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Report 6-81-23

Bolted beam to column knee connections with haunched beams. Tests and computations

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CONTENTS

1. INTRODUCTION

Results of 5 tests on bolted beam- to column knee connections with flush endplates and haunched beams are reported herein.
The research is an expansion of the research described in report 6-81-15.
The material properties as well as the main dimensions are the same as described in the latter report.
The aims of these tests were identical to the previous report, viz.:

1. To check a design method of flush-end-plates and stiffened-column flanges, when more than one boltrow is used.
2. To check new ways of stiffening. However, in this case the stiffeners are mainly web-doubler plates used to improve the shear-force capacity of the column-web in knee connections.
3. To show undergraduates several types of yielding mechanisms which develop during loading of the connections until failure.

These tests were also carried out in order:

1. To confirm the design criteria for a haunch without a flange as developed in report 6-81-15.
2. To check the influence of an axial force in beam- and column on the behaviour of the bolted connection.

Based on these testresults and results reported in |5|, |6| and report 6-81-15 |7| a design method has been developed.
The formulae of the method have been tabulated in appendix A5.
An explanation of the formulae is given in report 6-82-7. The computations of the tests are given in appendices A1 and A4.

2. TESTSPECIMENS

A review of the testspecimens with the dimensions is given in figure 1.

Welds

The throat-sizes of all fillet welds were made 4 mm with the exception of the welds shown in table 1.

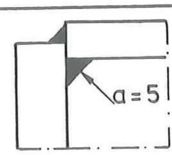
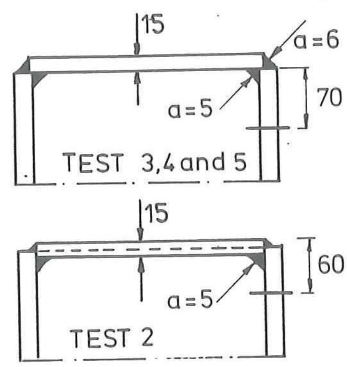
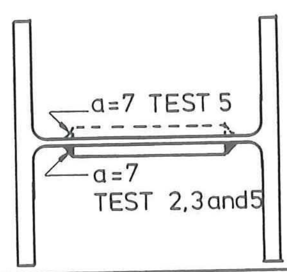
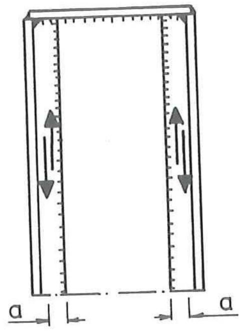
Location of weld	
Inner fillet weld between end plate and beam flange	
fillet welds between end plate and column-flange	
fillet weld between web doubler and column-web.	

Table 1: Locations of fillet welds larger than a = 4 mm

All specimens were knee-connections loaded as shown in figure 2.

Other tests showed that this is possible. However, in these tests the plate was welded as shown in figure 2b of report 6-81-15, thus the space between plate and flange was filled by weld-material. This way of welding appears to be rather expensive in practice. The impression existed that the shear-force capacity of the web-panel may also be increased if this panel is strengthened over a smaller part of the web as shown in figure 3.



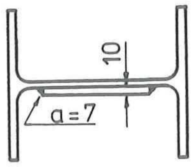


Figure 3: Large shear stresses occur in the parts "a" of the web when the web-doubler covers only the central part of the web.

The advantage is less welding-labour if this way of construction is used. Theoretically this method of stiffening is insufficient because failure occurs at the parts "a" where large shear stresses occur. Despite this phenomenon an improvement was expected.

2.3. Testspecimen 3

This specimen was meant to show the contribution of the end plate on top of the column in comparison with the result of test 1 and served as a reference for test 4 in order to check the influence of the web-doubler. The end plate as well as boltconfiguration and haunch dimensions are similar to those of test 3-21 of report 6-81-15. The bolts were bolts M24 grade 8.8.

3. TEST ARRANGEMENT

The specimens were loaded as shown in figure 4. The test specimen was fixed at support A. Lateral displacements were prevented by hinged supports at points B, C and D.

