DEMOUNTABLE CONCRETE STRUCTURES

A challenge for precast concrete

Editors:
H. W. Reinhard
J. J. B. J. J. Bouvy

Delft University Press
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DEMOUNTABLE CONCRETE STRUCTURES

A challenge for precast concrete

Proceedings of the international symposium, held at Rotterdam, The Netherlands, May 30-31, 1985

Editors:
H. W. Reinhardt
J. J. B. J. J. Bouvy

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INTRODUCTION

H.W. Reinhardt
Chairman of the Scientific Committee and
of the Local Organizing Committee, Delft,
the Netherlands

MOTIVE

Demountable building is a part of adaptable architecture, adaptable to other uses,
social changes, different environment. Adaptable means also expandable or reducible,
movable, changeable and flexible. The motives for an adaptable architecture are
numerous: human needs are changing during the lifetime of a building; the social
environment of a particular locality of a town develops and causes new requirements;
the philosophy of housing and urban development changes due to increasing wealth.
Industrial production techniques change; office organization is influenced by new
electronic tools; technical installations of hospitals and service centres become
obsolete and have to be replaced. There are goods which cannot be augmented, such as
raw materials, space and energy. They should therefore be conserved and/or recycled as
much as possible. And, last but not least, the environment should be kept free from
dust, noise, solid waste and air pollution.

Demountable building is a technical means to help to realize adaptable architecture.
The idea as such is not new. Tents and awnings have been used for centuries; half-
timbered buildings were erected, dismantled and reused for generations, and also many
steel structures are capable of dismantling and reuse. Demountable means always to
disassemble a structure in such a way that the parts are not damaged and can be reused.
It means usually to undo connections between structural parts. What has been achieved
with other materials can also be achieved with concrete, especially with precast
concrete. Since handling and lifting appliances and transport vehicles have increased
enormously in capacity during the last few decades, the weight of a structure is no
longer an obstacle to demountable concrete construction. On the other hand, new
techniques were invented, such as anchors and prestressed bolts which facilitate
changes in monolithic reinforced structures and therefore meet requirements with
regard to limited flexibility and variable use.

Of course, demountable construction is not confined to buildings. It is also appropriate
to bridges, industrial plants, transmission towers, storage facilities, safety walls, and
other structures. In fact, it is a question of acceptance rather than of technical
constructability.
The purpose of this Symposium is to discuss all aspects of demountable concrete structures. It is divided into four sessions. The first session is devoted to the objects and possibilities of demountable structures and comprises papers on service life of structures, changing functions, flexible use and economic aspects. In the second session, design and detailing of demountable concrete structures will be treated. Topics such as safety and strength are also covered. Since connections play an essential role in demountable structures, the whole third session is devoted to this subject.

Detailing of connections between beams, columns, slabs and cores will receive attention, as well as the diaphragm action of floors and the behaviour of joints under load. Many demountable concrete structures have been erected in various countries.

The last session covers examples of demountable buildings such as schools, dwellings and offices, as well as other structures, like bridges and towers. The first three sessions will be introduced by an invited speaker and will be complemented by selected papers. Session four comprises selected papers only. There will be sufficient time available for discussions.

ACKNOWLEDGEMENTS

We are indebted to many persons and organizations for having promoted this Symposium by their assistance and encouragement. Thanks are due to the members of the Scientific Committee, who have provided us with ideas and suggestions for the programme. International organizations have sponsored the Symposium by various activities. The Comité Euro-International du Béton (CEB) has informed the members of its committees and working groups. CEB has agreed to hold the Symposium in conjunction with its 24th Plenary Session starting two days later in the same city. The International Council for Building Research Studies and Documentation (CIB) has informed its members and has assisted us to find an appropriate venue for the Symposium. The Fédération Internationale de la Précâontrainte (FIP) and the International Association for Bridge and Structural Engineering (IABSE) have encouraged us by their interest in the subject.

The Netherlands Committee for Research, Codes and Specification for Concrete (CUR-VB) set up a research committee on demountable structures in 1978. Part of the results will be presented during the Symposium. Besides providing continuous support for the investigations, CUR-VB has stimulated this Symposium and contributed substantially to its preparation and to carrying it into effect. STUPRE, the Society for Studies on the Use of Precast Concrete in the Netherlands, discussed the topic in a working group and stimulated its members to take an active part in the Symposium. The Council of the Delft University of Technology and its Department of Civil Engineering provided funds for suitably organizing this event. Financial support has also been obtained from the Professor-Bakker Foundation, Ministry of Housing, Physical Planning and Environment and the City of Rotterdam. All these contributions are gratefully acknowledged.

The members of the Local Organizing Committee have accomplished a great task. We
wish to record our indebtedness to all of them and their assistant staff. Special thanks are due to Mr. J. Stroband, who served as general secretary, and to Mr. J.J. Bouvy, who served as technical secretary of the Local Organizing Committee.

Enthusiastic speakers, chairmen and reporters give a Symposium its real value. We express our thanks to all those who contributed by their time, effort and discussions to the promotion of demountable structures which may help to meet the human, social and environmental requirements of a changing society.

Finally we wish to thank the Delft University Press for its valuable cooperation in producing the syllabus of this Symposium.
OPENING OF THE SYMPOSIUM

by Mr. G.Ph. Brokx, Minister of State
Ministry of Housing, Physical Planning and Environment
The Hague, the Netherlands

Approximately 10 years ago the ideas of demountable concrete structures came into focus and received more and more publicity.

I may in this context refer to a lecture of the chairman of the scientific committee of this international symposium, held in Delft on May 19th 1976.

In a number of buildings, the method of demountable structures has already been put in practice, whether or not by request of the future proprietor, who wished to give his property an added value.

Also, one case is known where with the re-use of building components of an existing building, two new buildings have been erected.

Actually, demountable structures of relatively big building components may be considered as the best way of recycling rubble.

If done in a clever way, the re-use of demountable structures may be financially attractive, because the production of new elements is partly or totally superfluous.

By this, a saving of the use of energy and raw materials occurs.

Besides these advantages a reduction of rubble may be favourable for the environment.

All this provides sufficient reasons to advocate demountable structures.

The advantages mentioned can only be obtained if this way of constructing takes place in a proper way.

Therefore it is necessary that a certain "philosophy" will be introduced, a philosophy that demands of all involved participants another approach of the building process than was customary until now.

Such a philosophy is in my opinion the great challenge of this symposium.

I want to emphasize that new times demand new solutions.

What about the architect?

Until now he was used to start with a clean slate.

The starting point for the design could be placed by him in consult together with the future proprietor, like for instance on the distance between centre lines of loadbearing columns or walls.

The application of recycled building elements will confront him with fixed starting points. Will this be experienced as oppressing or as a challenge?
Also, if someone wishes to design a completely new building without the re-use of building elements, he will wonder whether the design is adequate to be carried out in demountable structures. Furthermore, he will see to it that a future architect, while re-using the building components, will be limited as less as possible by his design, in order to offer the future architect the best possibilities for his own creation.

What will be the attitude of the manufacturer of building elements towards this "phenomenon"?
Will he consider demountable structures exclusively as a reduction of his market share in the long run, or will he meet sufficient benefit of the added value?
Will he be sufficiently creative when designing the joints and anchorages, so that if need be demounting and re-erection will be possible without any problem?

Although there are several technical problems to which I think generally a solution will be found without doubt, the emphasis will be put on a new building concept, a building philosophy.

During this symposium you will probably not get a completely adequate answer. If this symposium enables to stimulate your way of thinking in the right direction, then I think the symposium will be successful.
After the Second World War a very interesting evolution process started in the field of concrete construction. Prestressing technique achieved full progress, not only in structures concreted in-situ, but also in the industrial production of prestressed concrete elements - especially beams. For example, in the United Kingdom, Denmark and the Netherlands, factory-made prestressed concrete was developing rapidly.

Due to the introduction of prestressed concrete it was possible greatly to exceed the "magic" span limit (8 - 10 m) in ordinary reinforced concrete beams. Also the use of high-strength concrete was of economic interest, thus limiting the cross-sections of beams to a minimum.

With factory-made - pretensioned - prestressed concrete the use of steel moulds in conjunction with steam curing processes was an impetus to standardization of beam cross-sections. For example, in the Netherlands one factory had already standardized the cross-sections in the early nineteen-fifties. The precast beams were transported to the site and erected by cranes. In those years the beams were mainly freely supported at their ends on felt or rubber bearings.

Many single-storey buildings for factories, garages, etc. were built in this way. However, in most cases the columns were rigidly connected to the foundation slab. In several countries these buildings erected in the fifties and the early sixties are still in use. Since the roof construction often consists of freely supported slabs carried by freely supported secondary beams, most of these buildings can be claimed to have fully demountable roof structures!

In the development of the technique of prefabrication in concrete, there was an increasing need for rigid beam-to-beam connections over supports and rigid beam-to-column connections. Multi-storey buildings could be built more economically if the concrete structural framework possessed stability of its own. Therefore sufficient strength and rigidity of such connections were essential. They were achieved by means of slabs concreted in-situ over beams, the introduction of - in-situ -prestressing, and specially developed connection details. These details were based mainly on "wet joints" - using concrete placed in-situ.

In the Eastern European countries welded steel connections were moreover used. In this way it was possible to continue the erection of building frames even during the winter periods.

It can be stated that by the end of the nineteen-sixties only few concrete structures
constructed with precast concrete units were - more or less - demountable. In the mid-seventies there occurred a marked change in the general feeling in this field. The increasing use of concrete structures was causing a shortage of raw materials - especially of sand and gravel. So it was studied whether and how recycling of concrete was possible, especially from an economic point of view. Concrete can be recycled by dividing the structure into loose reinforcement bars and concrete rubble on demolition. The rubble can be comminuted in such a way that it can be re-used as "gravel" coarse aggregate in the concrete mixer. However, the other way is to re-use the structural concrete units themselves. But then the concrete structure must be "demountable".

It was Prof. Reinhardt who, in 1975, introduced this idea in his inaugural lecture as Professor in the Delft University of Technology. Since then he has been very active in this field, especially in the Stevin Laboratory, in promoting the development of "demountable" wet and dry connections.

The aim of the research in this field of concrete structures is to apply rigid connections in the structure in such a way that they can be removed without damaging the structural members.

For example, longitudinal joints between members and connections over supports must be so designed that they contribute to the stability of the structure during its lifetime but can be easily removed if the structure is dismantled.

This means that joints and connections must not simply be as rigid as possible but must possess controlled strength and rigidity.

Control of the materials and of the structural quality is possible only on the basis of comprehensive knowledge of the behaviour of our structure, based on scientific and practical research projects.

It is very important to assemble the knowledge which is already available. Therefore we appreciate it very much that so many of you have accepted our invitation to attend this symposium. I hope that it will - for all of us - provide wider scope to convince the general public that concrete structures need not be demolished with noisy hammers, but can be taken down and re-erected elsewhere with nothing more objectionable than the gentle humming of busy cranes!
PURPOSE AND POSSIBILITIES OF DEMOUNTABLE CONSTRUCTION

Ir. W.J. van den Boogaard
Chairman of the Research Committee D 7
"Demountable Construction",
CUR-VB, Zoetermeer, the Netherlands

ORIGIN OF THE IDEA OF DEMOUNTABLE CONSTRUCTION

It is not always possible to pinpoint the actual cause of a particular process of evolution.

In so far as the theme of this Symposium is concerned there are the developments in technology and the engineering sciences to consider. Both the scientific researcher and the engineer engaged in the preparation and execution of construction projects in the course of his day-to-day professional work are participating - whether they are fully aware of this or not - in developments in their field of technology: concrete/precast concrete, concrete/prestressed concrete. In devoting further thought to these subjects their attention will also be directed to the demolition of buildings and other structures, and they will consider whether those prefabricated units might also be suitable for demolition in a simple manner. Thus the idea of demountable construction evolves in response to the scientific engineering approach.

The other approach is through the aspect of social developments.

Since 1945 a great deal has been built, much of it in concrete. There has been a growing realization that some day all these buildings will reach the end of their lives, sometimes in the technical, but much more frequently in the economic sense. So long as the number of buildings to be demolished was not unduly large, the traditional method of demolition was acceptable or could, at any rate, be tolerated. Actually it has always been a rather unacceptable method, demanding a great deal of energy (both human and mechanical), besides being environmentally very disagreeable - primarily for the men on the job, but also for the surroundings. Noise and dust are the abundant by-products of these demolition activities. But if the number of buildings to be demolished greatly increases - and that is bound to happen - then the existing method will become totally unacceptable.

So it is from these two directions, i.e., technical and social developments, that the conception of specific investigation into the possibility of demountable construction has emerged. That is what this Symposium is about.

THINKING MONOLITHICALLY

For a great many years reinforced concrete construction has been characterized by a number of specific facets. In the context of this Symposium the following may more particularly be mentioned: placing concrete in-situ in formwork likewise constructed or
at least assembled in-situ, and a very high degree of monolithicity of the structure.

In the post-war years there have been major developments in concrete construction. Much attention has in the past already been paid to the tremendous advances achieved in prestressed concrete, making it possible to build structures that had previously been inconceivable. But today I am more particularly concerned with the rise and the great expansion of prefabricated construction in concrete, i.e., using precast components. In connection with this evolution, too, both the technical and the social relevance that I have already mentioned has been present.

In the technical sphere there was the development of handling possibilities with increasingly powerful and higher lifting appliances. Important from the social point of view was more particularly the conception that a large proportion of all concrete should be capable of being produced in factories instead of in the open air under variable weather conditions.

With in-situ concrete, monolithic construction is a fairly obvious thing to do. On examining the cross-section of a building constructed in this way we perceive that the floors and walls merge into one another, as it were. Where a beam is joined to a floor, too, a logical form of construction is obtained when these two members are concreted as a monolithic combination, so that the thickness of the floor slab forms part of the depth of the beam.

Which does not mean to say that achieving monolithicity does not require appropriate arrangements.

We all have some conception of the care with which joints and connections have to be executed: cleaning, removing irregularities, possibly applying cement slurry, etc. And then there are all those protruding bars that are always getting in the way; they have caused many a tear in clothing and, what is much worse, numerous injuries, including possibly even fatal ones (I know of at least one such accidental death). But the result is indeed a structure which, thanks to its multiple statical indeterminacy, embodies very substantial reserves of "hidden" safety - far more than is generally taken into account in the design calculations. Quite probably this is the reason - though not an expressly stated one - why, when prefabricated construction with precast units came to be very extensively applied, designers often still clung to the principle of forming, to a greater or less extent, strong structural connections with projecting reinforcement and in-situ concrete.

SAFETY

Here we encounter a curious episode in the history of building in concrete: precast concrete construction undergoes considerable development, but in applying it the predominating tendency is to carry on with the traditional method of building in this material: with projecting bars and in-situ concrete, resulting in structures of multiple statical indeterminacy and essentially monolithic in character.

The underlying reason for this lies in the designers' awareness of a great measure of
structural safety not fully taken into account in the calculations and in their reluctance to enter a range where this extra margin of safety is no longer available. Though this mental attitude is open to scientific criticism, it must be borne in mind that building is always concerned with protecting, with safety as an essential part of this. The designer who, at the expense of some inconvenience of execution of the job (and therefore some extra cost), opts for a higher degree of real built-in safety is surely not to be too severely censured.

BUILDING AND USING

On the other hand, even less blameworthy is the designer who critically scrutinizes and reassesses all the aspects involved and concentrates on three sets of criteria: the construction, the use and the demolition of the building. We shall confine ourselves to the structures themselves - though similar considerations are, broadly speaking, applicable in many ways to the structural components - and, in the context of this Symposium, to reinforced concrete structures. It is indeed notable that in the sphere of construction our interest as designers tends to be largely directed at the realization of a project. This is more particularly true of those of us who have been given a civil engineering training. Greater interest in the use of the building - and in its users - is more frequently encountered in those who have been trained as architects. Even so, at the inauguration of a building I once heard an architect remark: "We designers so concentrate on designing and on getting our designs carried out that we could now say: The building is ready, so let it be demolished, and we shall then design a new one and have it built".

Be that as it may, neither the more purely civil engineering nor the more purely architectural designer is particularly concerned with "the end" of the building, i.e., the manner in which it ends its service life. There is indeed a reason for this attitude of mind: the lifetime of a building is measured in tens of years, even in centuries, in a few instances possibly even in millennia. In any case, a period ranging from fairly long to very long in relation to the span of a human life. So the designer or the builder is not often confronted with the end of the building that he helped to create. The ageing, the decay and the end of his own products are not matters with which he is confronted every day or indeed every year. In other occupations this does occur: the market gardener who plants tomatoes or the farmer who plants potatoes knows that these are annual crops. This does not mean to say he consciously thinks about this all the time, but it is a fact that must influence his thoughts.

DEMOLITION

Let us return to buildings. We, too, are well aware that they are not everlasting. But, in contrast with people in occupations like those mentioned above, we must stop and reflect in order to reach full realization that buildings will have to be demolished some day.
In recent years this reality has begun to loom larger in our minds. From various sides we hear about research projects undertaken with a view to estimating how much demolition work is likely to arise in the future. Despite differences in their estimates, all these researches indicate that the quantities involved will be very large.

And, as it turns out, reinforced concrete is very difficult to demolish. Much more difficult than masonry, timber or steel.

Of course, we knew this all along, but I have explained to you that - and why - we are not unduly preoccupied with the need to dismantle our carefully designed and constructed buildings.

So we shall have to look for new demolition methods which demand less energy and cause less noise and dust nuisance. An entirely different approach is to investigate the possibilities of taking account, already in the design and construction stages, of the fact that the building will one day have to be demolished. This brings us to the sphere of research of the CUR-VB Committee "Demountable construction". It is almost a foregone conclusion that this research takes prefabricated construction as its starting point, as it can be presumed that this form of construction best lends itself to development into a demountable system. It should be pointed out, however, that in-situ concrete construction also deserves to be investigated with regard to the possibility of better demountability than is now the case; by appropriate detailing and methods of construction it should be possible to devise a system in which the eventual demolition of the building is taken into consideration already at the time of construction, so that, when the time comes, it can be taken down with less energy consumption and less nuisance.

For it is more particularly the approach to the problem, the attitude of mind brought to bear on it, that is decisive. Some mental adjustment is required to move from the conception of monolithic construction which, because of the sheer frequency with which it is applied, colours the designer's pattern of thinking and to progress to construction techniques in which the realization of the ultimate need for demolition receives its due attention already at the design stage.

With regard to this it is indeed justifiable to speak of some degree of retraining, as Mr. Brokx pointed out. I would suggest to designers and builders among us to ask themselves this question: "Do I actually realize that the building on which I am now working so enthusiastically will be demolished one day?". I believe that the answer will be "no".

It is certainly not to be regarded as reflecting a pessimistic state of mind when I now advise you to realize just that. For one thing, demolition will take place anyway. Furthermore, consideration of this aspect is an education in modesty. Thirdly, the certainty of demolition ensures that future generations can build again - since building is an activity to which we all, each in our own way, are greatly attached.

RESEARCH

Prefabricated building construction, more particularly with precast concrete units, is the obvious starting point for a specific study of demountability, though the conception of merely "installing the components and later simply removing them" is of course too
simplistic. Under a program of international co-operation between Belgium, Germany and the Netherlands the CUR-VB (Netherlands Committee for Research, Codes and Specifications for Concrete) set up Committee D 7 to conduct research into demountable construction. The investigations in question are being carried out by the Stevin Laboratory of the Delft University of Technology. In view of what has been pointed out above, the Committee decided to start its research with buildings assembled from prefabricated (precast concrete) components. In the trio "columns, beams and floors" it is the floors that account for the largest quantity of concrete used - between 60 and 70%. Besides, because of their size, floors are the most labour-intensive structural members to demolish. For this reason, floors were chosen as the first category of members to be investigated for demountability.

Now if the only function that floors have to perform consisted in the transmission of vertical load via the beams to the columns, the problem would be easy to solve. Loose floor slabs laid on the beams, with anchorages here and there, would in most cases provide a good solution. But floors also play a very important part in the stability of a building. Horizontal wind forces, the possible effect of out-of-perpendicularity of the columns, and other horizontal forces are transmitted by the floors to the columns, to the lift shafts and services cores or to the walls.

To use fairly strong joint fillings, tie reinforcement and more particularly a structural concrete topping in conjunction with precast floor units are features which are incompatible with demountable construction, i.e., the desire to be able to dismantle or "demount" the components in a simple manner. Therefore the research seeks primarily to investigate whether a good and sound floor structure can be built without having to install an in-situ concrete topping which is difficult to demolish. This will also involve investigating the quality of the joint fillings and the above-mentioned tie reinforcement connecting the floor to the edge beams.

The joint have been the subject of separate research by the CUR-VB Committee C 43, the results of which will certainly be given due consideration in the context of the investigation of demountable floor structures.

The investigations which have been carried out provide a good insight into the action of this type of floor, into the importance of proper functioning of the longitudinal beams, into the effect of the location of connections to shear walls, and into many other factors that are important to the functioning of the floor as a diaphragm for the transmission of horizontal loads. The principle of such a floor structure can be fulfilled in many different ways.

Obviously, it is not possible to carry out model research for all possible configurations on plan that may be encountered. But it is possible by a different method to gain a deeper understanding of this form of construction, namely, by utilizing the possibilities that the computer offers. The Stevin Laboratory secured the ZEFE program which had been developed in Germany.

Although this program was not quite ready for dealing with our set of problems, after the necessary adjustments had been made it turned out to be perfectly serviceable for the purpose.

From the investigations which have so far been performed it emerges that it is possible in principle to assemble floors from precast units in such a way that subsequent
dismantling can be carried out. This does not mean to say, however, that all the
problems have already been solved. In approaching a method of construction which
differs considerably from commonly used methods it is particularly necessary to pay
attention to the safety aspects.
These matters are dealt with in detail in J. Stroband's paper.
In fulfilment to the basic requirement that dismantling should be possible it is necessary
also to investigate the structural connections with regard to this. In his paper G.F.
Huyghe will consider the subject of demountable connections.
Finally, attention should be drawn to the work of two CUR-VB Committees which are
likewise engaged in research which is of major importance to the application of
demountable concrete construction to buildings. Efforts are directed at bringing
together the results of all the investigations referred to above and thus to arrive at a
self-consistent whole for the application of demountable construction to all buildings
which are suitable for this. It should be realized, however, that not all buildings qualify
in this respect. All the same, there remain many categories of buildings which are
suitable for demountable construction or which can be made suitable by appropriate
adaptation.

FLEXIBILITY

It would be too limited an approach to demountability to conceive it merely as a
simpler way to demolish the building than is now employed when it reaches the end of
its service life. In a society where many different conditions exist and sometimes
change rapidly it is very important that buildings should not, because of inflexibility or
lack of adaptability in their character, hinder such developments. Much more than in
the past it will, in designing and constructing a building, be necessary to take account
of the possibility that it will be used for a different purpose at some time during its
service life and that this may necessitate adaptations and alterations which it should be
possible to carry out without having to resort to drastic demolition work.
Here are some examples chosen at random: To ensure efficient functioning an office
department accommodated on two floors of a building will have to be provided with a
direct vertical connection between these floors; a simple staircase is easy to install,
and it should be possible to form the opening for it simply by removing one or two floor
slabs. Alternatively, the nature of the department's activities are such as to require a
two-story-high space over parts of its floor area; this, too, should be attainable in an
acceptable manner.
It would not be right, by giving more examples, to create the impression that with a
demountable construction system it will be possible to make alterations to a building as
simply as if it were built of a child's box of bricks. Actually, the building continues to
be an organism, as it were, and carrying out conversions and alterations is comparable
to the work of the medical specialist on the human body.
The care that must be devoted to such operations on a building will, with demountable
construction, certainly not become less, but the physical activities involved can be
carried out with much less difficult, energy input and nuisance. This is where the great
advantage lies.
COST

With regard to cost the following aspects are to be considered: If the demolition of a building is made simpler, it becomes a cheaper operation to carry out. But if extra expenditure has to be incurred at the time of construction in order to enable savings to be effected at the end of the service life of the building, the loss of interest over a long period of time is liable to tip the balance of the overall financial result to the negative side. So the basic principle will in general have to be that demountable construction must be not, or hardly, more expensive than conventional building construction. Only in special circumstances, e.g., for tall buildings in very densely built-up areas or where stringent environmental conditions have to be fulfilled, will it be justifiable to invest extra money at the very outset with a view to achieving advantages in taking down the building at the end of its life.

With regard to conversions or alterations to the building during the course of its service life the cost balance may turn out more favourably, but there remains the drawback that at the design and construction stage it is hardly ever known just how extensive later alterations will be. From what has been said here it follows that, besides seeking solutions for technical problems, the investigations will have to come up with methods of designing and detailing which are so conceived that demountable construction will add little or nothing to the cost of a building as compared with conventional construction.

The efforts of Committee D 7 aim more particularly at achieving this object. We emphatically consider that there is a great future for demountable construction. I would remind you what has already been stated earlier on, namely, that purpose-directed thinking in terms of demountability must come first, before demountable construction can be undertaken.
OBJECT AND POSSIBILITIES OF DEMOUNTABLE STRUCTURES

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SUMMARY

With the help of a number of examples in the Rotterdam area, an introduction will be given to the subject mentioned above.

In the course of history building has undergone big changes. Besides the development of the character of building materials and building methods during ages, an enlargement of scale in the realization of buildings, civil constructions and infrastructural works can be observed obviously in the last decades in particular. This is a direct result of wishes and requirements from society that more and more intrudingly asks for a larger flexibility in the use of buildings but also of the space in which the buildings have been erected.

This development, which seems to take place in an ever increasing speed, has caused that designers and builders have adapted and are adapting themselves to this fact.

Reuse of existing buildings, yes or no after renovation or structural modifications (dwellings), multifunctional designs (schools, halls etc.) and use of prefabricated elements are pointing thereto obviously.

Such an approach is not always possible and then the wish to use space for purposes other than originally meant for, must, by force, lead to the demolition of constructions not yet written off technically or economically.

From the point of view of society, this seems a solution which is difficult to accept, not only in view of economy (destruction of capital) but also in view of wasting materials and pollution of the environment (waste materials). For that reason new roads are being looked for to tackle this problem.

Demountable structures seem such a road; already used on a large scale for steel and timber structures, for concrete still a rather new challenge.

With the use of prefabricated elements which are joined in such a way that dismounting - demolition is no longer the proper word - becomes possible in an easy way, the flexibility wanted can be achieved. To what extent this solution is justified for a material as concrete with its specific structural problems (stability), may become clear in the course of the symposium.
SUMMARY

With the help of information and transportation systems, there is no need to depend on human or animal labor for transportation. The development of automated transportation systems has made it possible to transport goods and passengers efficiently and economically. These systems are not only cost-effective but also help reduce traffic congestion and pollution. However, the implementation of these systems requires investment and careful planning. The future of transportation will be shaped by the continued innovation and advancement in automated systems.
"FOUR-DIMENSIONAL" STRUCTURES

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SUMMARY

Developments in every field are now extremely rapid. The needs that a structure has to satisfy during its relatively long lifetime, and its conditions of service, may vary considerably and quite unforeseeably.

It is therefore advisable to build structures which can be modified and adapted without too much difficulty and without sacrifice of initial economy, aesthetic quality and ease of maintenance.

Demountable bearings, expansion joints, etc. are particularly desirable for such structures. Prefabrication - in precast concrete - facilitates demountability, but forming the structural connections for that purpose is difficult. Nevertheless, suitable solutions have been devised, especially those based on prestressing or bolting.
LES OUVRAGES À "QUATRE DIMENSIONS".

1. INTÉRÊT DE LA DEMONTABILITÉ

1.1 Rapidité de l'évolution. Difficulté d'une prévision valable.

Nous vivons à une époque d'évolution extrêmement rapide dans tous les domaines: technique, démographique, économique, social, culturel, politique. (fig 1).

Pendant la vie relativement longue, en général, des ouvrages que nous construisons, les besoins à satisfaire et les conditions à respecter risquent donc de changer très fort et de manière imprévisible.

Bien souvent, à peine a-t-on construit un ouvrage, qu'on voudrait le modifier pour l'adapter à des conditions nouvelles résultant de cette évolution.

A titre d'exemple, citons les difficiles et coûteux élargissements de routes, d'autoroutes et de viaducs urbains.

Le premier moyen qui vient à l'esprit pour éviter de coûteux travaux d'adaptation, est de voir large et de prévoir une réserve suffisante.

Mais le problème n'est pas si simple.

Tout d'abord, ce n'est pas toujours dans le sens d'une insuffisance que les ouvrages se révèlent mal adaptés à leur usage.

Dans le cas d'une surabondance, voir large n'aurait fait qu'aggraver le gaspillage que constitue un ouvrage surdimensionné.

Ensuite, une "réserve" représente un investissement qui peut être important et peu rentable.

De plus, il ne suffit même pas de savoir dans quel sens les besoins évolueront.

Il faut connaître l'importance des modifications prévisibles.

Je me souviens avoir visité aux États-Unis, un ouvrage dont l'auteur avait été particulièrement prévoyant et l'avait doté d'une infrastructure permettant un élargissement de 50%.

Effectivement, quelques années plus tard, un élargissement s'était avéré indispensable, mais 50% ne suffisaient pas, c'est 100% qu'il fallait, et l'adaptation de l'ouvrage a été fort difficile.

Enfin, l'évolution des conditions peut nécessiter plus qu'un simple élargissement ou rétrécissement. Une modification totale ou une démolition sont parfois nécessaires.

C'est notamment le cas des ouvrages dits provisoires. Mais on constate que, très souvent, des ponts provisoires sont maintenus aussi longtemps que certains ponts conçus comme définitifs.
LA RAPIDITE DE L'EVOLUTION DEMOGRAPHIQUE ACTUELLE
UNE VERITABLE EXPLOSION CATASTROPHIQUE

Fig 1

- An 1
  - Naissance du Christ
  - 0.2 milliard

- 1850
  - Age classique
  - 55 milliards

- 1850
  - Ere industrielle
  - 1 milliard

- 1914
  - Ere nucléaire
  - 23 milliards

- 1980
  - Ere spatiale
  - 44 milliards

- An 2000
1.2 Solutions proposées - Ouvrages " à quatre dimensions ".

Ce que nous proposons, c'est de réaliser des ouvrages adaptables, c'est-à-dire des ouvrages que l'on puisse aisément élargir, démonter, déplacer, bref modifier dans tous les sens.

Cela ne signifie nullement que la qualité et l'esthétique seraient sacrifiées. Elles ne doivent jamais être sacrifiées.

Une voiture automobile, par exemple, n'est construite que pour quelques années et, pourtant, son esthétique est très soignée.

Nous proposons de ne plus classer les ouvrages en deux grandes catégories bien distinctes, à savoir, d'une part, les luxueux ouvrages définitifs, théoriquement indestructibles et, d'autre part, les hideux ouvrages provisoires qui souvent menacent ruine peu de temps après leur construction.

Mais nous proposons de considérer plutôt la plupart des ouvrages comme plus ou moins définitifs, et de les concevoir de manière à ce qu'ils puissent être adaptés facilement aux conditions du moment chaque fois que celles-ci se modifient suffisamment.

C'est ce que nous appelons "ouvrages à quatre dimensions".

La quatrième dimension est évidemment le temps.

2. DIFFICULTÉ DE LA DEMONTABILITÉ ET DE L'ADAPTABILITÉ.

Pour être démontable, l'ouvrage doit être constitué d'éléments transportables et assemblés par des liaisons amovibles.

Pour que les éléments puissent être réutilisés dans d'autres ouvrages, ils doivent être standardisés et modulaires.

En ce qui concerne la réalisation des éléments transportables, des progrès spectaculaires ont été réalisés dans le cadre de la préfabrication de poutres.

Mais une standardisation modulaire valable est loin d'être aisée.

Le problème des liaisons amovibles fiables est également loin d'être simple.

3. LA STANDARDISATION MODULAIRE EVOLUTIVE.

3.1 Utilité de la standardisation.

3.1.1 Réduction du coût des études.

En plus de la réduction de coût évidente due au fait qu'une seule étude sert pour un grand nombre d'ouvrages ou de parties d'ouvrages, la standardisation permet de diminuer sensiblement le volume des études annexes telles que les études de soumissions, d'exécution, de vérification, d'équipements etc...
3.1.2 Amélioration de la qualité des études.

Etant donné que les frais d'études s'amortissent sur un grand nombre d'ouvrages, il est possible et rentable de consentir pour ces études, une dépense sensiblement plus importante, et par conséquent, de les pousser beaucoup plus loin au profit de la qualité, de la sécurité et de l'économie.

3.1.3 Réduction des délais d'étude.

Quand on réutilise un projet tout étudié, ou dont une partie est déjà étudiée, il est possible de réduire, voire de supprimer le délai habituellement nécessaire pour les études, et de commencer directement à construire.

3.1.4 Réduction du coût de l'exécution.

La possibilité de construire en série permet une réutilisation optimum du matériel, et une augmentation sensible du rendement du personnel.

3.1.5 Amélioration de la qualité de l'exécution.

La construction en grande série justifie l'utilisation d'un matériel de première qualité, puisqu'elle en garantit l'amortissement. Ce matériel permet d'obtenir une fabrication de haute qualité. L'expérience acquise après la réalisation d'un certain nombre d'exemplaires d'une construction standardisée permet également une amélioration de l'exécution.

3.1.6 Correspondance entre éléments et remplacement de ceux-ci, facilités.

A condition de tenir compte des problèmes de liaison lors de l'étude des éléments standardisés, la standardisation peut favoriser la correspondance entre les diverses parties des constructions, ainsi que le remplacement de ces éléments (par exemple, en cas de défectuosité).

3.1.7 Possibilité de stockage.

Le nombre d'éléments standards différents étant par définition fort réduit, il devient possible de les fabriquer à l'avance et de les stocker, de manière à toujours disposer des éléments dont on a besoin. Il devient également possible d'étaler la fabrication dans le temps, donc d'éviter le suréquipement et l'irrégularité d'emploi de la main-d'oeuvre.

3.2 Inconvénients de la standardisation.

3.2.1 Manque de souplesse d'adaptation aux conditions particulières.

Un ouvrage standard sera rarement aussi bien "ajusté" au site qu'un ouvrage sur mesure.

3.2.2 Surabondance de dimensions et de résistance.

On ne dispose pas toujours d'une solution standard correspondant exactement aux conditions minimales imposées, aussi y a-t-il forcément un gaspillage de matière.
MODULATION TRANSVERSALE

Dispositif de rive

Module de structure

Dispositif de rive

MODULATION LONGITUDINALE & VERTICALE

Portée isostatique = L \cdot \alpha

modulation du biais

H \cdot m_g

Gabarit

Fig 2

BIAIS

LIMITATION DE L'INFLUENCE DU BIAIS À UNE QUESTION DE PORTÉE

Fig 2bis
3.2.3 Danger de stagnation technique.

Quand on dispose de projets tout étudiés, la tentation est forte de continuer à les utiliser, même s'ils présentent des défauts par rapport à des projets meilleurs que l'expérience acquise et les progrès de la technique permettraient de réaliser.

3.2.4 Monotonie d'aspect.

L'esthétique des ouvrages est plus satisfaisante quand ils sont en harmonie avec le site et bien adaptés à leur fonction et aux conditions particulières locales.

3.3 Difficultés de la standardisation.

Le grand obstacle à la standardisation est la diversité de ce que nous appelons les "données", c'est-à-dire des conditions imposées par la disposition des lieux, la nature du sol, les caractéristiques des voies portées et franchies, les caractéristiques du trafic etc... Les paramètres qui caractérisent un ouvrage sont très nombreux et le nombre de valeurs qu'ils peuvent prendre est illimité.

3.4 Solutions de principe pour résoudre les difficultés et réduire les inconvénients.

3.4.1 Limitation du nombre de cas différents.

Pour réduire le nombre de cas, donc le nombre de combinaisons, il faut réduire à tout prix le nombre d'éléments à un strict minimum.

Il faut réduire le nombre de variables et le nombre de valeurs que peuvent prendre ces variables. Ceci est obtenu très efficacement par la modulation.

3.4.2 Choix de solutions polyvalentes.

Quoique l'on fasse, le nombre de cas, même réduit au minimum, sera encore relativement grand, il faut donc rechercher des solutions polyvalentes, faire choix d'ensembles composés d'un nombre aussi petit que possible d'éléments différents et présentant une grande souplesse d'adaptation.

Les ensembles seront alors différents comme les cas mais leurs éléments seront identiques.

3.4.3 Division modulaire des projets et réalisations.

Il faut adopter une division modulaire des projets et des réalisations, c'est-à-dire en respectant des règles de coordination qui permettent aux différentes parties de se raccorder à d'autres (fig. 2 et 2bis).

3.4.4 Caractère évolutif à donner à chaque partie et au schéma d'assemblage.

Pour éviter la stagnation, on doit veiller à ce que chaque partie et le schéma d'assemblage puissent être améliorés indépendamment chaque fois qu'une solution meilleure sera trouvée. En ce qui concerne l'aspect, la souplesse de la formule proposée permet également une diversité suffisante pour éviter une trop grande monotonie.
Fig 3
4. LES LIAISONS AMOVIBLES Fiables.

4.1 Le béton coulé sur place.

Le béton coulé sur place enrobant des armatures d'attente est un des moyens les plus simples, souples, économiques et efficaces pour relier des éléments préfabriqués.

Mais son éventuelle démolition est laborieuse et dangereuse pour les éléments à réutiliser.

4.2 La précontrainte par câbles.

Une liaison par précontrainte peut plus aisément être supprimée si des précautions sont prises.

Ces précautions peuvent consister à placer des câbles à l'extérieur de la structure ou dans des gaines traversant librement des ouvertures ménagées dans celle-ci.

4.3 La précontrainte par boulons.

En construction comme en mécanique, les boulons peuvent être avantageusement utilisés pour réunir provisoirement des éléments que l'on désire pouvoir séparer ultérieurement. Mais là encore, des précautions doivent être prises pour que l'effort de précontrainte soit conservé et pour que le démontage reste possible malgré les salissures, corrosion et autres agressions.

5. EXEMPLES DE SOLUTIONS.

5.1 Expérience des ouvrages métalliques.

Contrairement à la plupart des constructions en béton dont une partie est, en général, coulée sur place, les ouvrages métalliques sont assez aisément démontables. La réalisation sur place de soudures valables étant problématique, les éléments préfabriqués en usine sont, en général, liaisonnés au moyen de boulons à haute résistance dont l'enlèvement ne pose, en principe, guère de problèmes. Même avec d'autres dispositifs de liaison tels que les rivets, le démontage ne présente pas de sérieuses difficultés.

Dans le domaine de la construction métallique, on dispose donc d'une longue expérience dont on a déjà profité pour réaliser des ponts et viaducs amovibles afin d'assurer la circulation pendant les travaux de construction d'ouvrages définitifs. C'est à l'acier qu'on a d'abord pensé pour fabriquer des ouvrages provisoires. D'ailleurs, qui n'a, en mémoire, le "mécano" de son enfance quand il pense à des ouvrages démontables. (fig 3).

L'acier présente l'avantage d'une grande résistance, notamment à la traction, permettant de réduire l'épaisseur, donc le poids des éléments et de bien s'accommoder de liaisons locales facilement amovibles.

Même dans le cas d'ouvrages pour lesquels la démontabilité n'avait pas été voulue, tels que les buses métalliques, cet avantage a été ajouté à ceux
Poutres préfabriquées
Dalles préfabriquées
Entretoise préfabriquée
Câblés de precontrainte

Fig 4
qui avaient été recherchés (rapidité d'exécution, souplesse, etc...).

5.2 Tabliers constitués de poutres et dalles préfabriquées renforcées et réunies par précontrainte transversale amovible. (fig 4, 5, 6).

En supprimant les contraintes de traction, la précontrainte a libéré les constructions en béton de l'obligation d'avoir leurs éléments reliés par des armatures adhérentes et des collages.

Pour que ces constructions soient aisément démontables, il suffit, dès lors, que les éléments constitutifs soient transportables et reliés par une précontrainte amovible c'est-à-dire, par exemple, réalisée par des câbles dont les gaines injectées ne sont pas solidaires de la construction.

Différents types d'ouvrages devenus classiques, se prêtent fort bien à cette solution, notamment ceux dont les tabliers sont constitués de poutres et de dalles préfabriquées renforcées et réunies par précontrainte transversale amovible.

Cette solution de démontabilité n'augmente pratiquement pas le coût de l'ouvrage et peut donc être généralisée sans investissement à rendement problématique.

On objectera qu'en supprimant l'adhérence des câbles à la structure, on réduit la sécurité de celle-ci mais, il convient de remarquer que la possibilité de vérifier et de remplacer aisément ces câbles est par contre favorable à la sécurité. Il y a donc une certaine compensation.

5.3 Tabliers constitués de voussoirs préfabriqués reliés par précontrainte longitudinale réalisée par des câbles extérieurs amovibles.

Les tabliers de multiples viaducs et ponts sont constitués par des voussoirs préfabriqués reliés par précontrainte longitudinale. Celle-ci peut être réalisée, comme dans le cas cité plus haut, par des câbles intérieurs dont les gaines enfilées dans des ouvertures cylindriques légèrement plus larges, n'adhèrent pas à la structure, et aussi par des câbles dits extérieurs, c'est-à-dire qui ne se trouvent pas dans la masse de béton des âmes et des dalles des caissons, mais bien en dehors de celles-ci.

Les câbles sous gaine injectée ne sont en contact avec la structure qu'aux ancrages et aux déviations sur entretoise. Ils sont donc aisément amovibles.

5.4 Dalles de platelage en éléments préfabriqués fixés aux poutres par des boulons. (fig 7).

Dans le cas des tabliers plus étroits, la précontrainte transversale est moins économique. La dalle de platelage peut être constituée d'éléments préfabriqués d'un seul tenant sur toute la largeur du tablier et reliés aux poutres par des boulons avec des dispositifs s'inspirant des fixations des rails de chemins de fer sur les traverses en béton.
Câble de précontrainte

Double gaine

Poutre caisson préfabriquée

Fig 6
5.5 Eléments préfabriqués reliés par l'intermédiaire de profilés métalliques boulonnés entre-eux.

Pour profiter de l'aptitude des structures métalliques aux liaisons amovibles, on peut, aux joints, disposer des profilés métalliques encastrés dans le béton et dont la partie en saillie peut être boulonnée au profilé fixé à un autre élément en béton.

5.6 Eléments préfabriqués reliés par armatures d'attente et béton pouvant être démolis sans trop de difficultés et sans dégradation exagérée des éléments.

