thickness ratio $r$ is set to 0.5, and for the joints with $\beta = 0.73$ and 1.0 $r$ is taken as 1.0. For the joints with a can, the geometric chord
length parameter $\alpha$ is set to 11.5, whereas $d_0/\delta_{\text{can}} = 25.4$ and the thickness ratio $\delta_{\text{can}}/\delta_0 = 2.0$. Note: the can length of joint X17 is reduced to 0.0, which results in a chord without a can and $2\gamma = 50.8$. The can length of joint X7 is equal to the chord length, which results in a chord without a can and $2\gamma = 25.4$. The nominal dimensions of the joints are given in table 1.

The steel grade for all tubular members is Fe 510 with $f_y = 355 \text{ N/mm}^2$ and $f_u = 510 \text{ N/mm}^2$.

3. Finite element analyses

The main characteristics of the finite element analyses are as follows:

- The finite element analyses have been performed using the general purpose finite element program MARC which can model the joint behaviour under material and geometrical non-linearity.
- Eight noded thick shell elements (MARC element type 22) are used to model the joints. Both the corner nodes and the mid-side nodes of these elements have six degrees of freedom. Quadratic interpolation functions are used for coordinates, displacements and rotations. Transverse shear strains are taken into account. Seven layers of integration points are used for integration through the wall thickness.
- Due to symmetry in the joint geometry and loading, only one eighth of each joint has been modelled. The total number of elements used to model the joints is about 200. The finite element meshes used for the different $\beta$-values are shown in figure 2.
- Prescribed increments of uniform displacements applied at the brace tip are used to model the compressive loading of the X-joints (displacement controlled loading)
- The updated Lagrangian procedure has been used to update the displacement field after each iteration.
- The material properties are represented by a logarithmic stress-strain curve which has been modelled as a step-wise linear relationship (figure 3). This true stress-true strain relationship has been obtained after converting an experimentally determined tensile curve [7]. Furthermore, the Von Mises yield criterion and isotropic strain hardening have been used.

4. Modelling of the welds

For all X-joints except the joints with $\beta = 1.0$ (where welds are not modelled), the welds have been modelled in accordance with the welding details as recommended by the AWS D1.1 [2]. Shell elements have been used to model the welds, which is shown in figure 4. The dashed lines indicate the mid-planes of the shell elements. The dotted lines indicate how the geometry of the weld has been moved from the outer surfaces to the mid-planes of the circular members. It can be demonstrated that after averaging the fillet thickness of the butt weld at the saddle point and crown point, the fillet thickness of the weld is about 0.4 $\delta$. Therefore, the wall thickness of the elements AC and A'C', used to model the weld fillet, is set to 0.4 $\delta$. 

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5. Calibration of the numerical models

In ref [8], an experimental and numerical research program has been carried out on 3 uniplanar and 9 multiplanar X-joints ($\beta = 0.60$, $2\gamma = 40$ and $\tau = 1.0$). The in-plane braces of these joints were either loaded axially or loaded by in-plane or out-of-plane bending. Good agreement was observed between the numerically and experimentally obtained load-deformation curves. The largest difference in ultimate strength for the axially loaded joints was 6%. The finite element strategy applied in the present numerical analyses, is similar to that in ref [8].

6. Design codes and recommendations

The values found for the ultimate strength of the X-joints can be compared with the predictions of several design codes. None of the design codes give recommendations for the influence of the chord length parameter $\alpha$. The static strength values resulting from the numerical analyses are compared with the following codes and recommendations:

- Kurobane [4,5] gives a formula for the mean values of the ultimate load for axially loaded X-joints, which has formed the basis for the IIW Recommendations [3].
- The IIW Recommendations [3] give design values for factored load design for axially loaded joints. The characteristic values can be obtained by multiplying the design values by $\gamma_m = 1.1$.
- The API RP2A [1] design code gives lower bound allowable strengths for axially loaded uniplanar joints. The allowable strengths incorporate a safety factor of 1.7. According to the API, the influence of cans on the strength of a joint may be taken into account if the can is extended a minimum of one quarter of the chord diameter past the outside edge of the brace (see figure 5).
- The AWS D1.1-84 [2] design code is also based on a lower bound interpretation of test data. The allowable strengths incorporate a safety factor of 1.8. According to the AWS, for circular cross connections reinforced by a can, the allowable brace axial load may be taken as:

$$P = P^{(1)} + \frac{1}{2.5} \left[ P^{(2)} \cdot P^{(1)} \right]$$

for $l_{can} < 2.5 \, d_0$

$$P = P^{(2)}$$

for $l_{can} \geq 2.5 \, d_0$

where $P^{(1)}$ is the allowable strength obtained by using the nominal member thickness $t_0$ and $P^{(2)}$ is obtained by using the can thickness $t_{can}$.

The small difference between the values obtained from API * 1.7 versus AWS * 1.8 is caused by the fact that for the API the load capacity formulae are used, whereas for the AWS the punching shear formulae are used.
7. Numerical results for the joints with short chords

In figure 6, the load-displacement curves are separately given for each group of X-joints with the same $\beta$-value and with different chord lengths. The non-dimensionalized axial load $F_1/ f_y,0 \times t_0^2$ has been plotted against the non-dimensionalized crown point displacement. (Note: for $\beta = 0.25$, 0.48 and 0.73 the results are given for the joints modelled with a weld.) The non-dimensionalized static strength values are given in table 2.

In figure 7 the non-dimensionalized results of the finite element analyses for the joints with short chords have been plotted as a function of $\alpha$ for each $\beta$-value. In addition, curves for the different design codes and recommendations are plotted in figure 7 for $\alpha$-values greater than 6.0.

Considering the load-deformation diagrams in figure 6 and the load versus $\alpha$ curves in figure 7, the following observations can be made.

For all $\beta$-values, it appears that a decreasing value of $\alpha$ results in a decreasing value of the non-dimensionalized ultimate load. This effect is most pronounced for $\beta$-values of 0.48 and 0.73.

For $\alpha$-values below 6.0, a significant drop in non-dimensional ultimate load (a drop up to 44% for $\beta = 0.48$ and $\alpha = 3.0$, compared to the joint with $\alpha = 11.5$) is observed for all $\beta$-values with the exception of $\beta = 1.0$. For $\beta = 1.0$ the chord ovalizing is much less than for other $\beta$-values.

Furthermore it can be observed that for $\alpha$-values above 11.5, the increase in ultimate loads is negligible. See also figure 8, in which for each $\beta$-value, the results for the joints with short chords are normalized with respect to the joints with $\alpha = 11.5$.

8. Numerical results for the joints with cans

In figure 9, the load-displacement curves are shown for the joints reinforced by a can. In figure 10, the non-dimensionalized ultimate loads obtained from the finite element analyses for the joints reinforced by a can have been plotted as a function of $l_{\text{can}}/ d_0$. Also, curves for the different design codes and recommendations are plotted in figure 10. The non-dimensionalized static strength values are given in table 3.

(For the formula of Kurobane and the IIW Recommendations, the curves are given only for $l_{\text{can}}/ d_0$ greater than 3.0).

The following points become clear from figures 9 and 10.

A decreasing can length shows a reduction in the ultimate load. Especially for can lengths less than 3 $d_0$, the drop in ultimate load is significant. For can lengths greater than 3.0 $d_0$, the influence of the can length reduces. These last two influences were also noticed for the joints with short chords.

9. Comparison of the numerical results with several design codes

Comparison of the numerical results with the available formulae are given in table 2 (see also figure 7). The following observations are made:

For the joints with $\beta = 0.25$ and $\alpha$ greater than 6.0, the results are close to the lower bound values of the API and the AWS and the characteristic values of the IIW.

For the joints with $\beta = 0.48$ and 0.73, the numerically determined ultimate loads vary from the lower bound values of the API and the AWS.
and the characteristic values of the IIW for $\alpha = 6.0$ to the values of the Kurobane formula for $\alpha = 18.0$.

For the joints with $\beta = 1.0$ and $\alpha$ greater than 6.0, the numerical results are close to the Kurobane formula.

For the joints reinforced by a can, it appears that the joint with $L_{\text{can}}/d_0 = 5.75$ agrees reasonably well with the Kurobane formula. For values of $L_{\text{can}}/d_0$ equal to and lower than 3.0, the results are close to the lower bound value of the AWS.

Comparing the joints reinforced by a can with the API lower bound values, it appears that for the joints with $L_{\text{can}}$ between the minimum required can length ($0.25 \cdot d_0$ outside the edge of the brace) and 3.0 $d_0$, the results are far below the API lower bound line. This means that API gives too small values for the minimum required can length, which results in unsafe design values.

10. Conclusions and recommendations

Based on the results of the 20 finite element analyses on uniplanar axially compressed X-joints, the following conclusions can be drawn:

- The Kurobane formula agrees reasonably well with the numerically determined ultimate loads for the joints with a large $\alpha$-ratio. However, the Kurobane formula is unconservative for $\beta = 0.25$.
- An increasing value of $\alpha$ results in an increasing value of the ultimate load. For $\alpha$-values above 11.5, the influence of $\alpha$ is small, especially for $\beta = 0.25$ and 1.0. For $\alpha$-values below 6.0, a sharp drop in ultimate load is observed, with the exception of $\beta = 1.0$.
- For the joints with short chords and $\alpha$ greater than 6.0, the numerically obtained ultimate loads are nearly equal to or above the lower bound values of the API, AWS and the characteristic values of the IIW recommendations.
- An increasing value of $L_{\text{can}}/d_0$ results in an increasing strength of the joints. For $L_{\text{can}}/d_0$ values below 3.0, a sharp drop in ultimate load is observed with the exception of $\beta = 1.0$. These phenomena are also observed for the joints with short chords.
- The API recommendations give minimum required can lengths which are too small, resulting in unsafe design strengths.
- The minimum can lengths required by the AWS recommendations are larger than for the API. For the $\beta$-value investigated (0.48), the AWS gives a good approximation for the reduction in joint strength with reduction in can length. Research on can lengths for other $\beta$-values is in progress.

11. Symbols

- $d_0$: Outer diameter of the chord
- $t_0$: Wall thickness of the chord
- $t_{\text{can}}$: Wall thickness of the can
- $d_1$: Outer diameter of the braces
- $f_{y,0}$: Yield stress of the chord member
\[ f_{u,0} \] Ultimate tensile stress of the chord member
\[ l_0 \] Length of the chord
\[ l_{can} \] Can length
\[ t_1 \] Wall thickness of the braces
\[ F_u \] Axial ultimate force on the braces
\[ \alpha \] The geometric chord length parameter \( 2*1_0/d_0 \)
\[ \beta \] Diameter ratio \( d_1/d_0 \)
\[ \gamma \] Chord radius to thickness ratio \( d_0/2*t_0 \)
\[ \tau \] Wall thickness ratio \( t_1/t_0 \)

12. References


<table>
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<tr>
<th>Joint</th>
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<th>Braces dimensions</th>
<th>Non-dimensional parameters</th>
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Table 1: Summary of the geometries investigated.
Table 2: Numerical results for the joints with short chords.

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Note: * - these numerical results are obtained from ref. [6].
- - not realistic to compare with design recommendations, because $\alpha = 3.0$

Table 3: Numerical results for the joints with a can.

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Note: * - these numerical results are obtained from ref. [6].
- - not realistic to compare with the IIW recommendations and the Kurobane formula, because $1/\text{can}^2 < 3.0$
\[ \alpha = \frac{2.l_0}{d_0} \]
\[ \beta = \frac{d_1}{d_0} \]
\[ \gamma = \frac{d_0}{2.t_0} \]
\[ \tau = \frac{t_1}{t_0} \]

Figure 1: Configuration of the X-joints.

Figure 2: Finite element meshes.
Figure 3: Engineering and logarithmic stress-strain relationships.

Figure 4: Welding details for crown and saddle point.

Figure 5: Minimum required can length according to the API [1].
Figure 6: Numerical load-displacement curves. For $\beta = 0.25$, 0.48 and 0.73 the results are given for the joints modelled with a weld.
Figure 7a: $\beta = 0.25$

Figure 7b: $\beta = 0.48$

Figure 7c: $\beta = 0.73$

Figure 7d: $\beta = 1.0$

Figure 7: Load - $\alpha$ curves.
Figure 8: Relative strength of the joints with short chords as a function of $\alpha$.

Figure 9: Numerical load-displacement curves for the joints with a can.

Figure 10: Load - $l_{can}/d_0$ curves for the joints with a can.
THE ULTIMATE BEHAVIOR OF CIRCULAR MULTIPLANAR TT-JOINTS

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Abstract

Twelve static tests on circular multiplanar TT-joint specimens were performed. The objective of the tests was to investigate influences of the diameter ratio $\beta = d/D$, the transverse gap to chord diameter ratio $g/D$ and the out-of-plane angle $\phi$ on the ultimate capacity of the joint. The parameters were varied in the following range: $0.22 \leq \beta \leq 0.73$, $0.04 \leq g/D \leq 0.73$ and $60^\circ \leq \phi \leq 120^\circ$.

The ultimate capacities of TT-joints are compared with T-joint predictions by the formula of Kurobane which forms the basis of the I IW and CIDECT design recommendations as well as with the formula for multiplanar joints of the AWS design recommendations. In the range $60^\circ \leq \phi \leq 120^\circ$ the ultimate capacity of multiplanar TT-joints is higher than the capacity of uniplanar T-joints, and therefore can be treated in design as two separate T-joints. The ultimate capacity of TT-joints can be predicted more accurately by a proposed formula which takes into account the observed multiplanar effects.

In the majority of cases the AWS design recommendations overpredicts the ultimate capacity of TT-joints to a maximum of 35%.

1. Introduction

The design of multiplanar joints of circular hollow sections is performed following guidelines in the various design codes, mainly based on experimental studies of uniplanar joints. However, initial investigations show that the actual capacity of multiplanar joints varies from their uniplanar counterparts depending on the geometry and loading.

Only one major code, the A.W.S. Structural Welding Code [1], gives advanced design recommendations for multiplanar joints. The $\alpha_0$-approach for chord ovalization, used in this code, is based on the elastic shell theory and can only be considered an initial step in multiplanar joint design.


Only limited ultimate capacity data are available for multiplanar joints of circular hollow sections [4] [5] [6] [7] [8]. Consequently, an experimental study treating multiplanar TT-joints with non-overlapping compression loaded braces was initiated.

The objective of the study was to investigate the influence of a compression loaded out-of-plane brace on the ultimate capacity of the joint with the variation of the out-of-plane
angle $\phi$ and the diameter ratio $\beta$.

2. Test programme

Twelve TT-joint specimens with identical braces were tested under axial compression (Fig. 1). The diameter ratio $\beta$, the out-of-plane angle $\phi$ and the transverse gap to chord diameter ratio $g/D$ were considered important on the TT-joint behavior and varied in the range of application. Three groups of TT-joints were analyzed each with a different angle $\phi$ between the braces; $\phi$ was taken as 60°, 90° and 120°. Within each group the diameter ratio $\beta$ was varied providing values for the diameter ratio $\beta$ between 0.22 and 0.73. The variation of $\phi$ and $\beta$ automatically implied the variation of the transverse gap to chord diameter ratio $g/D$ resulting in values between 0.04 and 0.73 for this parameter. In Fig. 2 the diameter ratio $\beta$ to $g/D$ ratio is given for the three values of the angle $\phi$ for the 12 joints of the test programme.

The geometric chord length parameter $\alpha$ and the chord thickness ratio $\gamma$ were kept constant throughout this study ($\alpha=9.81$ and $\gamma=18.24$) while the joints had no eccentricities ($e/D=0$). The wall thickness ratio $\tau$ varies from 0.66 to 0.88 but has a negligible effect on the ultimate behavior of the joint.

3. Specimen details

The measured dimensions for the specimens and the steel properties of the chord are given in Table 1. All chords had a diameter of 190.8 mm and a thickness of 5.23 mm. The brace diameter were varied between 42.8 and 139.7 mm. The chord sections were cold formed; steel was specified as STK41. Fillet welds and partial penetration butt welds were used and welding was carried out with illuminate electrodes type D4301 according to Japanese Industrial Standards (JIS) specifications.

Three tensile coupons were tested in accordance with JIS type 12B to obtain the material properties for all the tube sections.

4. Test set-up

The test set-up is given in Fig. 3. At the braces and chord ends plates were welded using fillet welds. The end plates welded to the braces were bolted to two hinges, which were bolted to a stiffened beam. The end plates welded to the chord ends were bolted to a frame.

For the tests a universal 200 ton press bench was used. The force was applied to the middle of the beam and transferred through the hinges giving axial forces in the braces. The axial forces in the specimen braces were reacted at both ends of the chord by a tension force in the plates. The tension force was transferred through the frame and reacted by the lower side of the press bench.

5. Measurements

The load was recorded by a load cell placed between the cylinder and the beam. Displacement of specific points were measured using linear variable differential transformers (LVDT) and strains were evaluated using 5 mm long electric resistance strain gauges. The readings were collected using a high speed scanning amplifier and sent to a computer which stored the data on diskettes.
In this study the brace penetration on the chord surface was defined as the displacement of the crown points relative to the centerline of the chord cross section in the direction of the brace axis. In order to calculate this brace penetration the following measurements were made (Fig. 4):

1. At two positions along the brace the horizontal brace displacement relative to the other brace was measured by two LVDT's connected to a sliding frame.
2. The vertical displacement of the upper sliding frame was measured at both sides of the braces by two LVDT's attached to a measurement frame.
3. At two cross sections of both the braces the strains were recorded, in order to calculate the moments and axial forces in the braces.