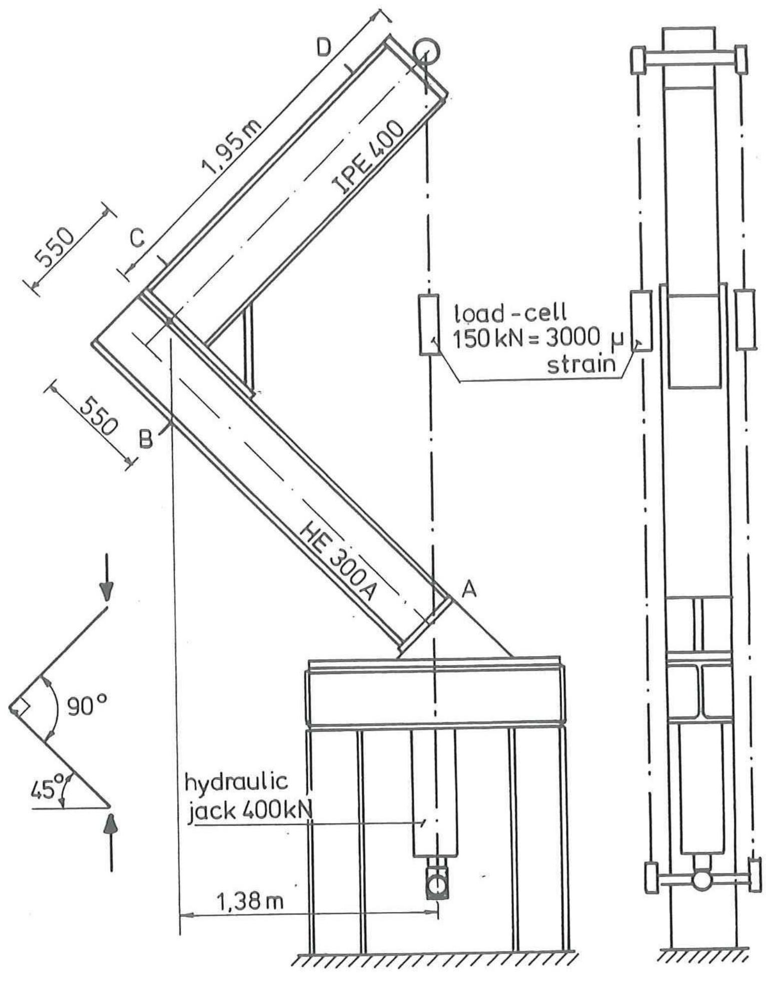


Figure 4: Test arrangement

The loading was executed with a hydraulic jack of 400 kN. The hydraulic pressure was increased with a handpump. The loads were measured with load-cells in series with the hydraulic jack.
The loads were increased with steps. After each step the measurements were carried out.

4. MEASUREMENTS

4.1. Deformations

The deformations were measured with transducers located as shown in figure 5, 6, 7 and 8.