La liaison par armatures d'attente et béton présente les avantages du monolithisme et d'un coût modéré pouvant peser lourd dans un bilan prenant d'autre part en compte les inconvénients d'une démolition locale quelque peu laborieuse, délicate et désagréable pour l'environnement.

Si cette liaison est conçue de manière à limiter suffisamment les risques de dégradation exagérée aux éléments à réutiliser, on peut considérer la construction comme passablement démontable.

5.7 Infrastructures et murs de soutènement.

La "terre armée". Les poutres préfléchies.

Les tabliers de ponts sont soumis à des charges extérieures relativement modérées et on s'efforce de réduire le poids mort qui sollicite parasitairement la structure.

Il n'en est pas de même des infrastructures et murs de soutènement dont le poids est favorable à la stabilité et qui sont, en général, soumis à des poussées considérables.

Il en résulte que ces constructions sont, la plupart du temps, extrêmement massives, que leur démolition en est excessivement laborieuse et qu'il ne peut plus être question de parler de démontage si on n'a pas pris, au départ, des dispositions tout à fait spéciales.

Pour rendre ces infrastructures et murs de soutènement démontables, on peut avantageusement recourir à la solution de la "terre armée".

Il est ainsi possible de réaliser un pont complet comprenant:
1) des culées en terre armée avec sommier en béton armé transportable;
2) des murs de soutètement également en terre armée;
3) un tablier constitué de poutres et dalles préfabriquées avec liaisons amovibles;
4) des équipements démontables (appuis, joints, garde-corps, etc...).

Le problème du surcroît de portée lié à l'emploi de la terre armée pourra être résolu élégamment par l'utilisation de poutres préfléchies quand la hauteur disponible sera limitée. Les poutres fort résistantes sont, de plus, particulièrement aptes à supporter des mouvements de support et des manutentions. De plus, leur longueur de vie est pratiquement illimitée.

(fig 5).
5.8 Les équipements amovibles: appuis, joints de dilatation, dispositifs de sécurité, dispositifs d'évacuation des eaux, étanchéité.

Les équipements sont, en général, les éléments les plus vulnérables des ponts et leur durée de vie est fort courte.

À eux seuls, les remplacements de joints représentent près de la moitié du coût de l'entretien des ouvrages.

Il est donc souhaitable que les équipements soient amovibles, même quand ils ne sont pas placés sur des ouvrages démontables.

Les équipements étant, pour une bonne part en acier, cette démontabilité ne pose guère de problème.
AIMS AND POSSIBILITIES OF DEMOUNTABLE STRUCTURES FOR INDUSTRIAL USE

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SUMMARY

At present, industrial organisations do not think in terms of demountable structures. As lifecycles of industrial processes are becoming shorter, there is an increasing demand for versatility of housing accommodation, including the structural elements. Furthermore building installations have to be replaced within a much shorter time than the lifetime of the building, and this again requires the possibility of demounting and adapting structural elements.
Demountable concrete structures: it is obvious that from the viewpoint of an industrial company this is not the first point of attention when a new building has to be erected.

Just as an industrial product also a building has a limited lifetime. The difference is, that an industrial product is mostly used for one purpose, whereas a building has to serve more purposes during its lifecycle.

A factory, office building or laboratory starts its life, because it is needed for a well defined purpose. The design is made on the base of a program of requirements, and though a certain versatility is always part of the program, rather soon after the completion of the facility it is used in a different way than where it was planned for.

A consequence is, that the functionality decreases and also the efficiency in the operational use. As the costs of maintenance and all other costs related to the use of the building remain the same, the relation between the housing costs and the operational result are upset. This process goes on for a time, and the situation deteriorates. Depending on the type of use the building remains unchanged for a period of time, but never till the end of its technical lifetime (exceptions are monumental buildings as churches, townhalls, operabuildings, but I am talking about industrial housing facilities).

At a certain moment it is no longer feasible to use the building in its original state, and then an adaption process starts.

This is done in different ways. Sometimes the building is devaluated: a factory becomes a warehouse, a high-value-production is replaced by a simple production etc.

The other possibility is to restructure the building, but for this the possibilities are very poor today, and that is where the difficulties start.

In more recently built facilities partitions can be removed, but to change the structure or the walls some real demolishing work has to be done, and in most cases it is more attractive to pull down the whole building and erect a new one instead of investing a considerable amount of money in the existing one.

Summarising so far the conclusion is, that from the point of view of the industrial use of a building there is indeed a challenge for demountable structures, on the condition that with the parts of the structure the building can be rebuilt or adapted to another way of use.

Important to be aware is, that changes in the industry are coming faster today.

A normal lifecycle for a production type in the electronic field is three to five years. This means, that the profitability of such a project is very short.

Mostly a series of the same kind of productions can take place in the same environment, but after two to three years changes in floor plan, production equipment, organisation are coming already.
A general experience is that within twenty years the production has changed completely, and the building is obsolete. Only by investing a considerable amount of money the building can be kept operational for new types of production.

So there are important economic reasons to have demountable structures. Till now we are used to lifecycle times of 35 - 50 years, and our administration people keep their depreciation terms based on this, realising that during this time at least 2 to 3 times more or less drastic changes are necessary.

In most cases this is justified, specially because most industrial buildings are part of an infrastructure, and even when the costs of adaption are of the same magnitude as the cost of a new building, maintaining the relation with this infrastructure is so valuable, that the adaption is preferred.

One other point asks attention. The level of installation-provisions in industrial housing facilities is increasing fast. A modern plant for integrated circuits is qualified for a great deal by the installation equipment for airconditioning, utilities etc. And this equipment has a lifetime which is never longer than 25 years. Mostly within this period parts have to be replaced already, and it means anyhow, that long before the end of the technical lifetime of the building the installation equipment has to be replaced. This always means that also the structure of the building has to be adapted. At this moment it often happens that a building is just pulled down, because the newly required installation equipment cannot be placed.

General conclusion is, that at this moment there is a discrepancy between the lifetime of our industrial buildings and their possible use. Versatility not only of the partitioning but also of the structure as a whole would be welcome, and would also increase the economic value of the facility. Demountability and re-use should go hand in hand, and should include the main mechanical and electrical installation equipment.

In my lecture I will give examples of the changes in the use of buildings, the efforts required to do this, of the relation of a building with its environment and of the increasing reduction of lifecycle times in industrial processes.
DEMOUNTABLE AND RE-ERECTABLE PRECAST REINFORCED CONCRETE BUILDINGS

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SUMMARY

From 1967 to 1968, W. Hasslinger constructed three medium-sized precast reinforced concrete office buildings in cooperation with "Südbau-Schleussner" in Vienna. In 1980, these buildings were dismantled, as contractually scheduled, in a non-destructive manner and re-erected in the towns of Moedling and Traiskirchen.

This experiment in dismantling and re-erecting prefabricated buildings was successful - for the first time in Europe.

This paper reports on general principles, economic questions and experience, 18 years after the introduction of such structures. It also makes a forecast of the future of concrete buildings in towns.


Ein von mir bereits 1965 entwickeltes kombiniertes Skelett- und Tafelsystem, für Schulgebäude wurde durch spezielle Gestaltung der Verbindungsmittel demontierbar gemacht.

Bei der obigen Ausschreibung war dem Anbieter die Wahl des Bau- stoffes freigestellt.

Eingereicht wurden 8 Angebotsprojekte 2 in Holzkonstruktion, 2 in Stahlkonstruktion, 4 Betonkonstruktionen. Mein Betonfertigteilentwurf wurde für die Ausführung ausgewählt.

Die Ausführung erfolgte, die Planungsleistung inbegriffen innerhalb von 5 Monaten nach Auftragserteilung.


Inzwischen möchte ich Ihnen einige Fotos der Gebäude nach Ersterrichtung und nach deren Umsetzung zeigen.


Letztlich möchte ich eine kurze Vorausschau in die Zukunft von Betonbauten versuchen.

1.) ANALYSE DER UMSTÄNDE WELCHE 1967 ZUR ERSTMALIGEN ERRICHTUNG VON DEMONTIERBAREN BETONBAUTEN IN WIEN FÜHRTE

a) Die Republik Österreich hatte in sehr kurzer Frist Büroräume für die Unido-Organisation in Wien zur Verfügung zu stellen. Da nur ein Teil dieses Bedarfs durch Verwaltungsgebäude der Stadt Wien in Rathausnähe gedeckt werden konnte, mußten in deren Nähe
Fig. 1 Unido - Büro - Block I, Wien, Montage (Stiegenhaus), Juni 1967

Fig. 2 Unido - Büro - Block I, Wien, nach Fertigstellung, Oktober 1967
Fig. 3 Unido - Büro - Block I + II + Verbindungstrakt, Wien, nach Fertigstellung, Oktober 1967

Fig. 4 Ankündigung der Umstellung, Herbst 1980, Wien, Demontage bereits erfolgt
Neubauten schnellstens errichtet werden.

b) Nur ein geeigneter Bauplatz in entsprechender Nähe stand hierfür zur Verfügung; dieser konnte nur durch eine Ausnahmegenehmigung vorübergehend bebaut werden, da er gemäß Flächenwidmung als Park vor einem wichtigen historischen Gebäude vorgesehen war.

c) Diese Bedingung konnte mit der für 5 Jahre vorgesehenen Nutzung des Büros in Übereinstimmung gebracht werden.

d) Gefordert wurde die Bauqualität eines modernen Bürogebäudes mittleren Standards.

e) Baracken waren aus diesem Grund und wegen der städtebaulich bevorzugten Lage unzulässig.

f) Für den Bauherrn (Republik Österreich) war es aus volkswirtschaftlichen Überlegungen unzumutbar eine relativ neuwertige Bausubstanz nach 5 Jahren abzutragen.

g) Schließlich mußte einer allgemeinen Erfahrung über die Langlebigkeit von provisorischen Bauten, glaubwürdig entgegen getreten werden.

h) Dies waren die Gründe, warum vom Bauherrn demontierbare Gebäude verlangt wurden.

i) Daß es sich hiebei um ein Experiment handelte, war allen Beteiligten bewußt.

2.) SÜDBAU-SCHLEUSSNER WURDE BESTBIETER

Mit meinem Beton-Fertigteilentwurf wurde Südbau-Schleussner Bestbieter und erhielt den Auftrag zur Ausführung von 2 Büroblocks mit Verbindungsbau im Umfang von ca. 22.000 m³ umbautem Raum.

Für die Auswahl des Südbau-Schleussner-Projektes war außer dem günstigen Preis, die Betonbauweise und die Glaubwürdigkeit der Demontierbarkeit ausschlaggebend.

Der Auftrag umfaßte die Generalplanung und schlüsselfertige Herstellung. Die Gebäude wurden genau 5 Monate nach Auftragserteilung schlüsselfertig übergeben.

Der erste Teil des Experimentes war gelungen.

Für die gleiche Organisation wurde ein Erweiterungsbau von ca. 8.000 m³ umbautem Raum erforderlich und dieser problemlos 1968 von Südbau-Schleussner ausgeführt.

3.) DEMONTIERBARE SCHUL- UND INSTITUTSBAUTEN

In der Zeit von 1967 - 1972 bestand in Österreich ein enormer Bedarf für Schulbauten. Ursache hiefür: die wachsende Kinderzahl, sowie die Erfüllung eines Nachholbedarfs nach dem 2. Weltkrieg, ebenso wie die örtliche Verlagerung der Schülerzahl durch das Ent-
Fig. 5 Wieder errichteter (Teil) von Block I in Mödling, 20 km südlich von Wien, Gendarmerie-Schule fertig übergeben 1981
stehen großer neuer Wohnhausanlagen, die mit jungen Familien be-
siedelt wurden.

Wie eingangs erwähnt, habe ich bereits 1965 einen Typenentwurf für vorgefertigte Schulen entwickelt.

Offensichtlich bedingt durch den Erfolg bei der Errichtung der ersten umstellbaren Bürogebäuden, erfolgte 1969 eine Ausschreibung des Bautenministeriums für ganz Österreich, für demontierbare Schul-
gebäude.

Die Besonderheit dieser Ausschreibung bestand darin, daß eine mehrmalige Demontage und Wiedererrichtung innerhalb von 15 Jahren gefordert wurde. Offensichtlich rechnete man damals mit einer ständig steigenden Schülerzahl und einer starken örtlichen Fluk-
tuation.

Die Ausschreibung verlangte diesmal nur Stahlbetonkonstruktionen, da kurz vorher negative Erfahrungen mit Stahlkonstruktionsschul-
gebäuden gemacht wurden.

Südbau-Schleussner war auch bei dieser Ausschreibung mit gleichem Konstruktionsprinzip erfolgreich. Vorgegeben wurde ein Raumpro-
gramm mit Ausstattungskatalog. Die Detailplanung oblag dem Bieter.

Das Leistungsverzeichnis war gegliedert in
a) Planung und schlüsselfertige Herstellung
b) Demontage
c) Transport (für mehrere Entfernungskategorien)
d) Wiedermontage einschließlich Nachbeschaffung der bei der Demon-
tage zerstörten oder beschädigten Teile und Einrichtungen.

Von 1969 bis 1973 wurden insgesamt 8 Schul- und Amtsgebäude im Um-
fang von ca. 60.000 m³ umbauten Raumes von Südbau-Schleussner aus-
geführt.

Umsetzungen dieser Gebäude erfolgten bislang nicht.

4.) ERFAHRUNGEN BEI DER UMSETZUNG DER UNIDOGBÄUDE


In diesem Jahr wurde der Auftrag für die Demontage erteilt, da der besagte Bauplatz nicht mehr länger zweckentfremdet benutzt werden durfte.

b) Bei der Auswahl von Projekten für die Wiedererrichtung ergaben sich Probleme.

c) Bei der Auftragserteilung wurde vertraglich vereinbart, daß die genau gleiche Wiedererrichtung auf einem anderen Bauplatz erfolgen soll. Weder gab es den Bedarf für Bürogebäude, noch den geeigneten Bauplatz für die Wiedererrichtung gemäß Erstplanung.
Fig. 6 Wieder errichteter Block II in Mödling mit Rest von Block I in wesentlich geänderter Grundrissanordnung (abgewinkelt), fertiggestellt März 1982

Fig. 7 Stiegenhaus – Ansicht innen Block II in Mödling nach Wiedererrichtung
d) Benötigt wurde eine Schulbauerweiterung der Gendarmerieschule in Mödling und ein Wohnobjekt als Erweiterung zu einem Gebäudekomplex in Traiskirchen, beides Städte in einer Entfernung von 20 bzw. 30 km vom ursprünglichen Standort.

Für das Wohnobjekt war der Block III vorgesehen, die Umplanungserfordernisse waren geringfügig.

Block I und II waren für die Schulbauerweiterung vorgesehen.

Erhebliche Umplanungen waren erforderlich. Die eine Bauplatzgröße gestattete nicht die Umsetzung eines Blockes in dessen ursprünglicher Länge, der andere Bauplatz konnte das übrige Raumvolumen aufnehmen, erforderte jedoch eine Ausführung in einer abgewinkelten Bauform.

Umplanungsprobleme gab es im Klassentrakt. Die gewünschte Klassentiefe von 6,5 m war durch den 5 m Raster der Bürogebäude nicht erreichbar. Eine Anpassung an das Rastermaß ergab dennoch eine befriedigende Lösung.

e) Innerhalb der 12 Jahre nach Ersterrichtung haben sich einige baubehördliche Bestimmungen geändert, hauptsächlich bezüglich der höheren Anforderungen an die Wärmédämmung der Außenwände und örtliche Bebauungsbestimmungen.


g) Durch die Probleme mit dem Wiedererrichtungsprojekt ergab sich die Notwendigkeit, der Zwischenlagerung der demontierten Bauteile, die normalerweise hätte vermieden werden können.

Schließlich wurden die 3 Gebäude zur Zufriedenheit des Bauherrn und der Nutznieder (und der Baufirma) 1981 übergeben.

5.) SCHLUSSFOLGERUNG AUS DER ERFAHRUNG MIT DIESEM PROJEKT

a) Die Wiedererrichtung von demontierbaren Gebäuden in genau gleicher Weise wird in den seltensten Fällen gegeben sein, am wahrscheinlichsten ist dies noch bei den Schulbauten zu erwarten oder im Falle von demontierbar errichteten Wohnbauten.

b) Im Fall der Umstellung nach längerer Zeit, müssen Anpassungen und Umänderungen durch baubehördliche oder sonstige technische Bestimmungen in Betracht gezogen werden und sind dementsprechende Umplanungen unvermeidlich. Dies ist ein zu berücksichtigender Kostenfaktor.
Fig. 8 Wieder errichteter Block III in Traiskirchen, 30 km südlich von Wien, übergeben und fertiggestellt 1981. Fensterparapete und Fensterpfeiler künstlerisch gestaltete Flachreliefs in Betonfertigteilkonstruktion.

Fig. 9 Teilausschnitt Fassade am umge- stellten Block III in Traiskirchen.
6.) ALLGEMEINE WIRTSCHAFTLICHE ÜBERLEGUNGEN

Eine wesentliche Bedingung der Ausschreibung und eine dementsprechende Bestimmung des Auftrages war die Verpflichtung zur Demontage und Wiedererrichtung zu den bereits im Angebot festgelegten Einheitspreisen jedoch mit Indexanpassung.

a) Das Leistungsverzeichnis war wie folgt gegliedert:

Planungs- und Errichtungskosten ohne Fundierung 76,5 %
Demontagekosten 7,5 %
Transportkosten 2,5 %
Kosten der Errichtung, einschließlich Nachschaffung von zerstörten oder beschädigten Bauteilen, Einrichtungen und Installationen 13,5 %

Ausgenommen hiervon waren, Wiederbeschaffung von Bodenbelägen, Maler- und Anstreicherarbeiten, Passadenanstrich, Dachhautneuerung und Spenglerarbeiten. Es sind dies jene Leistungen, welche normalerweise als Instandsetzungsarbeiten bei jedem Bau erforderlich sind, besonders in diesem konkreten Fall, nach 12-jähriger Bestanddauer in welcher überhaupt keine Erhaltungsarbeiten vorgenommen wurden.

b) Der Anteil von Bruch und Beschädigungen an Fertigteilen und sonstigen demontierten Bauteilen und Einrichtungsgegenständen betrug insgesamt nur 4 % der Herstellungskosten dieser Teile.

c) Die Demontagekosten betrugen ca. 5 % und waren geringer als kalkuliert. Eine wesentliche Demontagehilfe war der Weichkalkmörtelvergüß, welcher nach Anheben der Teile selbst abfiel. Das Lösen der Stahleinbauteilverbindungen erfolgte problemlos durch aufschneiden der Schweißverbindungen und Lösung der Schraubenbolzen.

7.) KOSTENÜBERLEGUNGEN VOM STANDPUNKT DES BAUHERRN

Der Bauherr hat nach Abrechnung festgestellt, daß die Wiedererrichtungskosten dieser 3 Gebäude im Durchschnitt 65 % der vergleichbaren Neubaukosten im Jahr 1980/81 betragen haben.

Hiezufolgende näherungsweise Aufgliederung:

a) Wiederverwendung der demontierten Bausubstanz zum seinerzeitigen Auftragspreis = 76,5 %

b) Preiskomponenten für:

Demontage 7,5 %
Transport 2,5 %
Wiedererrichtung 13,5 %

23,5 %

Zuzüglich Baukostenindexsteigerung von 162,4 %

Multiplikator 2,624 x 23,5 %

= 61,6 %

138,1 %
c) Der vergleichbare Neubaupreis im Verhältnis zu 1967 beträgt:

\[
76,5 \times 2,624 = 200 \%
\]
zuwändig Arch., Ing. Leistungen + Bauleitung 9 \% = 18 \%

Kostenvergleich \( \frac{138,1}{218} = 0,633 \text{ ca.} \)

63 \%

d) In diesen Kosten sind die Fundierungskosten ebenso nicht enthalten wie die Kosten für höhere Wärmedämmung und bessere architektonische Gestaltung.

Somit ist die Wirtschaftlichkeit dieser Bauweise in diesem Projekt anschaulich gemacht. Es ergibt sich die Frage, ob diese Ersparung genügend Anreiz für weitere Projekte dieser Art bietet.

8.) ÜBERLEGUNG VOM STANDPUNKT DER BAUNTERNEHMUNG

Trotz der Umplanungserfordernisse und der Zwischenlagerung der Bauteile war der Auftrag aus mehreren Überlegungen interessant.

a) Der Auftrag war in der Zeit eines enormen Rückganges von sonstigen Fertigteilaufträgen sehr willkommen.

b) Durch die erfolgreiche Wiedermontage wurde die Sinnhaftigkeit der Bauweise bestätigt und sollt des zu weiteren Umsetzungen Anlaß geben (Schulobjekte).

c) Durch den seit mehr als 5 Jahre anhaltenden Verfall der Baupreise, zufolge von Auftragsrückgängen und härtestem Wettbewerb, war der Auftragspreis für die Wiedermontage durch die Indexanpassung kostendeckend uns somit preislich günstiger als ein Neubaupreis.

Durch den tatsächlich geringen Abfall und Bruch, sowie durch Ersparungen bei der Demontage war dieser Baufaktor insgesamt lohnend geworden.

9.) ZUSAMMENFASSUNG UND AUSBLICK

Schon bei der Einführung der Demontagebauweise vor 18 Jahren, wurde darauf hingewiesen, daß eines Tages die Nutzungsdauer der enormen Betonbaumanmassen, die in fast allen Städten errichtet wurden, ein Ende finden wird. Dann sind diese Betonbauten abzutragen.

Die schon damals erkannte Umweltbelastung durch Abbruch, Staub, Lärm und Schmutz hat heute eine wesentlich größere Bedeutung erlangt. Daher ist das Abtragen von größeren Betonbaumanmassen heute kaum mehr vorstellbar und durchzusetzen.

Wäre ein Teil dieser Betonbauten bereits früher demontierbar ausgeführt worden, hätte dies ein bedeutender Beitrag zum Umweltschutz sein können, selbst dann, wenn nur die Demontage erfolgen würde.
Aus heutiger Sicht betrachte ich die Wiedererrichtung von Beton­bauwerken in ähnlichen Ausmaßen und Größenordnungen wie bisher, für nicht wahrscheinlich und wünschenswert.


Zum Beispiel:

Kosten für die schlüsselfertige Herstellung von 1 m² Wohnnutzfläche in Österreich 1984

<table>
<thead>
<tr>
<th>Kostenbereich</th>
<th>Min. Kosten</th>
<th>Max. Kosten</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wohnnutzfläche</td>
<td>S 10.000,--</td>
<td>S 12.000,--</td>
</tr>
</tbody>
</table>

ohne Grundstückskosten und Nebenkosten

Bei einer Einsparung von nur 20 % würden die vergleichbaren Kosten

<table>
<thead>
<tr>
<th>Kostenbereich</th>
<th>Min. Kosten</th>
<th>Max. Kosten</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wohnnutzfläche</td>
<td>S 8.000,--</td>
<td>S 9.600,--</td>
</tr>
</tbody>
</table>

/ m² WNR

betragen.

Daraus lässt sich die Höhe der möglichen Mietkostensenkung abschätzen.


Ich sehe eine neue Herausforderung an den Bautechniker; nämlich die Sanierung zwecks Erhaltung der enormen Bausubstanz von Beton­bauten so lang als möglich.

Verbesserung der Wärmedämmung und Schalldämmung, Sanierung von Fassadenflächen und Fenstern, Verbesserung der Wiederstandsfähig­keit gegenüber Schadstoffen und Schmutz, Verbesserung der architektonischen Gestaltung der Fassadenflächen, der Großtafel­bauten der 60er und 70er Jahre, weiters die Verbesserung der Umweltbe­dingungen solcher Bauwerke im weitesten Sinn und vor allem positive Werbung für den hervorragenden Baustoff Beton, sollen nur einige Anregungen sein.

Diese Sanierungsaufgabe könnte ebenso erfolgreich werden, wie die mit großem Einsatz und mit beachtlichem handwerklichen Können in vielen Städten in Gang befindliche Althaussanierung.
SUMMARY

The capacity of Munich's airport is no longer sufficient, due to the increasing traffic. Its transfer from Riem to Erding was planned as far back as the 1960s. The conclusion of the lengthy approval procedure for the new airport could not be awaited, however, and so in 1970 additional buildings of demountable precast concrete construction were erected as stopgap measures at the old airport. These new buildings were: departure and arrival hall, luggage and equipment hall, as well as multi-storey connecting buildings.

In 1985 structural alterations to the old airport with a further extension of these buildings became necessary. During this alteration work the condition and the demountability of the 14-year-old buildings was investigated.

The construction method, details of execution, condition and demountability are described and illustrated in this paper.
The Munich Airport is of great importance to domestic and international air traffic of the Federal Republic of Germany. Due to the increasing number of flights the existing buildings of the airport always turned out to be too small again. In addition, the airport is too near to the eastern outskirts of the city. Since the sixties it has been planned to relocate the airport from Riem to Erding. The approval procedures, however, have constantly been delayed by objections. When Munich was prepared for the Olympic Games of 1972 it was no longer possible to anticipate the outcome of the lengthy approval procedures required for the absolutely necessary extension of the airport facilities. Therefore, as temporary measure, extensive makeshift annexes were built at the present airport.

In detail these annexes comprise the following structures (Fig. 1):
- extension of the departure terminals for domestic and international flights
- multi-storey airport buildings with connection buildings
- baggage distributor
- hangar
- arrival terminal

These structures comprised a total of 113,000 m³ building volume.

At that time it had been expected that these structures would be used only a few years until the new airport was built. Therefore, a demountable construction system was looked for via a design competition. Dyckerhoff & Widmann AG was awarded the contract for a demountable reinforced precast concrete structure.
Figure 1   Airport - Terminal Munich
Perspective of Enlargement 1970
Figure 2
Airport - Terminal Munich, Departure Hall

Figure 3
Departure Hall, Demountable Connection Detail 1
The construction was executed in the years 1970 and 1971. In the following the details of this construction are explained as far as demountability is concerned.

The departure terminal shows a two-field loadbearing system with cantilever and drop-in girders (Fig. 2). The prefab columns are each fixed in the foundations. The roof girders are placed in a demountable way on the columns and the cantilever girders (Fig. 3). The connection of these members is formed by screwing by means of DYWIDAG-threadbars St 90/110 dia, 15.1 mm. The anchor holes at the supports of the roof girders were not grouted with mortar as usual. The steel parts not encased in concrete were only covered by a corrosion-protection coating. This method is only sufficient if the service life of the facilities is limited as in this case. The terminal was roofed with gas concrete slabs as is common practice, and the external wall was also produced with gas concrete panels.

The arrival terminal has a four-field loadbearing system with a column grid pattern of 10.00 x 11.25 m² (Fig. 4). In this case two kinds of demountable systems centres were used (Fig. 5). The support of the girders is effected by covered brackets made of section steel to which anchor bars are welded. At the supports the girders have recesses suited for these anchor bars. After the assembly the girders are bolted with the anchor bars. The anchorage holes are not grouted at these systems centres either. In this case the steel parts not encased in concrete were also coated with a corrosion-protection compound.

The baggage distributors and hangars have prestressed concrete roof girders with a width of 32.25 m (Fig. 6). These girders had to be installed demountably too (Fig. 7). For lateral stabilization of the girders set screws were provided at the support pockets of the precast columns instead of the usual grouting of the support pockets.
Figure 4
Airport Terminal, Arrival Hall
Figure 5
Arrival Hall Demountable Connection Details

Detail 2

St 37-2, φ20mm
weld seam

Detail 3

Neoprenebearing

Neoprenebearing

I 140
Length $23 \times 5.625 = 129.375$ m

Figure 6
Baggage - and Equipment - Hall

Steel Pipe with internal thread
M 20

screwed bolt
M 20 with internal hexagon

Neoprene Bearing

Grout

Figure 7
Baggage - and Equipment - Hall, Demontable Connection Detail 4
The two-storey airport-gate building and the three-storey connection buildings were built using the same construction method as the above described buildings. All these annexes had to be built while the air traffic at the airport went on. This demountable reinforced prefab concrete construction method with ease of assembly has been proven a success.

In 1984, the planned new airport of Munich has still been delayed by objections to the approval procedures. Thus, another makeshift extension of the airport facilities was necessary (Fig. 8). The structural alteration of the departure terminal, which was erected at the first extension in 1970, is particularly interesting (Fig. 9). At first 750 m² of demountable roof structure were pulled down. This roof structure consisting of gas concrete slabs and reinforced concrete girders was pulled down by 5 men by means of a mobile crane in 3 working days. In doing so, the demountability and the state of conservation of the 14 years old structure could be checked. The ungrouted system centres were in excellent state, uncorroded and easily detachable (Fig. 10). In order to extend the headroom of the terminal from 3,00 m to 5,00 m frame-like roof girders were then placed on the old reinforced concrete columns (Fig. 11). All detached roof girders could be reused at another place. The new frame-like roof girders were connected with the old columns by the existing threadbars (Fig. 12). At some points these threadbars had to be extended by means of threaded couplings.

For the entire second alteration and enlargement measures only 8 weeks were available for pulling down and new assembly, only another 17 weeks for the turn-key interior work. On 31st May 1985 this project will have to be finished to schedule.
Figure 8  Airport - Terminal Munich
Perspective of Modifying 1984
Figure 9: Demolition of the roof structure

Figure 10: Demountable Connection after demolition of the roof structure
Figure 11, Departure Hall, Modifying 1984
This demountable reinforced precast concrete construction method applied at the buildings of the Munich airport has proven a success at the assembly, demolition and extension in a both constructive and economic way. The state of conservation of these buildings while being altered was found to be excellent after 14 years. For structures with limited service life as in this case, this construction method is thus excellently suited.
This paper deals with structures which fulfil different demands for demountability: demountability as a precondition for multiple use, demountability for adapting a structure to changing boundary conditions, and demountability for the replacement of severely stressed structural members for the sake of safety. Traditionally, demountable structures were built of steel. In order to utilize the experience gained with steel structures and to combine the advantages of the connection possibilities of steel with the short erection time of precast concrete, composite constructions appear to be most promising for the development of demountable precast concrete structures.
1. INTRODUCTION

There exist different reasons why a structure should be demountable. The demountability of structures is a fundamental parameter of design for structures which are built for a limited time only and which should be used at different sites. In this case, already the first draft of the construction can be designed according to the above mentioned demand. This refers especially to the segmentation of the construction into single precast elements, the measurements and weight of which are determined by the means of transport and the existing lifting device. In addition, attention can be drawn from an early stage to the detailed design of the connections between the different elements of the structure. Thus, solutions can be elaborated which guarantee on the one hand a multiple, fast and exact mounting and dismounting, and on the other hand a sufficient corrosion protection of the connecting elements especially in case of prestressed constructions.

The call for demountability for a structure can be based on the possible adaptability to changing boundary conditions. This aspect refers especially to industrial buildings. Often, the durability of a building is not reached when, due to the conversion of the change of the production, reconstructions of the building are necessary. In such a case, the type and the quantity of the altered demands are not known in the stage of planning and design. Here, a demountable construction may have economic advantages compared to solutions which do not allow an adaptation to the altered boundary conditions, thus the whole structure has to be demolished in order to be rebuilt again.

Normally, the single elements of a construction are stressed unequally within the lifetime of a building. A demountable construction enables the exchange of construction elements without the renewal of the whole structure in case when some construction elements have lost their serviceability or are not safe enough. In these cases one has to take care that the stability of the building is guaranteed in the design during all stages of the exchange of single construction elements. In addition the construction has to be designed in a way that the exchange can be made within limited time and with small restrictions of its use.

In the following, buildings and drafts will be presented which are planned as demountable constructions because of the different reasons mentioned above.
2. DEMOUNTABLE STRUCTURES FOR MULTIPLE USAGE

2.1 Contact bridges

Up to now, demountable concrete structures have mainly been realized in bridge constructions. Since the seventies it has been possible in this field to contrast the up to then dominating steel structures with competitive precast concrete solutions. Using the DYWIDAG prestressed concrete contact construction /1,2/ the possibilities which these bridges offer will be shown.

In order to be able to bridge over different spans with a small number of basic elements, three different superstructure elements were designed (Fig. 1):

Type 1 : Single span girder, span 20,0 m, normal concrete, weight 20 t.

Type 2 : Single span girder, span 25,0 m with one cantilever beam (5,0 m), light-weight concrete, weight 30 t.

Type 3 : Single span girder, span 30,0 m, light-weight concrete, weight 30 t.

The girder type 1 and 3 can also be supported by a notch at the cantilevered end of the girder type 2. Fig. 1 illustrates an example of a bridge with three spans with variations of support positions and spans are possible if one takes these basic types.

The cross sections of the single girder are 0,75 m in width and 0,90 m in height with a circular cavity of 0,50 m in diameter in the center in order to decrease the weight. In transverse direction, the different girders are stressed together by tendons ø 26 mm, St 835/1030 at intervals of 0,50 m. The different elements come into contact only in the upper region of the girder at a height of 0,20 m.

When, in Munich, the inner circle was constructed in 1971/72, the Brudermühl Bridge and the Schäftlarn Bridge were built with the above mentioned demountable precast elements. Fig. 3 illustrates the column position and the use of the different girder types at the Brudermühl Bridge. In order to attain a short construction time, precast elements which by means of prestressing were connected and form a frame system, were used for the column piers and column beams. At the columns near to the main opening, the beams were fixed below the superstructure. In the remaining columns, the beams reached into the superstructure because of a notch of the girder ends (Fig. 2). In transverse direction 10 superstructure elements are arranged side by side and they are stressed together (Fig. 2). Starting from both abutment, mounting works and prestressing works are done alternately. In order to fix the final precast elements above the main opening, both cranes stood on the cantilever arms of the neighbouring spans. It took one day to fix one span, thus, the total mounting time of the 190 m-long bridge amounted to 8 days.
Fig. 1  DYWIDAG System girder types

Fig. 2  Brudermühl bridge, Munich Cross section

Fig. 3a  Types of support

Fig. 3  Brudermühl bridge, Munich Longitudinal section
2.2 Antenna tower

In 1977, a 50 m-high antenna tower was tendered out as a demountable precast reinforced construction in Germany. The offer for a turn-key mounting was given to DYWIDAG. The tower was erected on an in-situ concrete foundation and was made of 16 rolled concrete tubes (inner diameter 2.46 m) the concrete strength classification of which is B 65 (Fig. 4) /3/. The wall thickness of the tubes is 0.26 m. The tubes were stressed together by vertical prestressing tendons (ø 36 mm, St 1080/1230) without bond. The parabolic antennas were fixed on five precast platforms which are supported by consoles at the outside of the tubes. A symmetric arrangement of the prestressing tendons around the circumference was not possible due to the existence of doors, antenna openings and recesses for the sockets and end anchorage of the reinforcement (Fig. 6). As a determining load case, a wind load of 1.45 KN/m² had to be considered. The parabolic antennas (max. diameter 3.70 m) which had to be taken into account in the calculation, caused 2/3 of the wind moment if the wind direction was most unfavourable. In order to cover the largest moment, 24 prestressing tendons are necessary at the bottom. The gradation of the tendons along the tower height as well as the arrangement of the socket joints are represented in the schematic drawing of the prestressing reinforcement (Fig. 5). As for the recesses for the end anchorage one had to take care, that the pulled out excess length of the tendons should not be cut in order to allow a later dismounting.

The antenna tower was erected in January/February 1978. At that time, all required equipments with the exception of the electric installation had been installed into several tubes. After the mounting of the third tube and then after every two tubes, the prestressing tendons were threaded from above, anchored and prestressed. With the exception of the joint or anchorage region, the tendons were grouted in the factory in smooth metal sheaths in order to attain a permanent corrosion protection. The corrosion protection of the joints and the anchorages was executed in situ after mounting (Fig. 7). In spite of the bad weather conditions, it was possible to mount two tubes per working day on an average. The entire construction time from the beginning of the technical preparations to the end of the mounting took only 120 days.
Fig. 4  Demountable antenna tower

Fig. 5  Schematic arrangement of the prestressing reinforcement
Fig. 6  Position of the prestressing tendons
Anchorage in the foundation

Fig. 7 Details of the prestressing reinforcement
3. DEMOUNTABLE CONSTRUCTIONS WITH ALTERNATING DEMANDS

As to industrial buildings, an adaption of the construction due to the change of the production is often required. These changes refer to the arrangement of openings, the loads and load positions. The altered demands, however, are usually not known when the construction is drafted. Thus, up to now, such industrial buildings have been erected as steel constructions which guarantee a good flexibility due to the possible screwed or welded joints. With regard to the structural fire protection, expensive additional measures are required for pure steel constructions in contrast to reinforced concrete structures. Composite structures represent the best solution for buildings which should be adaptable and which should also suffice the enhanced demands for the structural fire protection. Remarkable examples for this kind of construction have been erected during the last few years by Stahlbau Lavis, Offenbach.

3.1 Opel varnishing plant

The carcass of the new 405 m-long, 80 m-wide and 32-m high Opel varnishing plant in Rüsselsheim was erected in 79/80 within nine months. The loads are carried from the composite slab (0.20 m thick) to the continuous composite beams of the transversal frames. The three one-storey transversal frames (Fig. 8) which are above each other, consist of one frame column and four socketed columns. The flexibility of the composite construction becomes clear by regarding the transversal frame in axis 21 (Fig. 9) with the numerous intermediate floors. The columns were realized as steel columns with fire protection (column joint see Fig. 10) because composite columns which would have been the more economic solution could not be constructed due to time limitations. For fire protection, the largest part of the floor girders were sprayed with shotcrete. As a damage of the protection layer should be prevented even in case of conversions during the production, the following solution for the suspension of the crabs and greater loads was provided: 50 mm lining plates were welded to the composite beams at a distance of 2.50 m. After the application of the 25 mm protection layer, the substructure could be fixed at the projecting lining plates (Fig. 11).

3.2 BMW motor-cycle production hall

A new production hall for the assembly of motor-cycles type K 100 was built by BMW. During the planning of the building, which is 120 m long, 60 m wide and 16 m high, great importance was attached to the fire protection. Due to a new conception for the fire protection which mainly applies constructive protection measures, a part of the steel construction could
remain unprotected. In transversal direction, the statical system of the Opel varnishing plant was used. The downstand beams of the composite girder were filled with concrete at the sides and were strengthened by two fire protection butt straps (Fig. 12). Thus the equipment can be directly fixed at the unprotected top boom girder. The socketed columns of the building are also realized as composite constructions.

Fig. 8 Opel varnishing plant, Rüsselsheim
Transversal frame in axis 13 (normal case)

Fig. 9 Opel varnishing plant, Rüsselsheim
Transversal frame in axis 21
with additional intermediate floors
Fig. 10  Opel varnishing plant, Rüsselsheim
Column joint

Fig. 11  Opel varnishing plant, Rüsselsheim
Fire protection

Fig. 12  BMW Factory, Berlin
Fire protection
4. CONSTRUCTIONS WITH DIFFERENT LEVELS OF STRESS IN THE SINGLE CONSTRUCTION PARTS

Drafts of the consulting engineers König and Heunisch for the engineering competition about the Werra valley bridge in Hedemünden represent examples of constructions, single construction parts of which are stressed especially high, thus, reaching the operating strength faster than the other loadbearing members.

4.1 Werra valley bridge Hedemünden, one bridge solution

As loadbearing system, the one bridge solution /6/ provides an arch with four single-cell-superstructures made of prestressed concrete. Due to the arrangement of the superstructures for the road-traffic above the railway superstructures, there results a two-storey traffic-handling (Fig. 13). The arch is built as a cantilever with additional auxiliary suspensions. In a next step, the hollow piers are erected by sliding form construction method. As to the superstructures the mounting is planned to be realized by a 'stroke-push-construction method' (Fig. 14). After the complete mounting of the superstructures, the filling of the so-called 'sliding shelter' is planned. If in the course of time, the superstructure loses its serviceability, the sliding shelter can be uncovered and the superstructure can be rebuilt and pushed from behind (Fig. 15). At the other abutment, the superstructure which has not enough bearing capacity can be dismounted at the same time when the pushing works are done. At this time the entire traffic on the railway is transferred to the remaining superstructure.

4.2 Werra valley bridge Hedemünden, two bridge solution

In addition to the one bridge solution, a draft was elaborated with two five-spans beam bridges, one for the railway and another for the road-traffic with a distance of 58.0 m. This solution /7/ offers the advantage that no complicated disentanglement construction for both traffic systems is necessary. The road bridge is of special interest because it is planned as a substitution and enlargement of the existing composite bridge. The superstructure of the new road bridge which consists of two double-webbed composite cross profiles with constant height, should be beared by the existing piers of the old bridge which have enough safety for the enhanced load. Due to the larger web-distance, an enlargement of the pier heads is required. As for the traffic handling during the construction time, two alternatives were elaborated. In alternative I, the transference of the highway traffic on the first new superstructure which is constructed on auxiliary piers parallel to the old bridge is provided for. In a next step, the old superstructures can be dismantled and the pier heads can be modified. The second superstructure is built in its final position. After the transference of the traffic to
the second superstructure, the first superstructure can be shifted in transversal direction from the auxiliary piers to its final position (Fig. 16).

If a superstructure has to be renewed without a transfer of the traffic to one superstructure during the construction period, the above explained procedure can be applied for an exchange of an entire superstructure.

In alternative II, the highway traffic is transferred to the preliminary mounted railway bridge which gets a temporary cover consisting of a reinforced concrete slab. After the dismantling of the old highway superstructure and the modification of the pier heads, the new superstructure can be built at its final position. This alternative requires a harmonization of the time schedules of the both bridges.

Fig. 13 Werra valley bridge (one bridge solution) Cross section
Fig. 14  Werra valley bridge  (one bridge solution)
Erection stages
Fig. 15  Werra valley bridge  (one bridge solution)  'sliding shelter'
Mounting of the first superstructure on an auxiliary pier.

Enlargement of the pier head of the old pier; dismantling of the old superstructures; mounting of the second new superstructure.

Transversal move of the first new superstructure.

Fig. 16  Werra valley-bridge (two bridge solution)  Erection stages, road bridge
5. CONCLUSION

Up to now, the different construction parts of demountable reinforced concrete structures have been stressed together by using prestressing tendons. As for bridge constructions and special structures, this represents a good possibility to build demountable constructions even in the future. An application for the normal structural engineering is less promising if one takes into account the necessary dimensions for the anchorage and joint elements and the large amount of erection work. Thus, for structural engineering and industrial constructions a composite structure consisting of composite slabs, composite downstand beams and composite columns, represents the better solution because the connection of the single parts by screws and welding allows a faster mounting and a greater flexibility. In contrast to pure steel constructions such a solution offers economic advantages and a better fire protection.