The measurement frame was bolted to the end plates at the centerline of the chord end. A pin at one side and a pin/slide system at the other side allowed distortions of the chord without loading the measurement frame.

The sliding frames consisted of two parts and allowed for a uni-directional movement between these parts. Each part of the sliding frame was fixed to one brace by a needle system, which consisted of two needles between which the brace was clamped.

The strain gauges were pasted to the specimens at two cross sections of the braces, 90° apart. The cross sections were selected to avoid local strain increase due to local effects. They were at least one and a half brace diameter away from the chord/brace intersection and at least one brace diameter away from the connection with the hinge.

For the calculation of the brace penetration it was assumed that the braces were rigid bars which rotated around the non-fixed hinge center. The rotation of the braces is denoted as $\theta$ in Fig 4. The brace rotation $\theta$ was calculated for both braces, in the following way:

$$
\theta = \arcsin \left[ \frac{-0.5[(\Delta_1-\Delta_{lb})-(\Delta_2-\Delta_{lb})]+(l_a-l_c)\sin(\frac{\phi}{2})}{(l_a-l_c)-(\delta l_a-\delta l_c)} \right]
$$

(1)

$\Delta_1$ and $\Delta_2$ are the values of the LVDT's of the upper and lower sliding frame respectively, while $l_a$ and $l_c$ are the distances between points a and c and the hinge center. $\Delta_{lb}$ and $\Delta_{lb}$ are the correction of the LVDT values for elastic beam bending, while $\Delta l_a$ and $\Delta l_c$ are the elastic elongation of the brace between point a and c and the hinge center. The elastic deformations were calculated under the assumption that the braces can be modeled by a beam which is loaded by moments and axial forces at the brace/chord intersection and hinge side and were based on the strain gauge data.

The displacement in horizontal direction of point e, the crown point, relative to the chord center line denoted as $u_{he}$ was calculated in the following way:

$$
u_{he} = -0.5(\Delta_2-\Delta_{lb})+(l_e-l_c)+(\delta l_e-\delta l_c)\sin(\frac{\phi}{2}+\theta)-(l_e-l_c)\sin(\frac{\phi}{2})
$$

(2)

The displacement in vertical direction of point e, the crown point, relative to the chord center line denoted as $u_{ve}$ was calculated in the following way:
\[ u_{x} = \frac{\Delta_3 + \Delta_4}{2} - \Delta_{3,ab} + [(l_c - l_e) + (\delta l_c - \delta l_e) \cos(\frac{\phi}{2} + \theta) - (l_c - l_e) \cos(\frac{\phi}{2})] \]

\[ - \frac{1}{48} \frac{P_v (L - d)^3}{E_{ch} I_{ch}} + \frac{1}{16} \frac{P_v (L - d)^2}{E_{ch} I_{ch}} l_{pin} \]

\[ \Delta_3 \text{ and } \Delta_4 \text{ are the values of the LDVT's fixed at the measurement frame while } \Delta_{3,ab} \text{ corrects the mean of the two LVDT values for elastic beam bending of the braces. The two last terms take into account the chord deflection and the rotation of the chord ends. } \]

\[ P_v \text{ is the total load in vertical direction, } L \text{ is the chord length, } d \text{ is the brace diameter while } E_{ch} I_{ch} \text{ is the stiffness against bending of the chord. } l_{pin} \text{ is the distance between the end plate and the pin center of the measurement frame.} \]

The mean of the horizontal and vertical displacements were taken for the two crown points e and f and the brace penetration was calculated.

Before loading the specimens to the ultimate load, several elastic cycles were made to allow for bolt sliding and to record data to calculate the elastic stiffness. The elastic stiffness was calculated using data of the decreasing part of the last elastic cycle employing the least-square method.

6. Results

Load-brace penetration curves

The overall behavior for the TT-joints can be seen in load vs. brace penetration curves given in Fig. 5.

The first peak in each curve is considered the ultimate capacity of the joint as the failure mode of the joints were associated with plastic deformation of the chord. Although joint TT-1 failed by brace buckling before plastic failure of the chord occurred it was not omitted because significant plastic chord deformation already took place.

The values for the ultimate capacity, the brace penetration at the ultimate load and the elastic stiffness of the joints are given in Table 2.

Ultimate capacity

In Fig. 7 the ultimate capacity is given as a function of the \( g/D \) ratio, the out-of-plane angle \( \phi \) and the diameter ratio \( \beta \). For the values of \( \beta=0.22 \) and \( \beta=0.32 \) the ultimate capacity decreases with an increase of the angle \( \phi \). The decrease is 10% for \( \beta=0.22 \) between joint TT-1 and TT-8 and 14% for \( \beta=0.32 \) between joint TT-2 and TT-9. While for \( \beta=0.47 \) the ultimate capacity of the joint with \( \phi=60^\circ \) is higher than the one with 120\(^\circ\), an optimum for the ultimate capacity is observed when \( \phi=90^\circ \). The ultimate capacity differences are more pronounced: joint TT-6 is 21% stronger than TT-3 and 31% stronger than joint TT-10. The ultimate capacity differences increase even more for \( \beta=0.60 \), where joint TT-7 is 51% stronger than joint TT-11.

Elastic stiffness

In Fig. 8 the elastic stiffness is given as a function of the \( g/D \) ratio, the out-of-plane angle \( \phi \) and the diameter ratio \( \beta \). For \( \beta=0.22 \) the elastic stiffness decreases with an increase of
φ while for the values of β=0.32 and β=0.47 and optimum is observed for φ=90°. Except for β=0.32 qualitatively the same changes in ultimate capacity and elastic stiffness are observed.

**Failure types**

All the joints except joint TT-1 failed by plastic failure of the chord. Two different types of plastic chord failure were clearly distinguishable and shown in Fig. 6. For failure type 1 occurring for g/D ratios smaller than 0.20, the braces acted as one member, with no local deflections in the gap area. Failure mode 1 was observed for joints TT-2, TT-3 and TT-7. For g/D ratio values larger than 0.22 failure type 2 occurred with significant local deflections in the gap area, forming a fold in radial direction. Failure mode 2 was observed for joints TT-4 to TT-6 and joints TT-8 to TT-12. Joint TT-1 failed by brace buckling. However, significant local deflection of the chord wall in the gap area suggested the occurrence of failure type 2.

After the joints of type 2 sustained the maximum load, the load dropped quickly and a minimum and second maximum were recorded. After the joints of type 1 the load sustained the maximum load, the load dropped much more gradually and no second maximum was recorded. Exceptions were joints TI-2, TI-3 and TI-7. For joints TI-8 to TI-12 and joint TT-1 which failed by brace buckling and TT-7. After joint TT-7 reached the ultimate load an unsymmetrical failure pattern occurred in which the brace penetration of one of the braces was significantly larger than the other, showing a rapid decline of the load.

For the tests on TI-joints by Scolla [4] the same types of plastic chord failure were observed. For specimens with g/D values smaller than 0.10 type 1 failures were reported (V-3 and V-5) while for specimens with a g/D value larger than 0.17 type 2 failures occurred.

Based on both the studies it can be concluded that the g/D value at which the failure mode changes is not a constant value.

**7. Comparison with T-joint**

The ultimate capacities of TT-joints were compared with the capacities of T-joint predicted by a formula proposed by Kurobane et al [3]. In Fig. 9 the TT-/T-joint ultimate capacity ratio is given as a function of g/D, β and φ, while in Fig. 10 it is given as function of β and φ. All the TT-joints have ultimate capacities larger than those predicted for T-joints.

For joints with a constant value of the diameter ratio β an increase of the capacity ratio with the decrease of the g/D ratio (or φ) is observed as long as failure mode 2 occurs. The increase in the capacity ratio increases with the increase of β. A smaller increase or even a decrease in the capacity ratio with the decrease of the g/D ratio (or φ) is observed when the failure mode changes to failure mode 1. As the capacity ratio drops with the decrease of the g/D ratio (or φ) when failure type 1 occurs to a minimum of 0.5 in case of a 100% overlap (φ=0°), an optimum in the capacity ratio is expected to occur at the g/D value (or φ value) where a change of failure mode is expected.

For joints with a constant value of the out-of-plane angle φ an almost linear increase in the capacity ratio is observed with the increase of β (or the increase of the g/D ratio) for joints with φ=90°, from 1.10 for β=0.22 to 1.54 for β=0.60. For joints with φ=60° the
capacity ratio increases with the increase of $\beta$ (or the increase of the $g/D$ ratio) as long as failure type 1 occurs and drops with the change of failure type. Capacity ratios range from 1.15 to 1.26. For the joints with $\phi=120^\circ$ no clear pattern in the capacity ratio can be seen although an increase of the capacity ratio with the increase of $\beta$ (or the decrease of the $g/D$ ratio) can be observed. The increase in the capacity ratio with the increase of $\beta$ (or the decrease of the $g/D$ ratio) is smaller than for the joints with $\phi=90^\circ$ and results in ratios from 1.02 to 1.16.

For the TT-joints tested by Scolla [4] qualitatively the same observations can be made.

8. Comparison with AWS

The values for the ultimate capacity of the TT-joints found in this study and the study of Scolla [4] are compared with the predictions for TT-joints according to the AWS design recommendations. In Fig. 11 the test/AWS capacity ratio is given as a function of $g/D$, $\beta$ and $\phi$.

The AWS overpredicts the capacity of 10 of the 12 joints in this study with a maximum of about 35%. The overprediction is largest for the joints with $\beta$ values smaller than 0.32 with $\phi=90^\circ$, while the overprediction for the joints with $\phi=120^\circ$ is larger than for the joints with $\phi=60^\circ$. The best prediction is given for the joints with $\beta$ values higher than 0.47 or where $\phi=60^\circ$. Thus, the AWS formulae do not cover the multiplanar effect of TT-joints adequately, demonstrating systematic errors.

The small gap factor proposed by Lalani et al [9] based on ref. [4] reflects only the influence of the $g/D$ ratio which is observed when failure type 2 occurs: an increase in capacity with the decrease of the $tg/D$ ratio. However, for $g/D$ values smaller than 0.2 for which this factor is proposed failure type 1 is more likely to occur showing an increase in capacity with the increase of $g/D$. Therefore it is not recommended to use this factor.

The higher capacity ratios of the joints tested by Scolla [4] may be contributed by the lower chord length factor $\alpha$ (4.44 to 5.54 vs. 8.91) and the higher tensile to yield strength ratio (1.20 to 1.40 vs. 1.15). Both effects are not covered by the AWS design recommendations.

9. TT-joint formula

To develop a formula for the ultimate capacity $P_u$ for TT-joints a model based on failure type 1 was adopted. This failure type was seen for values of $g/D$ smaller than 0.2 and is also assumed for negative values of $g/D$ when the braces overlap each other. In this case the resultant of the brace forces in vertical direction at the ultimate load of the TT-joint denoted as $P_{u,v}$ is assumed to be close to the ultimate capacity $P'_u$ of a planar T-joint with a large brace diameter $d'$. See Fig. 6. For values of $g/D$ larger than 0.22 failure type 2 occurred with local deflection between the braces. Therefore when compared with the T-joint capacity $P'_{u}$, a decrease of the resultant $P_{u,v}$ of the TT-joint with the increase of $g/D$ was assumed. When for the prediction of $P'_u$ the formula of Kurobane et al [3] was adopted, the decay of the $P_{u,v}/P'_u$ ratio was best described by an exponential function in which $g/D$ and $\phi$ were the independent variables. Based on these relations and performing a non-linear regression analysis on both the test data of ref.[4] and this study a prediction formula for the ultimate capacity $P_u$ of multiplanar TT-joints was obtained:
The variables in this formula are shown in Fig. 1, while $F_y$ is the yield stress of the chord material. The validity of eq. 4 is restricted to the following range: $0.2 \leq \beta \leq 0.6$, $60^\circ \leq \phi \leq 120^\circ$, and $c/D=0$.

Fig. 7 also shows the predictions of the ultimate capacity by eq. 4 for the joints considered in this study while Fig. 12 shows the relation between the predicted and actual values of the ultimate capacity for the joints which form the basis of this formula.

For TT-joints with $\phi$ smaller than $60^\circ$ it is advised to predict the capacity using T-joint formula of ref.[3] in which $d$ is represented by $d'$ in Fig. 6. Applying the T-joint formulae in the normal way will not be conservative when $\phi$ approaches $0^\circ$, as the TT-joint capacity is theoretically reduced to half the T-joint strength. If $\phi$ is larger than $120^\circ$ the capacity of TT-joints can be predicted by X-joints formula of ref.[3].

10. Conclusions

- For the range of $60^\circ \leq \phi \leq 120^\circ$ the ultimate capacities of TT-joints is larger than those of T-joints, and can therefore be treated as two separate T-joints in design. However, this is a conservative approach especially with large $\beta$ values and $\phi=90^\circ$.
- With the presented eq. 4 the ultimate capacity of TT-joints can be predicted in a close range taking into account the multiplanar effects observed for TT-joints.
- In the majority of cases the AWS design code overpredicts the capacity for TT-joints especially for small values of $\beta$ with $\phi=90^\circ$ with a maximum of 35%.

Acknowledgement

An acknowledgement is made to Nippon Steel Corporation for financial support.

References


### Table 1. Measured values.

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<th>D</th>
<th>T</th>
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### Table 2. Test results and comparison values.

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FIGURES

Figure 1. Joint Configuration.

Figure 2. Test programme.

Figure 3. Test set-up.
Figure 4. Measurements.

Figure 5. Load – brace penetration curves.
Figure 5. Load – brace penetration curves.

Figure 6. Failure types.

Figure 7. Ultimate capacity.

Figure 8. Elastic stiffness.
NUMERICAL INVESTIGATION INTO THE STATIC STRENGTH OF STIFFENED I-BEAM-TO-COLUMN CONNECTIONS

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Summary

This paper presents a numerical investigation into the static strength of I-beam-to-tubular-column connections stiffened by diamond plates and subjected to bending moments. A number of geometric parameters combined with a number of different load cases are chosen for a parametric study. The results are compared with current design practice, which is based on the AIJ formula for diamond plates connected to tubular columns.

Acknowledgement

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1. Introduction

This paper deals with the stiffened multiplanar connection made of I-beams and a circular tubular column. The stiffening consists of so-called diamond plates, commonly used in offshore platforms, where it connects the tubular columns to the deck construction. Because of the limited amount of research that has been carried out on this kind of connection, no formulae exist for determining the strength or the stiffness of the connection. In practice, the strength is determined by multiplying the height of the I-beam ($h_w$) and the strength of diamond plates to tubular columns, as given in the AIJ recommendations. This formula is a modification of that proposed by Kamba [5]. A large number of parameters are relevant for the strength of the connection. Apart from the thickness ratio ($r$), the width to diameter ratio ($\beta$) between I-beam flange and tubular column, and the tubular column radius to thickness ratio ($\gamma$), there are the shape and width of the stiffening and the height of the I-beam to be considered. The connection is loaded by in-plane bending moments.

The general purpose finite element computer program MARC is used for the numerical work. Material and geometrical non-linearity are taken into account. Pre- and postprocessing are done with use of the program SDRC-IDEAS.

As no experimental studies are carried out in this investigation and verification of the FE-model is desirable, a comparison is made with an experiment done by Wakabayashi et al [3] in Japan. Furthermore, an examination is made as to whether the AIJ formula for stiffened multiplanar connections between plates and tubular columns can be used for I-beam-to-tubular-column connections.
2. Research programme

In order to gain some insight into the parameters that are most influential with regard to the ultimate strength of the connection the AIJ formula [1] which describes the ultimate load and the yield load of a single diamond plate to tubular column connection is studied.

\[ N_{y, AIJ} = 0.65 \times N_{u, AIJ} = 0.65 \times (6.6 \times \frac{D_f}{d_o} + 2.9) \times t_o \times \sqrt{\frac{t_o}{h_o} \times (t_o + h_o)} \times f_y \]  \hspace{1cm} (1)

The validity ranges of this formula are: \(15 \leq 2\gamma \leq 50\), \(0.3 \leq \beta \leq 0.7\), \(0.07 \leq h_o/d_o \leq 0.3\) and \(t_o/t_o \leq 2\). With \(b_f\) depending on \(\beta\) (figure 1), \(t_o/t_o = 1\) and \(r \leq 1\), for a given \(d_o\) and \(f_y\) three parameters remain to be examined: the width of the stiffening (\(h_o\)), the width of the flanges (\(b_f\)) and the thickness of the column (\(t_o\)).

The first part of the paper is an investigation into the influence of the ratio of the stiffening width (\(h_o\)) and the flange width (\(b_f\)), nine analyses in all (table 1, HBl to HB9).

In the second part the influence of the thickness of the tubular column (\(t_o\)) is studied. In addition, four analyses are carried out with various prestresses on the column (table 1, TTL to TT7).

Finally the influence of multiplanar loads is investigated, both with and without a prestressed tubular column (table 1, MPL to MP6). In total, 19 analyses are carried out for the actual parametric study. A number of parameters are kept constant all through this paper: the thicknesses of the flanges (\(t_l\)), diamond plates (\(t_d\)) and web (\(t_w\)), the height of the I-beam (\(h_t\)) and the diameter of the tubular column (\(d_o\)).