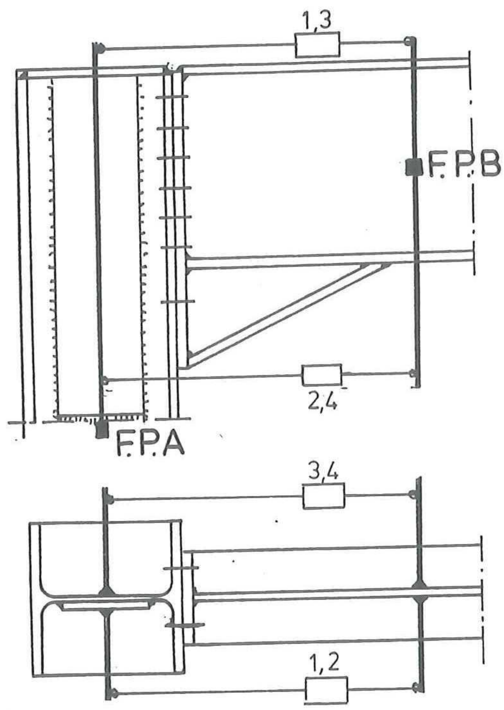


Figure 5: Transducers used for the measurements of the total rotations

With transducers 1-4 the displacements were measured between two bars fixed at points indicated as F.P.A. and F.P.B. in figure 5. The exact locations of these points are given in figure 6.
The results obtained with these transducers are used to compute the total rotation of the connection.
Transducers 5-18 registered the displacements of webs and flanges with respect to the bar fixed in point F.P.A. as shown in the figures 7 and 8.