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UNSTABLE VIBRATIONS OF DEMOUNTABLE CONCRETE BOX GIRDERS

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SUMMARY

The paper is concerned with an attempt to apply the Finite Strip Method to the description and analysis of the unstable vibrations of simply-supported iso- and orthotropic demountable concrete box girders. The advantage of the method consists in using orthogonal series for the longitudinal approximation leading to the separation of the equations of equilibrium for the particular harmonics, resulting in considerable economy of calculation effort in comparison with the Finite Element Method.

The analysis takes into consideration the bending state as well as the in-plane forces; the calculations were based on dimensionless co-ordinates, which allowed a wider-ranging and more general analysis.

It is exemplified by a concrete box girder bridge span for one symmetrical loading arrangement in the form of a vertical concentrated force moving at a constant speed in the longitudinal direction of the structure. At the same time the analysis takes account of variable speed and acceleration, as well as various values and damping hypotheses.
1. INTRODUCTION

Spans of demountable bridges made of concrete elements joined with bolts are of different character when comparing their work with that of prefabricated or monolithic spans. It results from a specific way the elements are joined among each other which, as a rule; does not bring about lateral cooperation of all main beams. Moreover, spans of this type have no pavement and the traffic takes place directly on the demountable elements. Direct transfer of load from wheels on bearing elements of a span makes a substantial difference in a dynamic work of such bridges in comparison with identical monolithic constructions.

In this paper an attempt has been made for application of Finite Strip Method /FSM/ [4], [6] to the description and analysis of the unstable vibrations of the iso- and orthotropic demountable concrete box girders simply supported used for bridges. Especially structures of constant cross-sections and mechanical parameters over their whole length can be analysed with the aid of the above-mentioned method.

The advantage of this method is making use of the orthogonal series to the longitudinal approximation leading to the separation of the equations of equilibrium for the particular harmonics which allowed for remarkable economy time of calculations in comparison with the classical Finite Elements Method /FEM/ [9].

The analysis takes into consideration the bending state as well as the forces in plane and the calculations were made on the dimensionless co-ordinates which allowed for the wider and more general analysis.

2. SIMPLIFIED SOLUTION

The displacement functions for the strip scheme shown in Fig. 1e are assumed [4], [6], [7], [8] as follows

\[ u = \sum_{m=1}^{r} \left[ \left( 1 - \frac{x}{b} \right) u_{im} + \left( \frac{x}{b} \right) u_{jm} \right] \sin \alpha_m y, \quad /1/ \]

\[ v = \sum_{m=1}^{r} \left[ \left( 1 - \frac{x}{b} \right) v_{im} + \left( \frac{x}{b} \right) v_{jm} \right] \cos \alpha_m y, \quad /2/ \]

\[ w = \sum_{m=1}^{r} \left[ \left( 1 - \frac{2x^2}{b^2} + \frac{2x^3}{b^3} \right) w_{im} + \left( x - \frac{2x^2}{b} + \frac{x^3}{b^2} \right) \theta_{im} + \left( \frac{2x^2}{b^2} - \frac{2x^3}{b^3} \right) w_{jm} + \left( \frac{x^3}{b^2} - \frac{x^2}{b} \right) \theta_{jm} \right] \sin \alpha_m y, \quad /3/ \]
Concrete Plate  
Asphalt Putty

Fig. 1
where
\[ \alpha_m = m \pi / l, \quad m = 1, 2, 3, \ldots, r. /4/ \]

These equations can be written in a general form [2], [7] for moving loads:
\[ u_p, v_p, w_p(x, y, t) = \sum_{r=1}^{m} N(x) \alpha_{pm}(t) \sin \alpha_m y (\text{or } \cos \alpha_m y) = \]
\[ = \sum_{r=1}^{m} N_m(x, y) \alpha_{pm} = \alpha_m \Theta \alpha_p, /5/ \]
in which:
\( N(x) \) - are Hermite's polynomials \([4], [6], [9]\),
the symbol \([\cdot]^{T}\) - denotes transposition,
the symbol \( \otimes \) - denotes a simple matrix multiplication,
the symbol \( \otimes \) - denotes a description in the base, of all components \( r (\delta m) \).

The strip nodal displacement vector is given for the function "m" by
\[ \vec{q}_{pm} = [u_1, v_1, w_1, \theta_1, u_j, v_j, w_j, \theta_j]^T. /6/ \]

The equation of the dynamic equilibrium of the strip is expressed
\[ \ddot{N}_p \frac{d^2 \vec{q}}{dt^2} + \ddot{K}_p \frac{d\vec{q}}{dt} + \ddot{\vec{q}}_{pm} = \ddot{F}_p. /7/ \]
where:
\( \ddot{N}_p \) - mass matrix,
\( \ddot{K}_p \) - stiffness matrix,
\( \ddot{R}_p \) - damping matrix,
\( \ddot{F}_p \) - the vector of the equivalence of the external load.

The damping matrix is assumed according to Voigt analogically to the calculation of the bearing surface girder [9] and of the beam girder [2].

The analysis was conducted using the dimensionless co-ordinates for the description of this case:
\( l_0 \) - length of the span,
\( m_o \) - uniform mass of the strip,
\( t_0 \) - thickness of the strip,
\( D_0 = \frac{E t_0^2}{12(1- \nu^2)} \) - stiffness of the plate in the direction "y".

The assumed stiffness and mass matrices of the strip are given in dimensionless form for \( m \)th harmonic:
\[ K_m = C [k_1^m + k_2^m + k_3^m + k_4^m] C, /8/ \]
where

\[ C = \text{diag} (1, \beta, 1, \beta) \]

The matrices \( K_m \) to \( K_m^4 \) and \( M_m' \) are given by eq. (11) where \( \gamma \) is Poisson's ratio and \( \beta = b/l_0 \).

\[
K_m^1 = \frac{1}{\beta^3} \\
K_m^2 = \alpha_m^2 \beta \\
K_m^3 = \frac{\alpha_m \sqrt{\gamma}}{\beta} \\
K_m^4 = \frac{\alpha_m^2 \sqrt{\gamma}}{\beta} \cdot \frac{1-\nu}{2}
\]

\[
M_m' = \beta
\]

In this analysis the load of construction is considered as a vertical concentrated force \( P \) moving with constant speed \( v \) along one of the nodal lines of the plate or the box girder.

The vector of external load for \( m \)th component is given as follows

\[
\hat{F}_{pm} = \left[ \frac{P_1}{D_o} \sin (\alpha_m \phi t), 0, 0, 0 \right]^T
\]
Fig. 2
for the edge where
\[ \tau = \frac{t}{T_0}, \]
\[ \sigma = \frac{v}{T_0}, \]
\[ T_0 = \frac{l^2}{\sqrt{m_0/D_0}}. \]

The essential parameters, which are characteristic for the described problem are the speed \( \sigma \), the damping parameter \( \tilde{\gamma}_v \), \[ \tilde{\gamma}_v, \]

The equations of dynamic equilibrium for the \( m \)th component are obtained by building up the matrices and vectors as in classic FEM \[ 9 \]. The equations are solved by means of integration using the SPK variant of the constant acceleration method \[ 3, 7 \].

3. NUMERICAL EXAMPLES

Numerical calculations were made for both types of box spans. Case I is a demountable span consisting of box girders placed beside each other and joined with reinforced concrete deck plate by means of bolts /Fig. 1a/. Case II is a span consisting of identical box beams but placed from each other at a distance equal half width of this girder /Fig. 2a/. Detailed description of typical box beam constructions within the span of 9-24 m was presented in the works of Węgrzyniak \[ 1, 5 \]. At the same time, for comparative purposes for both cases identical prefabricated box spans joined monolithically with reinforced concrete plate were analyzed /Fig. 1c and 2c/.

Discretization of spans for each analytical case was shown on Fig. 1b, d and 2b, d. The calculations were made for boxes of effective span \( l = 17.50 \) m and of width \( b = 7.00 \) m. In this analysis the load of construction is considered as a vertical concentrated force moving with constant speed along one of the nodal lines of the plates and the box girders. The calculations were made for the span of the width \( 0.4 l \) with the division for various number of strips /Fig. 1b, d/ and 2b, d/, for the force \( P = \frac{1}{D_0/1} \), taking the speed parameter \( \sigma = 0.8 \), two damping parameters \( \tilde{\gamma}_v = 0.00 \) and 0.02, and various numbers of the harmonic \( m = 5 \) and 19.

A wider dynamic analysis of box girders and plates of concrete and steel bridge span are given in papers \[ 6, 7 \]. Here, only some results pertaining to the concrete box girders are presented.

Fig. 3 shows deflection distribution in the mid span with the concentrated force moving along the middle nodal line in the middle of span's width for all cases considered. The time patterns of the displacements in the longitudinal direction are shown in Fig. 4 /Case I/ and in Fig. 5 /Case II/.
Demountable Span i Prefabricated Span

8 = 7,00m l = 17,50m

The number of box girders

b) Dynamical Solution

Statistical Solution

Fig. 3
Dynamical Solution
Statistical Solution
Demountable Span
Prefabricated Span

$B = 200 \text{m}$
$t = 17.50 \text{m}$
4. CONCLUSIONS

The presented abridged algorithm of the solution and the programme developed on its basis prove that the method of finite strips can be used for the analysis of unstable vibrations of demountable concrete box girders and plates.

The program was written in the language FORTRAN, the calculations being made in the Computer Center of the Wrocław Technical University.

Considerable differences in deflections and unit strains /stresses/ were found out between demountable spans and prefabricated monolithic ones. The values of deflections and deformations in the analysed cross-sections of a demountable span are about 18-25 % greater than those in a monolithic span in case when the beams are placed beside each other /Case I/, and about 27-38 % in case of beams placed at the distance /Case II/. It is of considerable importance for designing of such objects and should be considered in values of dynamic coefficients.

The conclusions which could be drawn on the basis of the presented results would have a limited character since the problem of moving loads is a highly complex one because of the great number of parameters involved in the description of the phenomenon. The programme covers the different hypotheses of damping, different types of moving loads, different velocities of the loads, various static schemes of the structure, etc.

5. REFERENCES


DEMOUNTABLE CONCRETE STRUCTURES WITH STEEL ELEMENTS OUTSIDE THE CONCRETE SECTION

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SUMMARY

The paper presents very light demountable reinforced concrete structural systems for roofs of industrial and other buildings, details of connections, results of theoretical and experimental investigations, as well as experience gained in application of such systems.

The upper chord of the system is a reinforced concrete member, subjected to relatively high compressive forces and small bending moments. The bottom, axially tensioned chord, is outside the basic concrete section and may be constructed either of ordinary reinforcement or rolled steel elements. For larger spans, prestressing tendons may also be used. The systems are successfully applied to spans of 8 to 30 m, and even longer.

Due to the relatively low weight, simple precasting, small quantities of requisite materials, easy transportation, simple connections and quick assembly and demounting, this structural system has shown significant technical and economic advantages over other systems usually applied in Yugoslavia.
DEMOUNTABLE CONCRETE STRUCTURES WITH STEEL ELEMENTS OUTSIDE THE CONCRETE SECTION

1. INTRODUCTION

The paper presents very light-weight demountable reinforced concrete systems of roof girders for industrial and other buildings, connection details of the system members, the results of theoretical and experimental investigations, as well as the experience gained in the application of these systems in Yugoslavia.

The presented demountable structural system is a two-chord system. The upper chord 1 is made of reinforced concrete members, subjected to relatively high normal compressive forces and low bending moments, while the lower, tension chord 2, which is outside the basic reinforced concrete cross-section, consists of steel elements subjected to axial tensile forces, Fig. 1. The required distance between the upper and lower chords of the structure is attained by compressed lateral members 3, designed at suitable distances along the span, Figs. 1 and 2. These bracing members may be made either of steel or reinforced concrete, demountably connected to chords 1 and 2 of the system. Their distances \( \lambda_i \) depend on the required geometry of the lower chord, flexural rigidity and geometry of the upper chord, as well as on the other serviceability parameters of the structure. Obviously, a larger number of braces enables the designer to adopt smaller upper chord cross-sections, which results in the significant reduction of the total self-weight of the structure.

The reinforced concrete part of the structure, depending on the span, may consist of one, two or more members, with arbitrary cross-sections. Because of the simple prefabrication, the cross-section of the upper chord is usually rectangular. For larger spans, in order to ensure lateral stability of the system during the erection or demounting, as well as in the service, two parallel members can also be used, interconnected by bolts at the required number of nodes, Figs. 2 to 5. Generally, the interconnection between all individual members of the system is made with bolts. Depending on the span and the static forces which should be transferred by the connections, the ordinary or high-strength bolts, prestressed to a desired degree, may be used.

The lower, axially tensioned members are formed either of ordinary steel for reinforced concrete, or of suitable rolled steel elements. For larger spans, prestressed tendons may be successfully used for tensile chord. In that case the demountable anchors must be applied. The corrosion protection of anchors is provided by special caps upon anchors. The tendons have to be suitably protected against corrosion, too.

In some cases it is very convenient to prestress this system to the desired degree, by which, in considerable way, the stress-deformation state of the
The system under service loads may be affected. The required degree of tensile force in the chord 2, which, in fact, represents a tie in the system, can be realized very simply in case of the ordinary reinforcement too, using nuts at the ends of bars, or moving apart the upper and lower chords at the places of bracings 3.

2. BASIC ASSUMPTIONS FOR CALCULATION AND ANALYSIS OF RESULTS

Generally, the considered structural systems, Figs. 1 to 5, are treated as systems of bars of corresponding geometrical and rheological characteristics. While the steel members of the system, in the domain of service loads, behave elastically, the linear creep theory is valid for reinforced concrete members. In general, due to relatively high deformability of the system, geometrical non-linearity has to be taken into account. The relation between stress and strain in the reinforced concrete part of the system is taken in the usual form

\[ \Delta \varepsilon_{c}(t, t_0) = \frac{\sigma_{c}(t_0)}{E_{c}(t_0)} \Phi(t, t_0) + \frac{\Delta \sigma_{c}(t, t_0)}{E_{c}(t_0)} + \varepsilon_{sh}(t, t_0) \]  

where the meanings of symbols are in agreement with those adopted by CEB [2], and \( E_{c}(t) \) is Bazant's Age-Adjusted Effective Modulus (AAEM) of concrete. The reader can find more details about the algorithm for the calculation of stress and strain states of the analysed system in [3].

These structural systems, particularly for large spans, can be treated as two-chord catenary systems, in which the upper chord 1 has, apart from the axial rigidity, also the flexural rigidity, while the lower, tensile chord 2, has practically only axial rigidity, Fig. 1. However, at the same time, these structural systems may be understood as systems in which the upper chord is a reinforced concrete girder, supported by discrete elastic supports - bracings 3, whose stiffness depend on the axial rigidity and the configuration of the tensile member 2. The following assumptions are introduced into the calculation:

- axial deformation of bracings 3 is negligible, so the deflections of the upper chord are equal to the deflections of the lower chord at the contact nodes;
- members of the upper 1 and lower 2 chords are sufficiently shallow that the following relation is applicable

\[ ds_i = [1 + \frac{1}{2} (\frac{dy}{dx})^2] dx, \quad (i = 1, 2) \]  

- horizontal displacements of elements 1 and 2 under the action of vertical loads are neglected;
- equilibrium conditions has to be satisfied on the deformed system, and 'conditions of lengths' are reduced to a known form

\[ L_i = L_{i,0} + \Delta L_i \]  

where

- \( L_i \) - length of element \( i \) under external loads, shrinkage and creep of concrete,
- \( L_{i,0} \) - length of element \( i \) before the action of loads,
\[ \Delta L_i \] - increment of length \( L_{i,0} \) due to external loads, creep and shrinkage of concrete.

![Diagram](image)

An example of such a structural system is shown on Fig. 2a). This type of system has been already many times applied by the Authors of this paper as a roof girder in various industrial buildings. The connection between compressed reinforced concrete part (1) and ordinary reinforcing tensile steel element (2) of the system is provided by bracing elements (3) only at two points in the span. The geometrical characteristics of the system and the rheological data for materials used, as well as the dead and live load, are presented in Fig. 2a).

Fig. 2b) shows the diagrams of bending moments \( M \) and deflections \( w \) of the reinforced concrete girder (1) in time \( t = 0 \) and \( t \to \infty \), calculated by the First Order Theory, when the system, in fact, behaves as an elastically supported beam. On the basis of analysis of the behaviour of such a structural system, and the obtained results, it can be concluded that:

- a significant reduction of bending moments is achieved in relation to bending moments obtained in classical girders of the same span and approximately equal internal forces lever arm;
- total deflection due to the action of dead and live loads is within the allowed deflection limits \( L/400 \), likewise in classical reinforced concrete girders. Therefore, it would be possible even to reduce the cross-section of the reinforced concrete element;
- time-dependent deflection for \( t \to \infty \), is, in this case, only 1,5 times the elastic deflection for the initial moment of time \( t = 0 \). This relatively
small increase of deflection with time is the consequence of a very small change of the curvature of the element with time, which is easy to explain as the bending moment are not significant;

- due to concrete creep in such a structural system the force in tensile element does not decrease with time but is even slightly increased, while relatively little loss of the tensile force results from concrete shrinkage. As high compressive stresses act in reinforced concrete element, it can be understood why the tensile force, due to both creep and shrinkage, does not decrease at all but is practically time independent;

- for spans not exceeding about $l = 30\, m$, stresses and deformations calculated under assumptions of either the Second or the First Order Theory are not essentially different. Only after a considerable reduction of the stiffness of the reinforced concrete element, and possible use of prestressing steel for the tensile element, these differences become noticeable, so that the Second Order Theory must be applied.

The results of experimental investigations of such linear systems, with spans of $l = 12.5\, m$ and $l = 15\, m$, of the type shown on Fig. 3a), as well as $l = 20\, m$, Figs. 2 and 4a), were in a very good agreement with the theoretically obtained results, at all stages of loading, including the ultimate load.

Fig. 2c) shows the measured values of the deflection of the investigated structure, shown in Fig. 2a), under various levels of short-time loading.

3. POSSIBLE STRUCTURAL SOLUTIONS
WITH PRESENTED DEMOUNTABLE CONCRETE SYSTEM

Figs. 3, 4 and 5 show some of the possible structural solutions of roofgirders designed in the presented demountable reinforced concrete system. For spans of approximately 10 to 18 m, optimum solutions are achieved by applying the system shown in Fig. 3a), and for larger spans, from 12 to 20 m it is suitable to apply the system presented on Fig. 3b).
For significant roof slopes, one can also apply the system presented on Fig. 3a) for spans greater than 18 m. "Double supported" demountable systems, shown in Figs. 4a), b), and c), are applied for spans of approximately 18 to 30 m, and even up to 35 m. Fig. 5 shows the possibility of applying the system for spans over 30 m. The systems presented in Figs. 5a) and b), are applied in industrial structures, where a great free height is required in the central part of the structure, while the system in Fig. 5c) enables spans of 50 to 80 m, and even considerably greater.

The angle of inclination of the upper chord of the system has to be designed to suit the contemporary roof coverings, and can be one-sided or two-sided system. As a rule, the slope is 2 to 10%, but it can be greater. Greater inclinations enable a higher rise \( f \) to be designed, which gives a larger arm of internal forces, and may considerably reduce the cross-sections of the upper and lower chords, thus affecting also the total weight of the girder. Otherwise, \( f \) amounts to limits of \( l/15 \) to \( l/10 \), where \( l \) stands for the span. In structures where the roof slope gives the possibility for an increased rise, the relation \( f/l \) usually exceeds \( 1/10 \).

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Figs. 6, 7 and 8 show the systems presented in Figs. 3a), and 4c) under construction.

For usual roof coverings and for distances between roof girders of 8 to 12 m, the cross-section of the reinforced concrete part of the system is a single rectangle, of the dimensions \( (20-30)/(35-45) \) cm, as shown in Fig. 3. In systems presented in Fig. 4, where the reinforced concrete part is most frequently made of two parallel composite members, the width of each member is within \( (15-20) \) cm and the height ranges from 40 to 70 cm. For systems shown in Fig. 5, the dimensions of each part of the upper chord cross-section are \( (15-20)/(45-75) \) cm, depending on loads, span, rise quality of concrete, etc. The concrete is usually \( C 30 \) or higher, made of light or normal aggregates.
4. DEMOUNTABLE MEMBER CONNECTIONS

The joints of individual members of the presented structural system are made with bolts, which secure an easy and quick assembling and demounting of the structure. The dimensions, particularly the lengths of individual members, may be easily adapted to the means of transportation, erection and demounting. Lengths of reinforced concrete members are approximately 5 to 15 m, in some cases reaching even 20 m.

The connection detail of the reinforced concrete part and the lower, steel chord, is shown in Fig. 9.

In this case the tensile chord consists of the ordinary steel reinforcement, used in reinforced concrete. Reinforcement bars are led through the openings left in the upper chord. There are the impressed threads at the ends of the
reinforcement and the connection between members is achieved with nuts. If it is convenient to prestress the structural system to the design force, it may be realised by prestressing the nuts, for instance by means of a "moment-key", controlling the transferred tensile force. For spans up to about 25 m, the required prestressing force is most frequently relatively small, sometimes only such as to give equal initial tensions of several MPa to all reinforcement bars. However, it is also possible, and in case of greater spans sometimes necessary, to prestress the system to a greater degree in order to reduce the deflection for service loads. For very large spans, exceeding 40 to 60 m, the lower tensile chord is usually made of prestressing tendons with special anchors which enable the force correction in the system at a desired moment.
5. CONCLUSIONS

The presented demountable reinforced concrete roof structural systems are characterized by relatively small weight, simple prefabrication and serial production, small amount of material, easy transportation and simple connections between members, which enable safe transmitting of static influences, and quick erection and demounting. They are also very suitable for structures in earthquake areas. Because of such properties these structural systems have shown significant technical and economical advantages in comparison with other systems applied in Yugoslavia.

It is of great importance for the presented structural system that the time-dependent deflections, resulting from creep and shrinkage of concrete, practically do not influence changes of forces in the system elements. It enables the system to be prestressed to a required degree, even in case the ordinary reinforcement is used for the lower tensile chord. But even a very low degree of prestressing may improve the stress-deformation state of the structure, particularly in the serviceability domain, which is very essential.
REFERENCES


VERTICAL JOINTS IN PRECAST CONCRETE CORES

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SUMMARY

It is logical to construct the stabilizing structures of a building (stiffening walls, cores) with precast elements if the loadbearing structure is precast, too. The horizontal joints will not cause particular problems if tension is avoided. But the vertical joints will influence the deflection of the core and the magnitude and distribution of the stresses caused by horizontal loading. Certainly the shear stiffness affects deflection and stresses (compare the monolithic case with the case where shear stiffness = 0).

The shear stiffness of the joint can be characterized by the modulus $K = \frac{F}{\delta}$ (Pommeret, Schwing, and others).

A parameter study was carried out to ascertain the influence of $K$, the location of the vertical joint and the boundary conditions, by means of differential equations, derived and solved for a linear-elastic static model.

At the same time the effect of plasto-elastic behaviour of the joint material and the difference between a continuous connection and a connection at discrete points were investigated.

The magnitude of the $K$-modulus and the location of the vertical joints were found to be very important. The connection consisting of steel plates anchored in the concrete and electrically welded together by means of an auxiliary feature (strip or bar) comes within the scope of demountable construction.

This simple connection contains the possibilities of standardization. It is important to know the force-deflection relation ($\sigma$-$\delta$-relation, therefore $K$) of this connection. Research is in progress.

Preliminary investigations on 1:1 scale models have been conducted in the Stevin Laboratory (Delft University of Technology). The results have been interpreted and evaluated.

A mathematical model (DIANA package) has been developed, with which the non-linear behaviour can be simulated. A parameter study has been started. A first impression of the $K$-modulus has been obtained. This justifies the confidence placed in the usefulness of the connection.
VERTICAL JOINTS IN PRECAST CONCRETE CORES

Introduction
The stability and stiffness of precast concrete buildings, particularly the higher ones (h > 10 to 12 m) is very often provided by walls, single or composed, or by cores. These walls and cores are often cast in situ. In general this appears to be an expensive solution because of the more complicated organisation on the site and/or because these walls or cores are determining the critical path of the execution. In many cases the prefabrication of these stability constructions may form an attractive solution; they can be assembled floor by floor at the same time as the rest of the construction. Prefabrication means the applications of joints between the precast concrete elements. By this question is rising up, which influence these joints will have on the strength and stiffness of the core. In the past a lot of attention has been paid to the joints on their own (among others Pommeret, Mehlhorn, Schwing), but no one has ever treated the integral precast concrete wall together with the influence of the joints. It is easy to show, that the horizontal joints, executed as joints of mortar (dry packed or grouted), have little influence on the behavior of strength and stiffness of the stability construction, provided that no tensile stress appears in the joint. In a good design of the building, a care is taken, that an as big as possible part of the total vertical load is going to be supported by the stability-element, so that no tensile stresses occur. Next to this is the possibility of prestressing.

Fig. 1
However the strength and stiffness properties of the vertical joints are very important for the magnitude of the stresses and their distribution in the stability elements, as well as the deformations of the shear walls or cores.

In fig. 1 is shown a monolithic construction and a construction provided with one vertical joint. The factor K of the joint is equal to zero, so that two separate monolithic structures exist. The influence of the vertical joint is evident and has been the aim of the study.

First of all the continuous joint, executed as a wet connection, has been examined. After that the influence of the concentrated joint on the strength and stiffness has been checked. The last one can be applied as a dry connection (for example by means of electric welding), which fits better in the frame of demountable building.

The behaviour of the joints is described by means of a differential equation analysis. The differential equations system gives the relation between the degree of displacement and the loading. A complete resolution is drawn for a core/shear wall with a non-symmetrically placed joint and one for two non-symmetrically placed joints. The displacement at the top of the core/shear wall and the maximum stress in the foundation are examined. These values are compared with those of a monolithic core/shear wall.

The mechanical model:

![Mechanical Model Diagram]

Fig. 2
To draw up the different differential equations, a mechanical model (fig. 2) was set-up for a precast core/shear wall with a non-symmetrical continuous vertical joint. The core/shear wall has been schematized as a system of two co-operating beams. In the model the two beams are connected with a great number of infinitely rigid, hinged bars between the beam neutral axes. The connection makes both beams behave in the same way in sway or deflection. An assumption is made that the beam theory is valid. Herewith the shear force is neglected, which is a disadvantage. The joint is described by means of an infinite number of vertical springs, which make possible the transfer of shear stresses from one beam to the other. The shear stress ($\tau$) is a linear function of the differential displacement ($\Delta x$) of both sides of the joint: $\tau = k \Delta x$.

The system of differential equations:

The beams are subjected to extension and bending by horizontal and/or vertical loadings. The extension and bending are related to each other. This gives a great number of coupled differential equations.
In order to derive the relation between the loadings and the displacements, three different kinds of equations are used:

1. Static equilibrium equations which describe the equilibrium for a differential element with a length $dx$ (fig.3).
2. Constitutive equations, which used to give the relation between the force per unit area and the deformation.
3. Kinematic equations; giving the relation between the deformations and displacements.

Bij eliminating the forces per unit area and the deformation we arrive at the following system:

$$\begin{bmatrix}
E.A_1D^2 - K.d & K.d & K.d.a.D \\
K.d & E.A_2D^2 - K.d & -K.d.a.D \\
-K.d.a.D & -K.d.a.D & E.I.D^4 - K.d.a^2D^2
\end{bmatrix} \begin{bmatrix}
[u_1] \\
[u_2] \\
[w]
\end{bmatrix} = \begin{bmatrix}
-fx_1 \\
-fx_2 \\
-fz
\end{bmatrix}
$$

Where $D$ = Canchy's differential operator: $D^4(\ldots) = d^4(\ldots)dx^4$

The general solution, the particular solution and the integration constants can be determined. For deriving the integration constants are 8 generally holding endconditions available: four at the foundation level and four at the top. (fig.4)

The boundary conditions are:

- $x=0$ \quad $w=0$ \quad $M+R\cdot dw/dx=0$ \quad $N_1=C_1\cdot u_1$ \quad $N_2=C_2\cdot u_2$
- $x=L$ \quad $M=T$ \quad $Q=H$ \quad $N_1=V_1$ \quad $N_2=V_2$

After deriving the above constants, a complete behavior of the beams can be formulated and it is possible to perform further parametric study.
The behaviour of a core/shear wall with one joint

The solution is only valid for a section symmetrical to the loading axis or else have to be considered the effects of torsional moment. An analysis learns one is confronted with a boundary condition problem for the shearstress in the joint differs of that in the monolithic core/shearwall at the ends \((x=0, x=l)\) only. The deviation is the greatest at thefoundationside, which means that the increase (in \%) in stress at the foundation is greater than the increase (in \%) in deflection at the top. Also, it can be concluded that:

- if the length of the core increases the relative decrease in amount of deflection and stress develop. This is also valid in case the joint stiffness increases, because the height where the joint is noticeable, decreases in that case.
- the deflection of the core increases exponentially with decreasing shear strength of the joint
- the influence of a joint on the deflection is less important for slender cores
- the deflection increases by placing the joint up to the centre of the core.

In the first instance, the core/shear wall is considered being fully fixed in the foundation. Providing the separate beams have a certain stiffness against rotation then the reduction in increase of the stress at the foundation is important and the decrease in deflection is only small. This is correct incase the rotation stiffness is not small and in most cases the latter is true. The joint stiffness plays a great influence here, however. The increase in the deflection for a H-shaped or box-shaped core is much greater than for a single wall because the ratio between the summation of the moments of inertia of the individual beams and the moment of inertia of the whole section is much smaller. (fig.5).

\[\text{Fig. 5}\]
In a H-shaped core consisting of two T-shaped parts, the greatest stress occurs at the largest T-shaped part. Depending on the joint stiffness $K$ and the location of the joint, the occurrence of the maximum stress can be either in the outermost fiber of the whole section or at the joint location. When the joint stiffness $K$ decreases, the effectiveness of both beams in acting co-operatively also decreases. This is also the main reason why the greatest stress in the web of the T-shaped part can occur. (fig.6).

The core/shear wall with two joints

In general, more than one joint are present in a core/shear wall. Beside, it is also possible to take a row of vertical openings as a joint because there exists an analogy between the joint and a row of vertical openings. For that reason, in analogous way as done for the core/shearwall with one joint the formulas are derived also for a core/shear wall with two joints. In order to get rid of the torsional problem, the section must by symmetrical in relation with the X-Z-plane. However, the joints need not to be located symmetrically and to have the same degree of stiffness. The findings and conclusions drawn up for the behavior of core/shear wall with one joint are also valid for those one with two joints present.
Additional conclusions are drawn up basically typical for core/shear walls with two joints.
The shear stresses which result from a horizontal uniform loading are plotted in fig. 7.

Both joints have different joint stiffness, accordingly, due to this difference, the shear stresses proved to be unequal too.
The joint with a greater stiffness is relatively better or in other words this joint is stiffer and therefore receives more loading whereby a greater shear stress develops. The shear stress over a certain part of the core/shear wall is even greater compared with that of a monolithic case, the other joint behaves on the contrary. In fig. 8, the joint stiffness of one joint diminishes while the other one remains the same.
Let's now compare a core/shear wall having a "good" (K=12N/mm³) and a "poor" joint (K=2N/mm³) with another core/shear wall having two "fair" joints (K=7N/mm³). The increase in deflection for the latter wall proves to be two times smaller than the other. It is very clear here that a precast core/shear wall with possesses two "fair" joints is better than the one with a "good" and a "poor" joint. A good approximation for deriving the total increase in deflection for a core/shear wall with two joints is to evaluate the behavior of a core/shear wall having just one joint and afterwards summing up together the increase in deflection for a case with one "poor" joint and the other case with one "good" joint.

The increase in stress is bigger in case the joint stiffness in one joint decreases and in the other remains the same then when the joint stiffness of both joints decreases in the same way. This means that a precast shear wall with two "poor" joints, speaking in terms of stress at the foundation is better than a shear wall with one "good" and one "poor" joint. (fig. 9).
The unfavorable joint locations in a H-shaped core when taking into consideration the increase in stress is in the neighbourhood of the flange/web transition. By a decrease in joint stiffness, the location of the joints is less important. The increase in stress is nearly equal for a precast core/shear wall having two "poor" joints, either the joint location is placed in the flange/web transition or a joint location is placed at one-fourth of the width of the wall (see fig. 10).

In a precast core/shear wall with a "good" and a "poor" joint, the increase in stress is mainly dictated by the influence of the "poor" joint.

Cores, locally connected

In the preceding pages the joint is handled like a continuous system, in which the joint is simulated by an infinite amount of vertical springs. In the frame of demountable building the local joint however fits better, because it makes applying a dry connection more real.
The question is can this be done without negative influence on strength and stiffness. The literature (Mehlhorn, Schwing) already gives indications in positive sense. The greatest problem will be, to have enough stiffness over the relatively small length of a joint to develop a strength, which is equal to the force which appears in the continuous system in the distance (VH) between the local connections. In this research now is stated, that the stiffness per length VH (Kvd) in the discreet system is equal to the joint stiffness of the continuous system (K), multiplied with the product of the joint depth and the distance VH: 

\[ Kvd = K \cdot d \cdot VH \]
By means of a program from ICES the control research has been done. The result is illustrated in fig.12.
The indicated shear force is the shear force per distance VH. In comparison with the continuous system the value of this shear force has to be divided by the distance VH. This calculated shear force per unit of length appears to be practically equal to the shear force, which follows from the continuous system (largest difference of strains 5%, of deflections 2%). The stiffness $K_{vd}=90 \times 10^5$ kN/m and $K_{vd}=15 \times 10^5$ kN/m corresponding with the joint stiffness of a "good" or a "bad joint" in the continuous system. Concluding there can be stated that the continuous system indeed can be replaced by a discreet system, provided that the stiffness $K_{vd}$ per distance VH is equal to the stiffness of the joint (K) in the distance VH.
Research to a discreet connection

A research is started to check how far the condition, mentioned above is a real problem.
In general the strength behavior of the local connection can be determined quite accurately. This cannot be said about the behavior of the deflection (the stiffness). For the research is chosen for the connection construction, as shown in fig. 13.

Fig. 13

Arguments for this choice were among others the simplicity of the connection and the possibility of standardizing. The approach is as follows. First the connection will be tested in the laboratory. To restrict this, because of very extensive test research, the problem will be also approached by a numeric way.
A set (12 pieces) of orientating tests in the laboratory is already carried out, by which 3 types of execution have been tested. (see fig. 14)
The interpretation of the results is in progress now. Preliminary conclusions are, that the strength of the connection is considerably larger than by calculations revealed, that the deflections are large, that the behavior of the connection is very complicated and in any case plastic. In order that the connection will have sufficient stiffness, a greater margin will have to be taken in respect of the increasing supporting strength. For the numeric approximation of the problem the finite element method with 3.D-elements from the DIANA package will be used. With this program can be calculated according to the nonlinear construction mechanics (as well geometrical as physical non-linear) with a smeared-crack-draft. At the moment the program is operational. The first results give a good curve fitting in the elastic zone.

A few first conclusions, very careful, could be, that the type, "on site mounted" has no sufficient stiffness and that for both other types is found Kvd=40.10^5 kN/m for the stiffness.

Based on stiffness to reach an acceptable stiffness at least one connection at every 1.35 m of the joint length is necessary to speak about "good" joint.

The research will be continued.
SYMBOLS

\( a \) \( m \) distance between centres of gravity core 1 and 2

\( A_1, A_2 \) \( m^2 \) cross section area core 1 and 2

\( C_1, C_2 \) \( \text{kN/m} \) displacement stiffness core 1 and 2

\( d \) \( m \) joint depth

\( E \) \( \text{kN/m}^2 \) Young's modulus of elasticity of core parts

\( f_{x1}, f_{x2} \) \( \text{kN/m} \) equally distributed load one core 1 and 2 vertical

\( f_z \) \( \text{kN/m} \) equaly distributed load one core 1 and 2 horizontal

\( H \) \( \text{kN} \) horizontal point load at the core top

\( I \) \( m^4 \) total amount of the moments of inertia of the separate core parts

\( K \) \( \text{kN/m}^3 \) joint stiffness

\( R \) \( \text{kN/m/\text{rad}} \) rotation stiffness core

\( T \) \( \text{kNm} \) bending moment at the core top

\( u_{1,2} \) \( m \) x-direction displacement of the core part 1 and 2

\( V_{1,2} \) \( \text{kN} \) vertical point load at the top of part 1 and 2

\( w \) \( m \) displacement of core

\( \delta \) \( m \) deflection precast core

\( \delta_m \) \( m \) deflection monolithic core

\( \sigma \) \( \text{kN/m}^2 \) stresses in the precast core

\( \sigma_m \) \( \text{kN/m}^2 \) stresses in the monolithic core

\( \tau \) \( \text{kN/m}^2 \) shearstress in the precast core

\( \tau_m \) \( \text{kN/m}^2 \) shearstress in the monolithic core
SUMMARY

This introductory paper is concerned only with buildings. Demountable construction in concrete increases the functional flexibility of buildings and simplifies their subsequent demolition. Indirectly this can thus contribute to conservation of energy and raw materials and to environmental protection. Prefabrication, more particularly in precast concrete, provides a suitable basis for the development of demountable structures.

A suitable choice of the structural system and good detailing of the connections are especially important. From the point of view of demountability it is essential to devise suitable solutions primarily for the floors (as load-transmitting assemblies of precast units) and for the structural cores of buildings. Demountable structures will in general be statically determinate. In order to ensure safety, it may be necessary to adopt arrangements to increase the robustness of such structures. Demountable structures must not be more expensive than comparable conventional structures. For further development, various aspects will require closer investigation.
1. INTRODUCTION.

In origin, concrete structures are monolithic in character. This fact has had considerable influence on construction in precast concrete and is still discernible in the application of in-situ concrete and grouted connections and of reinforced concrete toppings. In modern precast construction, however, attention is increasingly being focused on the application of "dry" erection joints in order to reduce on-site work. In this evolution it would be a logical step to detail the structural connections in a way that will enable the structure subsequently to be dismantled by a simple procedure. There are good reasons for stimulating further developments in that direction.

Present-day society is characterized by rapid changes both at social and at technical level. In consequence of these changes buildings become obsolete more rapidly, i.e., they cease to satisfy the changed demands of their users, so that they often have to be more or less drastically altered, or indeed demolished, after a relatively short time. This trend is expected to become even more pronounced in the years ahead. It is therefore necessary to design buildings in such ways that they can be adapted fairly simply to changed requirements as to their use. Of primary importance in this respect is flexibility of internal layout and technical installations, together with replaceability of limited-life structural members by new or different ones. With a view to subsequent conversion or extension of the building it also may be advantageous to design the structural system for demountability, so that any such alterations can in due course be carried out more quickly and with less nuisance to the environment.

Another reason to make concrete structures demountable is the necessity to simplify demolition. The demolition of conventional concrete structures demands much energy input and is accompanied by noise and dust nuisance. In view of the rising energy prices and the increasingly strict noise and dust control regulations, the cost of demolition will rise in the future. Moreover the increasing quantities of waste material that will have to be dumped (Fig. 1), are bound to cause a major environmental problem in the future, especially in

![Graph showing estimated concrete production and anticipated demolition in the EEC](image)

A: concrete production

B: quantities of concrete rubble

Fig. 1 Estimated concrete production and anticipated demolition in the EEC /1/
densely populated regions. Efficient methods of recycling concrete will therefore have to be devised. This is made all the more necessary by the increasingly rapid rate at which the available resources of raw materials are becoming exhausted. One possible way to help solve these problems consists in so detailing the structures that they can be dismantled by simple methods, while the components that thus become available are potentially reusable. Prefabrication - more particularly: precasting - is a suitable basis for developing such demountable structures.

Fig. 2. Precast concrete construction with in-situ concreted structural connections.

2. DESIGN OF DEMOUNTABLE CONCRETE STRUCTURES.

2.1. Basic principles.

The purpose of demountable construction is to build structures which can later be dismantled ("demounted") with the least possible amount of demolition effort. To what extent a certain amount of demolition is acceptable will depend on the possibilities for reuse of the structural components. If the building has merely a temporary function to perform and will, after a time, have to be re-erected on a different site, so that reuse of the components is definitely intended, it must be possible to dismantle the structure without damage and to reassemble its components as required. In general, however, the scope for reuse will be determined by the future development of building construction and by the prices of raw materials and the cost of production. In designing the structure it is therefore necessary primarily to consider its flexibility - more particularly the possibilities for alteration and extension - and to aim at simplification of eventual demolition. A limited amount of "destructive" demolition is acceptable, especially where particular components are concerned which are closely project-related and whose reuse is therefore rather unlikely. The following basic principles are applicable to the design of demountable precast concrete structures: modular design; taking account of the specific conditions of production, transport and erection; aiming to produce sufficiently large series of components and using standard components where
possible. With a view to subsequent dismantling a few additional conditions have to be fulfilled:
- reinforced concrete toppings and other in-situ concrete liable to make dismantling difficult should not be used;
- the structural connections should be properly accessible and, if possible, capable of being dismantled without damage to the precast components;
- in view of subsequent dismantling, lifting attachments and other erection aids should as far as possible remain intact.

Having regard to these conditions, it is preferable to use dry erection joints, in which the transfer of force is effected by means of bearing materials, or mortar joints and steel connecting devices (welded or bolted connections, dowel-and-socket connections). An important requirement is that the steel connecting elements must be permanently protected against corrosion and fire, which should be achieved, if possible, without the use of in-situ concrete fillings which are difficult to remove at a later stage. The choice of structural connections is in part determined by the production possibilities of the precast concrete components. If lifting inserts or attachments cannot be cast in, as in the case of extrusion-cast floor units, for example, then other solutions will have to be sought. Connections can also be formed by means of prestressing with unbonded tendons, provided that an effective method of lasting corrosion protection of the prestressing steel is employed and that intermediate anchorages are installed in order to limit the consequences of possible tendon fracture.

The choice of the structural components is determined by the design of the structure and by the requirements upon which the design is based. Where possible, an "open" building system of modular design should be adopted, allowing the use of commercially available standardized components. There is no objection to casting certain parts of the structure in situ, however, provided that subsequent dismantling is not unnecessarily hindered by this.

2.2. Structural design.

The loadbearing structure of a building is characterized more particularly by the way in which stability is ensured. For this purpose the following structural systems are to be distinguished (Fig. 3):
(a) columns fixed at the base, with simply-supported beams;
(b) unbraced framework with rigid joints;
(c) rigid core with pin-jointed attached framework;
(d) framework formed with loadbearing external wall units.

Fig. 3. Structural systems.
The structural system of the building also determines the function of the floors with regard to the transfer of horizontal loads and the requirements that this entails for the connections. For demountability it is advisable to choose a system with hinged joints, because flexurally rigid connections are rather unsuitable for construction in a demountable form.