The steel grade of all members of the connection is Fe510, with \(f_y = 355\) N/mm\(^2\) and \(f_u = 510\) N/mm\(^2\) [2] (figure 2).

3. Finite Element Analyses

- The finite element analyses are performed using the general purpose finite element program MARC which can model the connection behaviour under material and geometrical non-linearity.
- Pre- and postprocessing is carried out with use of the program SDRC-IDEAS.
- Four node thick shell elements (MARC element type 75) are used to model the connections. The nodes have six degrees of freedom.
- Due to symmetry in loading and in the geometry of the connection only a quarter of each connection is modelled. For some load cases an eighth is sufficient. The total number of elements used to model the connections is 1400 for the quarter and 700 for the eighth model. Figure 3 shows the element distribution for a typical model.
- Prescribed increments of displacements are used to model the loading (displacement controlled loading).
- The updated Lagrangian procedure is used to update the displacement field after each iteration.
- The material properties are represented by a logarithmic stress-strain curve which is modelled as a step-wise linear relationship. This true stress - true strain relationship is obtained by converting an experimentally determined tensile curve (figure 2). Furthermore, the von Mises yield criterion and isotropic strain hardening is used.

4. Verification of numerical model with experiment

To verify the accuracy of the numerical description of the model a comparison is made with an experiment carried out by Wakabayashi et al.
The measurements of the specimen in the experiment are also used in the numerical model. As no stress-strain curves of the materials used in the experiment are available, these are acquired by adapting an existing curve to fit the known material data of the materials used. The experiment was stopped when local buckling in the lower diamond plate commenced, instead of continuing beyond ultimate load.

Comparing the force-displacement trajectories (figure 4) and the information on the yield mechanisms of the numerical and the experimental model leads to three conclusions:

1. The numerical model is more flexible than the experimental model.
2. At the last known point of the experiment the load was about 6% higher than the load of the numerical model at the same deformation.
3. The failure mechanism is the same for both models: local buckling failure of the lower diamond plate.

The numerical model is therefore considered to give a sufficiently accurate description to be able to determine the behaviour of this connection. The difference in stiffnesses in the linear elastic region can be due to lack of sufficient information on experimental measurements.

5. Results of FE analyses

All connections studied showed the same mode of failure i.e. local buckling of the lower diamond plate near the connection to the beam. In all cases analysed plastic strain remains low at ultimate load and seldom exceeds 5%.

Besides the ultimate strength of the connection a rotation limit based on [6] is used. In [6] an example is worked out for a beam (stiffness EI) in a frame, with an uniformly distributed load q. The beam is connected at both ends to a tubular column by connections with a plastic moment \( M_{p,\text{con}} \), at which a rotation \( \phi \) occurs. When the deformation of the beam at mid-span is not to exceed 0.02 rad, the failure mechanism is determined. The following condition can be derived for \( \phi \):

\[
\phi = 2 \frac{\delta_{fl}}{h} \leq \frac{1}{8EI} \cdot (M_{p,\text{I-beam}} - M_{p,\text{con}}) + 0.04
\]

Therefore, with \( M_{p,\text{con}} = M_{p,\text{I-beam}} \), \( \phi = 0.04 \) is a lower bound.

Results of the analyses are shown in table 2, from which it can be seen that the deformation limit has a significant influence on the ultimate load \( M_u \) only at load cases with \( M_2/M_1 = -1 \). In all other cases the ultimate load is reached before or slightly after the deformation limit. Load deformation diagrams are given for a number of combinations (figures 5 to 8), to be able to consider the influences of different parameters.

The moment on the connection and the rotation of the connection are determined at the junction of the web of the I-beam and the tubular column.

The bending moments which occur at this position are divided by the plastic moment of the adjoining I-beam. As the transition from I-beam to diamond plate is at some distance from the tubular column, the ratio \( M/M_p \) can be considerably higher than 1.

Figure 5 shows the influence of different values of \( h_g \) for a given \( b_1 \) while all other parameters remain unchanged. The deformation capacity can be seen to be about 0.04 rad. Figure 6 shows the influence of different thicknesses of the tubular column. Figure 7 shows the influence of different loads on the tubular column for a given \( t_0 \). Figure 8 shows the influence of multiplanar loading.
6. Analysis of the results

In order to gain more insight into the influence of the different parameters, a number of diagrams is presented. Each diagram shows the relationship between the non-dimensionalised ultimate load \( M_u/M_{p,beam} \) and the various non-dimensional parameters of this study.

In the diagram of figure 9 the relation between the ultimate load and the \( h_u/b_1 \) ratio is given, using the data obtained from model HBl to HB9. For a given \( \beta \) the relation can be considered to be linear, with the lines for different \( \beta \)'s being parallel. It should be noted that \( M_{p,beam} \) by which the ultimate load is divided, goes up as \( b_1 \) increases.

The diagram of figure 10 shows the failure envelopes for multiplanar loading, obtained from the data for MP1 to MP6, with a prestress on the tubular column of 0 and 0.6 times the yield stress.

Finally some analyses are done in order to investigate the relation between the ultimate loads which are found in this study and the formula for stiffened plate-to-tubular column connections (eq. 1). The ultimate force \( M_{u,AlJ} \) is multiplied by the height of the I-beam \( (h_t-t_1) \), resulting in an ultimate moment \( M_{u,AlJ} \), by which the ultimate loads of HB1 to HB9, TT4 and TT6 are divided. The results are shown in the diagram of figure 11. While at low values of the \( h_u/b_1 \) ratio \( M_u \) corresponds well with \( M_{u,AlJ} \), at larger, more practical values applying formula of eq. 1 in the above mentioned way gives results which are too optimistic. None of the ultimate loads for non-prestressed tubular columns is lower then the proposed yield load in [1], being 0.65 times \( M_{u,AlJ} \). However, the AlJ formula has to be used with care, especially for relatively small \( \gamma \)'s and prestressed columns.

7. Conclusions

- The failure mechanism of all connections considered in this study is local buckling of the diamond plate under compression near the junction with the I-beam.
- Increasing the width of the flanges \( (b_1) \) has little influence on the ultimate strength of the connection \( (M_u) \) (see tables 1 and 2). However as the plastic moment of the I-beam increases, the \( M_u/M_{p,beam} \) ratio decreases.
- Increasing the width of the stiffening \( (h_u) \) increases the ultimate strength of the connection considerably, but not as much as would be suggested by the AlJ formula (eq. 1). For a constant flange width \( b_1 \), the increase of \( M_u/M_{p,beam} \) is almost linear.
- Increasing thickness of the tubular column \( (t_0) \) increases the ultimate strength, but again not by as much as the AlJ formula (eq. 1) would suggest.
- The presence of a compression load on the tubular column decreases the ultimate load. The influence of this load is larger with increase in the \( \gamma \) ratio.
- Symmetrical multiplanar loading slightly increases the load capacity, while antisymmetric loading reduces \( M_u \). For \( M_2/M_1 = -1 \), \( M_u \) is reduced by about a third in relation to the uniplanar loaded situation.
- The rotational capacity of this connection is 0.03 to 0.05 rad for \( M_2/M_1 = 1 \) or 0.
- Determining \( M_u \) by calculating the ultimate force in a flange plate with the AlJ formula for the ultimate load and multiplying this by the height of the beam \( (h_u) \) gives a too optimistic result in almost all cases.
- Further experimental confirmation is required to see whether premature cracking occurs.
- Additional research is required to be able to establish a more refined design formula for the ultimate strength of these type of connection, e.g. the influence of $h_s/b_1$ for other $\gamma$ and $r$ values and the influence of bending moments in the column.

8. Symbols

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<th>Symbol</th>
<th>Description</th>
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<td>cross-sectional area of a tubular column</td>
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9. References


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structures. Conf. on joints in structural steelworks, Teeside U.K.


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Table 1. Geometrical parameters

constants: \(d_0 = 1000\) mm, \(h_t = 1000\) mm, \(t_1 = 30\) mm, \(t_w = 12\) mm, \(t_s = 30\) mm
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Table 2. Ultimate loads and ratio's for variations of $b_1$ and $h_a$, for variations of $t_0$ and $N_0$ and variations of $M_u/M_1$ and $N_0$.

$M_u^*$ - ultimate load without taking into account the deformation limit.
figure 1 parameters of the connection

figure 2 stress-strain relationships

figure 3 finite element model

figure 4 comparison experimental and numerical model
Figure 5  Load deformation diagram for variations of $h_2/b_1$ for $b_1 = 400$ mm

Figure 6  Load deformation diagram for variations of $t_2$ for $N_0 = 0$ kN

Figure 7  Load deformation diagram for variations of $N_0$ for $t_2 = 30$ mm

Figure 8  Load deformation diagram for variations of $M_2/M_1$ for $N_0 = 0$ kN
A die-casting technique has been developed to cast lead-tin alloy models accurately to their finished size. Brace angles of the non-overlapped, unreinforced joints were 45° and 90°. Three values of $d/D$, $D_T$, and $d/t$ were used. 103 YT models were tested in compression or tension applied to one or both braces.

The results are presented non-dimensionally, as Ultimate Strength Reduction Ratios (USRR). USRR = $F \sin \theta / \pi (d-t) \sigma_u$, where the maximum sustainable brace force $F$ is divided by the actual cross-sectional area of the brace and $\sigma_u$, the UTS of the material of each model. Simple parametric equations are presented for compressive and tensile loading of one and both braces. The latter also fits published steel data. If $t < 0.6D_T$, failure also occurs in the braces (not in the chord) particularly in tension.

The equations published by UEG are shown to overestimate the strength of joints with thick-walled chords and to overestimate the tensile strength of YT joints.

1. Introduction

K joints are basic joints which incorporate most of the important features of more complex joints. Axial loading in tension or compression of the braces is the primary loading caused by the external loads acting on the structure and has been used here. Other loading modes will be dealt with in future papers.

The static strength was determined from the maximum force which the joint could carry during its first, monotonous loading in the chosen loading mode. The results are presented in non-dimensional form as ultimate strength reduction ratios, USSR.

The chord is a thin-walled cylindrical shell, which is much weaker in (local) bending than in compression or in tension. Because of this, it is usual (1) to relate the behaviour of tubular joints to the COMPONENTS of axial brace forces which are perpendicular to the chord axis. Accordingly, USRRs are defined as the component of load acting perpendicularly to chord, divided by the predicted strength of the plain brace tube. The latter is the ultimate tensile strength of the material of the model, multiplied by the cross-sectional area of the brace. Hence

$$\text{USRR} = \frac{F \sin \theta}{\pi (d-t) \sigma_u}$$  \hspace{1cm} (1)

Where $F$ is the force applied to the brace and $\sigma_u$ is the age-corrected tensile strength of the casting. Hence compressive forces give negative USRRs. The USRR characterises the failure, independently of the size and material of the model. It is a function of the shape and loading mode only.
A new experimental method which consists of the testing of small-scale, precision-cast models has been used for the work reported here. Small-scale models are used because this is the only practicable experimental method and extensive experiments are essential; it would be reckless to rely on finite element results without experiment verification. The models are made as die-castings in a low-melting-point alloy because welds cannot be scaled. A casting is homogeneous, like a properly heat-treated weldment made with full-penetration welds. Low-melting-point alloys can be cast in mild steel moulds; this method produces models of the necessary accuracy which require no machining. The techniques have been developed and proved for X joints (2).

2. Notation

\[ d, g, l, t, D, T, L, \theta \text{ see Fig. 1} \]

F force applied to end of brace
USRR ultimate strength reduction ratio \[ F \sin \theta / \pi (d-t) \sigma_u \]
YSRR \[ F \sin \theta / \pi (d-t) \sigma_y \]

3. Shapes and Dimensions of Models

As may be seen from Fig. 1, the shape of unreinforced YT joints (K joints with one brace perpendicular to the chord) with equal braces is defined by \( d/D, D/t, d/t, \theta \) and \( g/D \) and the lengths \( L \) and \( l \). The dimensions of the models are defined by the values of \( d, L \) and \( l \) shown in Fig. 1 and \( \theta = 45^\circ \), which are constant, and the ratios shown in Table 1; the values of \( g/D \) were 0.14, 0.18 and 0.25 for \( d/D = 0.33, 0.49 \) and 0.66 respectively. The small flanges at the ends of chord and braces are used to mount the models in the loading rigs.

Models were made with three brace diameters \( d \) to give \( d/D \) ratios of \( 1/6, 1/2, \) and \( 1/3 \). For the middle ratio \( (d/D = 1/2) \), models were made with three chord thicknesses \( T \) to give \( D/T \) ratios of 21, 37 and 65. For the middle ratio of these \( (d/D = 1/2, D/T = 37) \), models were made with three brace wallthicknesses \( t \) to give \( d/t \) ratios of 9, 19, and 30. In this way the three important shape parameters were varied systematically so that their effect on the static strength under different loadings could be determined at least for the one shape defined by \( d/D = 1/3, D/T = 37, d/t = 19 \). This required 7 shapes. A complete study of these parameters would require \( 3 \times 3 \times 3 = 27 \) different shapes. There were insufficient resources for this, but a few models were made to one of these 20 untested shapes, as shown in the last column of Table 1.

4. Loading Modes

All loads were applied to the ends of the braces. Single brace loading is important in partially failed structures and was used in addition to simultaneous loading of both braces. The reactions to all loads were carried at the ends of the chords. Axial loading was called positive if tensile and negative if compressive.

In axial loading of the 45° brace (singly or together with the 90° brace) the component of force parallel to the chord axis is carried at that end of the chord which produces compression of part of the chord, never tension. This is the condition most likely to lead to collapse of the model. Both ends of the chord are simply supported as shown in.
5. **Model Material**

A tin-lead alloy (50% Sn, 3% Sb, 47% Pb) was chosen as the model material because its stress-strain characteristic and ductility are sufficiently similar to those of offshore structural steels and models (and specimens) of convenient sizes and sufficient homogeneity can be produced in an 'ordinary' engineering laboratory at a reasonable cost.

Because it melts at 186°C to 206°C, this material (3) creeps at room temperature and has to be tested rapidly to avoid creep effects. The yield strength and UTS vary little above strain rates of about 1%/s (2). Tensile calibration specimens were tested at 2.4%/s. The average straining rate of the 140 mm long braces was 0.6%/s, but significant plastic strains were localised, extending over less than 30 mm, thereby increasing the effective strain rain.

Many tensile tests showed that the yield and ultimate tensile strength decreased with age, eg.

$$\sigma_u = (74.0 - 11.8 \log_{10} t) \text{ N/mm}^2$$

(2)

where $t$ is the time in days from casting to testing. To minimise errors, tensile specimens were cast with each model and usually tested at a similar age as the model. However, 'age-corrected' material properties from equations like Eq 2 were always used to determine USRRs, using Eq 1. At room temperature this material has a Young’s modulus of 33GPa (4) and ductility of about 30%, independent of age.

6. **The Models**

All models and associated tensile specimens (4 with each model) were chill-cast in our laboratory. The manufacture of the mild steel moulds and cores, melting, pouring and solidification have been described (5).

The accuracy of the models depends on the accuracy of the mould, cores and the locations of the cores within the mould. The chord core was located at both ends but the brace cores could only be located at one end. Because mould and cores had been annealed, they should not distort due to uniform heating, but the solidification of the castings by quenching produces large temperature gradients and some inevitable distortion of mould and cores. This does not affect the straightness of the chords as shown by the examples below.

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<th>Model No</th>
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<tr>
<td>82</td>
<td>0.000 in</td>
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The compass directions were used to define directions looking at the end of the chord and calling the brace axis North. The chord diameters D were measured in planes near
each end of the chord and in three planes near the braces. In each plane, measurements were taken in the four main compass planes, N-S, NW-SE, W-E, SW-NE. These 20 measurements are summarised below for two models, showing them to be round to ± 2%.

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The chord wall thickness T was measured near the junctions of the braces, at E, NE, N, NW and W. These 15 measurements are summarised below for twelve models.

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<th>0.49</th>
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<td>48</td>
<td>70</td>
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<td>117</td>
<td>140</td>
<td>148</td>
<td>mean</td>
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<tr>
<td>mean T, mm</td>
<td>2.08</td>
<td>2.13</td>
<td>2.18</td>
<td>2.12</td>
<td>2.12</td>
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<td>2.12</td>
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<tr>
<td>Std deviation mm</td>
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<td>0.087</td>
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<td>0.091</td>
<td>0.081</td>
<td>0.075</td>
<td>0.072</td>
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<th>0.49</th>
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<tbody>
<tr>
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<td>82</td>
<td>93</td>
<td>120</td>
<td>133</td>
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<tr>
<td>mean T, mm</td>
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<td>3.75</td>
<td>2.21</td>
<td>2.34</td>
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<td>Std deviation mm</td>
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<td>0.099</td>
<td>0.094</td>
<td>0.110</td>
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Except for the 1.2 mm thick models, the standard deviations are less than 5% of the chord wall thicknesses. Model No 117 was not tested; it was cut up to take these measurements. All the other models were (cut up and) measured after testing. No significant wall thickness changes occurred during testing because all the failures are primarily due to wall bending.

For twelve models, chord thickness T and brace thickness t measurements were also taken at the crown and saddle positions i.e. about 5 mm from the junctions of chord and braces to allow access of the micrometer anvil to the brace and chord stubs after the tubular members had been sawn off. Eight thickness measurements were taken at each of the 24 brace-chord junctions, four in the chord and four in the brace, at the crown and saddle positions. For each set of four measurements the maximum variation ΔT or Δt were recorded and analysed. The mean values were T mean = 0.16 mm and t mean = 0.30 mm. The variations in brace wall thickness were greater than those in the chord because the brace cores are only located at one end.