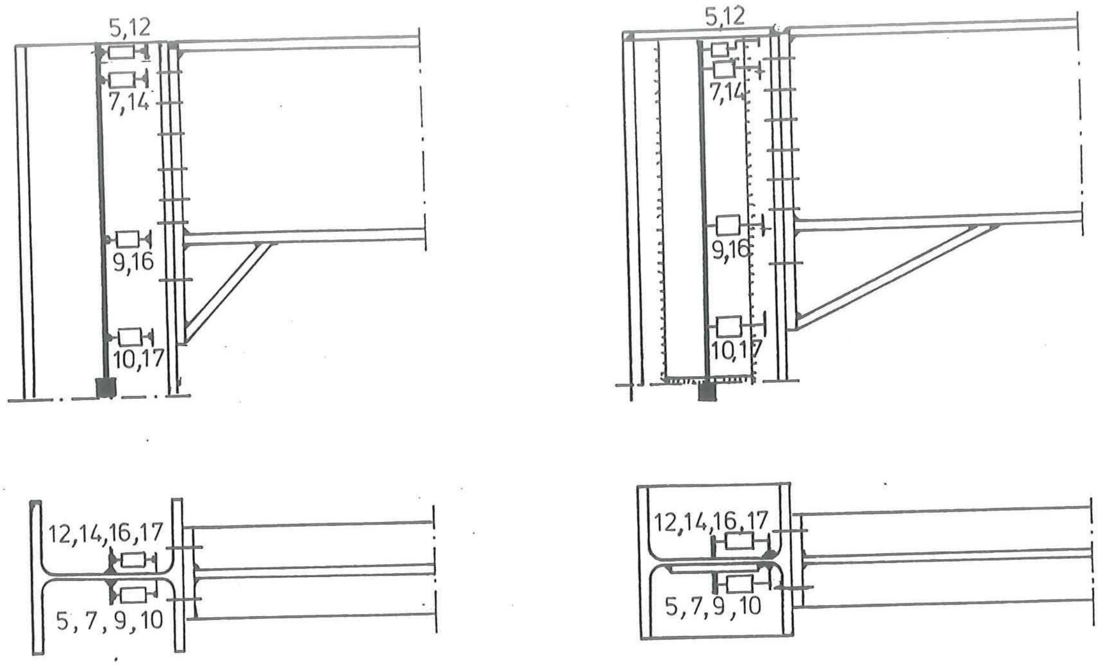


Figure 7: Transducers used for the measurement of the web deformations

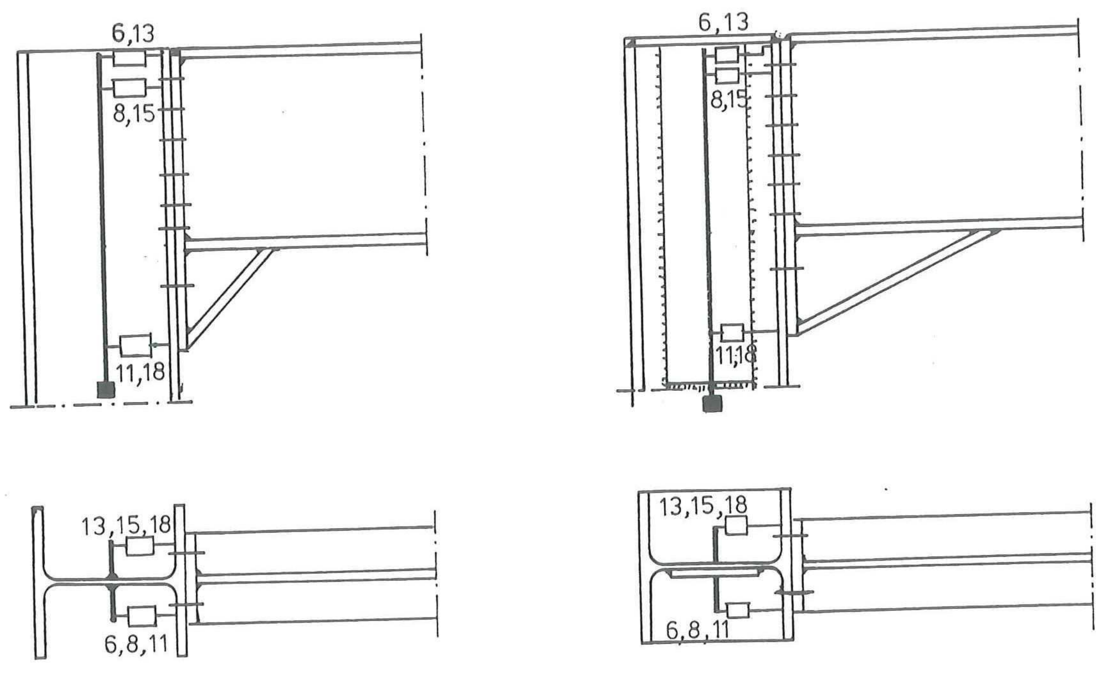


Figure 8: Transducers used for the measurement of the flange deflections.

TEST NUMBER	LIMIT STATE DESIGN LOADS							LEVER ARM	LIMIT STATE DESIGN MOMENTS			ACTUAL ULTIMATE MOMENTS
	Force Notation	Combination of bolt and Flange	Combination of bolt and End plate	Result	Shear Column web	Compression Column web	Reduced values		Column flange and end-plate	Beam web Shear	Beam web Compression	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
		kN	kN	kN	kN	kN	kN	m	kNm	kNm	kNm	kNm
1	F_1	222(12)	273(2)	222			222	0,53	118			
	F_2	92(11)	212(12)	92			92	0,47	43			
	F_3	92	140(11)	92			92	0,41	38			
	F_4	92	140	92			5	0,35	1			
	F_5	92	140	92								
	F_6	no contribution										
	F_c, F_s, F_h				$\boxed{411}$	598	411		$\boxed{200}$	222	258	274
2	F_1	243(2a)	273(2)	243			243	0,53	129			
	F_2	214(12)	212(12)	212			212	0,47	100			
	F_3	92(11)	140(11)	92			92	0,41	38			
	F_4	92	140	92			29	0,35	2			
	F_5	92	140	92			-					
	F_6	no contribution										
	F_c, F_s, F_h				617	$\boxed{598}$	598		$\boxed{283}$	418^{**}	373	278
3	F_1	278(1)	582(2b)				278	0,52	144			
	F_2	175(11)	330(12)				133	0,45	60			
	F_3	92(11)	189(11)					0,38				
	F_4	92	189					0,31				
	F_5	no contribution										
	F_c, F_s, F_h				$\boxed{411}$	598	411		$\boxed{204}$	418^{**}	373	278
4	F_1	278(1)	(2b)				278	0,52	144			
	F_2	175(11)	330(12)				175	0,45	79			
	F_3	92(11)	189(11)				92	0,38	36			
	F_4	92	189				54	0,31	17			
	F_5	no contribution										
	F_c, F_s, F_h				617	$\boxed{598}$	598		$\boxed{276}$	$\boxed{234}$	258	295^{*} $\boxed{336}$
5	F_1	278(1)					278	0,59	164			
	F_2	175(11)					175	0,52	91			
	F_3	92(11)					92	0,45	41			
	F_4	92					73	0,38	28			
	F_5	no contribution										
	F_c, F_s, F_h				$\boxed{617}$	1020	617		$\boxed{324}$	417^{**}	361	412