The choice of the structural system will also depend on the height of the building. In a building of limited height, structural stability can be derived from fixed-base columns, a solution frequently adopted in the construction of single-storey industrial and other similar buildings (Fig. 4), but suitable also for two- or three-storey structures. The limiting height for this solution is often governed by the magnitude of the horizontal displacements that occur. If the structure is required to be demountable for subsequent re-erection in a different location, it may be advantageous to design the columns as rocker columns and to derive the stability from cross-walls.

![Fig. 4. Single-storey industrial building with fixed columns.](image)

In the case of a multi-storey building it is possible to use a rigid-jointed structural frame. The flexurally rigid joints are difficult to construct in a demountable structure, however, so that this system is not suitable for demountable building construction. On the other hand, it is possible to assemble the frame from composite units, such as TT units (Fig. 5), which may be interconnected by hinged joints.

![Fig. 5. Structural frame composed of TT units.](image)
Alternatively, stability may be provided by loadbearing external wall units (Fig. 6). These units, which are one storey or possibly several storeys high, must be so interconnected as to form a rigid framework. The connections for this purpose only have to transfer direct forces and shear forces in the plane of the external wall. The rigidity of the connection is of major influence on the distribution of forces in the external wall. It is therefore important to know the deformation behaviour of the connection.

Fig. 6. Structural frame composed of external wall units.

The most frequently used structural system for precast concrete multi-storey structures is the rigid core with pin-jointed attached framework (Fig. 7). The core is composed of one or more shafts and/or shear walls which transmit the horizontal loads to the foundation. The column-to-beam connections in the frame can be formed as hinges. Quite often it will be possible to design the structure in a way that allows hinged column-to-beam connections. In such structures the horizontal loads must be transmitted by the floors to the stabilizing members. It is, however, no simple matter to achieve a sufficiently rigid and integrated floor diaphragm without having recourse to a reinforced concrete topping and/or a reinforced edge beam concreted in situ.

Fig. 7. Pin-jointed framework with core.
2.3. Floor and core.

In a multi-storey building the floors, often composed of floor units and beams, contain more than 60 - 70% of the total quantity of concrete. From the point of view of demountability they are therefore the most important components of the building. In principle the floor has the following functions to perform:
- transmitting vertical loads to the supports;
- transmitting lateral loads (due to wind or out-of-perpendicularity) to the cores, taking due account of second-order effects;
- providing lateral support for the columns and ensuring adequate structural integrity.

With regard to demountability the transmission of the vertical loads presents few problems: any load distribution that may be necessary can be achieved through grouted keyways or, if steel connector elements can be cast in, through simple welded connections.

Lateral loads are transmitted to shear walls or cores by floor or roof members acting as horizontal diaphragms. The horizontal loading produces bending moments and shear forces in the diaphragm made up of precast elements. Furthermore, tensile forces may occur in consequence of wind suction and of tilting (out-of-perpendicularity) of columns, while it is necessary also to take account of possible stresses due to deformation restraint. The pattern of forces acting in the floor is determined by the location of the structural cores within the plan of the building and by the units of which the floor is composed (Fig. 8).

Fig. 8. Forces acting in a floor composed of precast units.

The bending moments produced by the horizontal forces can be resisted by a thrust-transmitting arch within the floor and by a tie-member near the edge of the floor. The tie-member can be formed by interconnection of the edge beams or with the aid of continuous unstressed strands or prestressed tendons. The connection between the floor units and the edge beam must be able to transmit the longitudinal shear forces. The shear within the horizontal diaphragm is transmitted by the grouted joint keyways between the elements, which are often cracked as a result of shrinkage.
stresses. Various investigations into the behaviour of cracked joints and floor diaphragms assembled from precast units have recently been carried out /2/. On the basis of the results thus obtained recommendations will have to be prepared for the design of suitable floor diaphragms. Finally, in order to ensure lateral support of the columns and overall structural integrity, tensile continuity between elements must be provided.

For the sake of demountability the connections between the components of shear walls and structural cores should also be given special attention. The object is to achieve sufficient rigidity to limit horizontal deflection, particularly in high buildings. This means that vertical tensile stresses in horizontal cross-sections of these members should be prevented. If necessary, this can be achieved with the aid of prestressing. Furthermore, a reduction in rigidity due to shear deformation of the connections in vertical joints should be taken into account (Fig. 9). It is necessary to develop suitably demountable connections for the purpose and to carry out research into the deformation behaviour of these connections.

![Fig. 9. Deformation of a shear wall composed of precast units.](image)

### 3. RELIABILITY OF DEMOUNTABLE CONCRETE STRUCTURES.

#### 3.1. Safety.

"Safety is the ability of a structure to sustain actions and other influences liable to occur during construction and use and to maintain sufficient structural integrity during and after accidents" (Joint Committee on Structural Safety /3/).

Absolute safety is unattainable, not only for economic reasons, but also because the hazards to which a structure may be exposed and the resistance that it can offer to such hazards are not accurately predictable. Some risk is therefore unavoidable. The magnitude of the risk is determined by the probability of local failure of the structure and by the extent of the damage or loss due to failure. Hence the failure probability should be lower according as the damage or loss is greater and the consequences are more serious. The social acceptability of the risk is determined chiefly by the nature of the consequences of structural failure: material damage, loss of irreplaceable objects, personal injury or loss of human life.
The assessment of structural safety is often based on codes of practice or other such regulations. On the evidence of experience, the rules laid down in the codes can be presumed to help in attaining an acceptable level of safety. These rules and the safety margins they comprise are, however, geared to conventional structures and must therefore be applied with due caution when new methods of construction are introduced. This is especially relevant to aspects which are only briefly dealt with in codes of practice, such as the detailing and the overall integrity of the structure.

3.2. Demountable structures.

In general, demountable structures will be statically determinate. This means that they are characterized by a clear-cut load flow. Hence they are suitably amenable to analysis. On the other hand, because of the absence of "hidden reserves" of strength, statically determinate structures are more sensitive to the effects of abnormal loads and other influences not taken into account in the design calculations. For this reason the designer will, rather more than for monolithic concrete structures, have to consider the hazards to which the structure may be exposed.

In this context it is of interest to examine the causes of structural damage that has occurred in the past. These causes have been the subject of various investigations (14), which show that in many instances the damage was not due to material defects or to overloading. In the great majority of cases it was attributable to human errors and oversights due to lack of adequate knowledge or lack of proper care. The causes involved were faults of workmanship, but also design faults such as poor detailing, unclear or incomplete drawings, and failing to take account of (foreseeable) loads and lack of maintenance.

Evidently the safety of a structure is determined to a great extent by factors which are not taken into account in the usual methods of analysis. Hence the actual probability of local failure to a structure is higher than the theoretical probability of failure on which the codes are based. In demountable design it is not meaningful to adopt a greater margin of safety than is usual in structural design. The desire for safety is much better served by precautions to prevent human errors in design and execution.

Although the probability of failure of the structure is extremely low, the possibility of local damage must be taken into consideration. Accordingly, the main structure should be so designed that it should not subsequently be damaged to an extent disproportionate to the extent of the initial cause. Local damage to the structure will result in a change in the load flow. If the structure is unable to bridge over the local failure, the damage will propagate as a chain reaction in one or more directions. This mode of failure is called progressive collapse.

A well-known example of such collapse occurred at Ronan Point, a high-rise block of flats in London (Fig. 10). The damage, which affected part of the structure, was caused by a gas explosion in one of the flats which blew out a loadbearing external wall unit. This resulted in collapse of the storeys above, and the falling debris on succeeding floors in turn caused a progressive collapse downwards.

Obviously, such disproportionately severe damage is unacceptable. Structures should possess an appropriate robustness to assure structural integrity after a certain amount of damage. The basic principle with regard to this is that failure or unserviceability of any one particular part or component must not result in progressive collapse. This aspect must receive due attention in the design of demountable structures.
Fig. 10. Partial collapse of Ronan Point high-rise building.

3.3. Design considerations.

In designing a structure it is necessary carefully to consider the various factors that determine its safety. If need be, measures should be taken to ensure structural integrity. For this purpose a distinction can be drawn between measures which can reduce the probability of failure, measures intended to prevent progressive collapse, and measures for the conservation of safety. An overview of these measures is given in Fig. 11.

Reducing the probability of failure:

Abnormal loads and other influences such as, for example, collisions, explosions, differential settlement, etc. must be taken into account. First, ways and means of eliminating these hazards should be considered, e.g., by installing collision barriers, knock-out panels for explosion venting, or settlement joints. If such measures are impracticable or ineffective, then the special loads whose occurrence is not be rated as unlikely will have to be taken into account.

The aim should be to obtain a properly balanced design. The probability that a structure will fall is determined by the failure probabilities of its components and can be efficiently reduced by giving extra strength to relatively cheap components. It is therefore advisable to make the connections in demountable structures stronger than indicated by the design calculations, the more so as relatively little is yet known on strength and deformation behaviour of connections.

The probability of damage to a structure is determined to a great extent by the probability of human errors. The only effective way to obviate such errors is to introduce a system of quality assurance /3/. In so far as the production of their components are concerned, demountable structures have the advantage of manufacture under factory conditions. This, however, relates only to one particular sector of the building process. Quality assurance should comprise the entire building process from the design to the final handing-over of the completed project.
Measures for reducing the probability of failure

- eliminate hazards (loads and influences)
- take account of specific abnormal loads
- incorporate extra strength in the connections
- quality assurance:
  - specify duties and responsibilities
  - provide complete and clear details
  - introduce effective control during design and construction

Measures for reducing the risk of progressive collapse

- install expansion joints or discontinuities
- good interconnection (tying) of the components
- establish alternate load paths
- provide a tying system within the structure on the basis of specified forces
- use only ductile connections

Measures for conserving structural safety

- instructions for inspection and maintenance
- instructions for dismantling

Fig. 11. Measures to enhance the safety of a structure.

Prevention of progressive collapse:

There are various approaches to reduce the risk of progressive collapse. The first requirement is to limit the primary damage, e.g., by installing expansion joints or by local strengthening. Such arrangements are based on the principle that propagation of a chain reaction will be blocked by discontinuities in the structure. For example, in a structure with loadbearing walls the damage liable to occur can be limited by the installation of joints or cross-walls.

In several serious instances of structural damage a chain reaction was initiated by falling debris. It is therefore necessary to provide tensile continuity across the connections in a way that will prevent a damaged component from falling. This has the further advantage that the affected component will not be completely eliminated from the structural system and may conceivably still make some contribution to the transmission of force.

The capacity of the structure to withstand a local damage caused by abnormal loads and actions to which it may be subjected, is determined by the possibility for the redistribution of forces. In order to investigate whether an alternate load path is indeed available, an analysis will have to be made.
of the load flow in the partially damaged structure. For this purpose a low margin of safety ($\gamma = 1.1$) can suitably be adopted and only a part of the variable loads need be taken into account. Some examples of alternate load paths are shown in Fig. 12.

In large panel structures the structural capacity of the walls can be utilized to bridge over local failure, by cantilever or beam action (Fig. 12a, b). For this purpose continuous horizontal and vertical ties must be installed. To avoid debris loading, an alternate load path should also be available for the floor elements, e.g., by providing vertical ties to the wall above or effective longitudinal ties (Fig. 12c).

![Fig. 12. Some examples of an alternate path.](image-url)

In structures with columns it will be necessary to utilize some form of membrane action as shown in Fig. 12d. The required tensile continuity across the connections can be achieved by providing ties within and around the floor diaphragm. Equivalent solutions for corner columns are not available (Fig. 12a).

Fig. 12f shows how, in a floor diaphragm in which the shear force is transmitted through the grouted joints between the precast units, an alternate load path can be established by the installation of transverse ties. The requisite peripheral and transverse ties for the restriction of collapse can be assessed on the basis of such models.

In practice it may be difficult or indeed impossible to prove that an alternate load path is available. A reasonable alternative in such cases is to specify minimum detailing requirements in such a way that the structure can be presumed to possess adequate integrity. These requirements should include an appropriate tying system of horizontal and vertical ties within the structure.
Design recommendations usually specifying tie requirements in terms of forces, have been issued in several countries.

Connections between the precast elements are weak links in the structure. It is of the greatest importance to ensure ductility within the connections, more particularly for absorbing the energy that is released in the event of abnormal loading. For this purpose the connections should have sufficient deformation capacity.

Conservation of safety:
In order to ensure that the safety of a structure is maintained throughout the service life, the structure must be correctly used and properly maintained. To this end, a "user document" should be compiled for the building owner's guidance, containing instructions on use and maintenance. Essential components should, if possible, require no maintenance; if this is not practicable, they should be conveniently accessible for inspection and maintenance. Such maintenance as is necessary for conservation of structural safety should be specified in the user document. In the case of demountable structures the owner should also be provided with instructions as to dismantling and alterations, as well as with the relevant structural drawings and calculations relating thereto.

4. ECONOMY.
Demountable structures can be adapted to changing requirements of use. This adaptability can enhance the service value of a building and have a favourable effect on its economic life. It is not known, however, what effect this will have on the operating costs. Demountable structures have the additional advantage of the lower cost of demolition, while the dismantled components may have a certain residual value. In the macro-economic context, demountable construction certainly offers advantages in the long term. For individual cases these advantages are not predictable, however. Owners will therefore not be disposed to commit themselves to extra capital investment for the sake of demountability unless it is certain in advance that the building will have to be dismantled and re-erected elsewhere in the foreseeable future. Therefore a demountable concrete structure must not be more expensive than a comparable conventional structure. And, indeed, demountability need not add to the cost; it may even be possible to reduce the cost by using simpler connections and achieving quicker and more efficient erection.

5. CONCLUSIONS AND RECOMMENDATIONS.
Precast concrete structures can in principle be detailed in a way that will allow them subsequently to be dismantled. It is thus possible to adapt a building to changing requirements, so that its flexibility may be enhanced. Demountable structures moreover have the advantage of simple demolition, with a minimum of energy input and nuisance to the environment.

In the design of precast concrete structures particular attention must be paid to the whole conception of the structure, the choice of structural system and the detailing of the connections as well as to the quality assurance throughout the building process. All possible hazards must be considered and it is essential to ensure structural integrity by means of a system of horizontal and vertical ties.
Demountable building construction in concrete is a new technique. For its further development, various aspects will have to be the subject of closer investigation, relating more particularly to:
- the development of simple demountable structural connections possessing adequate deformation capacity;
- solutions for establishing "diaphragm action" of the floors;
- preparing codes of practice for the design, construction and dismantling of demountable structures.

6. REFERENCES.

DEMOUNTABLE CONSTRUCTION OF MULTI-PURPOSE CONCRETE BUILDINGS
IN THE GERMAN DEMOCRATIC REPUBLIC

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SUMMARY

In the German Democratic Republic, components for prefabricated single-storey and multi-storey buildings are produced in large series. These are precast concrete units for two "open" building systems, so designed that they can be erected without the use of in-situ concrete. They thus also incorporate a substantial degree of demountability.

The main problem with regard to demountability and the re-use of the components is that of ensuring structural stability.

A number of practical details of both building systems are presented and described.
GESICHTSPUNKTE DEMONTIERBARER BETONBAUTEN BEI MEHRZWECK-GEBAUEN DES INDUSTRIEBAUES DER DDR

1. BETONFERTIGTEILANWENDUNG FÜR GEBAUDE DES INDUSTRIEBAUES

In der DDR wird der Betonfertigteilbau im umfassenden Sinne in der Baupraxis angewendet. Für die spezialisierten Nutzungsanforderungen wurden komplexe Gebäudesysteme entwickelt und ständig rationalisiert. Die Herstellung von 1,2 Millionen m² Hallenflächen und 0,6 Millionen m² Geschossbauten pro Jahr in Fertigteilkonstruktionen - vor allem für den Industriebau - erfolgt in hochspezialisierten Betonwerken des VEB Betonleichtbaukombinat. Die Montage wird von spezialisierten Betrieben territorial zugeordneter Baukombinate vorgenommen. Für die Projekтирование dieser Gebäudesysteme stehen umfangreiche Katalogwerke zur Verfügung, wobei in der Regel die Projektierung durch spezialisierte Projektierungsbetriebe computergestützt einschließlich Plotterzeichnungen erfolgt.

Für Hallenbauten wird das Bausystem "Eingeschossige Mehrzweckgebäude 83" - "EMZG 83" - angewendet (Bild 1) /1/. 

Für mehrgeschossige Gebäude des Industrie- und Gesellschaftsbaues werden zwei Bausysteme, und zwar der Vereinheitlichte Geschossbau - VGB - und die Stahlbetonskelettausse 75 - SKBS 75 - hergestellt (Bild 2) /2/.


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Einriegelsystem
Eingeschossiges Stützelement
Zweigeschossiges Stützelement
Randstütze (Einriegelstütze)
Außenstütze
Eckstütze
Mittelstütze (Einriegelstütze)
Kragriegel (Randriegel)
Normaldeckenplatte
Stützenumfassungsdeckenplatte
Installationsdeckenplatte, nicht abgedeckt
Randdeckenplatten

Bild 2a SKBS 75 - Einriegelsystem
1 Zweiriegelsystem
2 Eingeschossiges Stützenelement
3 Zweigeschossiges Stützenelement
4 Randstütze (Zweiriegelstütze)
5 Außenstütze
6 Eckstütze
7 Mittelstütze (Zweiriegelstütze)
8 Außenriegel
9 Normaldeckenplatten
10 Stützenumfassungsdeckenplatten
11 Installationsdeckenplatte, abgedeckt
12 Randdeckenplatten

Bild 2b SKBS 75 - Zweiriegelsystem

Die gegenwärtigen Weiterentwicklungen werden durch Forderungen der Materialökonomie und der Energieökonomie bei der Herstellung und Nutzung der Gebäude bestimmt, so daß einige Vorteile der gegenwärtigen Systeme aus der Sicht der Demontierbarkeit durch nachfolgende Maßnahmen reduziert werden:

- Statisch wirksamer Verbund zwischen einzelnen Bauelementen, wie Dachkassettenplatten mit Binder oder Deckenplatten mit Riegel /3/

- Anwendung statisch unbestimmter Systeme, wie Durchlaufriegel

Uber weitere Entwicklungen wurde von Queck /4/ berichtet.

Die Demontierbarkeit von Gebäuden wird im wesentlichen durch die konstruktive Lösung der notwendigen Stabilisierung der Gebäude bestimmt. Der Bausystementwicklung in der DDR kommt dabei entgegen, daß keine seismischen Beanspruchungen zu berücksichtigen sind, wobei jedoch Untersuchungen für die Anwendung dieser Systeme in entsprechenden Beanspruchungsgebieten gezeigt haben, daß eine ausreichende Stabilisierung durch entsprechende Zusatzmaßnahmen möglich ist.

Es wird eingeschätzt, daß eine Reihe von Bauelementen der hier behandelten Bausysteme demontierbar und wiederverwendbar sind; die Stabilisierungselemente sind jedoch in der Regel nach der Demontage nicht wiederverwendbar.

2. EINGESCHOSSIGE MEHRZWECKGEBÄUDE

2.1. Systemaspekte der Demontierbarkeit

Für eingeschossige Mehrzweckgebäude wird ein Tragskelett, bestehend aus Dachkassettenplatten 6 und 12 m, Binder 9 bis 30 m und in Hülsenfundamenten eingespannte Stützen der Systemhöhe 4,8 bis 14,4 m mit und ohne Konsol zur Aufnahme von Kranbahnen verwendet. Während die Dachkassettenplatten statisch bestimmt gelagert sind, wird die BINDerauflagerung gelenkig ausgebildet, so daß Kopplungskräfte zwischen den Stützen übertragen werden können.

Der Giebel wird als gesondertes Tragelement (Bild 3), bestehend aus im Fundament eingespannten Stützen, die sich nicht an den Randbinder anlehnen, ausgebildet, so daß die Windbeanspruchungen direkt in das Fundament abgeleitet werden und jederzeit der

Die Wandplatten der Systemlänge 6 m werden aus Gasbeton oder Leichtbeton selbsttragend vor die Stützen gestellt.

2.2. Spezielle Verbindungsdetails

2.2.1. Dachkassettenplatte - Binder


2.2.2. Binder - Stützen

Die Verbindung Binder - Stützen ist gelenkig, wobei eine gewisse horizontale Koppelkraft auf Grund der Binderauflager pressung von der einen Stütze auf die andere Stütze übertragen wird. Zur Sicherung dieser Horizontallastübertragung werden Bolzen einbetoniert (Bild 4), die Demontage ist möglich.
2.2.3. Stützen - Fundament

Grundsätzlich werden die Stützen in Hülsefundamente einbetoniert. Unter dem Gesichtspunkt der Demontierbarkeit sind hierzu Erfahrungen der Österreichischen Bundesbahn für die Gründung von Masten bis 18 m von Interesse, indem die Zwischenräume zwischen Mast und Hülse nur mit Splitt ausgefüllt werden, so daß die Demontage der Masten (Stützen) jederzeit möglich ist.

2.2.4. Wand - Stütze


![Bild 5 EMZG 83 - Vertikalfuge bei Gasbeton-Wandelementen einschließlich Befestigung](image)

3. MEHRGESCHOSSIGE MEHRZWECKGEBÄUDE SKBS 75

3.1. Systemaspekte der Demontierbarkeit

Zur Stabilisierung der mehrgeschossigen Gebäude werden zur Sicherung einer effektiven Montagetechnologie die Decken nachträglich mittels monolithischer Ringankerausbildung und Fugenbewehrung als Scheiben ausgebildet und spezielle Vertikalscheiben bzw. Aufzugsschächte angeordnet. Damit können für das Tragskelett (Riegel, Stützen) einfache Fertigteilverbindungen angewendet werden, die eine spätere Demontierbarkeit sichern (Bild 2).
Für die Einzelelemente (Decken, Riegel, Stützen) werden statisch bestimmte Lagerungen vorgesehen, wobei eine Besonderheit darin besteht, daß die Stützen teilweise bis zu 3 Geschossen durchgehend hergestellt werden. Ein besonderer Schwerpunkt der Elemententwicklung war das Deckensortiment (Bild 6).

**ELEMENTEGRUNDTYPEN**

![Deckenplattenarten](Bild 6 SKBS 75 - Deckenplattenarten)

Durch die Installationsdeckenplatten sowie die kombinierte Anordnung von Randdeckenplatten und Treppenhausdeckenplatten können beliebige Deckenöffnungen ohne Monolithprozesse realisiert werden (Bild 7). Die Randdeckenplatten haben die Wandplattenlasten aufzunehmen und besitzen Ausklinkungen zur Anordnung des Ringankers in Ort beton. Für entsprechende Anordnungen im Gebäude besitzen auch alle anderen Deckenplattenarten entsprechende Ausklinkungen für den Ringanker jeweils an der Stirnseite. In der Regel erhalten die Decken bis auf hohe Laststufen keinen Aufbeton.
Die Vertikalscheiben werden in der Regel aus Fertigteilen gebildet, indem in später ausbetonierte Kanäle Bewehrungsstäbe einbetoniert werden, die an Stoßstellen verschweißt werden (Bild 8).

**ELEMENTEFORMEN**

- Normalelement bei außenliegenden Kanälen
- Element mit Schweißfenstern bei außenliegenden Kanälen
- Element mit Schweißfenstern bei innenliegenden Kanälen
- Element mit Podestausklinkung

Die Vertikal- und Horizontalscheiben werden schubfest zur horizontalen Kraftübertragung innerhalb bewehrter Ort betonbereiche miteinander verbunden. Die Vertikalscheiben werden nach Möglichkeit als Treppenhaus scheiben verwendet.
Es wird eingeschätzt, daß die aus Fertigteilen ausgebildeten Horizontal- und Vertikalscheiben zwar demontierbar sind, eine Wiederverwendung der Fertigteilelemente in der Regel jedoch nicht möglich ist. Für die Riegel und Stützen besteht eine Wiederverwendungsmöglichkeit. Die Wandplatten – Leichtbeton-Mehrschichtaußenwandplatte mit speziellen Oberflächenstrukturen und dem Querschnittsaufbau (von innen nach außen):
- 15 mm Putz
- 50 mm Wärmeschicht (z. B. Polystyrol)
- 120 mm Tragschicht (Stahlbeton Bn 250)
- 40 mm bzw. 90 mm Vorsatzschicht
werden durch spezielle Befestigungsmittel direkt auf den Deckenplatten und an den Stützen (Riegel) als vorgehängte Fassade befestigt und sind voll demontierbar.

3.2. Spezielle Verbindungsdetails

3.2.1. Deckenplatten - Riegel/Riegel - Stütze


3.2.3. Stütze - Stütze

Die Stützen erhalten eine spezielle Zentrierungsaussparung, die der Sicherung der Montagegenauigkeit dient (Bild 9).

Bild 9 SKBS 75 - Zentriereinrichtung zur Stützenmontage
In der Einführungsphase des Bausystems vorhandene Mattenbewehrungen der Fugen sowie die Einfassung der Stützenköpfe durch Stahlrahmen konnten durch die gesicherte hohe Genauigkeit der Stützenabmessungen und der Montage /6/ entfallen. Im Stützenstoß wird nur eine Mörtelfuge vorgesehen. Eine teilweise Einspannung der Stützen im Fundamentbereich zur Erleichterung der Montage ist als Variante möglich.

3.2.4. Wandplatte an Decke/Stütze

In die Wandplatten werden Stahleinbauteile in Form eines Kragelementes (Pratze) einbetoniert, das voll justierbar auf ein entsprechendes Stahleinbauteil in der Decke abgesetzt wird. Gleichzeitig erfolgt eine Verbindung mit der Stütze, wobei eine weitere Justiermöglichkeit gegeben ist (Bild 10).

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DEMONTABLE PRECAST INDUSTRIAL BUILDINGS

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SUMMARY

Precast structures are well suited for demountability. Only the connections have to be adapted. The paper presents a system of precast demountable industrial buildings. The ERGON system for industrial building construction is based on four components: columns, beams, facade panels and roof panels. The structure is very simple: frames, fixed in the foundations, support the roof and wall cladding. The demountable connections are of course the most important link of the system. The different types of connections are described and illustrated in the report.
DEMOUNTABLE PRECAST INDUSTRIAL BUILDINGS

INTRODUCTION

Precasting from itself is extremely well suited to pass on to fully demountability of structures. Indeed, it is sufficient to adapt slightly the connections between the elements to get this goal. In case of industrial buildings, this adaptation is very simple.

In the following, a system of precast demountable industrial buildings is described.

THE ERGON SYSTEM

General

The ERGON construction system for industrial buildings is build up with precast prestressed and reinforced concrete components. The system thus makes use of series of standardised and modulated columns, beams, floor elements and wall and roof panels. They are available in different profiles and lengths, enabling to solve all current construction problems, whatever the building surface and height may be.

Precast components

Columns with rectangular or square sections are manufactured in prestressed concrete for normal loaded slender columns and in reinforced concrete for heavier loadings and sections. The preferential free column height is 7.20 m for single story industrial plants. With this dimension, the height under the beams is 6.00 m. All other dimensions are of course possible, the maximum overall length of the columns being 20 m.

In the Ergon system, the columns are clamped into the foundations.

Beams are of prestressed concrete, and have a I-shaped section and a constant height (I-beams) or a variable height (IV-beams). The IV beams are used for saddle roofs (6°) and standard single spans range up to 40 m. The I beams are used for flat roofs or roofs with larger slopes than 6°. The maximum span is 34 m.

Columns and beams are assembled to frames which are put at a standard distance of 6 and 12 m. Other distances are however also possible. These frames may be combined to form multi-span developments and need not be necessarily spans of equal width. ( fig.1 )

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150
Where primary and secondary beams are required, it is normal to use I beams for the primary beams and IV beams for the secondary beams. This solution is used e.a. for large free spaces with a minimum number of columns. (fig. 2)

Fig. 2: Primary and secondary beams

With a purlin roof, special trapeziform purlins are used with a maximum length of 12 m. The intermediate distance between purlins is normally 3 m. (Fig. 3)

Fig. 3: Primary beams and purlins

Roof linings

The precast frame structures have been designed to accept any form of roof lining and sheeting. The standard solution however consists of roof panels in cellular concrete of 200 mm thickness and 6 m length (see fig. 1), or panels of 100 mm thickness and 3 m length (see fig. 3). Provisions can however be made for other forms of roofing like metal, asbestos cement, or even hollow-core concrete slabs.
Wall cladding

Wall cladding can be done either in vertical decorative prestressed concrete sandwich panels, ERGODAL, or in cellular concrete panels. Ergodal panels are industrialised sandwich facade panels in prestressed concrete of 600 mm width and up to 9.50 m height. The two concrete leaves are completely separated by an insulation layer of 40 to 80 mm thickness in expanded polystyrene or in polyurethane foam. The finishing of the exterior concrete leave is of water washed aggregates, the interior leave is of smooth concrete. The K-value of the sandwich panels varies between 0.82 W/m²°K to 0.33 W/m²°K according to the type and thickness of the insulating material.

The cellular concrete wall panels have a standard thickness of 150 mm and a max. length of 6 m. They are normally placed in a horizontal position and fixed to the frame columns. The roof parapet is supported by a metal T-profile in the prolongation of the column and attached to anchor rails at the top of the column (Fig. 4).

Fig. 4: Exploded view showing a parapet detail

THE DEMOUNTABLE CONNECTIONS

Connections are undoubtedly the most important link in the demountability of precast structures. The different standard connections of the Ergon Demountable Industrial Building system are given hereafter.
**Column-to-foundations**

The standard solution is a bolted connection. The columns are provided with a metal foot frame welded to the main reinforcement. This frame is fixed on site to the foundations by means of holding down bolts (fig. 5). During erection care has to be taken to protect the bolt ends before casting the floor.

![Fig. 5: Column-base connection](image)

**Beam-to-column**

Beams are connected to columns by means of dowels with threaded rods projecting from the columns into conduits or steel tubes cast into the beams. The rods are covered with a plastic sheath. A metal distribution plate is placed over the threaded rods before bolting. The tubes are filled up with bitumen.

![Fig. 6: Beam-to-column connection](image)
The same solution is also used for connections between primary and secondary beams.

**Purlins-to-beams**

Purlins are fixed to the beams by means of a specially conceived metal supporting shoe. This shoe is bolted to the beam and small plastic wedges are driven between the joist end and the metal shoe (fig. 7). In case of metal roof sheeting, a complementary hold down metal bridge is fixed over the purlins to the same bolt.

![Fig. 7: Purlin-to-beam connection](image)

**Roof slabs-to-beam**

The cellular concrete panels are fixed to the beams by means of galvanised steel blades, anchored to the beams and nailed into the cellular concrete. The joints between the slab ends and the beams are grouted with dense or lightweight concrete. The longitudinal joints between the cellular concrete panels present a tongue and groove profile and do not need to be grouted (fig. 8).

![Fig. 8: Fixation of roof slabs](image)
The Ergodal panels are attached to a plinth element and an edge roof beam by means of clamps and bolts. The plinth elements are also bolted to the columns by angles. The panels present a tongue and groove profile and the joints are filled up with an elastomer mastic (fig. 9).

Fig. 9: Fixation of Ergodal panels

The Ytong wall panels in cellular concrete can be erected either horizontally or vertically. The fixation of the vertical panels to the structure is similar to the Ergodal panels. The horizontal panels are attached directly to fixings provided in the facade columns (fig. 10).

Fig. 10: Exploded view of the ERGON system
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SUMMARY

The CD 20 system is a system comprising columns and floor slabs. It is a demountable system used for schools and office buildings. Shear walls for stability are unnecessary in buildings up to two storeys in height. The overall stability and lateral rigidity of the building can be assured by means of rigid connections between the columns and floor slabs.

The lateral rigidity was tested on a full-size (1:1 scale) experimental part of a building. With a minor modification of the loads it was also possible to test the strength of the connections. The conclusion was that in a two-storey building, i.e., with a ground floor and a first floor having a depth and width not less than 14,4 m, no additional stability walls are necessary.
1. Introduction

The system is developed by Copreal BV, Weesp, Netherlands. The CD 20 system is a system comprising columns and floor slabs. The standard column has cross-sectional dimensions of 200 mm; its length is equal to the storey height minus the floor depth. The floor slabs have dimensions of 2400 mm x 7200 mm and are 200 mm in depth; they are prestressed in two directions. These floorslabs are T-shaped. Also floorslabs with a length of 5400 mm and 4800 mm are available, these slabs are hollow core slabs and not prestressed.

The system is used for schools and office buildings. For schools, in particularly, a demountable construction system is desirable for adaptation to changes in the age of the population in some areas. The buildings have columns spaced 2400 mm centre-to-centre in one direction and 7200mm, 5400 or 4800 mm in the other direction. No other column spacings are possible. The reinforcement in the floor slabs is always the same. Maximum permissible imposed load is 400 kg/m². The reinforcement in the columns depends on the height of the building.

A steel element consisting of two angle irons welded together is installed at each corner of a floor slab (Fig. 1). The prestressing tendons are anchored by means of nuts to these elements. The bearings of the slab on the columns are formed by steel plates.

At each end a column is provided with four cast-in threaded bars as well as a 180 mm x 180 mm steel plate applied to the end face. The threaded bars engage with holes in the steel corner elements of the slabs. After the upper column has been installed, the interior of the joint is filled with non-shrinking grout (Fig. 2).

This grout is used only in the column connections. The joints between the floor slabs are filled with a low-strength mortar (1 part of cement, 6 parts of sand). At the outer wall columns steel tubes are additionally installed to fill in the spaces between upper and lower columns at the edges of the floors (Fig. 2).
2. Intention of the test.

According calculations shear walls for stability are necessary in buildings up to two storeys in height. The overall stability and lateral rigidity of the building can be assured by means of rigid connections between the columns and floorslabs. The calculations showed, that the strength as well as the stiffness satisfied the requirements. Infinite stiff connections were assumed. The lateral displacement of the top floor was calculated as $\frac{1}{1100}$ of the height, long term effects included.

The influences of the manufacturing tolerances and the influences of the not infinite stiff connections were not known. This is why a practical test has been done. The primary intention was to test the overall lateral stiffness of the building, but a small modification of loading has made it possible to test also the strength of the connections. It was not the intention of the test to prove the strength of the concrete structure.
Fig. 2: Connection facade-column/floorslab.
3. Stresses in the joints.

In fig. 3 the theoretical moment-distribution is shown. The load is a lateral force at the roofslab and at the middle slab. Also an unequal vertical load gives moments at the joints. The slabs are prestressed, so the weight of the floor slabs causes no moments at the joints.

Fig. 4 shows the stresses in the connections. The joint itselfs is mainly loaded by shearforces. The moment above the joint gives pressure at the right side of the column. The vertical force in the column moves as far as possible to the right. The rest of the moment has to be provided for by the glued threaded bars. (forces F1 and F2). At the lower column the forces are the same, but here the vertical force in the column moves to the left.

So at least the complete vertical load in the column will be moved from the right to the left. This in the depth of the joint. This is why large shear stresses have to cross the non-shrinking grout between the steel parts of the joint. The grout has to be of very high quality. These shear stresses are the same in the horizontal plain. In this direction the steel parts of the joints can provide these stresses. The lateral force H will be distributed over the several parts of the joint by the steel plates at the ends of the columns. The threaded bars are dowels between steel plate and joint.
fig. 4 Stress in the joint.
1: floorslab
2: columns
3: cast-in-iron in the floor-slab
4: glued threaded bar
5: steel tube with plates at the ends
6: steel plate
7: non-shrinking-grout
8: welded bars.
The testmodel was a part of a building which consists of a groundfloor and a first floor. The model was scaled 1:1. The depth of the model being 14,4 m and the width 2 x 2,4 = 4,8 m (see foto).
The calculations showed that the most unfavourable load for the connections was a windload in a direction parallel to the longest span (7,2 m) combined with a vertical load, unevenly placed in a "chess board" way.
The foundation was simulating foundation on piles so as widely used in The Netherlands. The total amount of floorslabs was 2 x 4 = 8 and the amount of columns was 2 x 3 x 3 = 18.
The vertical load was applied by placing two prefab floorslabs and a number of columns on the middle slab and roof. A special frame was erected to apply the lateral loads. The lateral load was distributed in the right ratio over the several points by means of cables which were guided over pulleys.
The lateral forces were measured at five spots in the cable system by means of electrical loadcells. The lateral displacements were measured by means of a frame which was placed to Yl. This frame was separately placed on the foundation. Dial indicators were attached between this measuring frame and the testmodel at the groundfloorslab, the middle floorslab and at the rooftslab. To obtain an impression of the rotation inside the connections, angular rotation measurements were done at 12 positions. This was done by means of a strip fixed at the end of the column and an angle iron fixed at the floor or floorslab. Between these two points, two dial indicators 800 mm on center were brought in.

5. The test
The test was split up in two parts, the stiffness test and the strenght test. At the stiffness test the vertical load was an unequally distributed load which was equal to the normally to be calculated live load. The lateral load was applied in steps up to about 120 % of the normaly windload. Hereafter the lateral load was reduced to 0. At the strenght-test the vertical load was 1,2 times the vertical load at the stiffness test (own weight included).

6. Results and conclusions
The most important results are shown in grafics 100 and 101. The lateral displacement of the building under service conditions was about \( \frac{1}{1800} \) of the height.
STIFFNESS TEST

X-AXIS: LATERAL DISPLACEMENT MIDDLE SLAB
Y-AXIS: TOTAL LATERAL LOAD

CD-20, STABILITY TEST

X-AXIS: LATERAL DISPLACEMENT MIDDLE SLAB
Y-AXIS: TOTAL LATERAL LOAD

STRENGTH TEST

X-AXIS: LATERAL DISPLACEMENT MIDDLE SLAB
Y-AXIS: TOTAL LATERAL LOAD
STIFFNESS TEST
X-AXIS: LATERAL DISPLACEMENT ROOFSLAB
Y-AXIS: TOTAL LATERAL LOAD

CALCULATED LE
MEASURED

STRENGTH TEST
X-AXIS: LATERAL DISPLACEMENT ROOFSLAB
Y-AXIS: TOTAL LATERAL LOAD

CALCULATED LE
MEASURED

CD-20, STABILITY TEST
STIFFNESS TEST AND STRENGTH TEST

X-AXIS: TOTAL LATERAL LOAD
Y-AXIS: RATIO OF THE NONLINEAR PART OF THE DEFLECTIONS TO THE LINEAR PART

JOINT GRIDLINE Y1
MIDDLE FLOORSLAB
ROOFSLAB WITH REGARD TO MIDDLE SLAB

GRAPHIC NR: 103 A
In case this displacement due influence of creep etc. will increase twice as much, the total displacement will be then \( \frac{1}{900} \) of the height.

For this kind of buildings \( \frac{1}{500} \) of the height is allowed.

From this we can conclude that the system is in the lateral direction sufficiently stiff.

The influence of the not infinitely stiff connections is restricted, but not negligible.
At the middle floor about 15 to 35% of the lateral displacement is caused by the rotation of the connections.
About 65% of the lateral displacement at the roofslab with regard to the middle slab, is caused by the rotation of the connections.

The behaviour of the connections is not significantly different from the behaviour of the concrete structure as a whole.
The non-linear-elastic behaviour of the joint rotation is not significantly different from the behaviour of the structures, as is shown in grafic 103.

The remaining deflections after releasing are as large as can be expected by a cast in situ concrete structure.

The strength of the structure under lateral loads is sufficient.
The total lateral force which was applied was about 5 times the service load. This was not the ultimate state.
The structure which was used to apply the lateral forces was not strong enough to increase the load further more.
According a theoretical linear-elastic moment distribution calculation, the forces on the joints were 2,75 times the forces under service load (average of 36 connections).
The connections are sufficiently safe.

Summarized the conclusion is that in a building, made with the CD 20 system, with a groundfloor and a first floor, with a depth and width which are not less than 14,4 m, no additional stability walls are needed:
The overall stability can be provided by the system of rigid connections between columns and floor slabs.

7. References

THE DEMOUNTABILITY OF A VERTICALLY LAYERED STRUCTURE
OF (LOADBEARING) CONCRETE FACADE ELEMENTS

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SUMMARY

Buildings with a loadbearing and stabilizing facade are an alternative to demountable buildings in respect of prolonging their economic life.
As far as the physical building aspects are concerned, such a facade should have a layered composition. The present method employed in fixing the layers however does not provide demountability.

There are a number of reasons for seeking methods of so constructing a vertically layered facade that the outer layer can be demounted.
The advantages can be summarized as follows:
- the factory process is less sensitive if mistakes are made;
- it is possible to replace damaged elements while the building is in use;
- the architectural character of the building can be adapted to its new function by altering the outer layer.
THE DEMOUNTABILITY OF A VERTICALLY LAYERED STRUCTURE
OF (LOADBEARING) CONCRETE FACADE ELEMENTS

1. Bearing facade elements

The economic longevity of a building is far less than its technical longevity. Therefore from a financial point of view it is preferable to coordinate one with the other as much as possible. Two principally different possibilities exist to achieve this. The design would then have to consider:

1.1. future demountability;
1.2. a future change in function.

1.1. Future demountability

When a building is no longer considered economically viable it is possible for it to be demounted and built up again (elsewhere), either completely or partly. It is also possible to incorporate any such future plans for a building in the design stage and thus later adapt it to a changed requirements programme or even a different one. One example that can be mentioned here is an 11-storey block of flats in Middelburg built from a brick construction system (BWB). See fig. 1. Managerial considerations deemed it necessary to demount the upper seven stories and set them up elsewhere as a new block of flats. Although a possible dismantling was not incorporated in the design criteria during the planning stage, this building system proved its adaptability (fig. 2).

1.2. A future change in function

The risks run by the investor are greatly lessened if the design incorporates a change of function. When, during the planning stage allowances are made for a building to change its function in the future, a different type of design is achieved then that for a specific requirements programme.

To achieve maximum flexibility a bearing structure must be designed with a minimum number of cores, shear walls and columns. A building that has a bearing and stabilising facade goes a long way towards meeting the conditions for flexibility (fig.3). The main aim of this sort of design is to provide the utmost amount of flexibility. At present the demands for flexibility are mostly being made by institutional investors. Often at the start of the design stage the future occupant is unknown. Sometimes this is still true when the building has been completed.