7. **Loading**

The frame shown in Fig. 2 was designed so that it could restrain the chord appropriately and load both braces simultaneously in any likely combination of force, e.g. balanced with the 45° brace in tension (Fig. 2) or in compression, unbalanced (provided the braces were loaded in opposite directions), tension or compression applied to one brace only.
The frame A is bolted to the cross bar B of the testing machine. The ends of the chord are simply supported on steel balls; all axial forces are carried as compression on the left. Either brace may be compressed by a stirrup C. The 90° brace is always vertical and connected to the RHS of the load-sharing beam D, whereas the 45° brace is loaded through 2 chains, connected to the LHS of D. The beam D has several holes to permit different ratios of forces to be applied (the connection between A and B is infinitely adjustable to allow this to happen). Spherical bearings in the beam avoid unwanted restraints. Beam D is connected to E, the load cell of the testing machine.

The maximum gap of the testing machine limited the size of frame A and consequently the distances of the 'fixed' points of load application F and G from the junctions of braces and chord. Hence the direction of load application of the braces changed more than is likely to occur in complete structures. However, this does not affect the collapse or tearing loads because braces only bend significantly after failures to the joint.

A 25 kN Instron testing machine type 1193 with a 25 kN load cell was used for the tensile specimens; another, 100 kN, type 1195 with a 50 kN load cell was used to load the models. The load cells for both machines were calibrated with an N.P.L certificated, Standard proving ring. After calibration, the 10 volt pen recorder was set to indicate 1, 2, 5, 10, 20 or 50 kN full scale (250 mm of chart) as appropriate. The calibration was checked on first use on each day on which tests were carried out. All forces were obtained by measurement of the pen traces on the testing machine record chart. An example is shown in Fig. 3.

All brace load measuring devices used ERS gauges bonded to flat bars which were loaded in tension. The electrical signals from these gauges were used to drive two additional pens on the testing machine load recorder. They were calibrated by separate tests in the model-testing machine when the full chart height deflection of their pens was set in the same manner as the load-cell pen.

The accuracy of load readings from the charts was investigated. The charts are high quality graph paper with 2.5 mm squares. They could be read repeatably to ± 0.1 'square' ie. to ± 0.001 of full chart height. For the largest loads, the recorder pens were set to need 50 kN force for full chart width, giving a reading accuracy of ± 50N. For most of the tests 10 kN force was used for full chart width, giving greater accuracy of ± 10N.

8. **Evaluation of Records**

After the test of a model and its calibration specimens, relevant information exists on the testing machine record and in the failed model. The former is used to calculate the ultimate strength reduction ratio USRR, which characterises the failure, independently of the size and the material of the model, ie. as a function of the shape and loading only. The model can be used to determine the position of failure origin and plastic deformations. However, due to the high testing speed, significant plastic deformation often occurs after the maximum load has been reached. Plastic hinges can be identified and plastic strains measured in the braces, away from the plastic hinges, because these strains are unlikely to increase significantly after collapse or tearing at the joint.
9. **Results**

Failure in compression always preceded failure in tension. This was observed during axial load tests when both braces were loaded and in all bending tests which will be reported elsewhere. Plastic collapse occurred before cracking tension. There was no significant change of wall thickness before cracking. This was taken as an indication that the predominant loading at failure is bending of the walls of chord and braces, as would be expected from the known elastic stresses.

Table 1 shows which shapes were tested under the different loading modes. The numbers in the Table give the USRR values obtained. The shape ratios are the mean values for each batch of castings, obtained from measurements of models. Because the three different brace diameter models had to be cast in three different moulds with three different sets of brace cores and because T and t are very small, it was impossible to achieve exactly the same mean values of real model thicknesses.

The tests are conveniently divided into single-brace loading and those where both braces were loaded. In the single-brace tests each brace was loaded in compression or in tension while the other brace was unloaded. When both braces were loaded, one was always in tension while the other was in compression. The ratio of the loads was defined as the force applied along the 45° brace axis divided by the force applied along the 90° brace axis; tensile forces were called positive and compressive ones negative. Some tests were carried out with equal forces, i.e. +1/-1 or -1/+1 but most tests were in balanced loading i.e. +1.41/-1 or -1.41/+1 (1.41 = 1 sin θ, see Fig. 1 for definition of θ) because this produces equal and opposite forces perpendicular to the chord axis.

In addition to the results presented in Table 1, seven tests were carried out on models of the 'central' shape, d/D = 0.49, D/T = 36.9, d/t = 18.7. They are listed below.

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<th>Load ratio, 45°/90° brace</th>
<th>USRR</th>
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<tr>
<td>-1/+1</td>
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<td>-1/+1</td>
<td>-0.299</td>
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<tr>
<td>+1/-1</td>
<td>-0.309</td>
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</tr>
<tr>
<td>+1/-1</td>
<td>-0.292</td>
</tr>
<tr>
<td>+1/-1</td>
<td>-0.279</td>
</tr>
<tr>
<td>+0.53/-1</td>
<td>-0.163</td>
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</table>

Fig. 4 shows how the normalised failure loads for single brace loading vary with the three shape parameters. The three shape parameters chosen also give graphs which are linear over the range of shapes tested, a convenience for formulating empirical equations. All values are plotted +ve to assist comparison and save space. Where the results are so similar that the symbols could not be distinguished, the crowded-out symbols have been drawn to the left of those plotted first; to avoid confusion, the correct values are indicated by short lines above 'x' axes.

Although there are a few cases of large experimental scatter, most of the 'repeat' tests show very good repeatability. The normalised tensile strengths are significantly greater than corresponding compressive strengths, but there is no significant difference between the results from 45° and 90° braces for any of the seven shapes studied.

Fig. 5 shows the results when both braces are loaded in opposite directions. Only
compressive values are shown because, due to the method of loading, the tensile ones are the same. Most of the results lie near the straight lines drawn, except No. 6, the first model for which results are presented and No. 50.2, i.e. the second test of this model which was tested a second time because of a loading rig failure.

Comparison of corresponding graphs in Figs. 4 and 5 shows that all the 'both braces loaded' curves are generally slightly above the single brace compressive ones. This shows the amount by which the prevention of 'beam-bending' of the chord strengthens the joints. This demonstrates that the predominant cause of failure is 'wall-bending' of the chord and braces.

Using average values for the pivotal points on the graphs leads to the following empirical equations. For single brace compression,

\[
USRR = 0.667 - 0.12 \frac{D}{d} + 12.4 \frac{T}{D} - 0.19 \frac{d}{t}
\]  

and for single brace tension,

\[
USRR = -1.055 + 0.32 \frac{D}{d} + 16.5 \frac{T}{D} + 0.023 \frac{d}{t}
\]  

and when both braces are loaded in opposite directions

\[
USRR = +0.686 - 0.155 \frac{D}{d} - 13.8 \frac{T}{D} - 0.015 \frac{d}{t}
\]

These equations were evaluated for the shapes of the models and are compared with the experimental results in Fig. 6.

10. Position of Failure

Failures always occur at the junctions of chord and braces but it is important to determine which tube failed. Punching shear methods assume that the chord fails in shear, whereas the UEG handbook implicitly assumes that the chord fails in bending. If \( t/T \) is small, it is evident that the brace, not the chord will fail.

If failure is defined as significant plastic deformation, this is readily determined for the model braces in a quantitative manner because the length changes between cast-in projections can be measured with a travelling microscope. Plastic deformation of the chord is easily recognised qualitatively but the measurement of plastic hinge lines in the chord is tedious because it requires curvature measurements. Such measurements have been made, but generally, patterns of reflected light identify chord failures. Plastic brace strains of 1% or more, as measured on the outer surface, were defined as evidence of brace failure.

An alternative method of identifying the position of failure due to brace tension or bending is study of the position of cracks or complete fractures. Obviously the brace has fractured if it is severed while the 'plug' of chord which was inside the brace remains attached to the chord. For many models it is possible to determine whether this would have happened, even if the test was stopped before it occurred.
Most of the failures started in the chord. Brace failures only occurred in joints where \( T/T < 0.6 \) i.e. models listed in Table 1 in the 1st, 2nd and 5th column. For about one quarter of these models it was not possible to state the position of first failure because they had been remelted for further use before this was deemed worthy of study. The 46 models having one of these three shapes showed that there were more brace failures than chord failures and that for these shapes brace failure is likely in tension and chord failure is likely in compression.

11. Published Work

Reliable published work is collected and analysed in the UEG Handbook (2). This book distinguishes YT joints which have one brace with \( \theta = 90^\circ \) from K joints where \( \theta \neq 90^\circ \) for both braces. They report numbers of static strength results as follows:

<table>
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<th>Type of joint</th>
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<td>Other axial loading</td>
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25 of the YT joints had \( \theta = 45^\circ \). These important shape parameters are presented in Fig. 7; \( d/t \) values are not given. Fig. 7 also gives the shapes of the tin-lead model joints. It is apparent that direct comparison with steel models is impossible. However, the authors of the UEG Handbook have used results from Y and T joints to produce the following parametric equations for the mean strength of YT joints with \( d/D < 0.6 \). For axial compression,

\[
P_u \sin \theta / \sigma_y T^2 = (4.1 + 20.3 d/D) [\frac{1}{2}(1 + \csc \theta)]
\]

where \( P_u \) is the predicted maximum force. for axial tension,

\[
(P_u \text{ in tension}) = 2.15 (P_u \text{ in compression})
\]

Eqs. 6 and 7 are Eq. B37 on page B38 and Eq. B47 on page B46 in Volume 2 of Ref 1. The published steel results are compared with the predicted values in Fig. 8, using the definitions of shapes from Fig. 7 and \( \varphi = \psi \).

12. UEG equations used for tin-lead model results

Because of the wide acceptance of the UEG Handbook, Eqs. 6 and 7 have been used for comparison with the experimental results. The UEG factor \( Q' \) has been omitted from Eq. 6 because it is equal to 1 for \( \beta < 0.6 \) and for \( \beta > 0.6 \), \( Q' = 0.3 / \beta (1 - 0.833\beta) \) which is equal to 1.01 for \( \beta = d/D = 0.66 \), the biggest value used in our work.

The single-brace model results are compared with UEG predictions in Fig. 9. Considering the basis of the UEG equations, the prediction is reasonable for compression loading but seriously wrong for tension.

Fig. 10 gives the comparison of measured and predicted compressive strengths of joints when both braces are loaded in a balanced and unbalanced manner. It shows that the UEG equation gives about the same quality of prediction as for single brace loading in compression and that the agreement for unbalanced loading is as good as for balanced.
loading. However, the model which had a much smaller force in the 45° brace than would be required for balanced loading (+0.53 instead of +11.4) was found to fit in better with single brace compression results than with the 'both braces loaded' ones.

Because Eq. 6 is independent of brace wall thickness 't', it was suspected that the bad predictions might be for joints where the failure occurred in the brace instead of the chord. Fig. 11 shows the predictions for axial loading of the three shapes for which brace failures occurred. This shows that, although most brace failure loads are over predicted, they are not significantly worse than the chord failure load predictions. It is assumed that due to continuity, the large brace wall bending strains which cause brace failure cause chord wall bending strains of similar magnitude. This assumption is reinforced by the fact that in 3 of the 9 shape - loading mode combinations, chord and brace failures occurred for models within one combination.

The straight line graphs in Figs. 4 and 5 suggested that 2 may not be the best index for T in Eq. 6. To investigate the effect of chord thickness T on the quality of the UEG predictions, all results for d/D = 0.49, d/T = 19 were plotted in Fig. 12. This shows that UEG predictions are generally good for d/T = 37, low for D/T = 65 and too high for the thick-walled chords with D/T = 21.

13. Eq. 5 used for steel model results

After the brace wall thicknesses of most of the steel models had been discovered, it was possible to evaluate Eq. 5 for these shapes, if the published values of yield stress $\sigma_y$ were used instead of the UTS, $\sigma_u$. Fig. 13 shows that Eq. 5 underpredicts the strength of joints with very thin chords (D/T > 75). The overprediction of the others is similar to that due to the UEG equation, shown in Fig. 8.

14. Discussion of tin-lead model results

Figs. 4 and 5 show that $F_{\sin \theta}$, the component of brace force perpendicular to the chord axis causes failure in compression and in tension for single brace loading and when both braces are loaded. (The differences between corresponding loadings of the 45° and 90° braces are no more than the scatter of repeat tests, which is generally small).

The single brace loading tests presented in Fig. 4 showed that the joints are stronger in tension than in compression, but the ratio of 2.15 (see Eq. 7) assumed by UEG is an oversimplification, as shown by comparison of Eq. 3 and 4.

Failures are primarily due to local, tube-wall bending. Bending of the chord as a beam, which only occurs in single brace loading, does not weaken the joints much; this is shown by the differences between the compressive results in Figs. 4 and the (compressive) failure values in Figs. 5 and illustrated by the small differences between Eq. 5 and Eq. 3.

Although Figs. 4 and 5 clearly show that the linear equations 3 to 5 are a sensible way of summarising the results, it is obvious that these linear relationships can only be valid over limited ranges of shapes because the absolute value of USRR must always be between 0 and 1. However a prohibitively large number of tests would be required to
determine the exponential relationships which would be valid over the whole range of shapes. Extending the straight lines in Figs. 5 shows that the applicability of Eq. 5 must be less than $0.16 < \frac{d}{D} < 1.0$, $12.5 < \frac{D}{T} < 130$, $2 < \frac{d}{t} < 66$. As the steel model tests in Fig. 13 identified as DT = 93 and 76 have $\frac{d}{D}$ ratios of 0.19, it is not surprising that Eq. 5 produced overpredictions. The suggested range of applicability of Eqs. 3 to 5 is $0.3 < \frac{d}{D} < 0.7$, $18 < \frac{D}{T} < 70$, $8 < \frac{d}{t} < 35$.

Remembering the definition of USRR (Eq. 1), our equations (Eq. 3, 4, 5) contain four of the five shape parameters which characterise an unreinforced, non-overlapped YT joint when the weld fillets are ignored. The effect of variation of $g$ (see Fig. 1) the gap between the braces has not been investigated. The UEG equations (Eq. 6, 7) assume that the brace wall thickness $t$ has no effect on the static strength. Fig. 11 shows that when $t < 0.6T$, brace failures are probably more frequent than chord failures. For these shapes, the UEG equations are less direct because they are concerned with the effect of these brace failures on the stresses in the chord.

The UEG equations, using the yield stress of each model, overpredict the strength of many tin-lead models as shown in Figs. 9 and 10. Fig. 12 illustrates the reason for this; because it has to have an index of 2 for $T$ to preserve dimensional homogeneity in a simple equation, the UEG predictions are too high for thick chords and too low for thin ones.

15. Discussion of steel model results

Fig. 7 shows the $\frac{d}{D}$ and $\frac{D}{T}$ values of the steel models used to formulate the UEG equations and Fig. 8 shows that the resulting predictions are safe.

Fig. 13 shows that our Equation 5 also gives safe predictions for the steel model joints whose shapes are within the applicability range of Eq. 5. (Some of the steel models are missing because no brace wall thicknesses could be found for them). The over predictions are similar to those in Fig. 8.

16. Conclusions

The efficiency of the tin-lead model technique has been demonstrated by the large number of repeatable, consistent results which have been produced in 2 years; a significant part of the time was devoted to work not reported here.

The concept of Ultimate Strength Reduction Ratios (USRR) has been introduced. The component of every failure load which is perpendicular to the chord axis is expressed as a fraction of the load which would cause failure of the plain brace in the same loading mode. This makes the model results independent of the size of the model and of the model material; the ultimate tensile strength of each model and the actual brace dimensions are used. USRRs for static loading are analogous to Fatigue Strength Reduction Factors for cyclic loading. It is useful to think of each brace as a part of the structure which does not have its full plastic strength due to the shape of the joint and the way the reaction is transmitted by the chord.

Models of seven different shapes have been tested with one or both braces loaded. Most
models failed by plastic collapse or tearing of the chord, as has been found in steel model tests. However, when $t/T < 0.6$, significant numbers of failures occur in the braces, particularly under tension.

The results have confirmed the soundness of using the component of force perpendicular to the chord axis for assessing the strength of tubular joints in brace tension and brace compression loading.

The 61 tests of either single brace in tension or compression showed that the tensile strength is much greater than the compressive strength. The 42 models in which both braces were loaded by axial forces in opposite directions all failed due to brace compression. These joints were stronger than those loaded by single brace compression, probably due to absence (or reduction) of 'beam-bending' of the chord.

New parametric equations are proposed for the above three loading modes of YT joints. They are also 'safe' for the steel model results except for very thin chords. The parametric equations in the UEG Handbook, based on the results of the 27 tests of steel models, have been shown to greatly OVERESTIMATE the strength of joints which have thick-walled chords.

Acknowledgements

This investigation was funded by a Marine Technology SERC grant. The authors wish to acknowledge the skilled, enthusiastic work of Mr Ray Pickard and other technicians. The Department of Materials Engineering gave metallurgical assistance and the Department of Mechanical Engineering generously provided financial support after the grant terminated.