* prior to stiffening

** M_p reached

Table 2 : Limit state design loads and limit state design moments of tests 1-5

5.1.2. Table 3

Here other design rules are applied than in report 6-81-15. That is why the limit state design moments of the tests described in the latter report are computed again in appendix A4. The results are summarized in table 3.
The newly computed limit state design moments are larger than the former ones as far as the column flanges are concerned. This is caused by the following changes.
In report 6-81-15 it is assumed that the effective length is restricted by the pitch between the boltrows.
Now it is assumed that the first boltrow in the unstiffened part of the flange cause to form a complete yield line mechanism with a corresponding effective length of $b_m = 2m + 0,625\ n'$
When the other boltrows are located within the latter effective length, then their effective length is restricted to the overlap which is equal to the pitch (see figure 10)

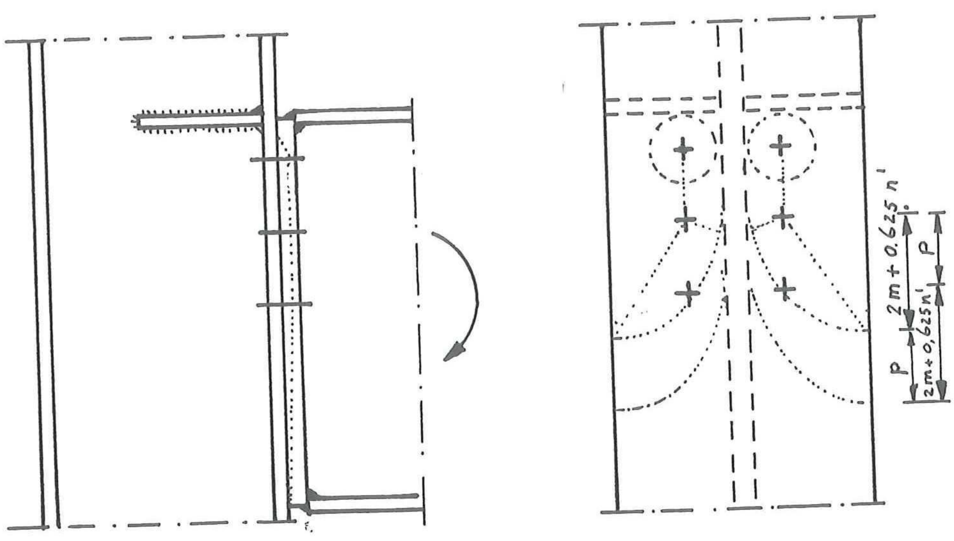


Figure 10 .

This way of design generally results in a larger limit state moment because the largest value of the effective length is assigned to the boltrow with the largest lever arm. A comparison of the computed results and test results should confirm whether this assumption is correct.

On the other hand, the contribution of a boltrow is neglected when the lever arm is smaller than two times the effective length $b_m = 2m + 0{,}625\ n'$ and the column flange (or end plate) yields before the bolts fail.
See the conditions for formula (11) and (12) in table A 5.4

The design criteria for the haunched part of the beam have been converted into some formulae (formulae (27), (28) and (34))
Formula (27) give smaller values than the method used in report 6-81-15.
The reason is that a comparison of the limit state design moments of test 4 and test 1-15; 1-20; 4-18 and 4-21 of report 6-81-15 with the limit state design moments of the other results gave the impression that the effective width of the end plate for shear ought to be restricted to two times the web thickness and not four times.
Furthermore, the term in the denominator of the formula has been simplified.
The forces transferred by the material in the fillet of the beam section are reglected whereas it is supposed that the beam-web completely yields. It is expected that both simplifications neutralize their effects.
Moreover formula (27) may be neglected when it is shown with a more simplified formula that failure due to shear of the beam cannot occur because the length of the haunch is sufficient.