Besides the demands for flexibility, physical building aspects play a great role in today's building world. Thus a minimum consumption of energy coupled with a comfortable interior temperature must be
Fig. 1 B.M.B. Facade element

Fig. 2 Flats in Middelburg
fig 3  bearing façade element

fig 4

fig 5.
considered. To meet this demand the bearing facade elements have to be composed of several layers, as follows:

- a bearing and accumulative inner layer;
- an insulating layer in between;
- a protective weather layer (fig. 4)

Moreover there is a preference to design an air cavity between the insulation layer and outer layer. The advantages of this are that the insulation stays dry and the outer layer dries more quickly. Dampness fosters the adhesion of dirt and probable biological growth. The inclusion of an air cavity aids in controlling the pollution process on the facade, that and the proper design and detailing.

The choice of material and the composition of the outer layer aspire to represent the visual aspects of the building's character. Functionally all materials are suitable for the outer layer so long as they meet the aesthetic and durable requirements and moreover are capable of resisting rain, wind and mechanical influences. Not so many types of materials are suited for the inner layer. Concrete mainly meets the requirements for strength, rigidity and accumulative properties.

Various sorts of bearing facade elements can be created by combining a concrete inner layer with any of the materials used for the outer layer. The most well-known are elements with an outer coating of cement or cement reinforced with glass fibre. Less well-known are, for example, masonry, natural stone or steel profiles (sections) filled with glass and wood. Metal is used to connect all the combinations that can be employed. As the durability of these connections is of utmost importance, stainless steel with standard specifications is mainly chosen.

The connections which are pre-prepared at the cement factory and fitted on the building site can be distinguished by the connecting method employed for the inner and outer layer. It does not always follow that demountable building is possible when the outer and inner layers are connected. Adjustments are often necessary if an outer layer has to be demounted when the building is in use. One example is connecting natural stone to concrete (fig. 5). Replacement is necessary if a crack appears in a slab of natural stone. The method used at the moment is to remove the damaged slab, saw through the anchor pins and drill holes in the same place. The new natural stone slab is provided with deeper holes and a spring is inserted together with a new pin. The new slab is placed in position, the pin fixative dissolved and the pin pushed into the facing pin hole by the elasticity of the anchor. Such necessary adjustments are caused by the present design method which concentrates on a once only realisation and not future changes.

The other connecting method involves the concrete inner layers being fastened to an outer layer, for example, also of concrete, during the production process at the factory. In a number of instances such as the "Paper Clip" building in Rotterdam, the Felleenoord Complex in Eindhoven and the University Hospital in Utrecht (fig. 6,7,8), design differences in the connecting method can be perceived but they are all wet connections. The pre-
Fig. 6 The "Peperclip" building in Rotterdam
- both layers of the Facade element are produced in one mould
- steel plates take care of the connection between the inner and outer layer of the element
Fig. 7 The Fellenoord Complex in Eindhoven
Fig. 8 The University Hospital in Utrecht
- inner layer of the Facade element
- the fixed connection is placed in the upper hole
- at the bottom holes thin steel plates take care the rotation-stability
fabricated outer elements cannot be removed unless demolished.

The principle question that can be posed here is "is it possible (or desirable) to design the connections between the inner and outer layers in such a way that the outer layers can be replaced?"

2. Demountable connections

The reasons for making demountable connections between the inner and outer layers are:

- sub-standard outer layers caused by damage, colour differences and tolerance excesses such as measurements and smoothness are usually discovered after both facade elements are already connected. The complete facade element then has to be discarded;

- the difficulty in repairing sufficiently any damage that occurs after the outer layer has been assembled. It is practically impossible to repair surfaces which have been designed as an impression of the form. Although an experienced workman can achieve a reasonably acceptable repair job with care and skill, the damage will become visible in the course of time. The problem is the compactness of the prefabricated concrete and the repair mortar differs;

- a renewed function for a building can mean that a new facade (outer layer) has to be realised. Usually management policy or the necessity of complying with insulation requirements deem it necessary. Moreover the architectonic character of the building can be changed to conform with the new function it is to receive.

3. Survey of the existing joint systems for outer layers

   . vertical joints
     a. open joint (fig. 9)
     b. closed joint filled with sealant in one or two layers (fig.10)
     c. closed joint sealed over by a plastic rain protector and combined with an expansion space (fig. 11)
     d. closed joint with plastic profile in a cemented sliding system (fig. 12)
   . horizontal joints
     e. open joint (fig. 13)
     f. open joint with weather cornice in the outer layer (fig. 14)
     g. closed joint filled with sealant in one or two layers
     h. closed joint with plastic profile in a cemented sliding system (fig. 15)
a. open voeg (fig. 9)

b. met ketting (fig. 10)

c. regenwater en ontspanningskelder (fig. 11)

d. kunststof profiel (fig. 12)

e. open voeg (fig. 13)

f. open voeg met waterslag (fig. 14)

h. kunststof profiel (fig. 15)
Nine realistic possibilities are achieved by combining the various systems for vertical and horizontal joints.

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When designing demountable connections the sort of connection is also dependent on the size and weight of the outer layer. If the outer layer is of slight weight, it is possible to use connections under strain of bending. An example of this can be observed in outer layers made of natural stone, brick or cement reinforced with glass fibre. For larger layers with a heavier load it is mainly possible to use fastenings with a normal force load. Here a difference can be made in connections that:
- bear the outer layer on the upper side while keeping the outer layer at a distance underneath (type A), and those that
- bear the outer layer underneath while keeping the outer layer at a distance on the upper side (type B). See fig. 16.

The choice of the type of connection (type A or B) depends on the joint system to be used. Thus all the f-joint combinations have a weather cornice which prevents a horizontal translation of the outer layer when demounting.

With a type A connection, which bears the outer layer on the upper side, it is possible to pull the outer layer from underneath, away from the facade surface. The upper connection can then be loosened and the outer layer removed.

The h-joint combinations that are being used a great deal at present in facade constructions, are not damaged so much when being demounted if the outer layer is demounted as described above.

4. Recommendation

As far as is known to me, demountable connections have not been used in bearing facades with a layered composition. Further
research into the subsidiary conditions applying to demountable connections is necessary. Such research would have to include the architectural consequences of the demountable connections that are to be employed and the influence brought to bear on the design of the outer layer by the material used.

Furthermore it is recommended that - in view of the present developments in the composition of joints - the outer layer be connected by hanging it from its upper side. The aspect of durability should also be taken into account by pre-reinforcing concrete outer layers so as to avoid tensile cracks.

Further research which would involve other disciplines, could lead to the perfection of the connection method and thus the realisation of a layered and demountable facade composition.
A perfect component of construction should have an unlimited ability for direct, instantaneous and unaided interconnections resulting in endless variety of assemblies. Nature itself, however, fell short of this ideal.

Precision of shape and ability to achieve direct and instantaneous connections are the two features which at the same time are the most essential and the most interdependent. These in general continue to elude the building industry. It is clear that precision in buildings would require an acceptance of mechanical engineering discipline and in addition allow, without producing adverse effects, a greater order of deformation than mechanical engineers normally do.

There always were, and still are, two basically correct approaches to this fundamental problem of connection between basic elements.
1. Connection with generous tolerances;
2. Connections approaching perfect fit.
In the first category the connection is always indirect, through an additional medium. While production of "inaccurate" elements is still reasonably simple and economic, their assembly is correspondingly slower and the quality of joints often suspect, to say the least.

The real prefabrication, with perfect fit, not only speeds up construction processes and improves the quality, but also decreases costs.

This paper discusses the philosophy of joint design and demonstrates how high-quality jointing could result in economies as well as open up the field for demountable concrete structures.
DESIGN AND BEHAVIOUR OF DEMOUNTABLE CONNECTIONS

1. INTRODUCTION

Before the First World War both houses and cars were made by hand, and at that time the price of a medium car was roughly double that of an average dwelling. The car industry embarked almost instantly on intensified industrialization, and as a result, by the time the Second World War started, the price of a car had dropped by half.

At present the average cost of a very modest dwelling is in the region of £ 35 000, while, at the same time, £ 3 500 is enough to buy a modest car. Thus, a mere three quarters of a century has witnessed a tremendous transformation of the car industry from its infancy to its present state, while the building industry, the oldest known to humanity, remains relatively primitive. The concentrated capital investment and intense competition in the car industry created conditions under which it was both possible and imperative to invent, conceive, and not only plan, but carry the plans out. As a result the price of a car was reduced twenty times relative to that of a house.

Unlike precast large-panelled construction, a car has a clear distinction between the skeleton and the body. This mirrors the difference between primitive, prehistoric creatures and highly developed mammals.

Progress in the car industry has not been achieved by avoiding difficult issues. One of the most basic and fundamental, was that which still bedevils the building industry, namely the problem of "fit", indivisible from "joint". A car has a distinct structural skeleton - the chassis; this in turn supports a non-structural body which gives it its weather protection. The structure also supports the digestive, energy creating, respiratory and traction systems.

It should be noted here that each of these systems is necessary and sufficient to perform only its own function. The structure, however, is a prerequisite for the very existence of the car itself.

Had the car industry persisted in making cars by hand or alternatively tried to make its structure the body and other systems as a single moulded unit, it would not be where it is now - it would have simply died a natural death at birth.

In the construction industry, such division of functions is equally essential and the efficiency of jointing is a reliable indicator of the degree of sophistication which has been reached.

"Fit" is of key importance in the practical implementation of dimensional coordination of standardized components. Traditionally it was a craftsman's responsibility to ensure that components fitted together. The designer could assume that as long as his principal details did not depart from customary practice, the final result would be satisfactory, provided that suitable craftsmen were employed in construction.
Building methods remained stationary over a long period of time. The established procedures ensured automatic compensation for the considerable dimensional and positional deviations arising in the assembly of structures. Traditional methods, however, are totally unsuited to the state of flux where new materials, new inventions and new procedures literally tumble over each other. In addition, with the gradual disappearance of craftsmen, the responsibility of accommodating the deviations falls on the designer. The design must thus become fully interdependent with construction. It becomes necessary, at the design stage, to give explicit consideration to tolerances and fits in determining the joints.

Prefabrication, in which precast structural concrete plays a primary role, cannot normally make economic sense unless mass production is achieved. This in turn requires a continuous demand for standard components which can be produced by the manufacturers for a reasonably stable market and which can be assembled by an average contractor without undue risk and was that which still bedevils the building industry difficulty.

The efficiency of connections relates directly to the dimensional accuracy of components and there are only two basic approaches to this fundamental problem:

a. Connections with generous tolerances.
b. Connections approaching perfect fit.

Of necessity, the connection in the first category is, in most cases, labour-intensive and indirect, through an additional medium, like mortar, in-situ concrete, mastic or resilient bearings.

Present-day techniques of production of components falling into category a) are reasonably simple and economic. Their erection, however, is correspondingly slower, as the elements require independent setting out, lacking self-positioning capacity.

Moment-resisting capacity of such joints is also difficult to achieve, unless by the use of in-situ concrete, and then a monolithic joint is produced. Monolithic joints by their very nature are not demountable. Consequently, with the exception of simpler forms of flooring, roofing or minor bridges, the connections with generous tolerances do not give a wide range of applications. However, this must not be confused with the fact that the field of these applications is considerable indeed.

2. "PERFECT FIT" - MECHANICAL ENGINEERING CIVILIZATION

The construction industry will not utilize all advantages of mechanical civilization unless and until joints like those illustrated in Figs. 1, 2 and 3 become commonplace. Strangely enough, but quite logically, jointing of this nature would create a breakthrough not only in modular coordination, but would similarly open doors to the highest quality mass production. It would also produce "a self-positioning structural joint which at the same time is totally rigid and demountable."
Modularly coordinated standard components in a small range of types and sizes can create a basis for real mass production. In that case a reasonable degree of accuracy for fit is achieved almost as a by-product, free of charge.
production a coarse tolerance will have to be accepted as the only common-sense measure, and joints will always require additional treatment with careful supervision.

It must be remembered that, considering accuracy of building components, civil or structural engineers cannot hope to have solved their problems completely by just resorting to machining like mechanical engineers. They must reflect, in addition, on a degree of dimensional stability of their materials and, even if these are as stable as steel, the deformations in relation to size are bound to cause yet another problem.

Concrete, the predominant material of the construction industry, is still, on the one hand, the same primitive material, dependent on so many factors for its success, as it was in Roman times, but it can also be highly sophisticated.

Fickle materials, and the size of construction elements, not the conservatism of civil engineers, are the basic reasons why the setting-out of buildings has remained an operation involving the use of instruments, since the components are not self-positioning. It is correct to say that it makes no fundamental difference whether the setting-out of a right angle is done by using string with equally spaced knots, as the ancient Egyptians did, or by using a sophisticated modern instrument.

If components are self-positioning, setting-out is inherent in the assembly. The advantages are enormous and the construction industry must advance in this field.

The application of precision structural components and joints between them can produce not only rigid joints which can also be easily demountable but can indeed open up, enlarge and enhance scope for all industries, trades and professions relating to construction, facilitate unlimited and rationalized output of basic buildings for home and overseas, secure free architectural expression and compliance with different regional requirements and provide a realistic basis for organic modular coordination resulting in a standardization of a high order with enlarged and enhanced architectural vocabulary.

It will create a new outlet for the steel and concrete industries combined, instead of an adversary relationship. A way would be opened not only for increased exports from industrialized countries, but also for providing low-cost housing for the Third World in a combination of a small amount of very high technology merged with a predominance of local resources.

It must be noted, however, that the economy of a concept as basic as this cannot be evaluated efficaciously by traditional methods, comparative cost tables or similar means. It must be seen and considered in the perspective of progressive changes, growths and processes of rationalization, which its promotion is bound to generate, and affecting the total spectrum of construction, the economy, social progress at home and overseas, and the generation of an architecture which will truly and significantly represent our age.
It is abundantly clear that precision in building would require an acceptance of a discipline, but this would be a small price to pay in comparison with a literal tyranny.
which would result from solutions similar in character to large-panel construction.

Similar problems were faced and solved by mechanical engineers during the industrial revolution. New industries stopped fitting components by hand and started controlling their accuracy by machining, thus achieving not only the necessary fit but also, and far more importantly, their interchangeability as well as demountability.

The principles discussed are simply illustrated by two prefabricated standard components loosely assembled yet capable of providing building structures of great variety. The design as well as proposed modular coordination was developed by the late Oscar Singer (architect) and the author. The two parts are a hollow floor-cum-ceiling slab (6m x 3m x 200mm) and steel columns which are jointed together with a male and female coupline. The female part (socket) of the precision joint is cast integrally with the concrete slab, into which the tapered spigots at each end of the steel column fit precisely. The columns, terminated with spigots, have a constant external dimension of 150mm x 150mm with varying heights and varying thickness of the walls. For low structures, the slab can be supported by one, two, three or four columns with very thin walls indeed, while by the time a column becomes a solid steel section a fifty-storey building can be constructed.

The erection of the structure at the site consists in the assembly of slabs and columns, totally dry, see Figs 4 & 5, in a way comparable to the assembly of components in a toy building set. There are no beams. With perfect fit the construction is self-supporting during erection.

The slabs would be manufactured in high-duty steel moulds fabricated or cast in one piece of steel, stress-relieved and then machined to a tolerance of +0.125 mm, -0.010mm. Casting of slabs would always take place at a constant controlled temperature, during casting and vibration the sockets would be secured by means of dowels to the mould itself to ensure total rigidity. Dimensional provisions would be made for creep and shrinkage, but in addition only the essential surfaces of the units already cast would be machined to a final tolerance.

To minimize deviations, due to flexure, horizontal slabs would only register along machined lines, corresponding to the neutral plane of the slab.

The steel columns would also be finally machined at the same constant temperature as concrete slabs after completion of prefabrication.

Suitable diagonal bracing between the columns can provide any degree of wind or even seismic resistance.

Under heavy loads the surfaces machined to fine tolerances will tend to seal. Therefore, whenever demountability is required it was suggested that a slightly coarser tolerance could be provided, and surfaces to be jointed could then be treated with graphite or suitable grease.
It is estimated that in order to achieve cost levels lower than the prevailing market prices a continuous production at a level of about 20,000 joints per annum has to be assured. This is a very small amount indeed when compared with the output of the construction industry, even at the present time of recession.

Once a precision structure is provided, then it could be connected with precision-manufactured non-structural components without any need for special adjustments or provisions.

The scope and capacity of contractors would be widened and their equipment and plant range as well as their work simplified. However, one of the great benefits would be the means for sheltering millions of homeless people in Asia, Africa and South America together with building the necessary hospitals, schools and factories for those areas. Such industrialized construction would provide more rather than less creative architecture, more buildings which could be rapidly erected and where necessary demounted and re-erected. This could mark a turning point in environmental design and the total building economy.

A bibliography is given at the end of the paper where those interested in the finer details of this particular concept can obtain more information.

3. BIBLIOGRAPHY


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PRODUCTION METHODS AND CONNECTIONS FOR DEMOUNTABLE PREFABRICATED COMPOSITE STRUCTURES

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SUMMARY

Speedy and flexible production using prefabricated elements gives rise to the need for the development of mechanical connections. The basic requirements for structural connections include easy installation of the connectors during the casting of the elements in the plant, and subsequently on site. High shear and considerable tension capacities are often required during use.

In this paper, a practical connection method is presented for the composition of elements using precast sub-elements, and for the erection of prefabricated building frames with precast elements. The connection system gives wide independence and flexibility to the production of elements. No precast brackets on the columns are necessary. The shear capacity allows for the use of connections within the whole range of spans used in prefabricated frame structures.

The connections used in this method are formed with high-strength prestressing bolts and dowel plates. The bolts provide the necessary rigidity, bending and tension capacity for the connection. The dowel plate provides the shear capacity. Two types of dowel plate have been developed. One type is based on the penetration of sharp nails into hardened concrete at the connection surface under the action of the prestressing force developed by the bolts, and of the dead weight of horizontal members. In the other type of dowel plate, the rather thick dowels are installed in drilled or preformed holes. The holes for the bolts can either be drilled or preformed during casting.

The application of connections to beam-column joints and to the joints of wall elements is presented. The report also describes the principles of production and design methods.
1. THE ROLE OF CONNECTIONS IN PRECAST CONCRETE TECHNOLOGY

The connections of elements play a central role in precast concrete construction technology. The necessity for a connection technology arises from prefabrication and mounting techniques, functional requirements of structures during use and the need for easy demounting following the period of use.

Traditionally, the connections were generally made from mortar and reinforcing ties. This technique is based on structural principals of the cast-in-place construction. Mortar connections of good quality design and production give good performance properties during use.

During production, traditional types of connections often cause difficulties both in the element factory and on site. The main difficulties in the production by element factories are caused by connection ties and other connection details, such as column brackets. These details cause difficulties in moulding and particularly in the development of modern production such as extrusion and sliding techniques.

On site, the main disadvantage of current connections lies in the great need for labour and prolonged mounting times.

Following the period of use, mortar-tie connections are always difficult to demount. Demounting cannot be effectuated without substantial damage to the elements, rendering eventual re-use of elements difficult or even impossible.

In developing techniques for demountable buildings, there is a great necessity for innovative development of new types of connections. In the case of suitable connection types, requirements for the production process in the element factory and on site should also be taken into account.

2. INNOVATIONS IN DOWEL PLATE CONNECTIONS

2.1 Types of dowel plate connections

At the Technical Research Centre of Finland the dowel plate connection technique has been innovated and developed /1/, /2/, /3/. The following three types of dowel plate connection have been developed:

- Rigid nail plate-bolt connection (Fig. 1)
- Rigid dowel-plate or combined dowel plate-bolt connection (Fig. 2)
- Limited deformable dowel plate or combined stud dowel plate-bolt connection (Fig. 3).

The functional role of the dowel plate is to take the shear force of the connection. The possible tensile forces are taken with high strength prestressed bolts. The force state at the dowel plate is presented in Fig. 4. The shear force between the concrete surfaces to be connected is moved out of the concrete surface into the steel plate, with shear nails or dowels, and again out of the steel plate into the other concrete surface. The shear force
Fig. 1. Nail plate-bolt connection.

Fig. 2. Rigid dowel plate or dowel plate-bolt connection.
1 dowel
2 hole in concrete
3 steel plate
Fig. 3. Limitedly movable dowel plate connection.
Fig. 4. Force state of the nail and dowel plate connection under shear.
Fig. 5. Compression force-nail penetration curver for the nail plate connection.
causes bending into the dowels and also through the dowels into the steel plate. The steel plate should therefore be stiff and sufficiently strong to take the integrated bending moment of the dowels.

The effectiveness of the dowel plate in comparison with the usual shear bars is based on the rigidity and moment capacity of the steel plate, which enables the effective shear resistance of the dowel connection (Fig. 4). The thickness of the steel plate depends on the forces and varies typically between 2 and 4 mm. The productional advantage of this is that the structural elements can be made without any special outstanding ties or other connection details. For the dowel plate connections, the drilling of holes for bolts and duct dowels can be made as a supplementary measure after or during the hardening of concrete.

2.2 Nail plate-bolt connection

The nail plate connection is built by means of the penetration of sharp nails into concrete, using the compression force brought about by the gravitational force of the structures or by the prestressing force of the bolts.

Several force-penetration curves are presented in Fig. 5. The required penetration limit is 2/3 of the nail length. The penetration force for each nail varies between 5 kN and 10 kN. The increase in the number of nails decreases the penetration force for one nail.

The length of the nail may vary, the usual length being 5 mm.

The shear force-slip curve for the nail plate-bolt connection of a column corbel with four prestressed bolts 16 mm in diameter, and with four nail plates each including 24 nails, is presented in Fig. 6.

The shear capacity can be calculated using the following equation:

\[
V_{ud} = \frac{n \cdot f \cdot K}{\gamma \cdot K_f} + \phi^2 \sqrt{f_{cd} \cdot f_{yd}}
\]

where

- \(V_{ud}\) is the design value of the ultimate shear capacity,
- \(n\) the number of nails
- \(f\) the ultimate shear capacity of one nail with the calibration concrete strength,
- \(K\) the concrete cube strength,
- \(K_f\) the calibration cube strength of concrete corresponding to the value \(f\),
- \(\gamma\) the capacity safety factor,
- \(\phi\) the bolt diameter,
- \(f_{cd}\) the design prism strength of concrete (\(\approx 0.5 \text{ K}\)),
- \(f_{yd}\) the design strength of the bolt.
Fig. 6. Force-slip curve for the nail plate-bolt connection of the column corbel.
In the example presented in Fig. 6, the capacity with four bolts of diameter 24 mm and concrete strength \( K = 50 \text{ MPa} \) corresponding to the capacity safety factor \( \gamma = 1.7 \) is about 310 kN.

As a rough estimation, the penetration force and the shear capacity of the nail plate connection have the same numerical value.

If the connection, as, e.g., in Fig. 6, has a bending moment, the bending moment capacity can be calculated by usual methods. The suitable prestressing force following anchorage, creep and shrinkage losses corresponds to the tensile force of the bolts under service loads.

In the tests, the sensitivity of the nail plate-bolt connection to variations in bolt prestressing force, to the rotation or unevenness of the connecting concrete surfaces, has been studied. The test results show that a decrease in bolt prestressing force, down to the final value of 10% of the design value, affects the ultimate capacity by no more than 15%.

Approximately one third of the nail length can be used as an estimation of the permissible unevenness or rotation of the connection.

The toughness of the nail plate-bolt connection is good, the ultimate slip measuring many tens of a millimeter.

2.3 Dowel plate connection

The dowel plate connector is presented in Figs. 2 and 3. The principal difference in comparison with the nail plate connection is the need for drilled or cast-in holes for the dowels. Practically speaking, the steel plate in the dowel plate connection should also be thicker because of the larger bending moment due to the forces at the dowels. Typically, the thickness of the steel plate varies between 3 and 10 mm, the diameter of the dowels ranging from 12 to 16 mm. The necessary thickness of the steel plate can be calculated on the basis of the bending moment presented in Fig. 4.

The shear capacity, stiffness and toughness properties of the dowel plate connector are qualitatively of the same kind as those of the nail plate connector. In the case where the length of the dowel is at least three times the diameter, the shear capacity of the dowel plate or combined dowel plate-bolt connection can be calculated using the following equation:

\[
V_{ud} = 1.2 \left( \Sigma \phi_d^2 + \Sigma \phi_b^2 \right) \sqrt{f_{cd} f_{yd}}
\]

(2)

where

\( \phi_d \) the diameter of the dowel,
\( \phi_b \) the diameter of the bolt,
\( f_{cd} \) the prism design strength of concrete (\( \sim 0.5 \cdot K \)), where \( K \) is the cube strength,
\( f_{yd} \) the design strength of the steel.

As an example, the design shear capacity of a connection with four bolts of diameter 16 mm and four dowels of diameter 12 mm, in concrete \( K 40 \) and steel quality Fe 520, is 219 kN.
Fig. 7. Dowel plate with limited movement as the support connection of the beam.

Fig. 8. Connection of the bending and shear resistant frame edge.
In order to avoid a change of the failure mechanism into a tension failure of the concrete cover, a necessary amount of reinforcement must be placed near the surfaces of the connecting concrete structures.

At the moving supports a limited movement in some direction is required at the connection. In that case, the dowel plate can be designed in order to allow for limited movements, as presented in Fig. 3. The holes of double steel plates are stretched in the direction of movement. After the permitted movement has taken place, the dowel will be interlocked against the connecting steel plate, thus moving the shear force again from one concrete surface to the other. Naturally, if required, a two-dimensional movement can also be allowed.

The combined dowel plate-bolt connection is preferably also made with prestressed bolts in those cases where the connection is subjected to a combined shear force and bending moment or where a high degree of stiffness at the working state is important. The calculation of the bending capacity can then also be effectuated by means of usual methods as in the case of nail plate-bolt connections.

3. EXAMPLES OF THE POSSIBLE USE OF NAIL AND DOWEL PLATE CONNECTORS

3.1 Assets of nail and dowel plate connectors

Nail and dowel plate connectors are suitable for use where a high shear capacity is required. Combined with prestressed high strength bolts, the shear capacity and the working state stiffness can be improved. The bolts also make possible a high bending capacity for the connection.

Advantages in production include the possibility of prefabrication in the element factory, independently of connection detailing, easy mounting on site and easy demounting and re-use of the structures.

Assets of nail and dowel plate connectors during use include statical safety, high safety against progressive collapse, and high safety against fire. Particularly important is the total elimination of progressive collapse, as this is often difficult when using traditional connection techniques. The bending capacity of connections also enables statically preferable multistorey framed structures to be used instead of the usual present-day cantilever column frames. This not only means considerable savings, particularly in column costs, but also some assets for the beams. The economy pertaining to columns can be significantly improved using new production techniques possible with the new types of connections.

3.2 Possible use of connectors

Examples of use include:

- column corbels (Fig. 6),
- beam supports (Fig. 7),
- edge connections of the frames with internal threads for the bolts (Fig. 8),
- wall joints (Fig. 9).

With innovative design, most of the connection problems, particularly between one-dimensional and often between two-dimensional elements, can be solved using the nail plate or dowel plate connectors. The increase in bolted connections
Fig. 9. Dowel plate connection of the wall elements.

is an important step towards a demountable prefabricated construction technology.

In prefabrication, the necessity for highly mechanised production and flexible products gives rise to the need for an increase in building with prefabricated elements, jointing them with sub-elements. This technique makes it possible to produce a set of sub-elements, which can be joined together in the element factory for different types of elements. For such techniques bolted, shear resistant connections are needed.

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THE BEHAVIOUR OF A DEMOUNTABLE FLOOR

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the Netherlands

SUMMARY

Under the international research project "Demolition, recycling and dismantling of concrete" which is being carried out with the participation of three countries the behaviour of a 1:5 scale model of a floor bay composed of precast concrete units was investigated in the Stevin Laboratory of the Delft University of Technology. The report describes the set-up and detailing of the floor, the method of loading and the method of measurement. Then, with the aid of the results of the measurements and a numerical analysis, it is investigated what effect the bending moment and the transmission of force in cracked joints have upon the behaviour of the floor. Attention is paid more particularly to the action of the forces in the floor, the lateral deflection of the longitudinal edge, the transverse deformation in the joints between the precast units, and the failure load.
1. INTRODUCTION

Under the international research project "Demolition, recycling and dismantling of concrete" which is being carried out with the participation of three countries (the Netherlands, Belgium and Germany) for the purpose of developing environmentally acceptable techniques which will moreover effect savings in raw materials, an investigation into the behaviour of 1:5 scale model of floor structure was undertaken. This investigation was performed in the Stevin Laboratory of the Delft University of Technology in co-operation with CUR-VB Committee D7.

In 1977 a start was made with the development of structures that are simple to dismantle. From among the many structures potentially deserving investigation it was decided to choose a floor bay, consisting of precast concrete units.

A floor bay of this kind is characterized by the fact that transmission of force must be effected solely via the joints and with the aid of simple connections between the units, not through in-situ concrete or an in-situ concrete topping.

The "diaphragm action" of this floor under horizontal loading in its own plane was investigated. The set-up and execution of the research, as well as the results of the tests, are described in Stevin Report No. 5-84-4 [1]. An analysis of the results is to be published later this year [2].

2. SET-UP AND EXECUTION OF THE RESEARCH

The research program relates to a floor bay composed of precast concrete units, constructed to a 1:5 scale, as shown in Photograph 1.

Photogr. 1: General view of the floor bay as tested in series D.
2.1 Structural features of the floor

The structural features are shown in Fig. 1.a. The overall dimensions are 9.36m x 2.88m. The grid dimensions in the longitudinal direction are 0.72m, and in the transverse direction 1.20m, 0.48m and 1.20m respectively. The floor is composed of beams extending in the longitudinal and hollow core slabs in the transverse direction. The slabs bear on the beams with the interposition of a strip of hair felt, while the beams bear on the column heads likewise through interposed hair felt strips. The details of the bearings are shown in Figs. 1.b and 2.a, b and c. The joints between the floor slabs themselves and also those between the slabs and the beams are filled with low-strength mortar.

![Diagram of floor layout and details](image)

Fig. 1 Plan and details of the floor bay.

The method of interconnection of the beams in the longitudinal direction varies in the respective series of tests.

In the first series (series A, comprising 7 tests) the longitudinal connection is established by means of 5mm diameter prestressing wires, steel grade FeP 1670, anchored at the ends of the floor (Fig. 2.a). The total prestressing force is 36kN.
In the second series of tests (series B, comprising 13 tests) the longitudinal connection is established by means of 8mm diameter reinforcing bars, steel grade FeB400, likewise anchored at the ends of the floor and moreover welded to the column heads (Fig. 2.b). In consequence of the welding a total prestressing force of about 16kN is developed in the longitudinal direction of the floor.

In the third series (series C, comprising 26 tests) and in the fourth series (series D, comprising 15 tests) the ends of the beams are longitudinally interconnected by M6 screw-threaded bars, for which purpose the ends of the beams are provided with anchor sleeves (Fig. 2.c).

The column heads are supported on the steel testing frame through roller bearings so as to obtain virtually frictionless bearing conditions (Photograph 2).

At the ends of the floor the slabs and beams are provided with the necessary attachments for connecting the floor to the testing frame.
2.2 Parameters

The influence of the following parameters was investigated in the four series of tests:
-- the type of longitudinal connection;
-- the boundary conditions at the ends of the floor;
-- the tensile force at the end of the floor;
-- the imposed deformation at grid line 2 or grid line 13;
-- the manner of loading.

The floor was loaded with a horizontal load acting in the plane of the floor as shown in Fig. 1.a. It was applied by means of pneumatic jacks. The reactions were transmitted at the ends to the loading frame (see Photograph 1).

The following modes of loading were applied:
-- monotonically increasing load;
-- repeating load;
-- alternating load.

Monotonically increasing load was applied in test series A, B and C, with the exception of test C 306 of series C, in which case a repeating load was applied. Alternating load cycles were applied in series D.

2.3 Program of measurements

The program of measurements comprises the following four groups:
 a) measuring the loads and reactions;
b) measuring the forces in the longitudinal connections;
c) measuring the lateral deflection of the longitudinal edge;
d) measuring the crack width and shear deformation of the joints.
2.4 Material properties

The composition of the concrete, made with gravel as coarse aggregate, and of the jointing mortar is indicated in Table 1. The average values of the strength properties and modulus of elasticity are also given there.

| Composition of the gravel concrete mix | sieve analysis of aggregates | retained on sieve cumul. kg/m³ | | kg/m³ | | kg/m³ |
|-------------------------------|----------------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|
| Portland cement PC-B | 325 | 4 | - | - | - |
| Aggregates | 1712 | 2 | 34% | 34% | - |
| Water | 195 | 1 | 58% | 58% | - |
| Plasticizer | 5 | 0.5 | 75% | 75% | - |
| Total | 2237 | 0.1 | 100% | 100% | - |
| w/c ratio | 0.60 | | | | |

| Composition of the jointing mortar mix | sieve analysis of aggregates | retained on sieve cumul. kg/m³ | | kg/m³ | | kg/m³ |
|-------------------------------|----------------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|
| Portland cement PC-B | 132 | 1 | - | - | - |
| Aggregates | 1471 | 0.5 | 45% | 45% | - |
| Chalk | 118 | 0.25 | 83% | 83% | - |
| Water | 330 | 0.1 | 100% | 100% | - |
| Total | 2051 | | | | |
| w/c ratio | 2,5 | | | | |

<table>
<thead>
<tr>
<th>average values of</th>
<th>gravel concrete</th>
<th>jointing mortar</th>
</tr>
</thead>
<tbody>
<tr>
<td>cube compressive strength (N/mm²)</td>
<td>33,7</td>
<td>2,1</td>
</tr>
<tr>
<td>splitting tensile strength (N/mm²)</td>
<td>2,6</td>
<td>0,3</td>
</tr>
<tr>
<td>modulus of elasticity (kN/m²)</td>
<td>27,3</td>
<td>3,1</td>
</tr>
</tbody>
</table>

Table 1 Composition and properties of concrete and jointing mortar.

The yielding force of the longitudinal connection in test series C and D is 26kN. The average cross-sectional area of this connecting steel is \( A_a = 22.6\text{mm}^2 \).

3. OBSERVED BEHAVIOUR

For the purpose of the present report the discussion of the results of the measurements will be confined to:

a) the effect of the bending moment on the longitudinal forces at grid line 7;
b) the effect of the total horizontal load on the lateral deflection of the longitudinal edge;
c) the effect of the shear force on the longitudinal forces and the deformation of the transverse joints;
d) the failure load.
3.1 Effect of bending moments on longitudinal forces at grid line 7.

The relation between the bending moment at grid line 7 and the tensile force in the longitudinal connection at point D7 is represented in Figs. 3.a and 3.b. Fig. 3.a compares the series A, B and C with one another. This diagram also compares the measured values for series C with the values calculated with the ZEFE computer program and with a simplified mathematical model. The computer program employed was based on the finite element method of analysis.

Fig. 3 Relation between bending moments and tensile forces in the longitudinal connection at D7.

Fig. 4 Comparison of tensile forces at grid line 7 for monotonically increasing and for alternating load.
Fig. 3.b compares a monotonically increasing load (series C) and an alternating load (series D) for 3, 200 and 400 cycles.

In Fig. 4.a the tensile forces in the longitudinal connections at grid line 7 for test series C have been plotted for a total horizontal load \( Q = 6 \text{kN} \). This diagram also compares the measured values and the values calculated with the ZEFE computer program and with a simplified mathematical model.

In Fig. 4.b the tensile forces in the longitudinal connections at grid line 7 for test series D have been plotted for a total horizontal load \( Q = 12 \text{kN} \) after 3, 200 and 400 cycles.

3.2 Effect of total horizontal load on deflection of longitudinal edge

The relation between the total horizontal load and the lateral deflection of the longitudinal edge at point D7 is represented in Figs. 5.a and 5.b.

Fig. 5.a compares the series A, B and C with one another. This diagram also compares the measured values for series C with the values calculated with the ZEFE computer program and with a simplified mathematical model.

Fig. 5.b compares a monotonically increasing load (series C) and an alternating load (series D) for 1, 200 and 400 cycles.

![Fig. 5 Relation between total horizontal load and lateral deflection of the longitudinal edge at D7.](image-url)
3.3 Effect of shear force on longitudinal forces at grid lines 2 and 13

Figs. 6.a, b and c show the relation between the bending moment and the tensile forces in the longitudinal connections at the grid lines 2 and 13. The values calculated with a simplified mathematical model are also plotted in these diagrams. Before the start of testing, the transverse joint at grid line 2 had been provided with a preformed crack about 0.3mm in width. At grid line 13 a flexural crack developed during the test. In this test a tensile force of 3.04kN was applied at the ends, distributed over the grid lines A, B, C and D.

Fig. 6 Relation between bending moment and tensile forces at grid lines 2 and 13 for monotonically increasing load.

3.4 Effect of shear force on deformation of transverse joints at grid lines 2, 11 or 13

Figs. 7.a, b and c show the relation between the shear deformation and the crack width in a transverse joint.
Fig. 7.a relates to the average deformations of the transverse joint at grid line 2 for a monotonically increasing load, with a preformed crack at grid line 2 and for a total tensile force of 3.04kN applied at the ends, distributed over the grid lines A, B, C and D.

Fig. 7.b relates to the average deformations of the transverse joint at grid line 13 for repeating load, with a preformed crack at grid line 13 and for a total tensile force of 3.04kN applied at the ends, distributed over the grid lines A, B, C and D.

Fig. 7.c relates to the deformation at point 11-BC in the portion BC of grid line 11 near grid line B for an alternating load, without a preformed crack and without tensile forces applied at the ends.

Fig. 7 Relation between shear deformation and crack width.

3.5 Failure load

The failure loads for the four series of tests are given in Table 2, which also indicates where the failure developed and what type of failure occurred.
### Table 2 Failure load data.

The relation between the load producing failure in shear and the logarithm of the number of load cycles in series D and the number of load repetitions in series C is represented in Fig. 8.

<table>
<thead>
<tr>
<th>Test</th>
<th>Failure mode</th>
<th>Failure load</th>
<th>Location of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A6</td>
<td>a</td>
<td>$Q_u = 30.5, \text{kN}$</td>
<td>Compressive zone at A 7</td>
</tr>
<tr>
<td>B11</td>
<td>b</td>
<td>$Q_u = 23.2, \text{kN}$</td>
<td>as 13</td>
</tr>
<tr>
<td>C206</td>
<td>b</td>
<td>$Q_u = 30.3, \text{kN}$</td>
<td>as 2</td>
</tr>
<tr>
<td>C306</td>
<td>c</td>
<td>$T_u = 10.2, \text{kN}$</td>
<td>(6200 cycles)</td>
</tr>
<tr>
<td>D5</td>
<td>c</td>
<td>$Q_u = 12, \text{kN}$ and $-12, \text{kN}$</td>
<td>as 12</td>
</tr>
<tr>
<td>D205</td>
<td>c</td>
<td>$Q_u = 12, \text{kN}$ and $-12, \text{kN}$</td>
<td>(2000 cycles)</td>
</tr>
</tbody>
</table>

**Failure mode:**

- a = failure in bending
- b = failure in shear due to sudden fracturing
- c = failure in shear due to plastic deformation

**Fig. 8 Relation between failure load and number of cycles.**

The dotted lines indicate the possible functional relationship between the failure load and the number of cycles or repetitions.
4. ANALYSIS OF THE STRUCTURE WITH A SIMPLIFIED MODEL

4.1 Bending moment

For calculating the lateral deflection of the longitudinal edge at point D7 in test series C the floor bay is schematized as shown in Fig. 9. It is divided into a number of 600mm wide flexurally rigid portions between the grid lines and a number of 120mm wide flexible portions at the grid lines.

![Diagram of floor bay](image)

**Fig. 9 Schematization of the floor bay.**

The analysis of the flexible portion is based on the usual stress analysis at a section. On the assumption that plane sections remain plane it is possible to calculate the distribution of forces at the section, from which the curvature \( \kappa_1 \) of the flexible portion can then be determined, namely:

\[
\kappa_1 = \frac{c}{x}
\]

For the flexurally rigid portions between the grid lines the distribution of forces can be calculated in similar fashion. The longitudinal beams are assumed to be uncracked and the joints to be unable to transmit tensile forces. The curvature \( \kappa_2 \) of the flexurally rigid portion can then be calculated from the strain distribution.

![Diagram of curvature line](image)

**Fig. 10 Curvature line of the floor bay.**

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The lateral deflection of the longitudinal edge can now be determined from the curvature area of the floor, as shown in Fig. 10.

The deflection at point D7 of the edge is equal to the moment of the curvature area at the point under consideration.

4.2 Shear force at a cracked joint

In a joint where both moment and shear force are acting the stress distribution will be as shown schematically in Fig. 11. The shear force will be entirely resisted by the compressive zone. The magnitude of the shear force that can be resisted depends on the $\tau-\sigma$ relation of the mortar joint. If this relation is known, the shear force magnitude which just fails short of producing shear deformation can be calculated.

![Diagram of stress distribution in a transverse joint without shear deformation](image)

Fig. 11 Stress distribution in a transverse joint without shear deformation.

If then, simultaneously with the development of shear deformation, the crack width also increases, additional tensile forces will develop in the longitudinal connections in consequence of wedge action. These tensile forces produce additional compressive forces in the joint, so that the shear loadbearing capacity is increased. This phenomenon is called shear friction. It can be represented schematically as in Fig. 12. The slope ($a$) of the crack model and the height ($a$) are a measure of the roughness of the cracked surface.

![Diagram of schematic representation of a cracked joint](image)

Fig. 12 Schematization of a cracked joint.
If the $\tau - \sigma - w - s$ relation of the joint is known, the transmission of force in a cracked joint can be described with the aid of this crack model, which was developed by Tassios [3].

The stress distribution in a transverse joint will then be as shown schematically in Fig. 13.

Fig. 13 Stress distribution in a transverse joint with shear deformation.

5. DISCUSSION OF THE MEASURED RESULTS AND CALCULATED VALUES

The following conclusions can be drawn from a comparison of the test results with one another and with the calculated values:

1) From Fig. 3.a it appears that the magnitude of the longitudinal forces depends on whether or not the floor is prestressed. It also appears from this diagram that at a joint with predominantly bending moment the measured values of series C are in good agreement with the calculated values.

2) From Fig. 3.b it appears that at a joint with predominantly bending moment the longitudinal forces are virtually independent of the number of load cycles.

3) From Fig. 4.a it appears that for monotonically increasing load there is a reasonably linear relationship between the longitudinal forces on grid line 7 at the lines B, C and D. It also appears from this diagram that the calculated and the measured values are in good agreement with one another.

4) From Fig. 4.b it appears that the number of cycles has little effect on the behaviour of the longitudinal forces. Only the occurrence of cracks in the longitudinal joints results in discontinuity in the magnitude of the forces in relation to one another.