References

1. 'Design of tubular joints for offshore structures' Vol 2, part B Static strength of simple welded joints, 1985, UEG Offshore Research, CIRA.
3. Tandem J101 manufactured by Fry's Metals Ltd, Glasgow, UK.
Table 1 USRR values, loading modes and shapes

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<td>0.668</td>
<td>0.228</td>
<td>0.839</td>
<td>0.630</td>
<td>0.173</td>
<td>0.437</td>
<td>0.668</td>
<td>0.228</td>
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<td>+1</td>
<td>0.783</td>
<td>0.861</td>
<td>0.173</td>
<td>0.427</td>
<td>0.647</td>
<td>0.234</td>
<td>0.783</td>
<td>0.861</td>
<td>0.173</td>
<td>0.427</td>
<td>0.647</td>
<td>0.234</td>
<td>0.783</td>
<td>0.861</td>
<td>0.173</td>
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<td>0.893</td>
<td>0.173</td>
<td>0.427</td>
<td>0.643</td>
<td>0.238</td>
<td>0.657</td>
<td>0.893</td>
<td>0.173</td>
<td>0.427</td>
<td>0.643</td>
<td>0.238</td>
<td>0.657</td>
<td>0.893</td>
<td>0.173</td>
<td>0.427</td>
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<td>0.238</td>
<td>0.305</td>
<td>0.305</td>
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<tr>
<td></td>
<td>-1.41</td>
<td>+1</td>
<td>-0.422</td>
<td>-0.514</td>
<td>-0.132</td>
<td>-0.394</td>
<td>-0.445</td>
<td>-0.096</td>
<td>-0.249</td>
<td>-0.351</td>
<td>-0.422</td>
<td>-0.514</td>
<td>-0.132</td>
<td>-0.394</td>
<td>-0.445</td>
<td>-0.096</td>
<td>-0.249</td>
<td>-0.351</td>
<td>-0.422</td>
<td>-0.514</td>
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<td>+1.41</td>
<td>-1</td>
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<td>-0.558</td>
<td>-0.132</td>
<td>-0.191</td>
<td>-0.498</td>
<td>-0.117</td>
<td>-0.257</td>
<td>-0.437</td>
<td>-0.425</td>
<td>-0.558</td>
<td>-0.132</td>
<td>-0.191</td>
<td>-0.498</td>
<td>-0.117</td>
<td>-0.257</td>
<td>-0.437</td>
<td>-0.425</td>
<td>-0.558</td>
<td>-0.132</td>
</tr>
</tbody>
</table>

Fig. 1 Model and co-ordinate system; dimensions in mm.
Fig. 2 loading arrangement

Fig. 3 Testing machine record

Fig. 4 Single brace failure loads

Fig. 5 Failure loads when both braces are loaded

483
6a Single brace compression

6b Single brace tension

6c Opposite loading of both braces

Fig. 6 Comparison of our equations with our results symbols defined in Table 1.

Fig. 7 Shapes of YT joints tested.

Fig. 8 Steel results
Fig. 9 single brace results
Fig. 10 'Both brace' results
Fig. 11 Comparison of UEG equation with brace failures

Fig. 12 Comparison of UEG equation with D/T variation
Fig. 13 Comparison of our equation with steel results
IMPACT OF INTERNATIONAL DEVELOPMENTS ON A.W.S. D1.1

Peter W. Marshall
Shell Oil Company, Houston, Texas

Summary

Tubular connection design provisions of the AWS Structural Welding Code have undergone a number of major revisions since they were first introduced in the early 1970's. In 1981-84, databases of Yura, Kurobane, and others were evaluated to formulate new criteria for circular tubes, including a first attempt at multiplanar connections. In 1980-86, results of a massive European program on fatigue were realized in weld profile and size-effect provisions. Two major revisions have just been approved for the 1992 edition. One deals with fracture toughness requirements, taking a traditional Charpy-based approach. The second is a major revision of the static strength requirements for box connections, based on the work of CIDECT and ITW s/c XV-E.

Yet work on the Code is not complete. Current topics for future revisions include: considering the effect of transverse gap in multiplanar connections, redefining hot spot stress to include the effect of weld toe position, more advanced fitness-for-purpose approaches to fracture control, and the addition of fatigue provisions for box connections.

A number of unique failure modes are possible in tubular connections. In addition to the usual checks on weld stress provided for in most design codes, the designer should check for the following (listed together with the relevant AWS Code sections):

- Local failure *
- General collapse
- Progressive failure (unzipping)
- Materials problems

* Overlapping connections are covered by 10.5.1.5 and 10.5.2.5

This paper will briefly comment on the basis and use of these criteria. Reference 1 covers the topic in more detail.

1. Circular Sections

Local Failure... AWS design criteria for this failure mode have traditionally been formulated in terms of punching shear. In addition, the 1992 edition of the AWS Code will include tubular connection design criteria in ultimate strength format. This was derived from, and intended to be
equivalent to, the earlier punching shear criteria. The thin-wall assumption was made (i.e. no t/d correction), and the conversion for bending uses elastic section modulus. Thus, the allowable punching shear \( V_p \) (including a safety factor of 1.8) is given by...

\[
F_{yo, \text{allow}} V_p = \frac{Qq \times Qf}{0.6 \times \gamma}
\]  

[1]

...where terminology is defined at the end of the paper.

Figure 1 (ref. 1) indicates a safety index of 3.6 for working stress design \( V_p \), appropriate for selection of the joint-can as a member (safety index is the safety margin of the design criteria, including hidden bias, expressed in standard deviations of total uncertainty). Since these failure criteria are used to select the main member or chord, the choice of safety index is comparable to that used for designing other structural members -- rather than the higher values often cited for connection material such as rivets, bolts, or fillet welds, which raise additional reliability issues, e.g. local ductility and workmanship.

The corresponding ultimate axial load is \( P \) given by...

\[
P = \frac{2}{\sin(\theta)} \times T \times F_{yo} \times (6\pi\beta) \times Qq \times Qf
\]  

[2]

When used in the context of AISC-LRFD, with a resistance factor of 0.8, this ultimate strength is nominally equivalent with the punching shear allowable stress design (ASD) for structures having 40% dead load and 60% service loads. LRFD falls on the safe side of ASD for structures having a lower proportion of dead load. AISC criteria for tension and compression members appear to make the equivalency trade-off at about 25% dead load; thus, the LRFD criteria given by AWS would appear to be more conservative for a larger part of the population of structures. However, some of this difference is nullified, since the t/d correction to punching shear was omitted for simplicity...

acting \( V_p = (\tau) \times \sin(\theta) \times f_n \times (1 - t/d) \)  

[3]

For further comparison, the ASCE Committee on Tubular Structures in reference 2 derived a resistance factor of 0.81 for similar Yura-based tubular connection design criteria, targeting a safety index of 3.0.

For offshore structures, typically dominated by environmental loading which occurs when they are unmanned, the 1986 draft of API RP2A-LRFD proposed more liberal resistance factors of 0.90 to 0.95, corresponding to a reduced target safety index of 2.5 (actually, as low as 2.1 for tension members). API also increased their allowable stress design criteria to reflect the benefit of typical t/d ratios.

In Canada (ref. 3), using these resistance factors with slightly different load factors, a 4.2% difference in overall safety factor results. This is within calibration accuracy.

General Collapse... In addition to the failure of the main member which occurs locally in the vicinity of the welded-on branch member, a more
widespread mode of general collapse may occur, e.g. by general ovalizing plastic failure in the cylindrical shell of the main member. To a large extent, this is now covered by strength criteria which are specialized by connection type, as discussed below under the heading of multiplanar connections.

Progressive Failure (Unzipping)... The initial elastic distribution of load transfer across the weld in a tubular connection is highly non-uniform, with peak line load (kips/inch) often being a factor of two or three higher than that indicated on the basis of nominal sections, geometry, and statics, as per AWS section 10.8. Some local yielding is required for tubular connections to re-distribute this and reach their design capacity. If the weld is a weak link in the system, it may "unzip" before this re-distribution can happen. The criteria given in the Code are intended to prevent this unzipping, taking advantage of the higher safety factors in weld allowable stresses than elsewhere. For example, the line load ultimate strength of an 0.7t fillet weld made with E70XX electrodes is 0.7t(2.67x0.3x70) = 39t (ksi), adequate to match the yield strength of mild steel branch material. IIW rules, and LRFD-based strength calculations, suggest larger matching weld sizes are required, e.g. 1.0t or 1.2t (1.07t in the draft Eurocode).

2. Multi-Planar Connections

For design purposes, tubular connections are often classified according to their configuration — for example T, Y, K, X and N-connections and the other "alphabet" joints (so-called because letters of the alphabet are used to evoke their configuration). Different formulas for strength design and fatigue stress concentration factor are applied for each different type. The research, testing, and analysis leading to these increasingly sophisticated criteria have for the most part dealt only with connections having their members in a single plane.

However, many tubular space frames, including most of those used in offshore structures, have bracing in multiple planes. For some loading conditions, these different planes interact, and when they do, criteria for the alphabet joints are no longer satisfactory. When one considers the number of possibilities, it seems unlikely that parametric design formulas will ever be developed to cover each and every one. Hence the attraction of approximate but general criteria which capture the major features of these multi-planar interactions, even if they must be less precise than criteria for the much-studied uni-planar joints.

Ovalizing Parameter... Figure 2 presents the author's 1984 Code formula for computing the ovalizing parameter (alpha), in a way which recognizes that loading pattern, rather than just geometrical configuration, is important to the behavior of tubular connections. Alpha is evaluated separately for each brace for which connection capacity shear is to be checked (the "reference brace"), and for each load case, with the summation being taken over all braces present at the node for each load case. In the summation, the circumferential cosine term and the axial exponential decay term express the influence of braces on ovalizing stress at the reference position. This simple, but repetitive calculation is suitable for a tubular connection design post-processor for a computerized structural analysis of the space frame, and avoids the necessity of arbi-
trarily assigning one of the alphabet classifications. Indeed, designers
often mis-classify connections based on their geometry, rather than load
pattern.

The ovalizing parameter \( \alpha \) is used in the \( \frac{Q_q}{\alpha \beta} \) expression which appears in strength design equations [1] and [2] as follows...

\[
1.7 \quad 0.18 \quad 0.7(\alpha - 1)
\]

Using the expression in Fig. 2 for computed \( \alpha \) gives the following
results for the classical "alphabet" types of planar connections:

<table>
<thead>
<tr>
<th>TYPE OF JOINT</th>
<th>VALUE OF COMPUTED ALPHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>balanced K-joint</td>
<td>1.0 to 1.4 (depends on footprint spacing)</td>
</tr>
<tr>
<td>T &amp; Y joints</td>
<td>1.7</td>
</tr>
<tr>
<td>cross joints</td>
<td>2.0 to 2.4 (spacing depends on angle)</td>
</tr>
</tbody>
</table>

Although the acting punching shear computed according to the rules for
strength design reflects the overall statics and geometry of the connection, a better reflection of the relative importance of various branch
member loads causing fatigue damage is given by the AWS expression for
cyclic punching shear, which also includes the ovalizing parameter \( \alpha \), as described in ref. 4...

\[
cyclic V_p = \text{tau*sin(\theta)} * (\alpha f_a + \text{SRSS}(0.67 f_{by}, 1.5 f_{bz}))
\]

Japanese Data... Shortly after the the AWS multiplanar criteria were
formulated, a series of 20 ultimate strength tests were conducted at
Kumamoto University, Japan (ref. 5). The configuration has K-connections
in two planes 60-degrees apart, as would be found in the single bottom
chord of an inverted delta truss.

AWS allowable capacity for these connections was computed, and the test
results plotted as a histogram in Figure 3. The observed capacities are
clustered on the conservative side of the AWS nominal safety factor of 1.8.
Indeed, the correlation is tighter than for the uni-planar connections of
Fig. 1. Some of this conservatism comes from invoking the rule that effective
\( f_{yo} \) should not exceed two-thirds the tensile strength (footnote 2 of
AWS Table 10.2).

Somewhat less satisfying are the comparisons of each multi-planar connection versus its uni-planar counterpart. An unusual failure mode was ob-
served in the transverse gap region, as shown in Fig. 4(a). The strength
of multi-planar connections was found to average 92.3% of the calculated
strength (according to previous correlations) of the corresponding uni-
planar connections. The circumferential influence term in the AWS criteria,
on the other hand, predicts that ovalizing is suppressed by this pattern
of loading, so that \( \alpha \) approaches its minimum value of 1.0, especially
for a large transverse gap which places the brace footprints about 90-deg-
rees apart. As the longitudinal and transverse gaps tended to increase
together in these experiments, and uni-planar K-connections are penalized
by larger values of \( \alpha \) for larger gaps, the Code formula's benefit of
suppressing the ovalizing increases with gap, contrary to the trend of
the observations, as shown in Fig. 4(b).
Inelastic Finite Element... Paul (ref. 6) describes the inelastic finite element analysis of a multi-planar "hub" connection. The finite element technique and mesh density, were representative of typical practice, except that the steel had UTS/YS ratio of 1.62 (reflecting specification values for mild steel). For calibration, two uni-planar, double T-connections were also analyzed, with the results agreeing closely with Kurobane's empirical best fit.

Ultimate loads correspond to plastic collapse of the chord, with local strains of about 15%. These are compared to the AWS nominal ultimate (1.8 x static allowable) in Fig. 5. As compared to design rules which ignore multi-planar effects, AWS criteria more correctly reflect both the adverse effects of increased ovalizing (alpha = 3.8) and the beneficial effect of suppressed (alpha = 1.0), as well as the greater importance of load pattern as opposed to connection geometry (compare uni-planar DT and hub at alpha of 2.4).

Although the match is not perfect, it is about as good as one gets for other classes of connections. Finite element ultimate loads range from 1.55 to 3.37 times the AWS allowable, which is comparable to the scatter shown in earlier correlations. For the case where ovalizing is suppressed (alpha = 1.0), as for the Japanese double-K experiments, we see a strong effect of the transverse gap, which is not reflected in AWS criteria based on angular footprint spacing.

Clearly, the ultimate strength behavior of a wide variety of multi-planar connections deserves further study. Criteria which explicitly consider the effect of transverse gap would seem to be promising. Meanwhile, despite its present shortcomings, the real value of the AWS alpha criteria lies in preventing gross under-design for the adverse loading patterns not yet covered by testing, such as those shown in Fig. 6. Computed alpha also continues to be useful as a means of automating the classification of connection type based on load pattern.

3. Tubular Design Revisions - Box Sections

In AWS D1.1-90 and earlier editions of the Code, treatment of box sections had been made as consistent as possible with that of circular sections. Derivation of the basic allowable punching shear $V_p$ for box sections included a safety factor of 1.8, based on a simple yield line limit analysis, but utilizing the ultimate tensile strength, which was assumed to be 1.5 times the specified minimum yield. This is why $F_{y0}$ in the design formula for punching shear was limited to $2/3$ times the tensile strength. A favorable redistribution of load was also assumed where appropriate. Localized yielding should be expected to occur within allowable load levels. Fairly general yielding, with connection distortion exceeding 0.02 D, could be expected at loads exceeding 120-160% of the static allowable.

For very large beta (over 0.85) and K-connections with gap approaching zero, yield line analysis indicates extremely high and unrealistic connection capacity. In such cases, other limiting provisions based on material shear failure of the stiffer regions, and reduced capacity for the more flexible regions (a.k.a. effective width) must also be
Although the old AWS criteria covered these considerations, for bending as well as for axial load, more authoritative expressions representing a much larger data base have been developed over the years by CIDECT (Comité International pour le Développement et l’Étude de la Construction Tubulaire, ref. 7) and by members of IIW subcommittee XV-E (ref. 8). These criteria have been adapted for limit state design of steel structures in Canada (Packer et al ref. 3). The Canadian code is similar to the AISC-LRFD format. In the 1992 edition, these updated criteria will be incorporated into the AWS Code, using the thickness-squared ultimate strength format and Packer’s resistance factors, where applicable.

Local failure... Load factors vary from equation to equation to reflect the differing amounts of bias and scatter apparent when these equations are compared to test data (ref. 3). For example, the equation for plastic chord face failure of T-, Y-, and cross connections is based on yield line analysis, ignoring the reserve strength which comes from strain hardening; this bias provides adequate safety factor with a resistance factor of unity. Another equation, for gap K- and N-connections, was empirically derived, has less hidden bias on the safe side, and draws a lower resistance factor.

General Collapse... To avoid a somewhat awkward adaptation of column buckling allowables to the box section web crippling problem, AISC-LRFD web yielding, crippling, and transverse buckling criteria have been adapted to tension, one-sided, and two-sided load cases, respectively. The resistance factors given are those of AISC. Packer (ref. 9) indicates a reasonably good correlation with available box connection test results, mostly of the two-sided variety.

Uneven Distribution of Load... For box sections, this problem is now treated in terms of effective width concepts, in which load delivery to more flexible portions of the chord is ignored. Criteria for branch member checks are based empirically on IIW/CIDECT work. Criteria for load calculation in welds are based upon the testing of Packer (ref. 10) for gap K- and N-connections; and upon extrapolation and simplification of the IIW effective width concepts for T-, Y-, and cross connections.

Overlapping Connections... By providing direct transfer of load from one branch member to the other in K- and N-connections, overlapping joints reduce the punching demands on the main member, permitting the use of thinner chord members in trusses. These are particularly advantageous in box sections, in that the member end preparations are not as complex as for overlapping circular tubes.

Fully overlapped connections, in which the overlapping brace is welded entirely to the thru brace with no chord contact whatsoever, have the advantage of even simpler end preparations. However, the punching problem that was in the chord for gap connections, is now transferred to the thru brace, which also has high beam shear and bending loads in carrying these loads to the chord.