5.2 Moment rotation curves.

In appendix A 2, in figures A 2.1 - A 2.15, the moment rotation curves are given.
Rotation caused by the deformations of various components of the connection are distinguished.
All rotations are averaged values of the measurements on either sides of the testspecimen.

Two solid lines are drawn in the moment rotation curves in which the rotations of web, flanges and the connection (total) have been plotted. The horizontal line indicates the limit design moment.
The line through the origin is the elastic relationship according to the theory of elasticity between moment and rotation for the part of the beam between the measuring points F.P.A. and F.P.B. (see figure 6) when a rigid connection is assumed and the influence of the haunch is neglected.

5.3 Deformations

Because in many cases a relationship is supposed between boltforces and deformations , the measured deformations of web and web + flanges at specific bending moments are plotted in the fig. A 2.16 - A 2.25 of appendix A 2.
The deformations are averaged values of the measurements on both sides of the column-web.

The deformations of the web were measured at four locations of which the distances to the lower edge of the endplate are plotted at the vertical axis.
It is supposed that the web deforms rectilinearly between the measuring points.

The same assumption is made for the deformations of web + flanges which are only measured at three locations.
In figs. A 2.21 - A 2.25 (pp. 86 - 87) the differences between the deformations of web and web + flanges are given as deformations of the flanges.

5.4 Boltforces.

The measured boltforces are given in moment-boltforce curves in figs. A 2.31 to A 2.35 inclusive.
The boltforces are averaged values of two bolts on both sides of the column-web.

In figs. A. 2.36 to A. 2.40 inclusive the boltforce distributions are given at specific bending moments.
These boltforces are also averaged values of two bolts in a boltrow.
At the vertical axis the distance with respect to the lower edge of the end plate are given.
The specific bending moments of the various specimens are approximately the same. This is done to facilitate comparisons.
In figs. A. 2.41 to A. 2.60, moment-boltforce curves are given too.
In the latter figures the computed forces are indicated with solid lines in accordance with the computed values of table 2.
The differences between the solid lines and the actual boltforces are due to either prying action or mistakes in the assumption of the force distribution or centre of rotation.

A review of the moment - rotation curves of tests 1-5 of report 6-81-15 7 is given in fig. 14. This is done because here an other method of computation is applied than in report 6-81-15.