5) From Fig. 5.a it appears that the rigidity of the longitudinal connection is of major influence on the lateral deflection of the longitudinal edge. It also appears from this diagram that in series C the measured and the calculated values of the deflection are in good agreement with one another.
6) From Fig. 5.b it appears that the number of cycles is of major influence on the lateral deflection of the longitudinal edge, partly in consequence of the shear deformation in the transverse joints.

7) From Figs. 6.a, b and c it appears that in joints with predominantly shear force - both in the case of a flexural crack and in the case of a preformed crack - the measured values of the longitudinal forces differ greatly from the calculated values.

8) From Figs. 7.a, b and c it appears that the deformation behaviour of a cracked joint depends very much on the manner of loading.

9) From Table 2 it appears that a floor bay, unless prestressed, always fails in shear.

10) From Fig. 8 it appears that the failure load depends very much on the number of load cycles.

6. TRANSMISSION OF FORCE IN A CRACKED JOINT

The calculated values of the longitudinal forces, as plotted in Fig. 6, relate to a stress distribution as shown in Fig. 11.

From Fig. 6 it appears that this stress analysis does not adequately describe the actual behaviour. When the structure is subjected to a load of about Q=12kN in conjunction with a preformed crack at grid line 2, the calculated values deviate considerably from the measured values. Evidently a different mechanism now operates which is possibly in closer agreement with the stress distribution shown in Fig. 13. This stress distribution can occur only if, due to roughness of the crack faces, the increase in shear deformation is accompanied by an increase in crack width. This mechanism is described in Section 4.2.

This means that the loadbearing capacity of the joint depends greatly on the roughness of its faces. Therefore a smooth-faced transverse joint subjected to predominantly shear force and a small moment will have a low loadbearing capacity. For this reason it must be recommended that the roughness of the sides of the floor slabs should be made as great as possible, e.g., by providing them with indentations to act as "shear keys", the depth of which should be suited to the expected crack width due to shrinkage, creep, imposed deformations, etc.

The shear loadbearing capacity of a joint can also be increased by the application of a prestress across the joint, as was done in serie A. In those tests the floor did not fail in shear, but in bending.
7. CONCLUSIONS

1) The longitudinal forces at a section where predominantly bending moments occur are, both for monotonically increasing and for alternating load, in good agreement with the values to be expected on the basis of a simplified analysis.

2) The lateral deflection of the longitudinal edge of a floor bay is, for monotonically increasing load, in good agreement with the values to be expected on the basis of a simplified analysis, provided that the differences in rigidity of the floor are taken into account. Under alternating load the deformations increase considerably, partly due to the shear deformations.

3) The stress distribution at a section where predominantly shear force occurs differs considerably from the stress distribution obtained by calculation, unless the shear friction effect is taken into account in the calculation.

4) The loadbearing capacity of a floor depends very much on the shear capacity of the joint and therefore on the roughness of the joint faces, i.e., the edges of the slabs. It is recommended that these should be formed with indentations, unless the floor bay is prestressed. Moreover the loadbearing capacity of a floor depends very much on the number of load cycles.

Notations

\[ N_s \] = tensile force in the steel

\[ N_c' \] = compressive force in the concrete

\( F \) = load

\( n \) = number of cycles

\( s \) = shear deformation

\( w \) = crack width

\( x \) = depth of compressive zone in the concrete section

\( \delta \) = (lateral) deflection

\( \varepsilon_c' \) = strain of the concrete
\[ \kappa \quad = \text{curvature} \]
\[ \sigma'_c \quad = \text{normal stress in the concrete} \]
\[ \sigma_s \quad = \text{normal stress in the steel} \]
\[ \tau \quad = \text{shear stress} \]
\[ \mu \quad = \text{coefficient of friction} \]

References


Connecting and disconnecting structural members are the decisive part of demountable precast concrete structures.

This paper introduces a system of joints comprising a lap-joint and a constricting steel pipe wrapped in heat-generating wire. The size of the joints and the method of connecting the members by using sulphur mortar are critical to the success of this system, and mark its departure from the conventional lap-joints of precast concrete. The mortar consisting of liquefied sulphur and sand can be filled into the pipe while heated by electricity. When cooled, the joint is ready for use. The reverse procedure is applicable to demounting.

The scope of this study was to determine the optimum lap length with respect to other conditions, such as grout strength. Technical background and experimental results will be discussed, followed by the case study discussion of the application of this method to a culvert structure.
APPLICATION OF CYLINDER LAP JOINTS TO BOX CULVERT

1. INTRODUCTION

Demountable structures would require a systematic method to disjoint members. This report is to propose a system of joints which consists of a lap-joint and its reinforcing steel pipe wrapped in heat generation wiring. The size of the joints and the method of connecting members are characteristic, and mark its departure from the conventional lap-joints. Fig. 1 shows the cross section of the system. When this joint is connected, sulphur mortar can be filled into the pipe while heated by electricity. In this way, grouting mortar can easily penetrate into the space within the constricting pipe. When cooled, the structure is ready for use immediately. The pipe and mortar can be heated again by electricity, when the structure is to be demounted. When the temperature of mortar gets to the melting point, the joint loses the strength to be disjointed. Sulphur mortar appears to have considerable promise in such use, especially when these structural members are intended to be reused. Experimental works were carried out on sulphur mortar mixtures and the joints made of appropriate mixture. The joint was applied to connecting twenty concrete box culverts at one busy cross roads, where the closing period is limited for the construction and these concrete boxes are expected to be removed in the future.

![Fig. 1 Cross section of joint](image)

2. EXPERIMENTS

2.1 Experimental Procedures

Two series of experiments were carried out. The first series was on mechanical properties of sulphur mortar and the second one was on the structural behaviour of joints. These tests were planned according to the factorial design of experiments, in which factors and levels were chosen from the preliminary experiments.

2.2 The first series on mortar

Experimental conditions are shown in Table 1. Figures corresponding to each level in factor A, the sand, were of unit weight to sulphur. The sand was processed with asphalt before mixing mortar in an asphalt cooker. Then the surface of a sand grain was covered with thin asphalt membrane. Both the factor B, asphalt, and the factor C, calcium carbonate, were in percentage weight to sulphur. These two materials were
used in order to better workability of mortar. All ingredients were kept at 110°C before mixing at 125 - 130°C. Experimental design and results are shown in Table 2. Information on sulphur use is available in Ref. 1.

Table 2. Design of experiments and experimental results of sulphur mortar

<table>
<thead>
<tr>
<th>No.</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>bending (kg/cm²)</th>
<th>compressive (kg/cm²)</th>
<th>unit weight (g/cm³)</th>
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<td>8.7</td>
<td>4.35</td>
<td>2.29</td>
</tr>
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</table>
2.3 The second series on joint

Three factors were chosen for the tests of joints, as shown in Table 3. In D factor, three kinds of diameter of deformed bar were experimented. D13 indicates the deformed bar of 13 mm diameter. In E factor, the effects of lap length were tested. The length is shown in the multiplying number of diameter. "8D" is eight times as long as bar diameter. In factor F, the effects of loading times of stress, 1750 kg/cm², were tested. Since the yielding stress of SD 30 steel tested was 3500 kg/cm², the half of the stress was loaded as many times as given in design before the ultimate stress of joint was loaded. The design of experiments and experimental results of joint strength in the tensile tests are shown in Table 4. In the preliminary tests (Ref. 2), the effects of grout mortar and the shape of cross section of the pipe was experimented. Little effects were found on the shape, if all other conditions were the same. Thus the tests of joints were carried out using a circle cross section of the pipe for the simplicity sake. A flattened pipe of oval cross section, as shown in Figure 2, can be more practical, when the cover thickness between concrete surface and steel surface is limited.

![Diagram of joint](image)

**Table 3. Factors and levels of joints**

<table>
<thead>
<tr>
<th>Factor \ level \</th>
<th>1</th>
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<tbody>
<tr>
<td>D. steel diameter</td>
<td>D13</td>
<td>D16</td>
<td>D19</td>
</tr>
<tr>
<td>E. lap length</td>
<td>8D</td>
<td>10D</td>
<td>12D</td>
</tr>
<tr>
<td>F. repetition</td>
<td>5times</td>
<td>10times</td>
<td>15times</td>
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**Table 4. Experimental design and results of joint strength (kg/mm²)**

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<th>E</th>
<th>F</th>
<th>strength (kg/mm²)</th>
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3 ANALYSIS OF EXPERIMENTS

3.1 Analysis of Sulphur Mortar

The analysis of variance was performed by F-test for tested factors in the experimental results. The F-values obtained by the procedure were used to judge the significance of a factor.

In regard of bending strength, only the value of asphalt to sulphur was over the significant level of P=0.05 or 5% point of F-table, as shown in Table 5, where "*", one star, indicates the significance.

Table 5. Analysis of variance table for bending strength of sulphur mortar

<table>
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<tr>
<th>factors</th>
<th>ss</th>
<th>df</th>
<th>ms</th>
<th>F</th>
<th>judgement</th>
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<td>A (sand)</td>
<td>737</td>
<td>2</td>
<td>368</td>
<td>5.18</td>
<td>*</td>
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<tr>
<td>B (asphalt)</td>
<td>1163</td>
<td>2</td>
<td>581</td>
<td>8.17</td>
<td></td>
</tr>
<tr>
<td>C (calc. c.)</td>
<td>124</td>
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<td>62</td>
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</table>

In regard of compressive strength, both sand and asphalt were over the level of 1% and calcium carbonate was over the level of 5%.

In regard of unit weight of mortar, asphalt was over the 5% level.

Main relationships between the measured data and levels of each factor are shown in the following figures, where the two-stars mark, **, is of significant level over P=0.01. When molten sulphur matrix hardens, shrinkage appears considerable. Grains of sand may prevent the hardening shrinkage. Other ingredients were employed in order to improve the workability of mortar between 125°C and 130°C. The melting point of sulphur used is 119°C.

Fig.3 Relationship between asphalt and bending strength of mortar

Fig.4 Relationship between asphalt and compressive strength of mortar
3.2 Strength of Joints

The F-test was performed for three factors and these interactions, as shown in Table 6. It is found that the joint strength depend entirely upon the lap length, shows that the constricting pipe plays the decisive role of the joint. Other factors showed little effects on the joint function within tested range of these factors. The relationship between lap length and joint strength is shown in Fig.7.

Table 6 Analysis of variance table for joint strength

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<th>s</th>
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<th>f</th>
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4. APPLICATION OF JOINT TO CULVERT

4.1 Design of Joints

Since the lap length should be over 15D for the fracture of steel bar from the extrapolation of Fig.7 and other experiments, the actual one was designed as 20D with some superfluous length for safety. Thus its relationship between stress and strain was almost identical to that of original steel bar itself up to the yielding point, as shown in Figure 8. The input tube for grouting was installed on the constricting pipe. The heating device of nichrome wire was wrapped on the constricting pipe, as shown in Photo 1. Thin gypsum layer was used for electrical insulation and fixing terminals as shown in Photo 2. These pipes were located in reinforcing steel bars in the female members as shown in Photo 3.
Fig. 7 Relation between lap length and joint strength

Fig. 8 Stress-strain relationship

Photo 1. Nichrome wiring on pipe

Photo 2. Insulation and terminals
4.2 Construction of Culvert

The culvert is located at the crossing of important routes, and it will be removed when the development of neighbouring area is completed in the future. This method of joints was applied to a culvert construction, in which twenty boxes of precast concrete were connected to prevent a non-uniform sinking of boxes on peat bed. The top of box was of steel gratings for disposing snow in winter. All the boxes were prefabricated in a factory 40 km distant from the construction site. Construction procedures are shown in the following photographs. A power generator of 3 kw portable type was used for heating pipes, each of which required 0.3 kw. Since the room for the projected steel bar to be inserted was enough within a pipe placed in the female member, site work was simple and short. Besides the thus jointed structures were ready for immediate use after the side space of each box was filled with soil and jointing was over. At first, deterioration of steel bars due to sulphur was presumed. Therefore partial galvanization of steel bars contacting directly with sulphur was discussed, although not executed. Three years have passed without any trouble, since the work was completed at -2°C of ambient temperature. Water tightness of grout mortar might prevent the corrosion. Easiness of eye-checking is of merit in this jointing, compared with welding. Demounting procedure is reverse to jointing one. In order to remove grout mortar from the pipe, heated liquid such as oil can be pumped into the pipe.

5. CONCLUSION

A combination of sulphur mortar with heating device enables to demount structures connected with cylinder lap joints and to recycle these in a systematic way. But, at the normal ambient temperature, the bond strength of grout can be increased considerably by the constricting pipe, since grout materials are subjected to triaxial compression when loaded. The function of joints depends on the lap length of bars as well as the mix proportion of grout mortar. Its workability and bond strength depend upon the mixture. The allowance room of the pipes for projected steel bars can reduce and simplify the work considerably at the field site for demounting and jointing. The stress-strain relationship of the joints can be adjusted by lap length and other conditions in order to make it similar to that of original steel bars, which may prevent stress concentration at particular joint.
Photo 4 Crane work for placing boxes side by side

Photo 5 Insertion of projected male bar into female pipe
Photo 6 Connection of electric wire to terminals

Photo 7 preparation of sulphur mortar for grout


PRESENT-DAY CONNECTIONS FOR REINFORCING BARS

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SUMMARY

After a short review of the rebar (reinforcing bar) connections available on the market (lap splice, contact splice, welded joint, fusion connection, screw connection, clamped connection, etc.) new developments are presented.

The most important among them are:

- screw connections with different thread designs, such as cold-rolled threads (e.g. W&T), cut tapered threads (e.g. Lenton) or hot-rolled thread ribs (e.g. grade 60 thread bar with or without lock nuts)
- cold-worked couplings
- turnbuckles

Requirements applicable to these connections are as follows:

- static and dynamic bearing capacity
- slip behaviour
- susceptibility to defects during assembly
- space requirement
- ease of assembly
- compensation of inaccuracies in the workmanship of precast concrete components
- material and labour costs

Because of the wide scope of the subject matter, only certain aspects can be highlighted in this paper.
1. INTRODUCTION

The labour-intensive reinforcement technology of the 60ies, where rebars had been connected only by means of overlapping or welding, was a challenge to the engineers to develop possibilities to carry out splices in a more economic, effective and universal way.

![Diagram showing economic importance of splices]

Fig. 1 reveals the economic importance of splices. Depending on the basic steel price, the costs of the splice and the number of splices, certain variations in efficiency occur, with the technical side not being taken into account. The search for alternative connecting methods also led to new possibilities for transferring the forces at the bar end in a more efficient way which, however, will not be discussed in the following.

At first the designs of the overlapping splice, the contact splice, the welding splice and the types of connection which had been created in the first developmental phase and which have more or less been accepted on the market for producing a more efficient reinforcement will be presented and explained in detail. In the second part of this paper latest splicing methods will be discussed.

2. OVERLAPPING SPLICE

Standards such as DIN 1045, Sect. 18, for all standardized steels (e.g. DIN 488) determine the design of this type of splice where forces are transmitted via the overlapping ends of ribbed bars.
The bar diameter, the position of the splice with regard to the concrete cross section and the portion of spliced bars against the total cross section of the reinforcement determine the always necessary transverse reinforcement, the length of the overlapping splice and the longitudinal staggering of two splices in the case of multilayer reinforcement. If thick bars are subjected to compressive stress there is danger of scaling due to face pressure. When concrete sets the bond can be impaired due to vibrations. The arrangement of hooks and loops allows the overlapping length to be reduced due to the influence of the hooks on the length \( a_0 \) but entails additional difficulties in application.

Due to the existing deflection stress in the curvature area special requirements must be met by
- the lateral concrete cover,
- the splitting tensile reinforcement and
- the minimum bending radii.

In addition, the more complicated placing of the curved reinforcement and the reduced capability of concreting due to reinforcement congestion and, thus, the existing concentration of bond stress have to be taken into account in the design stage.

The projecting bar ends at the working joints are often bent back into the formwork. The related problems are compiled in "DBV-Merkblatt" (German Concrete Association Code of Practice)/14/.

3. CONTACT SPLICES

In the case of cross sections predominantly subject to compression perpendicular bars \( \geq 20 \text{ mm} \) may be connected by contact splicing. The contact faces, however, must be worked and, prior to concreting, it must be possible to check and secure their centrical position. Moreover, additional static application requirements have to be observed:

The bar must not be in areas subject to buckling.

Each rebar may be spliced only once.

Only a maximum of 50 % of the reinforcement may be spliced in one cross section.

In the case of thick compressive bars, however, this splice is superior to the overlapping splice.
Possible applications are:
- The G-Lock-contact splice /1/, where the bars to be spliced are fixed by a metal sheet collar and a wedging plate,

![Fig. 3](image)

- the NOE-contact splice /2/, where the centrical seating and the firm guiding of the bars is effected by clamping 4 round bars by means of 3 clamping rings,

![Fig. 4](image)

- the threaded coupler contact splice (GEWI-Splice), where the bars having rolled-on thread ribs are screwed into a threaded coupler in such a way that force transmission via the bar faces is possible.

![Fig. 5](image)

**WELDED SPLICES**

By welding the bar ends by means of different types of seams and, if necessary, by additionally using bars with loops or semicircular tubes it is possible to produce solid splices at a minimum of material used. In the factory bars may be connected by flash butt welding on demand.
5. FUSION SPLICE

This connection is produced according to the alumino-thermic CADWELD-Rebar Method /3/, where positive and frictional bonding between the rebar and an internally serrated coupler is achieved by placing steel in an exothermic reaction.

Fig. 8

The advantage that minor differences in tolerance, deviations of angle and eccentricities can be compensated is in contrast to the big disadvantage for application consisting in the fact that the parts to be connected and the casting device have to be preheated. This renders the method dependable on meteorological conditions, produces long assembly periods, and handling with liquid metal involves an increased risk of accidents on site. To date only insufficient experience is available about carrying behaviour of the thermite casting subject to dynamic loading.

6. PRESSED COUPLER SPLICE (EPERSPÄCHER SYSTEM)

The bars are connected by means of a coupler tube which is pushed over the end of the bars and pressed against the ribbed rebars by means of a special jack in such a way that the comparatively soft material of the coupler tube is pressed into the bar ribs.
Depending on the coupler size a suitable jack has to be used, allowing the coupler tube to be pressed on in several steps, and in at least two passes where the jack is staggered by 30° to 90°.

Since the force is transmitted via the shear bond between the ribs of the rebars and the indented coupler the quality of the splice directly depends on the pressing operation. For this reason, the measuring of the coupler extension must be taken into consideration for quality assurance. Due to the resulting extension longitudinal shifting of a bar must be ensured. This type of connection allows the splicing of almost all commercial rebars $d_e \geq 12$ mm for applications with predominantly static loading and a small number of applications with predominantly dynamic loading. These connections are widely known by the designation "Eperspächer pressed coupler splice" /4/ or "CCL-Alpha Splice". By using special parts (RELI-Splice) half coupler connections may be pressed on, assembled or spliced by screwing after concreting.

7. SCREW CONNECTIONS FOR REBARS AND LATEST DEVELOPMENTS

7.1 General instructions and requirements to splices

This state of the art was no reason for the engineers to rest on their laurels since the increasingly severe competition called for still simpler, cheaper connections applicable in a still more universal way, combined with improved requirements to the splice design. The requirements to be met have been published in general instructions in West Germany, e.g., in "Principles for Approval and Surveillance Tests of Coupler Splices and Plate Anchorages of Rebars" /5/ by the Institute of Construction Engineering, Berlin.

According to this, the coupler splices must have:
- a yield load of $1.0 \cdot R_y \cdot A_s$ and
- an ultimate load of $1.2 \cdot R_s \cdot A_s$.

The deformation occurring in addition to the elastic elongation must not exceed 0.1 mm under working load.

The fatigue strength has to be determined for both static and dynamic splices.
In the case of dynamic splices the admissible amplitude under consideration of a safety coefficient \( \gamma = 1.5 \) has to be determined from several proofs of fatigue strength in concreted condition at an upper stress of \( 0.7 \beta_0 \) and an amplitude \( 26_A \) at which 2 million load cycles can be applied without causing a failure.

It is required that:
- unfavourable tolerances of the parts to be connected,
- possible eccentricities and
- sources of errors depending on processes
are taken into account for all these proofs.

In addition to the requirements of the general instructions the requirements demanded by site conditions in the case of new developments have to be increasingly taken into consideration in order to be successful on the market, e.g.
- limited space requirements of the necessary equipment,
- insensitivity during assembly under site conditions,
- ease of assembly, quick and simple assembly without large devices,
- low labour and material costs,
- maximum possible tolerance compensations (longitudinal displacement and transverse offset) for application in prefab construction,
- immediate loading of the splice,
- possibility of 100% splicing in one cross section,
- high dynamic bearing capacity.

These severe requirements could only be met by new concepts in splicing technology. Thus, new equipment has been developed, allowing the most various types of threads to be cut, rolled-on or pressed on the ends of the rebars. A new rolling method even allowed to produce rebars with continuous rolled-on thread ribs. This rebar available on the market under the tradename GEWI can be cut to length at any point and spliced positively and frictionally by means of a threaded coupler. In addition to the screw connections other fully applicable possibilities have been developed, such as:

- to produce grouted connections with mortar or resin for precast construction
or
- to press pipe couplers on the steel by means of the extruding process.

Moreover, special connections are designed in the field of reinforcement engineering for the most various marginal problems, such as turnbuckle connections. From this field some developments are explained in detail in the following.

7.2 Threaded coupler splice with standard thread

Since at cut threads only 80% of the cross section of the core can be taken into account, the thread of the connection known as WD-screw connection is produced by cold working. For this purpose the bar ends are peeled to the circular cross section of the core and the thread running out in a conical shape is rolled on by means of automatic rolling machines.
These threads are only produced at the factory due to the required equipment. The steel structure in the marginal zones is compacted by the rolling operation and, thus, the material strength increased in such a way that despite peeling the full (100%) ultimate bearing capacity is attained again. The conical ending of the thread allows that the coupler in the case of straight bars needs not to be secured by means of lock nuts for decreasing slip but that the coupler can be screwed on this conical thread with a defined starting torque.

Since notch stresses occur under dynamic loading in the transitional area - steel surface/peeling area - the dynamic bearing capacity of this splice is as comparatively small as of the cut LENTON tapered thread splice mentioned in the following.

Due to the fine pitch of the thread only small torque moments are required, however, assembly problems increase in case of contamination or damage of the threads; assembly inaccuracies cannot be compensated. During assembly it is always required that either the straight or the curved bar to be connected can be rotated or shifted in a longitudinal direction.

7.3 Threaded coupler splice with cut tapered thread

The LENTON coupler showing a tapered thread /7/ is based on the idea to adjust the couplers to the stress distribution in the splicing area bar-coupler-bar.

The thread at the bar end is produced by equipment suited for sites. The tapered fine-pitch thread is cut in one operation into the bar end by 4 knives without pretreatment of the steel ribs. The hexagonal coupler provided with the corresponding thread is optimally suited with respect to the material requirements to splices. This thread shape ensures full (100%) static bearing
capacity despite a cut thread, however, the sensitivity during assembly increases more considerably than in the case of screw connections with standard threads. If no special parts are used the bars to be connected of this splice must be rotatable and movable, too.

A variant available on the market is pressed thread shapes instead of cut threads, the former having a more favourable dynamic behaviour.

7.4 Threaded coupler splice for hot-rolled thread ribs

The expensive production of cut and rolled threads and the required preliminary planning for determining the individual bar lengths initiated the production of a rebar of the grade BSt 420/500 RU, having a thread over its entire length and, thus, can be spliced positively and frictionally at any cutting point by means of a coupler. Its extensive applications and technical properties are described in detail in /8, 9, 10. This GEWI-steel (grade 60 threadbar) is connected by round or hexagonal threaded couplers. Due to the continuous course pitch thread the coupler can be screwed from one bar end to the second bar end without the curved connection bar being rotated. Slip of the threads is eliminated by two nuts which are tightened by means of a torque wrench. The basic material GEWI-steel is slightly more expensive against a commercial rebar due to the special manufacturing procedure.

![Fig. 12](threaded_coupler_lock_nut_GEWI-steel_III_U)

In the case of straight connecting bars there is the possibility to produce a full tensile and compressive splice for both static and dynamic loads without lock nuts. In this splice the bars are screwed against each other, i.e. via "face" contact and locked by means of a thread profile wrench. This special torque wrench allows locking without any damage of the steel in such a way that the dynamic bearing behaviour of both the steel and the coupler joint is conserved. Due to its coarse-pitch thread this type of splice meets all site requirements. Like in the examples under 7.2 and 7.3 the bars must be rotatable against each other.

![Fig. 13](threaded_coupler_GEWI-steel_III_U)

The accessory program offers a turnbuckle coupler splice for the special application - connection of concreted fixed bars -.
7.5 Turnbuckle coupler

The development of a turnbuckle coupler for the GEWI-steel (grade 60 threadbar) allows in a simple way connection of concreted bars with longitudinal staggering and slight transverse offset. If it is applied, e.g. as beam tie reinforcement, a defined tensile load can be transferred.

Fig. 14

The splice consists of two parts, the hexagonal turnbuckle coupler and the double thread coupler. The matching of the thread shape - GEWI coarse-pitch thread/metric fine-pitch thread - and the opposed threads provide the possibility to screw positively and frictionally bars with differing turns.

8. COLD WORKED COUPLERS

The reinforcement technology has been increased by a new coupler splice, the extruded coupler, due to newly developed equipment. Like at the pressed coupler splice a coupler tube is used for connecting the bars. The coupler material, however, is not indented into the ribs of the bar by repeated transverse pressing but by a single extruding operation during which the material is uniformly cold worked over the circumference.

Fig. 15

A constant production quality is achieved by the automatic extruding process.

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The extruder is designed in such a way that the distances in the area of the coupler splice are the same as those for unspliced bars according to DIN 1045. This splice can be used for all ribbed rebars with the load being either static or dynamic. An approval from the site supervision /11/ is available for sections of the multitude of rebars.

9. GROUTED SPLICES

For considerable assembly tolerances of the connecting bars, like they occur in reinforced precast concrete construction, only a few of the above mentioned couplers can be applied but only under the condition that special assembly instructions are observed. Therefore, an idea of Eriksson /12/ has been taken up again, i.e. to connect rebars by means of a grouted coupler splice.

This splice is formed by an internally serrated steel coupler which is grouted with cement mortar. Due to the comparatively large inner diameter of the coupler and the coupler length, rebars which are not exactly placed in the bar axis or have a large transverse offset can still be grouted perfectly. Since Eriksson only provides load displacement diagrams and ultimate load values, engineers studied the bearing behaviour of the grouted coupler splice in an extensive test program /13/ and tried to establish a design procedure. In other countries a grouted coupler designated as NMB-Splice is already applied. However, several application oriented tests under site conditions have yet to be performed before the serrated grouted coupler can be employed for the first time in Europe.

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Fig. 16

10. SUMMARY

The various types of splices are presented. The criteria decisive for application are compiled as follows:

- full static bearing capacity
- negligible slip
- high dynamic bearing capacity
- simple, safe and controllable assembly
- little requirements of space and assembly equipment
- If necessary, attention has to be paid to special properties of the splices, such as small longitudinal displaceability, rotability. Tolerances at splices of prefab members have to be compensated.
- overall cost calculation required, not only material and labour costs but also influences on the complete procedure and quality have to be taken into account, e.g. no projections at formwork at working joints rigid rebar connection
good compactability of the concrete.

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SUMMARY

This paper is a summary of the work done by the working group which was formed within the CUR-VB Committee D 7 and was composed as follows:

Ing. J.J. Kolpa - Secretary
Ing. L.H. Verwey - Member
Ir. J.J.M. Font Freide - Member
Dr.Ir. G.F. Huyghe - Chairman

Interest is increasingly being focused on the construction of buildings which are partly or fully demountable.

If this method of building is to gain wider acceptance, it is necessary to give suitable solutions for designing the loadbearing framework, more particularly with regard to the joints, the structural connections and the installation of the reinforcement.

A study group "Demountable connections" within the scope of the CUR-VB Committee D 7 "Demountable Construction" has prepared a report covering this range of subject-matter and is at the disposal of designers.

The study is confined to load-bearing structures, comprising columns, beams, load-bearing external walls and floor slabs.
In so far as the connections are concerned, existing manuals dealing with structural connections were studied.

After describing the open precast systems for different types of buildings, the requirements for demountable connections are given and a number of structural connections are shown and described, which indicate how industrial, commercial and various other categories of buildings can be designed for demountability.
Demountable Construction in Precast concrete

Dr. ir. G.F. Huyghe

Interest is increasingly being focused on the construction of buildings which are partly or fully demountable. If this method of building is to gain wider acceptance, it is necessary to give suitable solutions for designing the load bearing framework, more particularly with regard to the joints, the structural connections and the installation of the reinforcement.

Since a demountable building can be presumed to be a prefabricated construction, the design principles of prefabrication (more particularly in precast concrete) can be adopted as the basis for tackling these problems. In this context it is necessary to take due account of the specific characteristics of such buildings (depending on the purpose for which they are intended), their overall stability and their demountability. Suitable connections are then chosen for the structural frame which satisfy the requirements applicable to them.

To help the designer, a study group has been formed within the scope of the CUR-VB Committee D7, to prepare a report covering this range of subject-matter and which is available. (fig. 1)

The study was confined to loadbearing structures comprising columns, beams, loadbearing panels, external wall units and floor slabs. (fig 2)

In so far as the connections are concerned, existing manuals dealing with structural connections were studied.

Two important reports (fig. 3) compiled by the Stupre (Study group for Prefabrications in the Netherlands) which have been issued in English and German translations, are presumed to be sufficiently widely disseminated to justify only to list those connections which can be rated as demountable. This list is taken up in the above mentioned report.

In the report a number of structural connections are taken up and some ones will be given here. It concerns only connections for open systems of prefabricated building construction with precast concrete components. Different firms in the Netherlands and abroad have developed closed systems which are currently applied and which are mentioned in the last chapter of the report. This will not be treated here, but different authors will report on their system during the symposium.
1. REQUIREMENTS FOR STRUCTURAL DEMOUNTABLE CONNECTIONS

The first group of requirements is associated with the function of the connection:

- the connection should possess sufficient strength.
- connections for the transmission of bending moments should have sufficient rigidity, but also enough deformation capacity to ensure that under extreme conditions a redistribution of moments can take place, and they should moreover be able to absorb deformations, due to shrinkage and creep.
- there should be sufficient freedom of movement at hinges and simple-supported ends of structural members.
- the connections should be durable and resistant to corrosion and fire.

The second group of requirements relates to production and construction:

- the dimensional deviations occurring in production and erection should be accommodated in the connection.
- it should be possible to form the connections quickly and simply, without holding up the progress of erection.
- in view of the vital role of a connection in a structure, effective checking of its execution is very important and the connection should preferably remain permanently accessible for inspection.
- in detailing the connection it is of course necessary to take account of production possibilities, esthetic requirements and the cost of the connection.

Finally, the third group of requirements relates to the demountable character of a structure:

- connections should be demountable as simply as possible.
- demountable structures should in principle not cost more than conventional ones.
- some local cutting and hacking may be acceptable (e.g. when low-strength in situ concrete is used locally in the structure)
- not every part of the structure need have a residual value after dismantling.
- instructions for the correct dismantling procedure should be attached to the design.
- the following operations are to be regarded as demountable:
  - undoing of nuts.
  - removal of mortar without damaging the concrete of the actual structural components.
  - severing of welded connections by grinding.
  - severing of ungrouted prestressing tendons by burning.
2. STRUCTURAL SYSTEMS

The structural behaviour depends to a considerable extent on the location of the connections in the structure and on their properties.

2.1. Single-storey buildings

The structural stability derives mostly from columns fixed in the foundations. The beams are simply supported on the columns. (fig. 4) The problems associated with demountable design arise almost exclusively on suitable connecting the columns to the foundation. (fig. 5)

2.2. Multi-storey buildings

In this kind of buildings, the vertical forces are taken up by the columns or the load bearing panels, the horizontal forces are transmitted by the floors to the columns, transvers walls or to the cores. Therefore the floors possess a certain amount of rigidity within their own plane, i.e. they function in much the same way in the horizontal direction as shear walls do in the vertical direction.

The horizontal forces in question comprise mainly:

- horizontal wind loads which are transmitted from the external walls to the floor, which in turn transmit them to the columns, shear walls or cores. (fig. 6)
- as a result of difference in rigidity between one core and another it may occur that horizontal forces have to be transmitted from core to core.
- forces due to tilting of columns and second order effects.

The diaphragm action of the floor in the horizontal direction comprises, in approximate terms, a bending moment acting within the plane of the floor and shear forces. Fig. 7 illustrates an example with a different arrangement of the cores. The bending moments and shear forces will of course also present a different behaviour.

In a demountable precast concrete structure it is excluded to provide the precast concrete floor with a reinforced in situ concrete topping, the connections between the floors and stabilizing elements have to be resistant to the bending moments and shear forces. Tensile connections (ties) to resist bending moment will have to be installed, while steel shear connections or low-strenght mortar fillings in the joints will be used for resisting shear.

In designing rigid floor assemblies it is necessary to investigate properly what horizontal forces are liable to develop in the floor, having regard also to imposed deformations caused by settlements, shrinkage, creep etc. and to situations that may arise in consequence of disasters.
steel plates (to be welded in situ)

section a-a

elevation

section b-b
detail I

vertical section

threaded socket or anchor sleeve

hollow core slab

wall

tie bar

horizontal section

variant with prestress

horizontal section
detail II

Fig. 8

Fig. 9
3.1. Cores and shear walls

For buildings with a limited number of storeys the structural stability is ensured by columns fixed on the foundations, for high-rise the stability is ensured by cores and/or shear walls.

In a demountable structure, cores and shear walls have to be precast.

Cores and shear walls are loaded in bending and shear. The governing criterion is mostly their rigidity, not their strength. They must be sufficiently rigid to keep the horizontal displacement at the top of the building within acceptable limits, so that no objectionable movements are felt by the occupants and no damage to finishes and fittings occurs. Moreover the rigidity should be sufficient to ensure that second order effects due to the horizontal displacements will not be too great. With regard to this a certain relationship with the mass of the building therefore exists.

The rigidity of a core or shear wall is influenced by flexural deformation and shear deformation.

The connections to be provided in a core or shear wall are determined by:

a. The ratio between axial force and bending moment: flexural deformation is governed mainly by this ratio.
   b. The extent to which additional shear deformation is permissible.

If tensions occur in the horizontal joints unacceptable cracks are the result and an important consequence is that the moment of inertia and therefore the rigidity of the core are substantially reduced.

There are two available solutions for coping with the problem:

- Increasing the axial force by means of a vertical prestress.
  (with unbounded tendons)
- Installing vertical reinforcements in the precast units and interconnecting the units by means of vertical ties.

Vertical joints are mostly necessary. These joints will be loaded in shear, requiring appropriate connections. With regards to these it is likewise necessary to distinguish between strength and rigidity. And to be rigid mostly a demountable prefabricated core will be overdesigned in relation to the minimum theoretically possible core and moreover so to design the building that the cores and shear walls are loaded by the floors in such a way that the horizontal sections, notable at the joints between units, remain uncracked.
vertical section

wall

mortar

plastic film or bitumen

concrete projection

floor slab

horizontal section

column

prestressing bar

floor units

prestressed beam

column

column

column

Fig. 10

detail 11
detail 12

elevation

isometric projection

Fig. 11
3.2. Connections between shear wall elements and core units (fig. 8)

Detail 1 relates to the connection at the horizontal joint. The precast units are joined together by the welding of two cast-in steel plates. An alternative solution based on unbounded prestressing bars is also shown.

Detail 5 shows the connection at a vertical joint, likewise formed by welding. The joints between the units are filled with mortar.

3.3. Connection between the floor-slabs and shear wall (or core)

As example (fig. 9) the hollow core slab is taken. The connection between the end of the hollow core slab to a shear-wall is realized by placing the slab on a corbel, in the longitudinal joint between the slabs a bar (10 mm or 12 mm diameter) is placed which is secured to the wall by means of a threaded socket cast in the wall. To ensure efficient functioning of the connection of floor to wall, the joint filling should be executed with proper care.

The connection between the side (lateral face) of a hollow core slab to a shear wall (fig. 10). The wall is provided with a projection which engages with a recess in the slab. The contact faces of projection and recess may be covered with plastic film or with a bituminous coating. The remaining space within this joint is filled with mortar.

3.4. Other connections

Beside the core and shear walls the structure can be combined with two types of frame.

- type 1: columns, beams and floor slabs.
- type 2: loadbearing external wall units and floor slabs.

For type 1 the problems are arising to fix the beams to the columns and to foresee the connections between floor slabs and beams to assure a good diaphragm action of the complete floor.

Rectangular shaped beams can be used, but a better solution (more expensive) will be to use inverted U-shaped beams (fig. 11). They are supported on corbels and the beams are likewise rigidly fixed to the columns and one or more prestressing bars are used to form the connection. If tensile forces are likely to occur at the junction between the underside of the beam and the top of the corbel, a welded connection can be provided there.
section a

detail 15

section b

top view

Fig. 12
From the viewpoint of dismantling the structure it is desirable to use unbounded prestressing bars. The hollow core slabs (fig 12) are connected to the floor beam by tie bars installed in longitudinal joints (with shear keys) of the floor assembly. The tie bar is treated at each end, and is inserted through a slot in a cast-in steel profile fixed in the beam. Each floor beam is tied in this way at two points. To resist shear in the joint between the slabs and the beams, the sides of the latter, as well as the end faces of the slabs, should be suitably profiled to form shear keys. So far as the slabs are concerned this can be achieved by closing the cavities and setting back the closures a few centimeters from the end faces so as to form shallow recesses. The joints between the precast components are filled with mortar, preferably of low strength for better demountability.

For type 2 the hollow core slabs are connected to the loadbearing wall by means of tie bars (fig 13). These bars have treated ends and are secured to treated sockets cast in the wall. Each wall unit is tied in this way at two points. The tie bars extend across the building from external wall to external wall or so anchored in their joints between the floor slabs that the force which the bars are designed to resist can be transmitted by the floor. For this purpose the longitudinal joints between the slabs should be provided with profiling features as shown in fig. 14. For the transmission of shear between the wall and the floor it is necessary to provide suitable profiling both on the wall face and on the ends of the slabs, so as to ensure a shear keying effect. In the slabs this can be achieved by closing the cavities and setting back the closures a few centimeters from the end faces. The loadbearing wall units are connected to one another by means of welds joining steel plates cast in these units. These last mentioned connections should be given protection against corrosion and fire where circumstances necessitate this. Similar connection has been studied for double T-slabs (fig 15).

In both types of systems the hollow core slabs have to form an interconnected whole by filling the joints. If the shear forces are transmitted from slab to slab chiefly through the mortar joints, the longitudinal edges (lateral faces) of the slabs should be provided with profiling (shear keys). (fig.14) The joints between the slabs are filled with mortar, preferably of low strength for better demountability. Other connections can be found in the different literature mentioned in the beginning and I like to draw your attention to the fact that it is not easy to invent good connections and that it is preferable, like in general in prefabrication, to use standardized connections and with your experience or tests to make them better and less expensive.

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Fig. 13

vertical section through floor slabs

Fig. 14

vertical section through floor slabs

Fig. 13

isometric projection of slab edge

Fig. 14

isometric projection of slab edge
Fig. 15
CLASSIFICATION OF JOINTS FOR DEMOUNTABLE CONCRETE BUILDINGS

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SUMMARY

The selection of joints and connections for demountable concrete buildings is considerably simplified if there is a properly developed classification. The present paper attempts a classification of structural joints.

The classification covers framed and large-panel buildings. It is based on:
- the nature of the forces acting within a joint,
- the position and type of the elements connected,
- the materials used for the connections,
- the method of connection and the force transfer mechanism.

The loadbearing element interconnections and connections between non-loadbearing elements and the structure are dealt with separately. The emphasis is on the most important properties of individual joints.
CLASSIFICATION OF JOINTS FOR DEMOUNTABLE CONCRETE BUILDINGS

1. INTRODUCTION AND GENERAL CLASSIFICATION

Selection of joints and connections for demountable concrete buildings is considerably simplified if there is a properly developed classification. Classifications are usually reduced to grouping of joints and connections according to the dominant properties. Classifications should enable a comparison of different design solutions with respect to structural and manufacture properties and taking into account their interaction. There is a number of classifications for precast concrete structures, with different criteria. In the case of demountable concrete buildings, the situation is reverse. Therefore, an attempt has been made to develop a classification for demountable concrete buildings. Classifications enable easier spotting of the essential properties of joints and connections, and working out adequate solutions in accordance with requirements posed before a designer.

According to their function, joints of demountable concrete buildings are classified as expansion, protective, and structural joints. Along with it a classification of joints loaded by external forces is characteristic and those which do not transfer forces.

Deformations of building structures caused by time and temperature variations call for designing expansion joints. Expansion joints enable the separate parts of a structure to deform independently. The spacing of expansion joints depends upon static indetermination of a structure, its rigidity and method of connecting. In the case of demountable buildings, as compared to the monolithic ones, the spacing can be increased by 30-40%.

The fundamental task of protective joints is to provide tightness with respect to temperature, air, moisture etc. These joints also have an important role as regards corrosion and fire protection concurrently meeting aesthetic requirements.
The purpose of structural joints is to connect individual elements into a structural whole and insure integrity of a structure. The structural value of a demountable building generally depends upon the quality of these connections with respect to any load that may appear during its exploitation. These joints must meet durability requirements depending upon the building importance and service life.

In the present paper, accent has been placed upon structural joints due to their importance for building stability and the need to provide easy incorporation into a structural whole, their demounting and mounting on another location.

Design solutions are considerably influenced by:
- form, location and function of the elements to be connected,
- desired type of structure, and its structural system
- nature of forces to be transferred,
- properties of materials used for connection, and
- mounting-demounting conditions, etc.

Joints and connections selected should enable easy mounting and demounting of a structure in terms of fast incorporation of elements into respective structure and accessibility for inspection purposes.

2. CLASSIFICATION OF STRUCTURAL JOINTS AND CONNECTIONS

2.1. Generally

Structural joints and connections can be classified according to:
- nature of forces acting within a joint
- materials of connection,
- types of elements connected, and
- way of connection.

With respect to the nature of forces acting within them, joints are classified as those subjected to: compression, tension, shear, bending, torsion, and combined forces.
In spite of the intention to design joints in a way which would clearly define flow of forces, forces rarely act in isolation producing only one state of stress. Therefore, above classification refers to the prevailing force. Selection and design of joints should take into account utilization of material properties in terms of load transfer, i.e. compression is transferred by concrete, whereas tension is transferred by steel parts.

Structural joints are classified according to the type of elements joined as:
- bearing elements' inter-connections,
- connections between non-bearing elements and cladding with structure.