Most of the testing of overlapped connections has been for perfectly balanced load cases, in which the compressive transverse load of one branch is offset by the tension load of the other. In such overlapped connections, subjected to balanced and predominantly axial static loading, tests
have shown that it is not necessary to complete the "hidden" weld at the
toe of the thru member. In real world design situations, however, local­
ized chord shear loading or purlin loads delivered to the panel points of
a truss result in unbalanced loads. In these unbalanced situations, the
most heavily loaded member should be the thru brace, with its full circum­
ference welded to the chord, and additional checks of net load on the
combined footprint of all braces are required.

Bending ... Since international criteria for bending capacity of tubular
connections are not as well developed as for axial loads, the effects of
primary bending moments are approximated as an additional axial load. In
the design expression...

$$M_{\text{additional}} = \frac{-P \cdot JD \cdot \sin(\theta)}{2}$$

...JD represents half the moment arm between stress blocks creating the
moment, analogous to concrete design -- half, because only half the axial
capacity lies on each side of the neutral axis. Various ultimate limit
states are used in deriving the expressions for JD in Table I. For chord
face plastification, a uniform punching shear or line load capacity is
assumed. For the material shear strength limit, the effective width is
used. General collapse reflects a side wall failure mechanism. Finally,
a simplified expression for JD is given, which may conservatively be used
for any of the governing failure modes.

Caution should be exercised where deflections due to joint rotations could
be important, e.g. sidesway of portal frames in architectural applications.
Previous editions of the Code provided a $\frac{1}{3}$ DECREASE in allowable connec­
tion capacity for this situation.

Other Configurations... The equivalence of box and circular branch mem­
bers on box chords is based on their respective perimeters. In effect,
CIDECT and IIW have applied the concept of punching shear to the problem,
even though these international criteria are always given in ultimate
strength format. The results are on the safe side of available test
results.

4. Fracture Control

This section describes an integrated approach to fracture control problems
-- applied first as a necessary adjunct to the author's own design work,
then in developing the US national Codes for tubular structures, AWS DI.1
and API RP2A.

A well engineered structure requires that a number of factors be in reason­
able balance. Factors to be considered in relation to economics and risk
in the design and steel selection for tubular connections include: (1)
static strength, (2) notch toughness, (3) fatigue resistance, (4) homogen­
eity and resistance to lamellar tearing, (5) weldability. Many of these
same factors arise again in setting up QC/QA programs during construction,
including such issues as weld profile control and allowable flaw size to
be applied during nondestructive testing. Human factors and organiza­
tional issues must also be addressed, such as personnel qualifications,
Most fracture control problems in offshore platforms occur in the tubular connections, or nodes. Use of the "hot spot" stress, as would be measured by a perpendicular strain gage adjacent to the toe of the weld in the region of localized plastic deformation, serves to bring fatigue and fracture problems for many different node geometries into a common focus.

In the chord...

\[
\text{hot spot stress} = 1.8 \times \text{cyclic } V_p \times \sqrt{\gamma}\]  

[7]

Use of elastic hot spot stress as an indicator of ultimate strength is only approximate. Local yielding, mobilization of plastic section modulus, triaxial stresses, and load redistribution via plastic deformation occur as tubular connections reach their practical ultimate capacity. These phenomena, which occur in the presence of weld-toe notches, place severe demands on the materials being used, particularly in the chord or main member of a tubular connection. Although elasto-plastic analysis methods are now becoming more widely available, design practice typically uses well-calibrated empirical formulas for ultimate strength.

Static strength has already been discussed in the preceding sections. Until this is properly taken care of, the other factors are largely academic. Indeed, early overload failures of tubular connections were often misdiagnosed as something more exotic.

Notch Toughness... For Gulf of Mexico offshore structures, the lowest anticipated service temperatures are 14 degrees-F air, 40-50 degrees-F water. Typical steel for "joint cane" (heavy wall chord section) is API Spec 2H, which calls for drop-weight nil-ductility transition at minus 40 degrees (ref.11). This is referred to as class "A" fracture toughness, and is used in fracture critical locations where the adverse conditions of thick sections, high restraint, plastic deformation, high stress concentration, and a lack of structural redundancy all co-exist. Thinner brace stub ends (at nodes) and fracture critical members (away from stress concentrations) may have a nominal Charpy requirement at service temperature -- class "B". Redundant members away from nodes still use ordinary mild steel -- class "C" -- and rely on a history of satisfactory service experience rather than specific toughness testing provisions. Toughness classifications "A", "B", and "C" may be used to cover various degrees of criticality as shown in the matrix of Figure 7.

Weld metal toughness... Weld toughness provisions of API RP 2A (since 1989) and AWS D1.1 (coming out in 1992) are quite similar in approach. When Charpy V-notch testing is required as part of welding procedure qualification, specimens would be removed from the test weld and impact tested in accordance with Appendix III, Requirements for Impact Testing, of AWS D1.1. The following test temperatures and minimum energy values are recommended, for matching the performance of the various steel grades as listed in API Tables 2.9.1 and 2.9.2. (AWS will include similar tables).

<table>
<thead>
<tr>
<th>YIELD KSI</th>
<th>STEEL CLASS</th>
<th>IMPACT TEST TEMPERATURE</th>
<th>WELD METAL AVG. FT-LBS (JOULES)</th>
</tr>
</thead>
<tbody>
<tr>
<td>up</td>
<td>C</td>
<td>0°F -18°C</td>
<td>20</td>
</tr>
<tr>
<td>to</td>
<td>B</td>
<td>0°F -18°C</td>
<td>20</td>
</tr>
</tbody>
</table>
Charpy impact testing is a method for qualitative assessment of material toughness. Although lacking the technical precision of crack tip opening displacement (CTOD) testing, the method has been and continues to be a reasonable measure of fracture safety, when employed with a definitive program of nondestructive examination to eliminate weld area imperfections. These recommendations are based on practices which have generally provided satisfactory fracture experience in structures located in moderate temperature environments (e.g. 40-deg-F sea water and 14-deg-F air exposure). For environments which are either more or less hostile, impact testing temperatures should be reconsidered, based on local temperature exposures.

For critical welded connections, the technically more exact CTOD test may be appropriate. CTOD tests are run at realistic temperatures and strain rates, representing those of the engineering application, using specimens having the full prototype thickness. This yields quantitative information useful for engineering fracture mechanics analysis and defect assessment, in which the required CTOD is related to anticipated stress levels (including residual stress) and flaw sizes.

Representative CTOD requirements range from 0.004-inch at 40-deg-F (0.10-mm at 4-deg-C) to 0.015-inch at 14-deg-F (0.38-mm at -10-deg-C). Achieving the higher levels of toughness may require some difficult trade-offs against other desirable attributes of the welding process — for example, we might have to forego the deep penetration and relative freedom from trapped slag of uphill weld passes.

Since traditional AWS welding procedure requirements are concerned primarily with tensile strength and soundness (with minor emphasis on fracture toughness) it is appropriate to consider additional essential variables which have an influence on fracture toughness — i.e. specific brand wire/flux combinations, and the restriction of SAW consumables to the limits actually tested for AWS classification. Note that, for Class "A" steels, specified energy levels higher than the AWS classifications will require that all welding procedures be qualified by test, rather than having prequalified status.

Heat affected zone toughness... In addition to weld metal toughness, consideration should be given to controlling the properties of the heat affected zone (HAZ). Although the heat cycle of welding sometimes improves base metals of low toughness, this region will more often have degraded properties. A number of early failures in welded tubular joints involved fractures which either initiated in or propagated through the HAZ, often before significant fatigue loading.

AWS D1.1 Appendix III gives requirements for sampling both weld metal and HAZ, with Charpy energy and temperature to be specified in contract documents. The following average HAZ values have been found by experience to be reasonably attainable:
As criticality of the component's performance increases, lower testing temperatures (implying more restrictive welding procedures) would provide HAZ's which more closely match the performance of the adjoining weld metal and parent material, rather than being a potential weak link in the system. The owner may also wish to consider more extensive sampling of the HAZ than the single set of Charpy tests required by AWS, e.g. sampling at 0.4-mm, 2-mm, and 5-mm from the fusion line. More extensive sampling increases the likelihood of finding local brittle zones with low toughness values.

Since HAZ toughness is as much dependent on the steel as on the welding parameters, a preferable alternative for addressing this issue is through weldability prequalification of the steel. API RP 2Z spells out such a prequalification procedure, using CTOD as well as Charpy testing. This prequalification testing is presently being applied as a supplementary requirement for high-performance steels such as API Specs 2W and 2Y, and is accepted as a requirement by a few producers.

5. Fatigue size and profile effects

There are several levels of analysis which may be considered for offshore towers and similar tubular space frames. Small regions receiving more detailed levels of analysis may be treated as substructures of the larger models in which they are imbedded. At the global level, we apply the design loads, e.g. wave forces, to a space frame model of the entire structure, and compute nominal member stresses of say 20-ksi. At the second level of analysis, a shell element model of tubular connections may be used to derive geometric hot spot stresses of say 60-ksi. Thick isoparametric elements have some advantages for this application, modeling the gross weld geometry, and giving hot spot stress results conforming to either European or American definitions -- linear trend or nearest integration point (GPSS), respectively. This approach has been used to derive improved parametric formulas for stress concentration factors (ref.12).

The third level of analysis has been used mostly for research, and for development of the weld profile and fatigue size effect provisions of the AWS Code; designers following these rules need only go to the second level. The third level involves a very fine mesh of the weld and weld toe region, used to derive local notch stress or stress intensity solutions. Both methods show (1) reduced fatigue performance in the presence of more severe notch geometries, and (2) for a given notch geometry, reduced fatigue
performance when the size is scaled up. Figure 8 shows both these effects, where fatigue performance is plotted in terms of second level geometric hot spot stress, as the designer would normally use. Disturbingly high notch stresses and low fatigue results are predicted for the root of single-sided Detail "D" welds in stiffened tubular connections (ref.13).

Following these trends, the AWS Code defines two levels of fatigue performance. It also defines several quality levels for weld profiles, as shown in Figure 9. The upper performance level (fatigue curves XI and KI) is more restrictive in its thickness limits for the various profiles. Where these limits are exceeded, fatigue strength is reduced as the minus 0.25 power of thickness. Figure 10 shows experimental confirmation of the AWS rules, from testing in seawater by Prof. W. H. Hartt at Florida Atlantic University (ref. 14).

In the years following the Cambridge and Paris conferences (refs. 15 and 16) much of ongoing the research on fatigue of tubular joints was centered in Europe and Japan. The American effort in which the author participated was primarily to digest this research and reduce it to practice. An interim report on this effort was given in the 1984 Houdremont Lecture (ref. 17) with the final results appearing in the 1986 AWS Structural Welding Code. Subsequent work (refs. 13 and 14) has confirmed the results first presented in Paris ten years ago, that BOTH fatigue size and profile effects were important. Unfortunately, some people seem to be stuck in positions they took 12 years ago, either seeing only profile effects (e.g. API RP 2A) or only size effects, e.g. the "blinders" method of plotting data, which makes Fig. 8(a) look like Fig. 8(b).

When the design analysis stops at the geometric hot spot stress, both European and American Codes have found it necessary to impose plate thickness or weld size/profile corrections to their design S-N curves. The need for such arbitrary and elaborate corrections may be taken as a symptom that it is time to search for a more fundamental design basis, which accounts for more of the problem during analysis and requires fewer corrections. There are several possible levels of analysis beyond hot spot stress which may be contemplated, as described in ref. 18.

AWS design criteria, along with the recent size and profile considerations, have been described. Test data have been presented which confirm these criteria for thicknesses up to 4-in (100-mm), with a variety of weld profiles and service environments. Hot spot stress which is an invariant function of connection geometry remains as the basis of today's design practice. Alternative design methods which go beyond hot spot stress deserve further attention.

6. Acknowledgements

The author is grateful to Shell Oil Company (with whom he is also still employed) for supporting this work over the years, and to the many AWS, API, and IIW committee members who stimulated and collaborated on these Code developments.
7. Nomenclature

- **D**: main member (chord) diameter
- **d**: branch diameter
- **fa**: axial stress in branch
- **fby**: branch in plane bending stress
- **fbz**: branch out of plane bending stress
- **fn**: nominal stress in branch, fa or fb, treated separately
- **JD**: half moment arm (see table I)
- **Fyo**: yield strength of main member (chord) at a tubular connection
- **M**: moment in branch member
- **P**: axial load in branch member
- \[ Q_{\beta} = \frac{0.3}{\beta(1-0.833\beta)} \] for \( \beta > 0.6 \), 1.0 for \( \beta > 0.6 \)
- **Qf**: derating factor for punching shear in presence of chord self stress
- **Qq**: factor for geometry and load pattern, see eqn. [4]
- **SRSS**: square root of the sum of the squares
- **T**: main member thickness
- **t**: branch thickness
- **Vp**: punching shear
- **alpha**: ovalizing parameter
- **beta**: ratio branch/chord diameters or widths
- **Beop**: dimensionless effective width for punching shear
- **Beoi**: dimensionless effective width for branch yielding
- **eta**: ratio footprint length/ chord width
- **gamma**: chord radius/thickness
- **tau**: ratio branch/chord thicknesses
- **theta**: angle between member axes
8. References


Table I. Values of JD

<table>
<thead>
<tr>
<th>Governing Failure Mode</th>
<th>In-Plane Bending</th>
<th>Out-of-Plane Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic Chord Wall Failure</td>
<td>$\frac{\eta D (\beta + \eta/2)}{2 (\beta + \eta)}$</td>
<td>$\frac{\beta D (\eta + \beta/2)}{2 (\eta + \beta)}$</td>
</tr>
<tr>
<td>Chord Material Shear Strength</td>
<td>$\frac{\eta D (\beta_{eop} + \eta/2)}{2 (\beta_{eop} + \eta)}$</td>
<td>$\frac{\beta D (\beta_{eop} + (1-\beta_{eop}/2\beta))}{2 (\eta + \beta_{eop})}$</td>
</tr>
<tr>
<td>General Collapse</td>
<td>$\frac{\eta D + 5\sigma_c}{4}$</td>
<td>$D$</td>
</tr>
<tr>
<td>Branch Member Effective Width</td>
<td>$\frac{\eta D (\beta_{eol} = \eta/2)}{4}$</td>
<td>$\frac{\beta D (\eta + \beta_{eol} - (1-\beta_{eol}/2\beta))}{2 (\eta + \beta_{eol})}$</td>
</tr>
<tr>
<td>Conservative Approximation For Any Mode</td>
<td>$\frac{\eta D}{4}$</td>
<td>$\frac{\eta D}{4}$</td>
</tr>
</tbody>
</table>

Figure 1. Correlation of AWS-84 criteria with WRC data base.
Figure 2. Computed Alpha.
(a) Equation.
(b) Definitions.

Figure 3. Comparison of AWS multi-planar criteria with Kumamoto test results.

Figure 4. (a) Typical failure mode at compression braces
(b) Ultimate strength of multi-planar joint compared to that of planar joint.
Figure 5. Interaction plots of ultimate strength for multi-planar hub connection, comparing AWS nominal ultimate (1.8 x allowable) with finite element method. (a) beta = 0.4. (b) beta = 0.6.

Figure 6. Adverse load patterns with alpha up to 3.8. (a) False leg termination. (b) Skirt pile bracing. (c) Hub connection.

Figure 7. Classification matrix for notch toughness applications.
Figure 8. Size and profile effects as given by notch stress theory. (a) Comprehensive format. (b) European size-only format.

Figure 9. AWS weld profile requirements and corresponding branch thickness limitations.
Figure 10. Invariant hotspot fatigue strength in comprehensive size effect format. (top) Welds meeting AWS level I profile requirements. (bot) Welds per AWS basic minimum requirements.
THE FIRE DESIGN OF SHS STRUCTURES USING BS 5950 PART 8

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SUMMARY

This paper discusses the history and contents of the new British Standard BS 5950 Part 8: 1990, "Structural use of steel in building, Part 8. Code of practice for fire resistant design". The standard has been written in sympathy with draft Eurocodes and is based on rational fire design principles, confirmed by fire tests. It also permits the use of fire engineering for external steelwork.

The paper demonstrates how the standard can be applied to the fire resistant design of externally protected Structural Hollow Section (SHS) members. In addition, it highlights how the standard enables the use of unique methods of SHS construction, e.g. concrete and water filled columns.

1. INTRODUCTION

National Building Regulations were introduced into the UK during 1964 (Scotland), 1965 (London) & 1966 (England & Wales). They were intended to provide a way to classify any building and then specify both the fire resistance period and the means of protection. In the main, protection was to be provided by using building materials (brickwork, concrete, plasterboard, etc). Sensibly, provision was also made for the relaxation of the regulations in non standard cases.

In 1985, Clauses specifying protection were removed from these regulations in anticipation of the publication of Part 8 of British Standard 5950 "Structural use of steel in building, Part 8. Code of practice for fire resistant design" [1], in June 1990.

The new Part 8 encompasses almost all the major developments in fire design, building construction and protective product manufacture that have occurred in the last 25 years. It is a limit state code written to complement BS 5950 Part 1: 1986 "Structural use of steel in building, Part 1. Code of practice for design in simple and continuous construction: hot rolled sections". A detailed explanatory handbook has also been published by the Steel Construction Institute [2].