The differences are dependent upon less weight and lower mechanical characteristics of non-bearing elements and cladding than those of bearing ones.

The analysis of possible directions intensity of some forces in joint type is essential in the selection of these joints as well as the taking and transfer from one element to the other. This analysis is easier when it is connected to non-bearing elements and cladding with bearing structure.

The external elements of a building, except of external loading, are exposed to the variations of humidity and temperature hence, their connections are much more complicated than those of the internal ones. Thus according to the position of elements to be connected the following classification can be given:
- interconnection of external elements,
- interconnection of external and internal elements, and,
- interconnection of internal elements.

These classification refers to the interconnection of bearing elements and to the connection of non-bearing elements and cladding with constructions.

2.2. Joints and interconnections of bearing elements

The selected system of joints and connections should not reflect
negatively to the elements manufacturing its mounting and demounting. The intention is to produce elements of large dimensions in order to decrease the number of joints and work in mounting and demounting. This is connected to the possibility of lifting elements during mounting under storing and transport conditions. Joint and connection should be able of adjusting tolerances in element dimensions. The type of joint considerably depends upon building structural system, position of the element to be connected, and intensity of the forces to be received and transferred by a joint.

Skeleton and large panel buildings require different joints. According to the position of elements to be connected joints are roughly classified as joints for:
- interconnection of horizontal elements
- interconnection of vertical elements, and
- connection of horizontal and vertical elements.

With large panel buildings joints are classified according to the type of elements connected, as given below:
- wall to wall (connection and continuation of wall panels),
- slab to beam or slab to column, stairs to beams or slabs,
- slab to slab (continuation of slabs), and
- slab to facade or interior wall, etc.

This classification can be considerably simplified into horizontal and vertical connections of large panel buildings. Horizontal connection connect vertical and horizontal panels or horizontal panels only, and vertical joints connect wall panels.

Horizontal connections transfer and deliver compression forces from upper elements to the supporting structure below. In the case of wall buckling, horizontal connections may be exposed to shear forces. Nevertheless, in these connection shear forces prevail.

Vertical connections take over shear forces resulting from panel interaction along the joints. Tension and compression forces may occur due to uneven settlement and temperature variations. However, the dominating forces that occur in such joints are those of shear.

With skeleton buildings joints are classified according to the type of elements connected, as given below:
- column to foundation,
beam to column and their crossing,
- beam to beam in span,
- main to auxiliary beam, etc.

According to rigidity joints are classified as rigid, loose and hinge.

According to force transfer mechanism in joints, they can be accomplished by:
- seating,
- dowels and inserts,
- bolts,
- special anchoring and hanging,
- epoxy concrete, etc.

According to the way of seating joints can be classified as those with direct, indirect and combined seating. With respect to force transfer mechanisms and structural properties of element connecting, there is a considerable difference between seat joints which transfer compression forces, and joints which in addition to compression forces also transfer tension and shear forces.

According to the force transfer mechanism, joints are classified as linear, which transfer forces along the entire length, and concentrated, which transfer forces only at certain points. Horizontal joints are usually linear, whereas vertical joints are accomplished as concentrated or combined concentrated-linear joints. Seat connections transfer loads to supports of foundations directly or indirectly. In the case of direct seating, the areas below and above contact surfaces are specially reinforced to receive local compression stresses, while in the case of indirect seating load is transferred by built in metal plates and angle irons.

Joints to receive tension and shear forces with demountable concrete buildings are by the rule performed as "dry" - metal joints. These joints transfer forces and deliver them through special anchored plates and angle irons welded to the supporting structure or anchors. Metal parts are interconnected by bolts to enable demounting of concrete elements and their mounting at different location. These joints have high strength and enable good inspection of connecting works.

There is no need to carry out in-situ concrete works nor wait for
concrete hardening, so that connecting works can be performed even at low temperatures.

In the case of small loads shear can be received by metal dowels. Such connections are simple to be accomplished, but can not stand greater forces in the direction of longitudinal axis which reduces the number of possibilities for their application.

Torsion is the most difficult force to be received and transferred by connection. To avoid torsion designers resort to special structural measures. Torsion is usually taken together with shear and calculation made for their combined action. Besides it is more difficult to receive forces of different signs than those of the same sign.

To make an estimate of the advantageous behaviour of the elements, except of structural characteristics with respect to the possibility of receiving forces, a suitting way of mounting and demounting manufacturing must be taken into consideration too. E.g. precast elements with protruding steel parts are suitable for mounting and demounting, however, their manufacturing is difficult. The manufacturing of the connecting parts should be highly precise too.

2.3. Connections between Nonbearing Elements and Cladding with Structure

In addition to the weight of supported elements connections between non-bearing elements and structure should transfer to the structure wind, earthquake and other element forces. In doing so, care should be taken not to exceed local stresses of cast in parts, anchor plates, which receive and transfer forces, or stresses of the structure.

Both stability of non-bearing elements and esthetic value of facade and interior of a building depend upon the quality of such connections. Connections between external elements and structure are much more complicated. Internal connections, such as connections between partition walls and structure are easier to solve.

Connections between non-bearing elements and structure can be performed by seating, fixing and hanging.
According to the type of elements connected, joints are classified to connections of the structure and:
- facade walls,
- parapets,
- edge beams, and
- attics, etc.

Non-bearing elements could be connected with horizontal and vertical bearing elements and directly with foundations.

According to the influence of forces and direction of their receiving joints can be classified as:
- joints in wall plane, and
- joints which receive forces perpendicular to the wall plane.

Direction and sign of forces determine the joint type applied and position of joint and connection with bearing structure. If the joint is accomplished below the gravity center of the elements connected then it is a seat joint whereas if it is accomplished above the gravity center then it is a hanger connection. If the force is perpendicular to the plane of the non-bearing element then it is a tie back connection. It is the case of tension forces in wall plane too.

In order to accomplish load transfer in the simplest way, there is a tendency to reduce the number of points transferring load from non-bearing elements to the structure to the smallest possible number, most often to two points. It is a frequent practice to combine a seat joint at the lower end and fixing joint at the upper one.

2.3.1. Seat Connections

Seat connections are designed at places where vertical loads are to be transmitted to supports and foundations. Seating is performed in the plane of surface below. Concrete corbels in the form of continual or local widenings are often cast in the seating zone and they can be used for direct seating. Angle irons can be used instead of corbels. However, angle irons are corrosion susceptible, which speaks in favour of corbels in spite of more complicated prefabrication. Direct seat connections with corbels transfer forces by
friction. Dowels are also used to prevent lateral movements. Shim plates are used in seat connections which are to transfer greater loads. If considerable movement is expected seat connections should include low friction or soft material plates. "Dry" pack could be used in cases where minor relative movements are expected. Angle irons and shim plates are connected to non-bearing or bearing panels by means of bolts with slot holes which enable bolt movement in the direction of the slot hole. Neoprene bearings can be used at seat connections. Such bearings enable larger tolerances. A vertical facade panel can be seated to a slab by an angle iron which is at one end connected to the panel by a cast in bolt, and at the other bolted to the slab. A slot hole should be provided to enable movements, whereas connecting should be performed by bolts.

2.3.2. Tie-back and Hanger Connections

Tie-back and hanger connections are usually used to enable movements in the direction of the panel plane axis, and transfer of load in the direction perpendicular to the panel plane. Movements perpendicular to the panel plane can be achieved by connector deflection.

The most frequently used practice is to hang or tie back lighter elements by bolts or dowels cast into bearing elements by means of epoxy concrete. Steel anchor plates at the surface of concrete bearing elements can be used instead of bolts and dowels. Such anchor plates are connected to the bearing elements by appropriate anchors - reinforcement steels of sufficient length. They should comply to the concrete forms in the connection zones in order to prevent movement. These elements should be performed of stainless material to reduce corrosion hazard.

Cast in bolts transfer tension forces by means of concrete and bolt adhesion force, while bending moment is transferred by casting in the bolts above or below bending axis. Bolts can have insert cups, in which case they are screwed into such insert cups. The force transfer mechanisms are similar to those met with cast in bolts. Hanger connections can be obtained by tie back connection modifica-
tions. Elements are hang against anchored bolts or plates. In order to insure adequate corrosion protection, joints are performed of stainless steel. Besides that, joints should be additionally protected from moisture and fire.

3. Final Comments

The selected type of joints and connections for demountable concrete buildings influence not only the economics of the same, but also have a considerable impact upon the state of stress and displacement in the structure. They should provide clearly defined transfer of load to the bearing elements and foundation. For this reason, the present paper refers to the structural joints between bearing elements themselves and joints of non-bearing elements and cladding with the structure.

The paper has attempted to classify structural joints according to their main properties stressing the fundamental ones. A comprehensive and satisfactory classification would, by all means, require further work.

References

Fig. 1 Classification of joints according to the nature of forces. Joints exposed to: a) compression, b) tension, c) shear, d) torsion, e) combined forces

Fig. 2 Classification of joints according to the position of elements to be connected. Joints of: a) skeleton and external walls in plane, b) skeleton and external walls at corner, c) internal walls in plane, d) partition walls at corner, e) external and internal walls

Fig. 3
Fig. 3 Joints of large panel buildings:
a) through d) horizontal joints,
e) vertical joints
Classification according to the way of seating: a) direct seating of slab to wall with upper wall interruption, b) indirect slab seating with direct column seating, c) combined upper wall seating, d) indirect slab seating without wall interruption at joint.

Fig. 4 Vertical element interconnections:
a) continuation of column,
b) continuation of wall.

Fig. 5 Column to column connections:
a) hinge
b) and c) quasi rigid

Fig. 6
Fig. 6 Bearing element interconnections: 
a) column of foundation, b) wall to foundation, c) beam to column, d) slab to wall, e) main beam to secondary beam

Fig. 7 Classification of joints according to force transfer mechanisms: a) direct seating with short corbel, b) seating with fixing by dowel, c) seating with shim plates, d) anchored inserts for hanging light concrete panels: 1. insert cups with bolt treading, 2. bolts anchored by epoxy concrete, e) anchored bolts and angle irons
Fig. 8 Classification of joints according to the type of forces acting within a joint: a) tie-back, b) tie-back and seat, c) tie-back and hanger

Fig. 9 Joint performed by a profile or panel hanging without tie-back 1. bearing element, 2. light concrete wall

Fig. 10 Combined seat and tie-back joint
The assembly of precast reinforced concrete units needs a certain degree of precision and dimensional stability.

Casting, curing and storage require care which are bound up with the further use of the units.

Conveying, handling and erection also have to be done with proper care. The different dimensional tolerances and the effects of warping, shrinkage and distortion must be checked and duly allowed for.

In that case post-tensioning will be no problem if the stability of the connectors is ensured.
1. THE CONSTRAINTS DURING CASTING

- The dimensional tolerances are often $\pm 1$ mm taking account of shrinkage over 20 days.
- The angular precision determines the quality of the fit.
- The moulds must not get out of shape during casting and should allow multiple re-use.
- Blowholes on exposed faces must be avoided.

2. REMOVAL FROM THE MOULD AND LIFTING

- Care must be taken not to pull the concrete away from surfaces to which it adheres.
- The stresses which occur during lifting and handling, often significantly different from those under working load, must be duly allowed for.
- The lifting points must be positioned well away from the corners or the edges, and lifting with straps or slings must not be allowed. (Figs. 1 and 2).

3. STORAGE

During storage of the units, inadmissible stresses due to support conditions, direct sunshine, and large differences in temperature between two sides, must be avoided. Stacking or juxtaposing the units gives them some protection; fragile ones should be placed in a workshop or a shed.
4. TRANSPORT

- Bad stowage, shifting of the units, and impact against the other units as well as the sides of the vehicle, must be avoided.
- Abnormal vibrational forces must be avoided.
- The units must be packaged to a greater or lesser extent, especially the edges. Skids, cushions and tires can be used.
- The longer units must be tightly secured in transit. (Fig. 3).
- Manoeuvring on uneven ground can bend the chassis of the vehicle and adversely affect on the units being transported.

5. INSTALLING THE UNITS ON THE BUILDING SITE

The maximum weight for units for continuous manual travelling is 40 kg. As mechanical lifting is now standard practice, it is more advantageous to produce heavy units, so that time is saved and the number of joints reduced.

In building construction the optimum unit weight appears to be 6-8 tonnes for structural works. In civil engineering, however, weights of up to about 40 tonnes are usual; indeed, in the Netherlands precast units of very much greater size and weight are used in some types of work.

6. ASSEMBLY JOINTS

The assembly joints can be divided into two main categories:
- vertical (Fig. 4 and 5);
- horizontal (Fig. 6).

When they are formed dry, i.e. without mortar, they must ensure:
- the overall stability of the structure;
- watertightness;
- distribution of the weight;
- allow slight movements, but restrain them if they tend to become large.

6.1 **Vertical joints**
Vertical joints are no problem if the dimensional and angular tolerances have been complied with. (Fig. 6).

6.2 **Horizontal joints**
The horizontal joints support the loads. Load distribution will be improved by the interposition of neoprene strips, which will also stop the capillary rise of dampness. Their thickness will be increased in areas exposed to earthquake hazard.
Dry jointing is used throughout.

7. **POST-TENSIONING**
Apart from the systems usually employed in civil engineering structures, it is better to use, for simple work, post-tensioning by means of threaded rods provided with nuts, tightened with torque wrenches (Fig. 7 and 8).

The effective diameter of a screw-threaded circular rod can be assessed at:

\[ d = D - 1.0825 P \]

- \(D\) effective diameter of the circle (mm)
- \(P\) thread (mm)
- \(d\) effective diameter (mm)

For a 1000daN tensile force on a 10 mm diameter wire:

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D = 10 mm \hspace{1em} S = 78.5 \text{ sq. mm (mm}^2\text{)}

P = 1 mm

d = 8.91 mm \hspace{1em} Su = 62.36 \text{ sq.mm (mm}^2\text{)}

T = 1603 \text{ Ba (on the thread)}

extension: \hspace{1em} E = 2100.000 \hspace{1em} T = 1603 \frac{Su}{S} = 1273 \text{ Ba}

A = 0.6 mm/mct (on D)

8. CONCLUSION

The dry assembly of large fully-finished factory-made precast concrete units by prestressing with post-tensioned steel must effect considerable economies and greatly speed up the work of construction.

Fig. 8
(a) Measurement of $p_x = X$, $p_y = Y$, $p_z = Z$

(b) Measurement of $\theta$, $\phi$, $\psi$

(c) Measurement of $\alpha$, $\beta$, $\gamma$

(d) Measurement of $\delta$, $\epsilon$, $\zeta$

(e) Measurement of $\kappa$, $\lambda$, $\mu$

(f) Measurement of $\nu$, $\xi$, $\pi$

(g) Measurement of $\rho$, $\sigma$, $\tau$

(h) Measurement of $\upsilon$, $\phi$, $\chi$

(i) Measurement of $\psi$, $\omega$, $\theta$

(j) Measurement of $\eta$, $\zeta$, $\iota$

(k) Measurement of $\kappa$, $\lambda$, $\mu$

(l) Measurement of $\nu$, $\xi$, $\pi$

(m) Measurement of $\rho$, $\sigma$, $\tau$

(n) Measurement of $\upsilon$, $\phi$, $\chi$

(o) Measurement of $\psi$, $\omega$, $\theta$

(p) Measurement of $\eta$, $\zeta$, $\iota$
Building is solving a housing problem:

1. "temporarily"
2. "permanently" for "utility" purposes (schools/offices/car parks)
3. "permantely" for "public" purposes (theatres/town halls)

The buildings in category 1 have always been demountable (e.g., site huts/temporary school class rooms).

There may be reasons to employ demountable structures for those in category 2. Based on the principles of a "closed system" for temporary buildings in wood and steel, a philosophy has been developed for a "sealed system" using concrete elements for "permanent", "utility" buildings which can be dismantled and re-erected. During the last five years the system has been used in school and office buildings; there are designs for larger scale structures such as car parks.

The system will be described in so far as it is of interest in giving information about mechanical principles, production, erection and dismantling.
1. INTRODUCTION.

Building is solving a housing-problem. In the field of "demountable building" a distinction should be made, based on to what extent and how long a building will remain unaltered in use, and to what extent and when the building is likely to become redundant.

With these distinctions in mind the following solutions may be possible:
1. "Temporary" buildings for building stores, emergency structures for schools etc.
2. "Permanent" buildings in the form of "utility" buildings for schools, offices, parking garages etc.
3. "Permanent" buildings for municipal or cultural functions such as town halls, theatres, etc.

Demountable building systems in steel and wood have been available for many years for the first group; for the second group there is a growing interest in demountable buildings which are easy to dismantle and reerect; for the third group there is little or no interest in demountable buildings.

The need for demountable buildings has arisen due to several reasons. Because of rapid social and technological changes it is preferable if the size and function of office buildings are readily adaptable and adaptability to the particular site is also required more and more. School buildings in newly developed areas are often superfluous, even after a relatively short time, or become too large because of the limited number of students. Examples can be quoted where a certain area in a town, for instance, is used for a parking garage and then after a few years, is needed for some other function. In short, in this type of building, the technical lifetime is generally longer than that needed for a particular function and the need, therefore, arises for a type of buildings which can be readily dismantled with as little energy and disturbance as possible. Ideally the components of the building should be such that they can be reused.

There have already been for some time, buildings in steel and wood which satisfy these requirements. There is the challenge now to provide the same type of building in concrete and, logically, to use prefabricated elements. If conventional components, such as columns, beams and floor elements are used in demountable systems, the connections required would be more expensive than those required for "orthodox" constructions. Demountable buildings would then cost money. This requirement has led to the development of "sealed systems" in which the connections are integrated in the elements, and the shapes of which are based on a starting point of "demountability". The construction described below is such a "sealed system".

2. DEVELOPMENT.

The stimulus for development was the requirement, in the 70's, of the Netherlands Ministry of Education and Science that, in certain situations, it should be possible to demount and reassemble school buildings elsewhere. A building industry, which for years had had experience in building schools from prefabricated, but not demountable, elements, answered this requirement with concrete. The first ideas were presented at the end of 1977; the first school was ready at the beginning of 1980. Since that date the system has been used for about 35,000 m² of schools and office buildings.

The philosophy of the system is aimed at an optimum balance between
construction time and construction costs.
In principle there are three main activities in building:
- the framework with or without facades
- finishing
- the installations
The starting point for the last two activities depends, in general, on when the framework is ready:

Objectives related to the framework.
There should be the minimum number of critical path activities and each of these should have the shortest possible duration.

Objectives related to finishing and installations.
With respect to execution, the various activities should not be interdependent. Finishing and installation technology should in no way effect framework construction.

3. DESCRIPTION OF THE SYSTEM. (figures see after page 9)

A load-bearing structure has been designed, taking into account the above considerations. The structure has the characteristics described below.

3.1. The load-bearing structure.

3.1.1. Floor elements rest directly on columns so that the minimum amount of work and costs are required for the connections, during production and assembly.

3.1.2. Positioning details are such that up to four floor elements can be laid on one column; the floor elements are fitted with in-built steel corner bearing elements. (figure 1)

3.1.3. Columns with interposed floor-units are interconnected by dowel-pins passing through the bearing elements of the floor-units. For this purpose, both ends of the columns are provided with steel plates with welded on close-fitting sockets which are cast in when the columns are concreted.

3.1.4. There is sufficient tolerance in the floor element positioning and fitting operation to take up, horizontally, any slight deviations resulting from the production and assembly processes; positional fixing is achieved, per floor element, by filling any spaces with mortar.

3.1.5. The reinforced concrete floor units can be installed in position with no preparatory operation. They form a surface with only negligible differences in level which is so flat, that the floor covering can be laid directly on them possibly after some smoothing. The absence of a floor-finish makes for greater speed of construction. (figure 2)

3.1.6. Because of reinforced concrete construction of the units the alignment of the system can be rotated with column erection along the longer edge, in the span direction, of floor units.

3.1.7. During assembly the precision work is limited to setting columns and walls vertical. This is all that is necessary because, from the foundations or the basement floor, the columns and the walls are set out very accurately, horizontally and vertically.

Given the column length tolerances the system can be used for up to three or four storeys without any releveelling. For higher buildings the columns could be releveelled at the third or fourth storey.
3.2. Stability.

3.2.1. The stability (of the structure) is provided by the horizontal rigidity of the floor sections tied at the columns, and by the vertical wall elements.

3.2.2. The wall elements are as thick as the columns and at the corners have similar fittings to those on the columns so that floor sections can be laid on the corners.

3.2.3. The capacity to take up horizontal loads through the walls is obtained by preloading the loads under their own weight and the weight of the floors which they support or from additional connecting ties extending from wall to wall and down to the foundation.

4. Dimensional Coordination.

4.1. System 2.40 m - elements. (figure 3)

The floor sections are 2.40 m wide with lengths up to 7.20 m, the length varying in increments of 1.20 m. For lengths between 4.80 m and 7.20 m, the floor sections are made with ribs 1.20 m centre to centre and a flange (slab) thickness (= bearing thickness) of 120 mm. The space from column to column is bearing thickness of the floor-elements (120 mm).

For lengths between 2.40 m and 4.80 m, flat slabs are used with a thickness suitable for the required span and with a bearing thickness of 120 mm. Units of reduced width (1.20 m) can be used as "specials" for dimensional adaption. The units are provided which shear-resisting joints which have to be filled with mortar.

The columns have a cross-sectional area of 0.20 x 0.20 m² and a variable height. For application in school buildings i.e. where the ceiling height is 3.20 m the column height is 3.08 m.

The length of the stabilizing wall sections is equivalent to the system length of 2.40 m plus two times half the column thickness, 0.20 m, that is, 2.60 m. The wall height is similar to the column height and the wall thickness is the same as the column thickness, 0.20 m.

4.2. System 3.60 m - elements. (figure 4, 5 and 6)

The floor sections are 3.60 m wide with variable lengths between 2.40 m and 7.20 m in units of 1.80 m. For spans of 5.40 m to 7.20 m the floor sections are made with ribs, with a center distance of ± 400 mm from the longer edge; the flange thickness along the edges is 160 mm, (= bearing thickness). The flange between the ribs is 100 mm thick. Floor elements widths of 1.80 m can be applied as "specials".

The columns have a cross-sectional area of 0.25 x 0.25 m² and a variable height. The length of stabilizing wall sections is equivalent to the system length of 3.60 m plus two times half the column thickness, 0.25 m, that is 3.85 m. The wall height is similar to the column height and the wall thickness is the same as the column thickness, 0.25 m.

4.3. Recesses.

4.3.1. Basically only standard recesses in floorslabs are entered. With a view of the reinforcement situation on the spot no boring is done as a rule of thumb in the bearing support areas and in the strip covering the width of the slab between bearings.
All remaining holes are to be drilled on the site. In places where recesses are manifold (such as lavatory groups) floor elements with small span width are recommended to choose. In that way ribless flat slabs can be applied. Horizontal sleeves through the ribs can be made available as a standard model however within limits. Conduit pipes are not to be cast in the concrete.

4.3.2. Vertical feed through passages.

(Rib)elements 2400 x 7200 and 3600 x 7200.
Area A.
Recesses up to either Ø 300 x 300 or Ø 300 x 300 can be applied at liberty but numbers in concert. These can be performed either in the plant or by drilling on the site. Openings of bigger size, either circular, quadrangular (square) or rectangular are exclusively to be performed in the plant, and in concert.

Area B.
Maximum 2 sleeves of Ø 50 mm in an even line (see plan), preferably symmetrical; minimum distance 75 mm in respect of each other; to be performed in the plant.

Area C.
No recess!
With the exception of elements along the facade in which as a standard, by the plant, can be provided (if required) a recess of Ø 25 mm in behalf of feed through passages for central heating (radiator conduits).

4.3.3. Horizontal feed through passages through (rib)element 2400 x 7200 and 3600 x 7200.
Area A.
Passage of conduit pipes along the columns within the height of the ribs.

Area B.
Preferably no sleeves.
Fabrication is technically hard to realize. If imperative however, it could might be carried in the plant as a standard for all elements. Dimensions of the sleeves required, should be held within the limits of conduits for electricity wiring and for telephone lines and such like.

4.4. Sound insulation.

The required vertical sound insulation can be achieved by increasing the mass (where necessary). Under normal conditions a mass of 250 kg/m² meets the insulation requirements between class-rooms.

Floor slabs of both systems do meet these requirements:
- System 240: Minimum flange thickness 120 mm
- Systems 360: Minimum flange thickness 100 mm

5. BEARING CAPACITY.

Both systems are, in base, dimensioned and reinforced on a load capacity (not including the dead-load of the element) of 3,15 kN/m² (315 kg/m²). Based on these loads the maximum building height is three to four storeys, using the standard reinforced columns.

By simply increasing the reinforcement the load capacity can be increased to:
- 4,65 kN/m² (465 kg/m²) for spans of 7,20 m.
- 7,50 kN/m² (750 kg/m²) for spans of 4,80 resp. 5,40 m.
- 10,00 kN/m² (1000 kg/m²) for spans of 2,40 resp. 3,60 m.

The bearing capacity of the columns can be increased to a maximum of 650 kN (system 2,40 m) or 1000 kN (system 3,60 m).

The dead-load of the floor rib-elements (span 7,20 m) is 3,58 kN/m² for the system 2,40 m and 3,66 kN/m² for the system 3,60 m.

6. MECHANICAL.

The principle of the connection column - floorslab - column, is based on transfer of force by steel to steel contact; this is imperative because of the limited dimensions of the contact areas and on account of the required accuracy of the connecting parts when mounting without relevellng and without any mortar joints. The floor slab bears two functions:

a. The transfer of the vertical loads to the columns on the four corners.
b. The transfer of the horizontal loads via shear forces in plane of slab to the wall slabs.

For both situations (functions), the element with a span length of 7,20 m applies the standard.

ad a. The ribs of the 7,20 m elements deal with the shear forces and bending moments caused by external vertical load and span. The problem lies in transfer of the shear force in the rib via the comparatively thin flange (120 mm respectively 160 mm) to the supporting columns.

In order to gain an insight in this transfer of force, the elements have been
calculated by a computer program, using the infinite elements method. To that end the slab element is split up into a network of small elements measuring 120 x 120 mm.

Figure 7 shows size and direction of the major transverse forces in the area between rib suspension and corner support with free end bearing for the 2.40 m system. The overhead load on the connection with the upright column on the floor causes a measure of fixing end support. Consequently the elements have been calculated based on an entire free end and with fixed end supports. The nature of the actualized connection allows the bearing in practice a certain rotation capacity:
- Little height (120 respectively 160 mm) with regard to the length of bearing (100 mm resp. 125 mm).
- A slight stiffness in flexure of the pin in the mortar filled cavity in the floorslab.
- Rubbers in the bearing connection (These rubbers are functioning as well for planing the essentially not exactly parallel steel contact areas of floor slabs and columns).

Calculation results as well as preceding considerations decide on reinforcement of the concrete flange and the anchorage of the corner steel elements there in.

Ad b. When transferring shear forces in plane of slab, tensile force, force of pressure, and shearing forces arise. The pressure is transmitted via the mortar filled joints between the mutual elements; the tensile forces and the shearing forces must be transmitted from element to element via the connection located at the column.

Figure 8 shows as to how the binding stay in the floor by way of shearing in the steel pins and via the steel plates of the columns acquires continuity; the same goes for the shearing forces and for the transmission of the horizontal forces in the floor to the shear walls.

7. FRONTS.

Sandwich panels are placed to the outside where they are hung against the columns. Concrete sandwich panels are used in standard sizes of 2.40 m to 3.60 m long and in standard heights based on the vertical lay out of storey height. The panels are bolt simply to the columns; the suspension can be re-adjusted; the concept lends itself to being dismantled. Combined with window frames and doorcases open and closed fronts are practicable.

Any other front (panel) can be applied; possibly not fit for dismantling.

8. DISMANTLING AND RE-BUILDING.

With the nature of connections used, the concept lends itself to being dismantled. After removal of the window frames the concrete sandwich panels are within reach of the crane and after releasing the necessary nuts the dismantling can begin. Floor slabs can be removed after having taken the dowels out of their corner position; now the floor slab can be shifted and detached from the mortar filled shear resisting joint and can be vertically hoisted out of the building. After removal of the floor slabs, the hoisting appliance in the steel end plate of the column has been set free, and the column can be pulled out of the dowels perpendicularly.

The wall slabs follow the same routine as the columns.

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All elements are ready for re-building.

As defined under the heading "Development", see page 2, the government required school-buildings to be removable and in such a situation the building company must guarantee during the next 15 years to move the building, if required, at 25% of the cost of construction adjusted to the index number. Any builder who is confronted with this situation, gives the guarantee, hoping that the school may stay for the next 15 years. It has been proven that removal at 25% of the cost of construction, can be effected for container building systems.

Buildingsystems in elements, as described, require approximately 50% for removal; the higher rebuilding costs are caused generally by the finishing (walls, ceilings etc.) and technical installations.

9. APPLICATIONS.

The method of constructing floors direct on columns, "floors without girders!", with the unique techinics of dowel connections, not only provides possibilities and opportunities for demounting and rebuilding but also has the quality to make permanent buildings in a very short time and at low costs. Very competitive indeed!

Hinged columns combined with stabilizing wall slabs and the relatively high rotation capacity of the floor and the column connection make this system very suitable to stand earthquake situation: relatively big deformations of the framed structure, lead only to minor additional forces in the connections.

The dimensions of the elements of both systems have been specifically guided by the standards used for office buildings, schools, hospital clinic departments.

From 1980 ± 35000 m² of floor space in this system have been built in the Netherlands, covering school- and office buildings; besides 2 schools have been built in KOEWEIT, totalling ± 15000 m² of floor space. Designs have been made, respectively constructions in preparation, with large span lengths for parking garages.

Double T prestressed floor slabs are applied here (width 2,80 m length ± 18,00 m) with direct support at the corners by console columns. Although the upshot will be a demountable building, this has not been intended to be one since we have been guided exclusively by economical motives:

- The floor slabs may be made in current moulds for double T on the long bench.
- The requirements of the manufacture of columns will be met by simple moulds.
- Stabilizing wall slabs require the same supporting provision as the columns.
- No girders required.
- Simple connection type.

The above leads to little investments, cheap production and fast erection also due to assembling without any mortar joints.

In short and in general: this way of doing is leading to:

- low building costs
- short building time
- possibility of dismantling and rebuilding for good measure
- great residual value invested in usable elements
FIG. 1 Typical connection four floorslabs on one column.
(system 3,60)

FIG. 2
Typical joint between floorslabs.

Typical joint between floorslabs.

FIG. 2
Typical joint between floorslabs.
system 2,40 m
flat slabs

system 3,60 m
flat slabs

system 2,40 m
ribbed slabs

FIG. 3

system 3,60 m
ribbed slabs

FIG. 4
FIG. 5 Possible arrangement floorelements and columns.

FIG. 6 Longitudinal section floorelements.
FIG. 7
Lay out of the network (120 x 120 mm).
- this lay out is showing:
  - positions of ribsuspension and ribsupport at the column
  - major transverse forces (kN/m)
  - the faces in which these forces appear

FIG. 8
Floor under horizontal load Q, with stability-walls.
A NEW TYPE OF URBAN BRIDGE, PREFABRICATED AND DEMOUNTABLE

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SUMMARY

Contrary to a normal bridge, an urban bridge may have a limited life, depending on the development of the city and its traffic. Dismantling the bridge should be possible at minimal cost and should also be carried out as fast as possible to avoid or to minimize traffic disruption. All this leads to the idea of a demountable bridge.

The AB bridge is a demountable bridge built of steel and concrete, i.e., a composite bridge, but different from other composite bridges.

The connection between steel beams and concrete is achieved, not by means of rigid connectors, but by elastic devices.

Thus the AB bridge permits rapid erection and dismantling, together with the possibility of subsequently reassembling the bridge in another location.
NEW TYPE OF AN URBAN BRIDGE PREFABRICATED AND DISMANTLABLE

The AB bridge, object of this paper has been conceived in 1973 and the first AB bridge was inaugurated in 1975.

Studies started from the following ideas:

Contrary to other traditional bridges, the urban bridge has no everlasting vocation. As a matter of fact, the development of a city and its traffic load may consequently lead to modify or even do away with an urban bridge. It is therefore desirable that an urban bridge has the possibility to be dismantled.

And, as it is a question of achieving works within a centre of existing traffic, the works must be carried out within the shortest possible delay.

Similarly, when a new urban bridge is to be built within a centre of existing traffic, it should be completed within the shortest period in order to avoid the least disturbance to the existing traffic.

To meet this objective prefabrication should be used: all parts are manufactured away from the building site and the work "in situ" is restricted to the assembling of the bridge only.

Finally the AB bridge has been conceived according to the following program.

"To achieve a prefabricated urban bridge that can be built and dismantled within a very short period. In spite of prefabrication, the bridge should possess the properties of a permanent bridge and particularly be continuous in order to suppress all expansion joints over the whole bridge's length. Particular care should be given to the aesthetic outlook and to its integration in the urban surrounding.

An urban bridge is intended for a "separation grade" i.e. to allow traffic over a cross road. Such a bridge in a general way consists of one main span and twice five approach spans making a total of eleven spans.
It is clear that such a layout calls for a continuous structure which carries the advantage of reducing the moments and the deflections and thus permits also to suppress the expansion joints over all the length of the work which is about 250 metres.

We have adopted the composite structure (steel + concrete) which differs from the traditional composite structure where the link between the steel and the concrete is performed by rigid links, particularly welded studs.

Owing to the continuous character of the girders, the negative moments' zone stretches over half the total length. On the other hand, as the concrete is used in the form of prefabricated slabs, the traditional composite technology cannot be applied.

To solve the problem, we substitute the rigid links (welded studs) by elastic fastenings of an original conception.
We obtain the continuity of the metallic girders by a system which diverges from the traditional method and which we call "simplified continuity".

The continuity is ascertained by joint covers only on the webs. This is combined with the fact that the continuity joints are set at places of minimum moments, that is at about 1/4 of the bearing.
All these simplifications have enabled us to take the girder at the rolling mill's exit, at a time when it is still at a low price, whilst it is already an operational element of high technical quality. This girder needs a minimum additional work which is to bore holes in the web at both ends, intended to receive the joint plates.

Finally, the AB bridge is achieved with simple elements, reliable, of high technical quality at cheap prices.

It is a question of rolled steel girders with wide flanges, taken away at the exit of the rolling mill, requiring a minimum additional work and a question of reinforced concrete slabs, 18 cms thick, for the bridge flooring, of curb-elements in prefabricated reinforced concrete and, finally, the fixing devices, that includes neoprene plates, bolts, sleeper clips, double Vossloh washers, all are supplied by specialised manufacturers.
Thanks to the simplified continuity and the elastic fastenings, the AB bridge, contrary to other prefabricated works, appears as a compact bridge with no expansion joints over the whole length of the bridge, which is about 300 metres. To make it short, the AB bridge may be considered as a permanent bridge with the advantage that it can be dismantled when necessary.

The structure of the AB bridge includes, according to the nature of the soil; either piles (stakes) or direct foundation slabs realised "in situ". On these "in situ" foundations are erected either prefabricated columns in reinforced concrete, or a central pier in reinforced concrete realised "in situ". Over the two columns or over the central pier is placed the prefabricated cap of reinforced concrete.

The abutments of the bridge may also be realised by means of prefabricated elements in reinforced concrete.

photo n° 2 : abutments in prefabricated elements
On this substructure is placed the superstructure in which the new, unconventional character of the AB bridge is revealed.

Metallic girders rolled in higher heights (from 600 to 1000 m/m) are used for the metallic part involving a minimum additional work.

Each circulation lane is supported by a group of two cross-braced girders.

![Fig. 5: cross sections](image)

The metallic frame work rests on the caps with interposing neoprene bearings that permit the thermal expansion of the bridge.

Lengthwise, the metallic structure is continuous from end to end of the bridge. The various metallic beams are organised in such a way that the continuity joints are located at places where, under permanent loads, the moment is nil.

On the metallic structure rests the reinforced concrete flooring composed by reinforced concrete slabs 3 m 50 x 3 m 00 having a thickness of 18 cm, each slab weighing 4, 5 T.

It is not possible to place the slabs directly on the girders for manufacturing tolerances occur.
To solve this difficulty, neoprene plates are placed between the reinforced concrete slab and the girder. The neoprene plate thickness must be superior to the sum of the tolerances of the girder and the slab.

Furthermore, the slabs are not simply deposited on the girders, they are solidly attached by means of assembling devices which include each two bolts, two sleeper clips, two double Vossloh washers and one neoprene plate.

The double Vossloh washer is a genuine silico manganese steel spring so that each washer gives a tightening force of about 5 T.

Six fastenings per slab are provided so that for each slab a global tightening of $6 \times 5 = 30$ T is obtained.
The neoprene plates solidly tightened against the steel create genuine chbc absorbers with all the advantages involved.

Prefab curb-elements are provided in which the railing will be fixed.

Immediately after the placing of the slabs and curb-elements the asphaltic wearing surface can be laid and the bridge is ready to be opened for the traffic.

As mentioned further above, the bridge is continuous over all the length; that means there are no joints over all the bridge's length which may reach 300 metres. The expansion joints are placed between the abutments and the bridge, that is to say at the entrance and the exit of the work.
This is most important, for the joints represent always a weak point, a source of destruction and maintenance.

It is necessary to point out that the first AB bridge is on service for over ten years.

In a general way, the prefabricated constructions take a character of stacking pieces one over the other with a large number of passive joints.

The AB bridge breaks away from this passive tradition; here, on the contrary, the approach is active.

The different girder’s sections are assembled in such a way as to create a continuous girder. The concrete slabs do not rest simply on the metallic structure but are energetically tightened against this structure.

To make it short, AB uses prefabricated elements but once it is terminated it is a sturdy and coherent construction.

Here is the real meaning of the AB bridge: to create a high quality bridge by starting from prefabricated elements.

Another aspect of the AB bridge is its high degree of standardisation.

Apart from the girders, all other pieces are identical and interchangeable from one bridge to another. It is a question of concrete slabs, concrete curb-elements, fastenings bolts, clips, double Vossloh washers and neoprene plates.

The only specific pieces for each bridge are the girders, shape and length.

Therefore one can say that by starting with standardised pieces the AB bridge is a bridge made to measure.

Finally, we emerge on the most important aspect of the AB bridge: the rapidity of execution; its importance is evident in an urban surrounding.
We can say, grosso modo, that with a good organisation, the period between the moment the decision is taken to build an urban viaduct and the moment this viaduct is put in service may be reduced to about four months.

When the construction of a viaduct is decided, it is necessary to proceed with the preliminary operations, topographic survey, study of the axis, identify the nature of the soil, the foundations and the possible obstacles in the subsoil, the gas, water and electricity conduits and the telephone cable lines, etc ...

The obstructing conduits must then be displaced and the foundations started "in situ".

These preliminary operations generally require about three months.

Now, the critical passage of the Pert diagram does not pass through the prefabrication of the bridge pieces but well enough through the period required for those interferences and preliminary works.

It is not necessary to wait for the end of the preliminary works to ascertain the location of the columns. As soon as such locations are known, it is possible to pass the order for girders within a very short time.

As for the reinforced concrete slabs and the curb-elements there are no problems as these items are always identical; orders can be passed immediately.

In other words, when the studies, investigations and preliminary works come to an end, all the parts of the bridge should be on the site, which naturally means a good organisation. From this time, eight days are required to set up the bridge.
We should like to stress the fact that the period of eight days required to set up the bridge does not imply eight days of traffic interruption in the thoroughfare to be crossed. As far as the AB bridge is concerned, the traffic for crossing that thoroughfare would be interrupted during four hours at night (between midnight and four o’clock in the morning) just the time required to mount the prefabricated columns and the caps and to deposit the metallic structure spanning the cross road over the substructure. After that, all the work is carried out quietly above, without the least obstruction to the traffic on the main thoroughfare.

We are giving hereafter some information about the first two AB bridges.

**AB bridge 1**
- total length 430 m
- length between the abutments 243 m
- total width 11 m 46
- main span 35,70 m
- reinforced concrete 499 m³
- metallic frame 273 T
- on service since 12th February 1975

**AB bridge 2**
- total length 430 m
- length between the abutments 343 m
- total width 7,96 m
- main span 37,11 m
- reinforced concrete 535 m³
- frame's weight 276 T

Three other bridges were realised in Algeria, two at Algiers on the entry runway of the airport to the centre, and one bridge at Annaba.

Two bridges were built in Belgium and a third one is under order.

In principle, the AB bridge has the vocation of an urban bridge. Nevertheless, the Belgian Public Works Administration is interested in the AB bridge and has requested us to study the application of the AB system to the superior bridges over highways.

Finally, we have received an order for three superior bridges for the Ardennes highway one of which bridge n° 34 is in service since two years, the second bridge is under completion.
The remarkable aesthetic success of bridge n° 34 should be underlined.

Its light and elegant outlook fits in harmoniously with the Ardennes landscape.
TEMPORARY PIERS CONSISTING OF PRECAST CONCRETE ELEMENTS FOR BRIDGES BUILT BY THE INCREMENTAL LAUNCHING METHOD

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SUMMARY

The incremental launching method needs temporary piers to support large spans for the construction of the bridge deck. The best solution for constructing these piers proved to be the use of precast concrete elements.

The main advantages were found to be:
- Reinforced concrete is the cheapest material to transfer the heavy loads to the ground.
- Consisting of precast elements, the piers are easy and quick to erect, dismantle and transport.
- This is therefore the most economical way to construct auxiliary piers for multiple use.

The design and detailing of the precast elements and their connections are reported.

Experience with piers up to 55 m in height is described, with particular reference to their erection.

The transverse launching of a complete auxiliary pier over a distance of 15 m is also described in the paper.
TEMPORARY PIERS FOR INCREMENTAL LAUNCHED BRIDGES
CONSISTING OF PRECAST CONCRETE ELEMENTS

For the construction of the superstructure of bridges the incremental launching system is known worldwide. Spans from 40 up to about 60 m are most economic. The consumption of prestressing steel depends on the normal span length of the bridge and if there are some larger spans, the best way is to use auxiliary piers in these spans. This saves pre­stressing steel and in general it is often necessary to allow the application of the launching system at all.

Often these temporary piers were constructed with cast in situ concrete using slip forms. After finishing the launching of the deck and final prestressing, these piers were removed by blasting. Also sometimes these piers were constructed using structural steel, but due to the heavy vertical loads, they are quite expensive.

Therefore from the beginning of the method we tried to use temporary piers consisting of precast concrete elements.