It was written by a team working in full awareness of the contents of the contemporary drafts of Eurocodes EC3 (steel) and EC4 (composite construction). Often, this has produced areas of similar content and procedure. As a result, Part 8 will ease the introduction of the Eurocodes into the U.K. and
the acceptance of the broader fire design philosophies and procedures they contain.

BS 5950 Part 8 is a significant document on two main counts:-

1) It contains both general and specialised design and assessment procedures, for example:-

**generalised**:
- limiting criteria for unprotected steelwork.
- estimation of Limiting Temperatures by fire design
- estimation of protection thickness requirements from Limiting Temperatures and protection data.
- specifying external protection for unique sections (i.e. castellated and hollow sections)
- evaluating protection performance data (Appendix D)

**specialised**:
- stability of portal frame columns (Section 4.5)
- concrete filled SHS columns (Section 4.6)
- composite beams (Section 4.9.4)

2) It is also an **enabling** document since it acknowledges the existence and permits the use of additional design approaches and construction types not covered in full in the Standard.

e.g. - water filled structures (Section 4.7)
- external bare steel assessed by fire engineering (Section 4.8)

In general, Part 8 fire design procedures can be used to assess the performance of elements of structure when heated to the ISO temperature-time curve and all have been confirmed by furnace tests.

Topics applicable to Structural Hollow Sections (SHS) are discussed in more detail, below.

2. FIRE DESIGN OF BARE AND EXTERNALLY PROTECTED SHS STEEL MEMBERS USING PASSIVE FIRE PROTECTION MATERIALS

(Sections 4.3.3 and 4.4 of the Standard)

The standard and the SCI handbook, when taken together, provide and justify a rational step by step design procedure that can be applied specifically to SHS members as follows:-

**STEP 1**: Calculate Design Loads for the fire limit state

e.g. Dead Load + Live Load + 1/3 . Wind Load

**Note**: additional fire limit state load types may need to be included. A full range of load factors is tabulated in the Standard.
STEP 2: Calculate the Load Ratio (R) using the relevant interaction formula from Section 4.4.2 according to member type and loading.

e.g. for a column in simple construction:

\[ R = \frac{F_f}{A_g \cdot P_c} + \frac{M_{fx}}{M_b} + \frac{M_{fy}}{P_y \cdot Z_y} \]

For any of the given equations;

If the calculated value of R is less than or equal to 0.6 and the required fire resistance time is 30 minutes, then proceed as follows;

i) Calculate the Section Factor \( (H_p/A) \) for the unprotected steel, where:

\[
UNPROTECTED \text{ Section Factor} = \frac{\text{Heated Perimeter of Steel}}{\text{Cross-sectional Area of Steel}}
\]

ii) Check that Section Factor < 50 or 90 m\(^{-1}\) according to the usage specified in the Standard. (i.e. column or beam)

If so, USE UNPROTECTED STEEL

If R > 0.6 or required resistance time > 30 minutes, then:

STEP 3: Find Limiting Temperature of the steel member (using the tabulated values reproduced in Table 1).

STEP 4: EITHER a) Calculate Section Factor, using Section 4.3.4 and Table 3 of the Standard.

For a PROTECTED section;

\[
\text{Section Factor} = \frac{\text{Heated inside Perimeter of Protection}}{\text{Cross-sectional Area of Steel}}
\]

OR b) Obtain Section Factor from published tables, e.g. the ASFP/SCI specifiers' guide 'Fire protection to structural steel in buildings'. [3]

STEP 5: Estimate the required thickness of passive protection using available product data, as follows;
EITHER a) Calculate required thickness using the procedure given in Appendix D of the Standard.

Note: to do so, the designer will need detailed thermal information about the passive product being assessed (see Section 3 of this paper)

OR b) Find the STANDARDISED thickness specified for a product tabulated in [3] (based on a steel temperature of 550°C). Next, modify this thickness as necessary, to allow for the actual limiting temperature, using the procedure explained in Section 3, below.

STEP 6: If needed, transpose protection thickness to allow for section shape and protection type, using the procedure given in Section 4.3.3.4 of the standard and explained more fully below.

3. PRACTICAL USE OF THE ASFPFCM/SCI PRODUCT TABLES (Ref. 3)

The UK Association of Fire Protection Contractors and Manufacturers (ASFPFCM) have collaborated with the SCI and the established UK test houses to produce a handbook [3] covering almost all the fire protection products readily available in the UK and prescribing their use with structural steelwork.

The required thicknesses for PASSIVE products are given in a tabular format and are based on a steel temperature of 550°C. They have been mainly derived from tests done using 1 Sections (a typical example is shown in Table 2).

The estimation formulae and associated Table 17 given in Appendix D of BS 5950: Part 8 are generalised for application across a range of steel temperatures. They require knowledge of the physical parameters of each protection system, such as density, percentage water content and effective conductivity over a range of fire times (derived from its thermal history). Obtaining this information will require the co-operation of the product manufacturer or distributor.

However, Section 8.3 of the SCI handbook gives an explanation of how Appendix D was constructed and shows that the procedure is based on published ECCS Recommendations [4]. These recommendations state that a passive product's performance can be quantified by using the expression:

$$ t_e = 40 \times (\Theta_s - 140) \times \left[ \frac{C \cdot d_1}{k_i \cdot (H_p/A)} \right]^{0.77} \quad (1) $$

provided that: $400°C < \Theta_s < 6000°C$

and $d_1$, $k_i$ and $H_p/A$ are in compatible units.
C is a modifier that allows for the fact that the protection also contributes to the total heat sink, i.e.:

\[ C = 1 + \frac{\frac{\rho_{\text{m}} \cdot C_1}{2 \cdot \rho_{\text{s}} \cdot C_6}}{1 + K \cdot d_1 \cdot (H_p/A)} = 1 + K \cdot d_1 \cdot (H_p/A). \quad (2) \]

These formulae formed the basis for drawing up the table (and its modifying equations) in Appendix D of Part 8. They can also be used to quantify the effect of altering the limiting temperature, as follows:

3.1 To Allow for an altered Limiting Temperature (Ref. 2; p. 28)

For equal fire resistance periods, the relative protection thicknesses can be expressed as:

\[ d_2 = d_1 \cdot \left( \frac{1 + K \cdot d_1 \cdot (H_p/A)}{1 + K \cdot d_2 \cdot (H_p/A)} \right) \cdot \frac{\Theta_1 - 140}{\Theta_2 - 140} \cdot 1.3 \quad (3) \]

where:
- \( \Theta_1 \) is a limiting temperature,
- \( d_1 \) is the thickness of protection needed to keep a steel member below \( \Theta_1 \) over a given fire time,
- \( \Theta_2 \) is a different limiting temperature,
- \( d_2 \) is the new thickness of protection needed to keep the member below \( \Theta_2 \) for the same time.

This equation can be quantified for a known structural section and given product, i.e.:

a) set \( \Theta_1 = 550^\circ \text{C} \); then read required protection thickness \( d_{550} \) for the known value of \( H_p/A \) and required fire life using the given product table in [3]; set \( d_1 = d_{550} \)

b) \( K \) can then be assessed using the product densities also given in [3]. Using the specific heat values cited in [4], \( K \) can conservatively be taken for spray and board products as varying linearly from 0.0367 to 0.1470 as product density varies from 250 to 1000 kg/mm².

Equation 3 can now be solved explicitly.

3.2 A Simple Estimate

In practice the relative change produced to the heat sink by altering the protection thickness only becomes significant when moving from one fire class to another and it can be initially ignored.

i.e., if \( \Theta_1 = 550^\circ \text{C} \), then:
\[ d_1 = d_{550} \cdot \left( \frac{410}{\Theta_2 - 140} \right)^{1.30} \]  

\text{NOTES:}  
1) If \( \Theta_2 < 550^\circ \text{C} \) then the simply adjusted thickness will always be conservative.  
2) Common Sense should be maintained when thicknesses are reduced! [if in doubt, calculate and compare heat sink modifiers; then, if necessary solve equation (3) explicitly]  
3) Physical limits of the protection system must still be maintained (i.e. minimum and maximum unreinforced thicknesses).  
4) Product suitability should still be verified by discussion with manufacturer.

3.3 Transposing Insulation Thicknesses Data for use with SHS

The transposition procedures given in the Standard (Section 4.3.3.4) are the result of a series of test campaigns mounted by British Steel [5], [6] to quantify the relative performance of protected SHS compared to I-Sections of similar serial size and Section Factor.

i) Board protection: NO TRANSPOSITION NECESSARY

Comparative testing confirmed that section thermal behaviour is related to the perimeter of the boxed protection, NOT the shape of the steel section within, i.e. when boxed, both section types perform identically [5].

ii) Passive spray protection: Transpose as follows:

for \( H_p/A < 250 \):

\[ \text{thickness} = d_1 \cdot \left[ 1 + \frac{(H_p/A)}{1000} \right] \]

for \( H_p/A = \text{or} > 250 \):

\[ \text{thickness} = 1.25 \cdot d_1 \]

Comparative testing showed that an SHS coated with a profile following spray protection has all four faces exposed to fire with equal severity. However, the severity of exposure of an I-Section varies from face to face, due to its re-entrant shape. These simplified transposition formulae were empirically derived from three test campaigns covering cementitious and felting sprayed materials. When applied to the original and additional independent test data, the resulting SHS performance became statistically indistinguishable from that of equivalent I-Sections [6].

4. INTUMESCENT PAINTS

The use of intumescent paints is allowed by the standard. However, there is no set procedure for specifying their use. Appendix D procedures cannot be used. This is not surprising since such paints are not standard or semi-generic products. Each must be judged on its own merits.
5. CONCRETE FILLED COLUMNS (Section 4.6 of the Standard)

5.1 Unprotected Columns (Section 4.6.2)

The design procedure and associated formulae given in Part 8 for unprotected rectangular and square columns is identical to that published by British Steel in the Design Manual for Concrete Filled Columns [7]. However, nomenclature and presentation have been modified to ensure consistency with earlier published parts of BS 5950. The procedure is semi empirical. It is based on the test performances of 110 square and circular columns tested in European furnaces in a series of CIDECT programmes (15 A/B/C) [8],[9] and independent U.K. tests. These covered a wide range of SHS sizes, loading conditions and types of concrete (i.e. plain, fibre reinforced and re-bar reinforced). The basic design procedure requires the engineer to:

a) calculate the squash load of the column core at room temperature [A set of these are already tabulated in the BSC Manual [7] for a range of construction specifications].
b) calculate the column core slenderness at room temperature. In Part 8 this is done using an effective radius of gyration (compared to [7], where an Euler length is used).
c) obtain a column core buckling factor by interpolation using a table of buckling factors (based on the identical buckling curve shown as a graph in the BSC Manual).
d) calculate the column buckling strength.

Axial fire design load

\[
\frac{\text{Axial fire design load}}{\text{Column core buckling strength}} < \eta \quad (5)
\]

where \( \eta \) is a lower bound time dependent load ratio that varies according to the desired fire life and infill type; in [7] it is called the design parameter for fire behaviour. Note: in Part 8, there are minor changes in some tabulated values of \( \eta \) compared to those given in [7].

If moments are present, the axial fire design load is increased by use of a modifier. The procedure for calculating this modifier varies according to concrete infill type (with or without re-bar). However, in both cases it is based on a linear interaction formula. In the case where re-bar is used this will involve the calculation of the room temperature ultimate moment of the concrete core. A range of precalculated ultimate moments are tabulated in [7].

Unlike the Manual, the Standard gives no advice on design detailing, such as re-bar cover.

5.2 Externally Protected Columns (Section 4.6.3)

When concrete filling alone will not produce an adequate fire resistance time, external protection must be provided. An in-
ial protection thickness is specified by reference to the Section Factor of the empty SHS. This thickness may then be reduced to take account of the additional heat sink provided by the concrete core, again, using a reduction factor related to the empty SHS Section Factor. This is done using a table that is quantitatively identical to a graph previously published in [7].

In principle, the function of the Standard will be to 'authorize' the Manual, since designers will still be able to use the squash load and ultimate moment tables in the Manual for preferred construction detailing.

6. WATER FILLED COLUMNS (Section 4.7 of the Standard)

Fire protection by water filling is a well established concept. In a fire, such columns will rarely reach temperatures in excess of 250°C. A detailed design method has been available in the UK since 1975 for interconnected, replenished columns [10] based on the ISO temperature-time curve. Since then, much further research on interconnected, isolated and unreplenished columns has taken place in continental Europe.

However, their use in the UK has been inhibited by their unique nature. An early decision must be taken by the engineer to use them, without any surety that the local response will be positive when applying for a local relaxation under the building regulations.

Hopefully, Part 8 will encourage their introduction by acknowledging their existence and permitting their use.

7. UNPROTECTED STEELWORK EXTERNAL TO A BUILDING (USING FIRE ENGINEERING CONCEPTS) (Section 4.8)

Fire engineering, in toto, lies outside the scope of Part 8. However, its use is specifically permitted for estimating the performance of unprotected steelwork external to a building (Section 4.8).

Fire engineering procedures attempt to predict the temperatures that would be reached by a structure exposed to a natural fire and are based on the temperature-time profiles of compartment fires, fueled by wood cribs. Its use requires knowledge of such factors as how a building is compartmented; each compartment's volume, window & floor area, fire loading (available fuel energy per sq. metre floor area) and the fire type (free burning or restricted ventilation). The possible deflection of flames by wind must also be considered.

An SCI publication [11] gives detailed guidance and a step by step design procedure for applying a fire engineering design method to external steelwork. This can include SHS both as columns and beams. Further applicable information on natural fires is given in the SCI Handbook [2].
REFERENCES


Table 5. Limiting temperatures for design of protected and unprotected hot finished members

<table>
<thead>
<tr>
<th>Description of member</th>
<th>Limiting temperature at a load ratio of:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.7°C 0.6°C 0.5°C 0.4°C 0.3°C 0.2°C</td>
</tr>
<tr>
<td>Members in compression, for a slenderness λ (see note)</td>
<td>510 540 580 615 655 710</td>
</tr>
<tr>
<td>&lt; 70</td>
<td>460 510 545 590 635 635</td>
</tr>
<tr>
<td>&gt; 70 but ≤ 180</td>
<td></td>
</tr>
<tr>
<td>Members in bending supporting concrete slabs or composite slabs:</td>
<td>590 620 650 690 725 780</td>
</tr>
<tr>
<td>unprotected members, or protected members complying with item (a) or (b) of 2.3</td>
<td>540 585 625 655 700 745</td>
</tr>
<tr>
<td>other protected members</td>
<td></td>
</tr>
<tr>
<td>Members in bending not supporting concrete slabs:</td>
<td>520 555 585 620 660 715</td>
</tr>
<tr>
<td>unprotected members, or protected members complying with item (a) or (b) of 2.3</td>
<td>460 510 545 590 635 690</td>
</tr>
<tr>
<td>other protected members</td>
<td></td>
</tr>
<tr>
<td>Members in tension: all cases</td>
<td>460 510 545 590 635 690</td>
</tr>
</tbody>
</table>

NOTE. λ is the slenderness, i.e. the effective length divided by the radius of gyration.

Table 1. Limiting temperatures for protected and unprotected structural steel members according to BS 5950 Part 8

Table 2. Typical specification table from the ASFPCC/SCI handbook [3] for a sprayed protection product.
Hollow section construction under predominantly static loading in Eurocode 3

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Summary

The Commission of the European Communities issued the "Construction Product Directive" in December 1988 with the aim to establish the single European market by the deadline of 1st January 1993. In order to formulate harmonized technical rules for the design and execution of buildings and civil engineering works, the Commission had initiated the drafting of structural Eurocodes since 1981. Eurocode 3 is the design code for steel structures.

This paper describes the development of Annex K of Eurocode 3, which deals with the design of hollow section structures under predominantly static loading. The basis of the proposed design rules are the documents of CIDECT and IIW and other national codes, in which the limit state concept had already been applied. For uniplanar lattice girder joints the rules agree with those recommended by IIW Subcomm XV-e. The Eurocode 3 rules are expected to be issued as European Prestandard (ENV) in 1991.

1. Introduction

In December 1988 the Commission of the European Communities published the "Construction Product Directive" that performs the legal basis for the establishment of the single European market envisaged from the 1st January 1993 on.

This Construction Product Directive specifies the "Essential Requirements" only and leaves the technical details to "Harmonized European Technical Specifications" (Standards or Approvals), which
in turn have a deemed to satisfy status and shall be accepted by free agreement.

For the design and execution of buildings and civil engineering structures the relevant essential requirement is the requirement for "mechanical resistance and stability". The "Harmonized European Standards" will be prepared by CEN (the European Standard Organization) which will be mandated by the Commission.

In the development of "Harmonized Design Rules for Buildings and civil Engineering Structures" much progress had already been achieved by the draft of the Eurocodes before the Construction Product Directive came into force and before CEN entered the scene.

These draft Eurocodes offer already an advanced concept of calculation and design together with up-to-date technologies and hence provide a means to prove compliance of civil engineering works with the essential requirement, form a basis for specifying contracts for the execution of private and public construction works and related engineering services and may be used as a framework for drawing up European Product Standards and approval guidelines.

In the field of steel structures the relevant Eurocode is Eurocode 3, for which Part 1 - General Design rules and rules for buildings has already been completely drafted and is expected to be released as an ENV-document (European Prestandard) for experimental use in 1991.

This Part of Eurocode 3 contains rules for the design of hollow section structures (Annex K). This paper deals with this particular subject.


The first draft of Annex K of Eurocode 3 Part 1 was prepared on the basis of a principal source document, the IIW Doc. XV-491-81 "Design Recommendations for Hollow Section Joints" issued in 1981 [1]. These recommendations deal with the design and analysis of predominantly statically loaded single plane joints in lattice structures composed of hollow sections with round, square or rectangular shape or combinations of these with open sections. These design recommendations were founded on the results of large research programmes, from which had been sponsored and coordinated by CIDECT.

During the last years these proposed strength formulae and design rules were calibrated against all available test results collected in an extensive databank and evaluated on the basis of Annex Z of Eurocode 3 [2] (see 4. Background Document to Annex K). By this calibration the best-fit strength models as well as the characteristic values and the safety factors could be determined.
Besides this work further experimental investigations and theoretical studies were carried out to establish guidance for the design of gusset plate joints, multiplanar joints under multiplanar loading, joints under bending moments and interaction between axial loading and bending moments.

In an extended draft of Annex K of Eurocode 3 [3] these latest research results are considered.

3. Design rules in Annex K of Eurocode 3

Annex K of Eurocode 3 contains design rules for the ultimate limit state verification for various joints (i.e. T, X, K, gusset plate joints etc.) with various member combinations (circular, square and rectangular hollow sections, I-sections, plates) loaded by axial forces, bending moments or both in predominantly statically loaded structures.

The chapters 8 to 10 of Annex K present strength functions and characteristic values for welded joints between circular, square and rectangular hollow sections and I-sections, from which in Fig. 1-7 some examples for different configurations of circular hollow section joints are given. These functions agree with those recommended by IIW Subcomm. XV-E in 1989 [I]

These strength functions are related to several failure modes:

- chord face failure
- chord web or wall failure by yielding or instability
- chord shear failure
- chord punching shear
- brace failure with reduced effective width
- local buckling failure

Furthermore the weld and parent material should meet certain requirements to avoid weld failure and lamellar tearing.

The test results of the experimental research have shown, that not all failure modes are relevant when certain validity ranges for parameters are given. Therefore the strength functions presented are only those necessary to be considered within the given validity ranges.

The required reliability of all these functions and characteristic values had been achieved by calibration with test results that was performed on a statistical basis as described hereafter. The statistical procedure had been harmonized by CEB and ECCS.

The results of the calibration are presented in a background document [4] to Annex K. This background document includes the justification of reduced buckling lengths of hollow section members in trusses that can be assumed conservatively to be 0.75 times the brace length. A more precise calculation method for the buckling length is given in chapter 11 of Annex K.
4. Background Document to Annex K

The standard procedure for the calibration of characteristic values, design values and partial safety factors $\gamma_M$ values for strength functions with test results based on a statistical evaluation is carried out in the following steps [5,6]:

**Step 1:** A strength function is proposed, that should be based on a simple mechanical model and contains all relevant parameters.

**Step 2:** All available test results are collected. The documentation of these tests should contain measured data for all parameters taken into account in the strength function; otherwise the tests cannot be considered.

**Step 3:** Comparison of the experimental test results with the results yielded from the strength functions with the measured parameters, see Fig. 8.

**Step 4:** Check of the sensitivity of the strength function in view of the different parameters and subsets, see Fig. 9.

**Step 5:** Determination of the mean value corrections and coefficients of variation taking account of the actual density distribution for the subset population that is plotted on Gaussian paper in order to find out how the actual distribution can be best approached by a log-normal distribution, see Fig. 10.

**Step 6:** Determination of the characteristic values, design values and the partial safety factors $\gamma_M$ in view of the target $\beta$-index ($\beta$=3.80) from the statistical parameters obtained in step 5.

The results of the sensitivity checks (step 4) as shown in Fig. 9 may be used:
- to check the sensitivity of the strength function to the different parameters,
- to find out whether the strength function should be improved by including additional parameters or test population should be separated into appropriate subsets.

A parameter may be taken as correctly modelled when the ratio of the experimental results to the results of the strength function is constant versus this parameter.

In Fig. 10 an example for an actual distribution (step 5) is given.
5. Symbols

\( N_{i, Rd} \) design axial resistance of the cross section of the chord member at the gap

\( N_{i, Rd} \) design resistance of a joint expressed in terms of a internal axial force of member \( i \)

\( M_{ip, i, Rd} \) design resistance of a joint expressed in terms of a internal in plane bending moment of member \( i \)

\( M_{op, i, Rd} \) design resistance of a joint expressed in terms of a internal out of plane bending moment of member \( i \)

\( N_{i, Li} \) internal design axial force of brace member \( i \)

\( M_{ip, i, Li} \) internal in plane design bending moment of brace member \( i \)

\( M_{op, i, Li} \) internal out of plane design bending moment of brace member \( i \)

\( V_{Rd} \) design shear resistance of a cross-section

\( V_{i, Rd} \) internal design shear force

\( f_{yi} \) nominal yield stress of member \( i \)

\( \sigma_{i, \sigma} \) normal stress due to internal axial force and bending moment in connected plate, \( I \) or RHS

\( \sigma \) maximum compressive stress in the chord due to internal axial force and/or bending moment

\( \sigma_{op} \) maximum compressive stress in the chord excluding the stress due to horizontal brace force components

\( \gamma_{M} \) partial safety factor

\( \gamma_{Mj} \) partial safety factor

\( \lambda_{ov} \) overlap, \( \lambda_{ov} = q/p \cdot 100\% \)

\( q \) length of overlap between braces of K- or N- joints at the chord face

\( p \) length of projected contact area between overlapping brace and chord without presence of the overlapped brace

\( b_{i} \) external width of a square or rectangular hollow section (RHS) for member \( i \)

\( d_{i} \) external diameter of a circular hollow section (CHS) for member \( i \)

\( e \) eccentricity of noding

\( g \) gap between the braces of a K or N joint

\( h_{i} \) external depth of a section for member \( i \)

\( i \) integer used to denote member of joint, \( i=0 \) designates chord and \( i=1+2 \) the brace members. Normally \( i=1 \) refers to the compression brace and \( i=2 \) to the tension brace

\( n_{0} \) index used for the overlapped brace

\( n_{i} = \eta_{i}/f_{yi} \)

\( t_{i} \) wall thickness of member \( i \)

\( l' \) buckling length

\( L \) system length

\( B \) (averaged) brace to chord diameter ratio or width ratio \((d_{b}/d_{c})\)

\( \gamma \) half chord width, or half chord diameter to wall thickness ratio \((d_{b}/2t_{o})\)

\( \eta \) depth of the brace to the diameter or width of the chord ratio

\( \theta_{i} \) included angle between a brace member and the chord or between the brace members

\( k_{i} \) function, defined in fig. 1 and graphical shown in fig. 2

\( K_{p} \) function which incorporates the chord stress in the joint strength equation

\( r_{e} \) experimental resistance

\( r_{t} \) theoretical resistance determined with the measured parameters
6. References

[1] IIW - Commission XV
Design Recommendations for Hollow Section Joints - Predominantly statically loaded -
IIW Doc. XV-491-81, Revised: IIW Doc. XV-701-89

[2] Eurocode 3 Part 1
Common Unified Rules for Steel Structures
Commission of the European Communities
Issue 5, November 1990

Annex K of Eurocode 3
Hollow Section Lattice Girder Connections
Draft 28.02.91

Background Documentation 5.07
Evaluation of test results on hollow section lattice girder connections
Eurocode 3 Editorial Group, July 1990

[5] Background Documentation 2.01
Background Document for Chapter 2 of Eurocode 3
Eurocode 3 Editorial Group, March 1989

Background report to Eurocode 3
Procedure for the Determination of Design Resistance from Tests
TNO-report BI-87-112; November 1987, update June 1988

7. Figures
<table>
<thead>
<tr>
<th>TYPE OF JOINT</th>
<th>DESIGN RESISTANCE (i=1,2)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>T and Y Joints</strong></td>
<td><strong>CHORD PLASTIFICATION</strong></td>
</tr>
<tr>
<td><img src="image" alt="Diagram of T and Y Joint" /></td>
<td>$N_{1,xx} = \frac{f_{y} \alpha_{y}^{2}}{\sin \theta_{y}} - (2.8 + 14.2 \beta) \gamma^{n-1} \frac{1.1}{Y_{y}}$</td>
</tr>
<tr>
<td><strong>X Joints</strong></td>
<td><strong>CHORD PLASTIFICATION</strong></td>
</tr>
<tr>
<td><img src="image" alt="Diagram of X Joint" /></td>
<td>$N_{1,xx} = \frac{f_{y} \alpha_{y}^{2}}{\sin \theta_{y} \left(1 - 0.81 \beta \gamma^{n-1}\right)} \frac{1.1}{Y_{y}}$</td>
</tr>
<tr>
<td><strong>K and N gap or overlap joints</strong></td>
<td><strong>CHORD PLASTIFICATION</strong></td>
</tr>
<tr>
<td><img src="image" alt="Diagram of K and N Joint" /></td>
<td>$N_{1,xx} = \frac{f_{y} \alpha_{y}^{2}}{\sin \theta_{y}} \left(1.8 + 10.2 \frac{d_{1}}{d_{2}} \right) \frac{1.1}{Y_{y}}$</td>
</tr>
<tr>
<td><strong>T, Y and X joints</strong></td>
<td><strong>PUNCHING SHEAR</strong></td>
</tr>
<tr>
<td><strong>K, N and KT joints with a gap</strong></td>
<td>$N_{1,xx} = \frac{f_{y} \alpha_{y}^{2} \pi d_{1}}{2 \sin^{2} \theta_{y}} \frac{1.1}{Y_{y}}$</td>
</tr>
</tbody>
</table>

**FUNCTIONS**

- $k_{p} = 1.0$ for $n_{p} \geq 0$ (tension)
- $k_{p} = 1 + 0.3(n_{p} - n_{p}^{*})$ for $n_{p} < 0$ (compression)

but $k_{p} = 1.0$

$k_{p} = \frac{\gamma^{n} \frac{1}{\cos^{2} \gamma} + 0.02e^{4} \gamma^{1.3}}{\exp(0.5 \frac{\gamma}{e_{y}^{*}} - 1.33) + 1}$

(see figure 4)

Figure 1: Design resistance of welded joints between circular hollow sections

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\[
\frac{d_t}{d_o} \geq 0.2 \quad \text{but} \leq 1.0
\]

| \( \gamma \leq 25 \) | \( \gamma \leq 20 \) (X joints) | \( \frac{d_t}{2t_1} \leq 25 \) |
| \( \sigma \geq t_1 + t_2 \) | \( \lambda_{ov} \geq 25\% \) |

**Figure 2: Range of validity**

<table>
<thead>
<tr>
<th>TYPE OF JOINT</th>
<th>DESIGN RESISTANCE (i=1,2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T, Y and X Joints</td>
<td>CHORD PLASTIFICATION</td>
</tr>
<tr>
<td>![Diagram of T, Y and X Joints]</td>
<td>( M_{pl,1,2} = \frac{f_{p,x} t^2 d_1}{\sin \theta_i} \left( \frac{1.1}{Y_{st}} \right) )</td>
</tr>
<tr>
<td>T, Y, X, K and N Joints</td>
<td>CHORD PLASTIFICATION</td>
</tr>
<tr>
<td>![Diagram of T, Y, X, K and N Joints]</td>
<td>( M_{pl,1,2} = \frac{f_{p,x} t^2 d_1}{\sin \theta_i} \left( \frac{1.1}{Y_{st}} \right) )</td>
</tr>
<tr>
<td>T, Y and X Joints, K and N Joints with a gap</td>
<td>Punching shear</td>
</tr>
<tr>
<td>![Diagram of T, Y and X Joints with a gap]</td>
<td>( M_{pl,1,2} = \frac{f_{p,x} t^2 d_1}{\sin \theta_i} \left( \frac{1.1}{Y_{st}} \right) )</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Range of validity see Figure 3. Special functions see Figure 1.

**Figure 3: Design resistance of sections loaded by bending moments**
Figure 4: Values of factor $k_g$
<table>
<thead>
<tr>
<th>TYPE OF JOINT</th>
<th>CHECK OF THE JOINT DESIGN RESISTANCE (i=1,2)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$N_{1,\text{ad}} \leq N_{1,\text{ed}}$ $N_{1,\text{ad}}$ from $X$ joint</td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$N_{1,\text{ad}} \sin \theta_i + N_{1,\text{ad}} \sin \theta_i \leq N_{1,\text{ad}} \sin \theta_i$ $N_{1,\text{ad}} \sin \theta_i \leq N_{1,\text{ad}} \sin \theta_i$ $N_{1,\text{ad}}$ from $K$ joint replace $d_i$ by $d_i + d_i + d_i$ in $K$ joint design resistance</td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$N_{1,\text{ad}} \sin \theta_i + N_{1,\text{ad}} \sin \theta_i \leq N_{1,\text{ad}} \sin \theta_i$ $N_{1,\text{ad}}$ from $X$ joint $N_{1,\text{ad}} \sin \theta_i$ is the larger of $N_{1,\text{ad}} \sin \theta_i$ and $N_{1,\text{ad}} \sin \theta_i$</td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td>$N_{1,\text{ad}} \leq N_{1,\text{ad}}$ $N_{1,\text{ad}}$ from $K$ joint in addition: check design resistance of cross section in 1-1 (gap joints only) $\left( \frac{N_{1,\text{ad}}}{N_{1,\text{ed}}} \right) \frac{Y_{\text{ad}}}{Y_{\text{ed}}} \leq 1.0$</td>
</tr>
</tbody>
</table>

Figure 5: Design resistance of special types of joints

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<table>
<thead>
<tr>
<th>TYPE OF JOINT</th>
<th>DESIGN AXIAL RESISTANCE</th>
<th>DESIGN IN PLANE MOMENT RESISTANCE</th>
<th>DESIGN OUT OF PLANE MOMENT RESISTANCE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$N_{ud}$ 0.5 $dM_{ud}$</td>
<td>$M_{ud}$</td>
<td>$M_{ud}$</td>
</tr>
<tr>
<td>XP1</td>
<td>$f_{pu} A_{t} \left( \frac{5.0}{1 - 0.8 \beta} \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td>$0.5 \beta M_{ud}$</td>
</tr>
<tr>
<td>TP1</td>
<td>$f_{pu} A_{t} \left( 4.0 \times 20.0 \beta^3 \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>XP2</td>
<td>$5.0 f_{pu} A_{t} \left( 1 + 0.25\eta \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td>$b_{s} N_{ud}$</td>
</tr>
<tr>
<td>TP2</td>
<td>$5.0 f_{pu} A_{t} \left( 1 + 0.25\eta \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>XP4</td>
<td>$f_{pu} A_{t} \left( \frac{5.0}{1 - 0.8 \beta} \left( 1 + 0.25\eta \right) \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td>$b_{s} N_{ud}$</td>
</tr>
<tr>
<td>TP4</td>
<td>$f_{pu} A_{t} \left( 4.0 \times 20.0 \beta^3 \left( 1 + 0.25\eta \right) \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td>$0.5 \beta M_{ud}$</td>
</tr>
<tr>
<td>XP5</td>
<td>$f_{pu} A_{t} \left( \frac{5.0}{1 - 0.8 \beta} \left( 1 + 0.25\eta \right) \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td>$b_{s} N_{ud}$</td>
</tr>
<tr>
<td>TP5</td>
<td>$f_{pu} A_{t} \left( 4.0 \times 20.0 \beta^3 \left( 1 + 0.25\eta \right) \right) k_p \left( \frac{1.1}{\gamma_k} \right)$</td>
<td></td>
<td>$0.5 \beta M_{ud}$</td>
</tr>
</tbody>
</table>

General:
Punching shear check:

For $TP5/XPS$ $(a_{s} + c_{p}) t_{d} \leq 0.58 f_{pu} A_{t} \left( \frac{1.1}{\gamma_k} \right)$

Range of validity see Figure 6.2 except $\beta \geq 0.4$ and $\eta \leq 4$
Special functions see Figure 6.1

\[ \eta = \frac{h}{d_{p}} \]
<table>
<thead>
<tr>
<th>TYPE OF JOINT</th>
<th>CORRECTION FACTOR TO UNIPLANAR JOINTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$60^\circ \leq \varphi \leq 90^\circ$</td>
<td>$1,0$</td>
</tr>
</tbody>
</table>

\[
\begin{align*}
1 + 0.33 \frac{N_{1,ed}}{N_{1,rd}} \\
\text{take into account of} \\
\text{the sign of } N_{1,ed} \text{ and } N_{1,rd} \\\n|N_{1,ed}| \leq |N_{1,rd}|
\end{align*}
\]

\[
\begin{align*}
60^\circ \leq \varphi \leq 90^\circ \\
0.9 \\
\text{in addition:} \\
\text{check design resistance of} \\
\text{cross section in 1-1} \\
\text{(gap joints only)} \\
\left(\frac{N_{1,ed}}{N_{1,rd}}\right)^2 + \left(\frac{V_{ed}}{V_{rd}}\right)^2 \leq 1.0
\end{align*}
\]

Range of validity see Figure 2.

Figure 7: Design Resistance of multiplanar joints

Figure 8: Comparison of theoretical strength $r_1$ with test results $r_c$
Figure 9: Sensitivity of the strength function in view of different parameters (here: $\beta = \frac{d_i}{d_o}$ for gap K-joints with CHS chords)

Figure 10: Representation of the test population on Gaussian paper
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