The design requirements could be described as follows:

- Obvious the expenditure for steel, concrete etc. should be as low as possible.
- Mounting and dismantling of the piers should be possible in a short time and at low cost.
- One important fact for this is that the heaviest element can be lifted into the final position with the crane already installed for other purposes on the site.
- To get a minimum of transport and storage cost the total weight of the construction had to be low.
- The pier had to be designed for the fairly high vertical load of about 10 MN. On the other hand the horizontal forces could be kept low in staying the piers.
- To have low expenditure for the joints, the compression forces had to be brought down on contact through the joints. To bring tension forces through the joints a few bolts should only be necessary.
- For the economic application of the forms, the dimensions of the precast elements should be the same most of the time. To allow different vertical loads the adaptation had to be done by additional reinforcement.

The best solution, to fulfill all the requirements, was an auxiliary pier with four precast concrete struts at the edges with a steel bracing in­between against buckling and wind loads.
For different heights we developed two different types.

Type A for heights up to 20 m.
The four concrete struts at the edges are inclined longitudinal like an A-shaped trestle. So this type of pier needs no staying and brings the horizontal forces, which are about 4% of the vertical load, direct to the foundations. Some structural steel is used for transverse bracing against buckling and for stabilization during construction. Horizontal forces in transverse direction are led to the superstructure by side guides at the top of the pier.

Fig. 1 shows a typical application of type A.

Type B is the common type, developed for heights from 20 m up to about 60 m. This type of pier consists of four vertical precast concrete struts at the edge and between the struts the bracing is done with structural steel. The temporary pier is stayed in longitudinal direction to the adjacent final piers or to their foundations. In transverse direction the horizontal force is led to the superstructure with side guides like type A.

Fig. 2 shows type B.

As this type of auxiliary pier is a typical example of a demountable concrete structure and also that type of pier which was used most of the time for incremental launched bridges, I want to describe it in detail.
Design and calculation of the temporary pier

The research showed that a stayed pier must be the most economic solution, but due to the elasticity of the staying bars the pier, too, should have a certain elasticity in longitudinal direction. One solution with longitudinal concrete plates showed for example that most of the horizontal forces had to be carried by the pier. The concrete plate, which was as thick as the concrete struts, was nevertheless not able to carry it within allowable stresses. So the chosen steel bracing between the concrete struts proved to be the best solution for a quick and easy mounting and for the necessary elasticity of the construction.

The spacing of the struts in transverse direction was 4.8 m, just so that the webs of the deck can bring the loads direct to the top of the struts. Longitudinal the spacing was 4 m to get enough stability during the time of erection.

It is important always to have equal vertical loads on the four struts. Between the sliding bearing underneath the soffit of the deck and the top of the struts we therefore arranged four hydraulic jacks which we connected and therefore we always had the same pressure and loading.

To avoid rotations, the sliding bearings were connected with steel girders to something like a grid system. To this grid system we fastened the staying bars consisting of up to 6 D 6 W bars with a diameter of 32 mm. We took advantage of the transverse slope on the soffit of the hollow box and of course the same on the top of the sliding bearings to lean the auxiliary pier against the superstructure.

![Grid system on the top of the pier](image)
Two side guides at the lower side of the pier were enough to hold the piers in transverse direction even during heavy storms, because of the horizontal force, resulting from the transverse slope and the big vertical load.

For the dimensioning of the precast struts maximum load during launching was decisive. To get the right value of the buckling forces we did the calculation for ultimate loads. We started in accordance to the German standards with a deflection of 200 mm for the 54 m high pier and in two additional steps the truss system of the pier showed that it was stable at a deflection of about 400 mm. This theoretical case of ultimate loading was used also for the dimensioning of the steel bracing.

Additional to this we had to calculate the free standing pier, for example just before staying, under full wind loads. For the steel bracing the forces under these loads were sometimes a little bit bigger than under the above described load. Also under this load we got tensile forces in the joints of the struts. To avoid more than six Ø 16 bolts there, we prestressed the lower half of the pier with 8 D 8 W bars. With the same bars we fastened the struts to the foundations.

**Detailing of the pier**

The basic element of the pier is a 4 m long precast concrete element with a square cross section of 420 x 420 mm. These rather slender dimension were chosen to get a maximum weight of 2 tons. Up to this weight it was possible to mount the precast elements into all positiones with the normal crane on the site.

On both ends of the element, that is on the top and the bottom, we arranged 15 mm thick steel plates. They permitted the multiple use of the elements without spalling at the joints.

The steel plates flush with the two outer sides and at the two inner sides they jut out 80 mm, to make it possible to bolt the two plates at the joints and to fasten the horizontal stay.

On the inner sides two longitudinal plates have been arranged which are welded to the top plate, to allow the welding of the gusset plates for the bracing. Stud-shear connectors which have to bring all horizontal forces into the precast concrete elements have been welded onto these longitudinal plates as well as on the top plate.

To retain always the same dimensions of the precast elements, the first elements at the foot of the pier needed a heavy reinforcement. The maximum compression force there was about 3,5 MN and therefore we had to use a reinforcement of 12 Ø 28. Bringing the force of this reinforcement to the top plate was only possible using direct contact. The reinforcing bars of Ø 28 were therefore cut with a steel saw to the exact length and stood direct on the top plate without welding. This design permits to have different allowable loads with always the same dimensions of the precast element.
For the bracing we installed two diagonal struts only for tension consisting in general of two angle 65 x 65 bolted to the top and bottom gusset plate and a horizontal strut for compression and tension forces consisting in general of two angle 120 x 80 bolted direct to the top plate. As with the precast elements we designed the bracing for three different load levels and with three different colours we marked the bracing and the accompanying precast struts.

Fabrication and mounting

All steel parts, the top and bottom part of the precast struts as well as the bracing, were manufactured in a specialized workshop. Then the top and bottom part were brought to the factory for the precast element and assembled in the forms together with the reinforcement. To have sufficient contact the top and bottom steel part were pressed against the 8 28 bars. Two elements were cast in one double form in one day. The following transport could be done at low cost, as the total weight of one 55 m height pier is only about 120 metric tons.

Then two specialized labourers mounted 8 precast elements together in one day with the bracing, that is a height of 8 m of the pier. To make such a quick progress possible we designed two wooden assembling platforms which stood on the horizontal bracing stays of the proceeding charge and
could be lifted for the next charge with the crane. Due to the weight we had to devide the platform into two parts. After mounting the last charge of the pier they stayed at the top and could be reused for the dismantling of the pier after finishing the launching of both superstructures.

For the serviceability of the auxiliary pier it was important to have an incompressible contact on the full section of the precast struts at the joints. A neoprene layer for example between the two steel plates would have brought too much displacement at the top of the pier during launching.

The best solution then proved to be an approximately 2 mm thin filler of epoxy resin between the steel plates.

As there was a primer on all steel parts the concrete struts could be separated easily in this layer during dismantling.

Launching of the pier in transverse direction

As our freeway bridges normally have two superstructures constructed and launched one after the other, the auxiliary pier had to be designed so that it could easily be launched transverse about 15 m to support the second superstructure. Launching could be done in 2 or 3 hours instead of dismantling and mounting at the new place which would take some weeks.

The footings of the pier on top of the foundations consisted therefore of two pairs of crossing girders. At the edges they jutted out forming two corbels. Under normal load conditions, these corbels were (Fig. 7) fastened with D & W bars to the foundations. For the transverse launching these bars were removed and with jacks under four corbels the pier was lifted about 20 mm and under the other four corbels special rollers were put. The rollers ran on steel angels cast into narrow foundations which led from one pier position to the other. With hydraulic jacks and D & W bars the total pier could be launched transverse as mentioned above in the time of about two hours. This could only be done on days with low wind. During movement no vibrations at the top of the pier could be watched.
Summing up it can be said that the described piers proved to be a very good solution. Referring to the cost of materials, they were very economic. Mounting, transverse launching and dismantling could be done easily and in a short time. The weight saving design of the pier made low cost for cranes, transport and storage possible and therefore it could be used again successfully.
EXODERMIC DECK - A DEMOUNTABLE INTERNALLY COMPOSITE MODULE

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SUMMARY

A new lightweight modular unfilled steel grid with composite concrete surface has been developed and the first installation is in place on a heavily traveled bridge in New Jersey, U.S.A.

The deck, which has been named "Exodermic", forms a two-way system in which the main bearing bars and the distribution bars of the grating combine with a precast concrete surface to form an internally composite deck. In use, the deck can also be made composite with the supporting structural framing members of the bridge. A major and significant attribute of the Exodermic Bridge Deck is its light weight: 53 lbs. psf in the case of the first installation, with an obvious potential for further weight reduction.
EXODERMIC DECK - A DEMOUNTABLE INTERNALLY COMPOSITE MODULE

The term "Exodermic" is suggested to define a lightweight modular unfilled steel grid deck with composite reinforced concrete surface. The term "Exodermic" is used as an analogy to the structure of the composite deck in that the steel grid is an external skeleton in much the same manner as the skeleton of an insect is external to the body. The term "Exodermic" is hoped will become generally used to describe structures of this nature whether used for bridge decks, possible building walls, or floors in buildings or even more likely, in parking garages.

Steel grids have been in use for at least fifty years and had been used in three configurations; as open grids, partially filled with concrete, or filled completely with concrete. The Exodermic Deck consists of a composite unit in which the reinforced concrete layer is formed on top of the steel grid, and is bonded to the steel in a manner which provides for the transfer of horizontal shear between the two elements of the slab: the reinforced concrete, and the steel grid.

Laboratory testing confirms that the behavior of the Exodermic Deck can be predicted considering it to be a two-way composite structure in which the transformed area of the reinforced concrete forms a "T" beam with both the main bearing bars and the distribution bars of the steel grid.

In addition to full size laboratory testing, over 500 prefabricated Exodermic modules were installed on a major bridge in the United States in which the total weight of the modules was 53 lbs. per square foot (259 kilograms per square meter). We believe that further investigation will demonstrate that a combination of a very thin heavily reinforced layer of concrete can be combined with either steel or aluminum or fiberglass grid to form an extremely lightweight and rugged structural unit which could be used in either the horizontal or vertical position in a building.

My presentation in Rotterdam is to review the events which led to the development of a new type of bridge deck. The story also includes the first use of the new deck in a project which went from the first step, an engineering agreement, to the last, the completion of construction, in only ten months.

I have been a bridge engineer for over 30 years. My early experience was affected by the beginning of the Interstate Highway Program under President Eisenhower, as well as the deferral of new construction caused by WWII. Relatively few rehabilitation projects were being designed during the late fifties and early sixties.

Subsequently, my practice has included a significant proportion of rehabilitation or replacement projects in addition to new bridges.

The next 20 to 30 years, will be remembered as a most challenging period for bridge engineers. We will rebuild or replace at least half of the bridge inventory in the U.S.
The most significant difference between a project involving rehabilitation, and a project to design a new facility, is traffic. After a facility is opened, traffic demand cannot be turned off. It is the need to maintain an acceptable level of service during construction which leads me to conclude that easily, replaceable bridge decks should be a requirement for any bridge project—new, rehabilitation, or replacement, because we know future traffic demand will be greater.

Advantages of any prefabricated modular bridge deck, other than replaceability, under traffic maintenance demands, include better control of manufacture, speed of completion, independence of weather and season for much of the project effort, and localized replacement required by accidental damage.

In thinking about an ideal type of bridge deck, it seems to me that weight is a matter of great importance, particularly in projects where the existing main framing members are to be retained even though some deterioration has occurred. Weight is also significant, of course, where the need to widen or upgrade the live load capacity of an existing bridge is an important goal of the project.

The modular deck which I developed consists of several components, each of which has been in use on bridges for from 25 to 80 years. All of the steel and concrete is stress-carrying, no material gets a free ride. The title of my paper describes the deck: a two-way reinforced thin slab which is composite with a steel grating; all main bearing bars and distribution bars are the stems of miniature "T" beams.

Testing at Lehigh University in Bethlehem, Pennsylvania confirms the accuracy of analysis of stress distribution based on AASHO rules for composite design. Data for wheel load distribution to main bearing bars indicate some liberalization of the AASHO rules for design of composite grid decks may be appropriate. Stresses in main bearing bars directly under the hydraulic actuators shown in photo 3 are, in the case of this three-inch deck, less than double the bearing bars located 21 inches each side of the load centerline.

In addition to static loading, testing of the experimental modules at Lehigh University for resistance to fatigue damage has been completed. The first panel, with a two inch thick, mesh reinforced surface was examined after 2,000,000 cycles of loading with HS20 plus 30 percent wheel loads (20.8k), no damage of any significance was observed. The second panel, with a two-way bar reinforcement mat in a three inch thick slab, has been tested to 3,300,000 cycles. The conclusion reached by the Lehigh tests is that the Exodermic modules have an unlimited fatigue life when designed and installed as specified.

At the time I discussed the proposed test program with Professors Daniels and Slutter of Lehigh University, they recommended we provide modules which had been kicked about a bit and had been in service if possible.

After fabrication, and while the modules were in place in a temporary roadway, engineers from the staff of the New Jersey Highway Authority, one of our clients, stopped by the field site to see the experimental modules. During their visit, random truck traffic passing over the modules impressed them so much that we were requested to utilize this type of construction on a major bridge project.
The photographs illustrate the following steps: manufacture of the experimental modules; the field "aging" site; the test setup of the first module at Lehigh University; production of 500 modules for the Driscoll Bridge and the work at the bridge.

Photo 1. The first test module being lifted off the floor. Both of the test modules were cast upside down and we have since found that other procedures are feasible which provide us with a satisfactory reinforced concrete surface and exclude any concrete from the grating interstices.

Photo 2. Field installation—the purpose of which was to subject both panels to random traffic, most of it trucks, for a period of over three months. The panel on the left has a three-inch reinforced concrete surface and on the right, the surface is two inches thick and is reinforced with a welded wire mesh.
Photo 3. The test setup at Lehigh University. This is the two-inch concrete-surfaced test module which is supported transversely on three steel stringers sitting on the floor. The two hydraulic actuators are, therefore, creating a two-span continuous loading of the main bearing bars which run longitudinally in the same direction as the long dimension of the pad under the hydraulic actuators.

Photo 4. An elevation of the Driscoll Bridge which is approximately 4,400 feet long and presently carries 11 lanes of traffic over the Raritan River. This bridge is the connection between the Jersey Shore and the mainland for the Garden State Parkway.
Photo 5. One of the production panels for the Driscoll Bridge being coated with epoxy after sand blasting to white metal.

Photo 6. Driscoll Bridge panels being stored after the epoxy had cured.
Photo 7. Production arrangement for adding the concrete surface to the deck modules for the Driscoll Bridge. The panels were cast right side up by being first placed in molding sand which completely fills the interstices of the grating. The workmen are completing the installation of the block outs so that concrete will not fill in areas which will be located above steel stringer flanges.

Photo 8. Finishing the concrete for one of the Driscoll Bridge modules. The surface was textured to receive waterproofing membrane and then a one-inch renewable asphaltic wearing surface.
Photo 9. A view looking south on the Driscoll Bridge showing the partial removal of the existing median and the provision for protection of traffic in the form of temporary precast concrete dividers.

Photo 10. Part of the framing reconstruction for the Driscoll Bridge. The work shown is the installation of a new floor beam bracket. The new joint between the two bridges will be located along the right side of the opening in the roadway.
Photo 11. Placement of the first module on the Driscoll Bridge. The entire 30,000-square-foot installation is complete; the bridge was opened to 12 lanes of traffic on June 13, 1984.

The theme of this conference is demountable structures. We have been advised that an inexpensive trace rod inserted into the top of the welded studs and left flush with the concrete modules would act as a satisfactory locator for future removal. The trace rod would serve as the center for a core drill which, by removing an annular area of concrete around each stud would, after cutting of joints, permit the removal for either reuse or replacement of the modules.
SEMI-DEMOUNTABLE MICROWAVE RADIO RELAY TOWER AT HOORN, THE NETHERLANDS

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SUMMARY

This paper describes a very slender microwave radio relay tower that can claim to be semi-demountable. The shaft was assembled from 20 precast concrete elements, joined together by means of prestressed Dywidag tendons. The tendon ducts in the elements were not grouted.

The functional requirements, the structural considerations, the design and its detailing, the foundation construction method, the precasting of the elements, the erection of the tower and its possible future dismantling are dealt with in this paper.
INTRODUCTION

In the past 30 years the section Construction of the Rijksgebouwendienst (Government Buildings Agency) has so far designed and built well over 30 towers. The greater part of these towers is intended for the PTT and they are micro wave radio relay, radio and television towers. They have been put down scattered over the whole of the Netherlands and often form outstanding points of recognition in the flat landscape.

Almost all concrete shafts of these towers have been constructed according to the sliding method. The height of the shafts varied from 35 to 140 m. With a view to functional as well as structural requirements the slenderness ratio (l/h) of the shafts varies from 8 to 15.

The tower described in this paper is the last link in a relay system. The design of this tower and the experiences from the last sliding object have resulted into a different structure of the shaft. Twenty prefabricated elements, the one stacked on top of the other, prestressed with Dywidag tendons, constitute the concrete shaft. So as to enable a later demounting as well as re-use of the elements and prestressing tendons, it has been decided not to inject the prestressing tubes. As there are so-called "wet" joints between the elements that constitute the tower, it must be considered semi-demountable.

The functional requirements, the constructive considerations, the project and its detailing, the foundation, the prefabrication of the elements, the mounting and demounting of the tower, are dealt with in this paper.

FUNCTIONAL REQUIREMENTS

The user had formulated the following items in the programme of requirements:
- the raising of a micro wave radio relay tower with a small building in which the equipment will be housed;
- the mounting of two antenna-supporting frames at 65 m above the ground level;
- the constructing of the tower in such a way that only low maintenance costs will be incurred;
- at a 140 km/hour wind speed the angular displacement at the location of the adjusted antennae structure must not exceed 0.6°;
- providing an outside platform for inspection and maintenance of the antennae;
- to arrange for the mounting of a wind generator so that the tower will be provided with a means to generate its own energy;
- to realize the project in a short period of construction.

CONSTRUCTIVE CONSIDERATIONS

The tower constructed replaced a previously demolished steel lattice tower, which did not meet the requirements of rigidity. Therefore, in designing the new structure, a steel lattice tower had not been taken into consideration.
The functional requirements of the tower are mainly: a rigid supporting structure for the antennae and the latter must be easily accessible for inspection and maintenance purposes. Consequently, the dimensions of the tower can remain restricted. The advisory architect wanted, from an esthetical point of view, a tower to be constructed as slender as possible.

Two forms of construction for the concrete shaft were compared.

A. The sliding of the shaft.
   In that case the shaft diameter is greater than at B. In case A the diameter in upward direction can remain the same or decrease (taper). Experience has taught that budgets for small scale sliding projects can be met only with difficulty.

B. The assembly of the shaft of prefabricated elements.
   Though experience with such a construction method has so far been only limited, nevertheless as reasonable insight could be gained into the way in which it had to be carried out. Budgetting the costs involved in this method was not difficult.

When constructive methods A and B were compared, it appeared that B clearly had a number of advantages over A, viz:
- a higher concrete quality than B 22.5 is possible;
- a greater concrete density and a smoother outside concrete surface are obtained. These are nowadays important advantages with a view to durability of the structure. In particular when the tower, as in this case, is not protected by a coat of paint;
- the higher concrete quality and the prestressing will result in a more slender tower (l/h 20);
- the budget can be drafted more accurately;
- the costs do not exceed those of the sliding method.

Considering the above advantages, it was decided to assemble the tower of prefabricated elements.

The wall thickness of the elements is determined by the depth required for the various recesses and the location of the prestressing tendons.

The thickness of the concrete beyond the recesses should be sufficient to ensure a good watertightness at the joint. Moreover, when the concrete layer is too thin, there is a risk of pieces of concrete chipping off during dismantling. Therefore, the wall thickness has been established at 300 mm, see Fig. 1.

It was considered how the joint between the elements could be provided best. Various ways were taken into consideration, i.a. counterjigs. The following method has been selected. The elements are constructed without special provisions such as cams or dowels.

A joint of 22 mm (theoretical measure) between the elements is grouted with a fast-hardening grout. The latter is protected by a sealed joint.

![Fig. 1 Detail of recess with intermediate anchoring and with coupling sleeve](image-url)
By not injecting the prestressing tubes, the following advantages can be obtained:
- less costs upon demounting the tower;
- re-use of elements and prestressing tendons is possible after demounting.

Then the prestressing tendons and the anchorages will have to be provided with an adequate anti-corrosive coating.

**PROJECT AND FURTHER DETAILING**

The project is constituted by a concrete tower of 61.25 m height. The tower has been assembled of 20 circular prefabricated elements whose dimensions are:
- external diameter 3.00 m;
- wall thickness 0.30 m;
- height 3.04 m.

The weight of each element is abt. 190 kN. The tower is mounted centrally on a bearing plate and a small building in which equipment will be housed is projected against the tower. A thermally zincplated and painted steel structure of abt. 10.00 m with an octagonal outer platform constitutes the top of the tower. To provide the tower with a means to generate its own energy, a wind generator can be placed on top of the steel structure, see Fig. 2.

Apart from the architect's wish for a tower as slender as possible, the joints between the elements had to be clearly visible, see Fig. 1. Another wish was the attainment of a uniform light colour of the elements. This can be obtained by adding titanium dioxide to the concrete mixture. Adding 3% titanium dioxide per weight to the total amount of cement yielded a good result. This addition has in particular an adequate effect when the surface is wet, e.g. after it has rained. The concrete surface remains clearly lighter in colour than that of blank concrete.

The tower was calculated according to the Netherlands regulations as to wind load, concrete stress and prestress. The admissible angular displacement of 0.6° is amply met. The eigenfrequency of the tower amounts to 0.46 Hz. Some known phenomena of wind load on slender structures were checked.

The prestress is constituted by smooth Dywidag tendons Ø 32 mm, quality FeP 1060/1230. Fifteen tendons are required at the base of the tower. Because of the parabolic course of the wind moment the number of tendons can be decreased in upward direction. On the 5th element five tendons are prestressed and another five on the 9th element. The remaining five tendons continue up to and including the 19th element.

The 20th element has been joined to the 19th element with 5 "short" tendons. At various levels there are intermediate anchorages and couplings, see Fig. 3.
The tendons run through tubes Ø 65 mm, which are obtained in the elements by means of impregnated cardboard sleeves. The non-injected tendons have been protected as follows against corrosion:
- the tendons were cleaned and provided with a layer of epoxy-coal-tar;
- over this layer a 1.5 mm thick layer of polyethylene was applied;
- the ends of the tendons are provided with rolled screw threads. Over the latter temporary shrink sleeves were shrunk.

The prestress level in the tendons decreased as the height increases to prevent previously stressed tendons from being pulled loose.

The reinforcement of the elements consists of an inner and outer mesh of tendons Ø 12-150 mm. The quality is FeB 400. The outer side of the reinforcement is covered by 35 mm thick concrete, whereas for the inner side 30 mm is used. In the tower after abt. every 6.0 m 10 resting platforms were projected, which are mutually connected by vertical ladders. This thermally zincplated steel structure is applied into the elements, before mounting takes place, see Fig. 9.

All elements have 3 recesses for jacks by means of which the elements are exactly adjusted to height and set plumb.

In 9 elements recesses have been provided for either the end or the intermediate anchorages as well as for the coupling sleeves.

The steel structure on top of the tower consists of a conical transition piece having a height of abt. 1,500 mm, and on the top of the latter piece abolted steel tube Ø 1,219 mm. The tube has a conical end on which later the mast of the wind generator can be placed. An octagonal outer platform is projected at abt. 1/3 of the height of the steel tube, see Fig. 2. Via an inner ladder and a small door in the tube the outer platform is accessible. To restrict maintenance to a minimum the entire steel structure has been thermally zincplated and thereupon painted.

After the steel structure has been assembled on the ground, it will be hoisted and placed on the top of the concrete shaft at one go.
The foundation of the tower consists of driven piles on top of them a cast bearing plate. A Hitachi K180 pile driving machine drove the twenty prestressed concrete piles - Ø 0.40x0.40 m, length 18.5 m, bearing capacity 850 kN - at depth without any problems. Apart from strut piles, there were also stay piles 6:1 and 10:1 in the design of the pile foundation. The dimensions of the bearing plate are 9.0x9.0 m at thickness of 1.40 m. At the location of the tower the thickness is 1.60 m. Fifteen Dywidag tendons, provided with anchorage bells and coupling sleeves, fixed in an adjusting frame, have been cast into the bearing plate, see Figs. 4 and 5. The coupling sleeves provided with a shrink sleeve protruded for half their length above the plate. After the plate has been cast, the temporary auxiliary tendons and the top frame were removed and the coupling sleeves temporarily sealed.

PREFABRICATION OF THE ELEMENTS

At "Hurks Beton", Veldhoven, the Netherlands, the 20 elements were made. As there was insufficient hoisting height in the production-hall, the elements were produced outside. They were cast vertically. For the production of the elements a steel mould was made consisting of a rigid base frame and an inner and outer formwork. On a piece of inner formwork welded to the base frame a vertical scratch line had been made. The impression of this scratch line visible on the concrete surface was used upon mounting so as to place the elements exactly on top of each other. On the outer formwork 6 formwork vibrators were provided, see Fig. 6.

The production of the elements proceeded as follows:

The reinforcement of the elements was prefabricated indoors into complete cages. The reinforcement cage could be put over the closed inner formwork. Steel tubes were mounted, provided with the impregnated cardboard sleeves for the prestressing tubes. Hereafter the outer formwork could be placed and closed.

The casting and compacting of an element was done late in the afternoon. The mould was covered by a tarpaulin and a steam treatment accelerated the hardening. The following morning the formwork was dismantled and the element was transported to the storage yard, see Fig. 7.
- 325 kg of portland cement (class C)/m³
- water cement factor 0.40
- sand (0 - 5 mm) 670 kg/m³
- gravel (4 - 14 mm) 805 kg/m³
- gravel (14 - 28 mm) 609 kg/m³
- 0.3 kg of titanium dioxide/m³

With this mixture, after 28 days an average strength of 63.1 N/mm² was attained with a standard deviation of 4.8 N/mm². With these values the quality of B 52.5 was amply gained. The steel structures, such as ladders, definite resting platforms with grids were mounted into the elements during the storage period. Provisions were also made for temporary auxiliary platforms. The transport of the elements to the building site took place by road by means of deep loaders, which each time conveyed two elements.

MOUNTING

The mounting of the tower was carried out in two phases. The first three elements were mounted, whereupon the skeleton structure of the small building was realized. Thereafter the other 17 elements were mounted, simultaneously with the completion of the construction of the small building. When the first three elements were being mounted – with a 60 ton hydraulic mobile crane – experience was gained, which was used at the later mounting of the 17 elements.

The mounting of the first three elements proceeded as follows:
- hoisting and placing of the first element, see Fig. 8;
- building up a Kwikform scaffolding around and to 3 m above the element;
- with 3 jacks exactly setting the element plumb and adjusting it to height, see Fig. 9;
- hoisting and placing the second element. Adjusting of the element took place by means of the scratch line previously stated. Four sets of coupled U-profiles guided and locked the element;
- building up the scaffolding etc.;
- after the 3rd element had been placed, the 3 elements were jacked up to 150 mm over the bearing plate, by means of the lower 3 jacks;
- mounting of the prestressing tendons and "screwing" them into the coupling sleeves;
- shrinking the shrink sleeves around the coupling sleeves;
- lowering the 3 elements and checking their vertical position;
- grouting the 3 joints with a fast-hardening grout and allowing the grout to harden;
- the following day expanding the jacks and removing them;
- prestressing 5 tendons, providing the other tendons with coupling sleeves and mounting temporary shrink sleeves, see Fig. 10;
- pulling down the scaffolding.

After the skeleton structure had been completed of the small building situated against the tower, mounting could be started again. A 200 ton hydraulic crane did all the hoisting up to and including the 13th element. A 125 ton mobile lattice crane was put in for the last 7 elements and the steel structure (that had to be put on top of the tower). Mounting of the 17 elements was carried out in sections of 2, 4, 3, 4, 3 and 1 element(s) respectively. See mounting scheme of Fig. 11.

The 2nd mounting phase was performed as follows:
- the providing of a protective guard over the roof of the small building. During hoisting of the elements the activities regarding completion of the construction of the small building had to be interrupted for a while;
- building up of the scaffolding;
- hoisting and placing of the 4th element;
- building up the scaffolding;
- exactly setting the element plumb and adjusting it to height by means of 3 jacks. When the elements were being produced, it appeared that their respective heights had a deviation of ± 3 mm. To avoid problems with the prestressing tendons supplied at exact length, each time the elements were put at a height of 3,062 mm. Exactly setting plumb was done by means of a long spirit level. During mounting, the vertical position of the elements that had already been mounted was a few times checked with a theodolite.

Fig. 9 Jacking equipment for adjustment of the elements

Fig. 10 Shrinking a temporary shrink sleeve around a coupling sleeve
A slight deviation of 7 mm, about half-way, was corrected during the further phases of mounting:
- hoisting and placing of the 5th element, see Figs. 12 and 13;
- putting in the prestressing tendons and "screwing" them into the coupling sleeves;
- grouting 2 joints and allowing the grout to harden;
- the following day expanding and removing jacks;
- prestressing 5 tendons and completing the anchorages.

According to the method stated above, the sections of elements were mounted, and the tendons were either prestressed or coupled, as the case might be. The last element was coupled to the 19th element with 5 tendons, see Fig. 14. In the meantime the steel structure for the superstructure had been conveyed and assembled. The complete structure (85 kN) was hoisted and placed on the tower, adjusted to height and then the joint was grouted, see Figs. 15 and 16.
The team consisted of a building supervisor, a foreman, a crane operator, and a jack or prestressing expert, as the case may be, and two scaffolding builders. On account of the good weather conditions — not much wind — the mounting of the 17 elements inclusive the steel superstructure could take place in 12 working days. Only the mounting of element 19 was delayed by 1.5 day owing to strong wind. On a Friday evening, after the wind had decreased, element 19 was mounted after all. In a later phase of this project third parties hoisted the 2 large antennae and fixed them to the steel structure, see Fig. 17.
DEMOUNTING

A future demounting of the tower can proceed as follows:
- building up a scaffolding around the tower by means of a crane;
- demounting the wind generator with mast and convey them;
- milling, both the inside and outside, of the joint between steel structure and 20th element;
- pulling apart the steel structure from the 20th element by means of 3 flat jacks and convey this structure;
- milling, both the inside and the outside, of the respective joints between the elements;
- pulling apart each element with 3 jacks to be placed in the jack recesses, and convey it;
- pulling down the scaffolding for the height of one element;
- in case of anchorages, prestressing the tendons to such an extent that the nut can be unscrewed and at couplings "unscrewing" the tendons;
- it was estimated that this demounting inclusive building up of the scaffolding can be carried in two weeks.

If one wants to re-use the elements, at first the remnants of the joint at their upper and lower ends will have to be removed. The prestressing tendons can be re-used, after having been checked on the soundness of the protection and in particular after the ends provided with screw-thread have been checked.
A research group "B.B.M." has developed an industrially prefabricated module of glass-fibre reinforced concrete. The module is based on the container dimensions of 2,40 m x 6,00 m, and the open modular system offers a multitude of application possibilities (field hospitals, motels, school buildings, emergency accommodation in disaster areas, houses, bungalows, offices, laboratories etc.).

The structural characteristics are very favourable. The insulation value of the wall (0,55 W/m²°C) is achieved with a sandwich construction of GRC and insulation core.

The thermal inertia is high so that living in the module is very agreeable, even in hot weather.

There is no danger of corrosion. These buildings can also be used in tropical countries with a high degree of humidity.

Modules can be transported as completely finished units or as separate parts thereof.
Prefab Housing Modules of glass-fibre reinforced concrete

Architect: L. DELHAISE     Dr. ir. G.F. HUYGHE

There is a need for complete prefab modules that can be shipped to underdeveloped countries to build in a fast way field-hospitals, dispensaries, offices, small laboratories, villages for construction-site-personnel etc.

If these prefab modules are not too expensive then they can be used for motels, schoolbuildings and bungalows and if they are attractive it is possible to find a market also in Europe.

A research group in Belgium, composed by an architect, a precaster and the Regional Investment Company for Flanders, has developed such a module.

It is an industrialy produced module of which the outside skin is a layer of glass-fibre reinforced concrete fixed to a metal frame covered at the inside of the module by plaster board and between the two layers an insulation.

The module is based on the container measure 2,40 x 6,00 m and the open modular system offers a multitude of applications, then leaving to the architect the freedom to develop in a flexible way his project in cooperation with the research group.

1. Technical data

The module itself is composed by different modulated pieces: (fig. 1)

- A roof element in one piece without joints, outside skin in glass fibre reinforced concrete of minimum 10 mm thickness which is in principle watertight. A lateral groove is foreseen to place a rubber joint if other modules are placed one against another.

- The floor element in one piece has its outside skin in glass fibre reinforced concrete with ribs for transmitting the forces to four points which are in contact with the foundation-blocs on which the modules are placed. After that the insulation is placed, a wooden flooring is covering the element.

- Four corner elements bear the roof and offer in this way a flexibility if different modules are combined.

- The other panels which fill up the walls are modulated on 1,20 mm, the height is 2,40 m or 1,20 m.
The structural physical characteristics are very favourable. As to the insulation value, a thermal conductivity value of the wall of 0.55 W/m² K is reached. This value can be increased or decreased according to the wishes of the client.

The thermal inertia is high so that living in the module is very agreeable, even by hot weather. These properties are highly favourable to the comfort and the heating and cooling costs.

The stability results from the complex forms of the module. The electric, sanitary and central heating conducts are built in and can be adapted to the requirements of the project.

2. Applications

A module consists of a floor-element, a roof-element, four corners and filling up elements. These elements can be assembled in any given composition, for applications in the country or abroad.

The modules can be used for the construction of permanent and mobile buildings, such as:

- dispensaries, hospitals and field-hospitals
- housing and additional services
- hotels, motels and bungalows
- emergency settlements at catastrophes
- schoolbuildings, after being used for several years, can be put up elsewhere
- offices and laboratories which have to be built out as a module with a permanent character or which are dismountable.

Fig 1 is an example of a building composed of 4 modules. These 4 modules form a housing accommodation with a living room and two single bedrooms with each a bathroom and toilet.

Figure 2 shows a draft of a village for construction site personnel in Irak.

Especially the medical world is interested in using the module for small hospitals in Europe and in developing countries. Figure 3 shows a field hospital.

3. Transport

Transport can be done as well in separated parts as in closed parts and completely finished modules. This depends upon the distances and the degree of finish.
A field hospital is transported in completely finished and equipped modules on trailers.

Weight of a module without equipment: ± 5 tons.

Large quantities can be shipped in separated elements or as closed modules.

4. Other applications and photographs present during the symposium will illustrate the system.
Prototype - 4 modules

Fig. 1
Availability of reasonably-priced parking in many urban centers with large automobile concentrations is at a premium. This paper describes the development of a low-cost precast concrete parking structure system with inherent demountability and explores both the design philosophy and the economic viability of such a structure. The possibility of an interim land use is offered, thereby not limiting the potential for future planning flexibility.
Worldwide, most urban centers are faced with problems of controlling the private automobile. Provision of adequate parking is a key to a system of control. In North America the high ratio of automobile ownership has accentuated the need for a well-conceived parking strategy.

Almost without exception, availability of land or the political and economic will to commit long-term land use for such a utilitarian purpose have been the major stumbling blocks towards the provision of parking facilities. Authorities have compromised good planning by accepting development amenities or payment in lieu of parking or have simply ignored the problem. Private enterprise has promoted these relaxations due to the high cost.

During the past 25 years, the advent of the large North American car and a public perception demanding ease of parking accentuated the cost of parking solutions, but now, with the virtual demise of the large car and a general public that feels more comfortable using structured parking, it is necessary to reanalyze the basic parameters.

1. PLANNING OPTIONS

From a planning perspective, "demountability" offers several benefits towards better control: firstly, as an interim land use, a demountable parking structure does not compromise the future ultimate and best use of land; secondly, to be able to relocate in response to changing and shifting demand; and, thirdly, as a mechanism to make parking a catalyst towards promoting early development rather than a restricting burden [i.e., higher cost of permanent parking can be deferred to later stages of a multi-phase development].

2. ECONOMIES

To achieve the least expensive solution an approach towards "ease of parking" must be committed. This structure represents a basic approach with level parking decks, two-way traffic circulation, 90° parking and minimum dimensions for aisles and stalls.

The premise is that the very existence and proximity of parking to a desired destination is more important than the size of stall.

The trend towards increasing numbers of smaller cars had lead authorities to allow up to 30% small-car stalls. Predictions are this will increase, and in anticipation of this increase, a 40% small-car ratio has been used.

Dimensionally, this structure incorporates 2370 x 4900 small car stalls with 2400 x 5800 larger car stalls and 7900 aisles. The clear span of 18.6 m enables flexibility in re-marking as stall sizes change in the future.

The efficiency ratio of structured area per car under these conditions is maximized at 27 m².
True economy in precasting techniques lies in simplification and repetition of elements. This solution reflects a rigid discipline in this regard. Redundancy of members was sought to be eliminated. The optimization of long-line prestressing facilities and flat bed techniques, maximum plant work and minimum field work and integration of architectural and structural functions were achieved.

The economies have resulted in a cost per stall of $5,000.00 CDN, a figure well below that for other comparable structures. This level of investment enables recovery on a marketable user-pay basis. Considering the high cost of land, it also creates structured parking at a comparable or lesser cost than developed surface parking [see Appendix "A"]. The structure incorporates maximum speed of erection with all-weather construction capability. This results in minimization of business interruption in redevelopment of a site.

Should the structure be relocated, the cost-effectiveness is further enhanced, since relocation costs are estimated at 60% of original cost. In addition, a demountable structure has significant retained value for future resale.

It is important to note that a tremendous opportunity exists for the precast concrete industry to lease these structures to private and public clients. The low cost makes it feasible to repay the investment on a reasonable user-pay basis. This, in turn, creates an expense to the client rather than a capital investment.

Fig. 1 Artist's Rendition of Structure
The basic structure comprises reinforced concrete load-bearing wall panels and untopped prestressed double-tee floor members.

The wall panel system includes a typical exterior and interior unit. The exterior spandrel units act simultaneously as vertical load-bearing columns, spandrel beams supporting the floors, lateral stability frames, vehicle guards and architectural facade.

The exterior panel has absolute minimum structural sections in order to obtain maximum openings to meet requirements for natural ventilation of the structure. Panels support three double-tee members and allow construction in modules of 7.2 m in length. The resulting solidity ratio is sufficiently low to allow end walls to be solid if necessary so that the structure may be located immediately adjacent to other buildings.

Interior panels are solid with pockets for support of floor members and are designed primarily for gravity loads only.

Ramp walls are the major lateral load resisting elements. They are basically solid panels with openings to provide visibility for access to and from the ramp.

![Fig. 2 Typical Floor Plan](image)

Standard pretensioned 750 mm deep double-tees are technically efficient members. They provide maximum span for minimum structural weight and depth. Produced in long-line pretensioning beds, they are a cost-effective product. An 80 mm deck and flange optimize shear and bending forces with effective reinforcing cover for an untopped deck.

Stairwells are modules of one double-tee width by one-storey height. Each module contains finished stairs, handrails and glazing. Glazing has been maximized to provide security control. Location is flexible.

The components allow for a "building block" approach to construction in modules of 7.2 m in length and up to six levels in height. The six-level limit on height was established for efficiency in structure and on the basis that two-way ramp systems are not convenient parking solutions above this height. Structures can be extended or reduced in length and height in future.
In detailing of all individual components of the system, maximum standardization is incorporated so that components are as interchangeable as possible.

Repitition in connections has also been achieved with the actual number of different connections kept to a minimum. Connections are designed to facilitate production in large quantities.

Corrosion control is firstly provided by positive slope of 2½ times prestress camber to evaporation/drainage troughs at the center panel. These troughs are sized to allow washing down of the structure and ease of cleaning by hand methods. All connections are galvanized. Epoxy-coated reinforcing is incorporated in critical areas of the double-tee floor members and a proprietary, proven joint detail between double-tee flanges ensures watertightness.

Design variables are wind and earthquake loads, these being a function of the location of the structure. The standard developed design has a capability of resisting wind load for any location in Canada. For optimum economy, design for earthquake loading is zone specific due to the magnitude and variation of loads.

Aesthetics of the structure has not been disregarded. To be truly functional, the structure is capable of integrating with adjacent quality developments.
4. DEMOUNTABILITY

Demountability, and, in fact, constructability, are economically achieved by the non-welded proprietary connections. These connections must provide simplicity, self-alignment and durability. Some of these parameters tend to be mutually exclusive, and invariably, adjustment is required in the direction in which final forces need to be resisted.

Transfer of lateral stability forces and integrity against progressive collapse in this structure are wholly dependent upon the connections since no continuous toppings or diaphragms are used.

5. CONCLUSIONS

This paper has outlined a demountable parking structure concept which has been fully-developed with respect to design and feasibility. Its inherent capabilities have been discussed, and seem to present a unique opportunity for precast manufacturers to develop both new markets and new marketing approaches.

The structure is a long-life solution incorporating flexibility and state-of-the-art corrosion protection. Predictable life is equal to or greater than that of a permanent structure.

A demountable parking structure offers the opportunity to obtain maximum interim use of land while retaining long-term land values and planning flexibility for re-use of land at a future point in time.

The favourable capital cost has been examined; any future increases in land values or reduction in available land will enhance the economic viability of a demountable parking structure.

Innovative financing and, in particular, leasing financing are rationalized on a user-pay basis.

It is acknowledged that the overall concept is not unique. Examples of demountable parking structures exist in the United States and other areas of the world. It is hoped, however, that the possibilities inherent in the concept have been approached in a fresh light to result in a unique and practical system making use of precast concrete.

6. ACKNOWLEDGEMENTS

Licensing: Genstar Structures Limited
Calgary, Alberta, Canada

Structural Design Consultants: Simpson Lester Goodrich Partnership
Calgary, Alberta, Canada.
APPENDIX "A"

APPROXIMATE ECONOMIES FOR CALGARY, ALBERTA, CANADA:

Area per stall for demountable parking structure - 27 m².
Cost per stall [300 or more] excluding land - $5,000.00.
Cost per m² - $185.00.

For a similarly efficient on-grade parking lot, the cost per stall excluding land - $1,500.00 to $2,000.00.
Cost per m² - $55.00 to $75.00.

Land costs are not less than - $250.00/m².
Therefore, on-grade parking costs a minimum of - $305.00/m².
Structured parking with 3 supported levels costs - $248.00/m².

Land costs must be less than $193.00/m² for on-grade parking to be less expensive than structured parking; then, availability of land becomes a governing criteria.