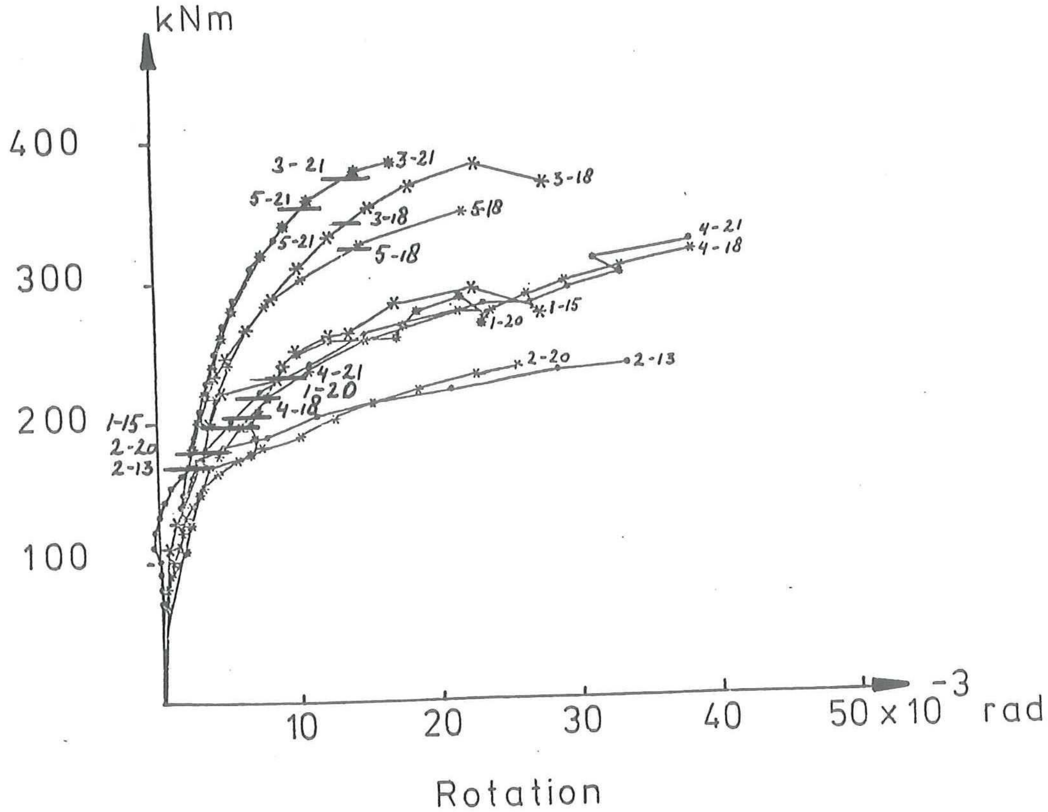


Fig. 14 : Review of the moment - rotation curves with limit state design moments of tests 1-5 of report 6-81-15.

7. Discussion.

7.1 Comparison of the old and new design methods

The results as shown in fig. 13 and 14 indicate that the proposed design method gives good results. The limit state design moments are always reached before large rotations occur.

The main assumptions of this design method are:

1^{0}. The first boltrow in the unstiffened part of the flange causes to form a complete yield line mechanism with a corresponding effective length of $b_m = 2m + 0{,}625n'$ in the unstiffened part.

determining mechanism and the limit state design moments are enlarged with 16 kNm and 23 kNm until 297 kNm and 306 kNm respectively.
A comparison with the moment-rotation curves shows that this increase for test 2 would be possible but for test 2-20 certainly not. Mainly it may be concluded that these tests are not appropriate to prove the adequacy of the reduction necessary for prying action.

The other four assumptions of the new design method are completely different from the old design method where the boltforces are taken linearly proportional to the distances to the point of rotation.
Moreover for the old design method different versions exists.
In the first version an infinitely stiff column flange and end plate is assumed, so that the design moment can be computed with the formula:

$$\hat{M}_v = \hat{F}_1 \frac{\Sigma h_i^2}{h_1} \qquad (7.2)$$

where: $\hat{F}_1$ is the design strength of the bolts in the first boltrow without any reduction for prying action and the values h_i similar to those in formula (7.1)

The assumption of an infinitely stiff column flange (or end plate) may be discutable. That is why in |3| a modification of the old design method is advised.
In this second version of the old design method:

$$\hat{M}_v = \hat{F}_2 \frac{\Sigma h_i^2}{h_2} \qquad (7.3)$$

where: $\hat{F}_2$ is the design strength of the bolts in the unstiffened part of the column flange just below the first boltrow.

The latter design strength is computed with the formulae of the unstiffened column flange with an effective length taken equal to the pitch between first and second boltrow.
The latter method had been advised because no method existed in which failure of the stiffened column flange was taken into account.
Now this method exist |5| the question may arise why formula (7.2) is not maintained with the value of $\hat{F}_1$ computed with the graph in

Test number	1^0 version, only boltfailure			2^0 version, combination of bolt and flange failure			3^0 version			failure of column web				Conclusion of versions 1	2	3
	$\hat{F}_1$	$\frac{\Sigma h_i^2}{h_1}$	$\hat{M}_v$	$\hat{F}_2$	$\frac{\Sigma h_i^2}{h_2}$	$\hat{M}_v$	$\hat{F}_1$	$\frac{\Sigma h_i^2}{h_1}$	$\hat{M}_v$	$\hat{F}_s$ shear	$\hat{F}_s$ compr.	$\frac{\Sigma h_i^2}{\Sigma h_i}$	$\hat{M}_v$	$\hat{M}_v$	$\hat{M}_v$	$\hat{M}_v$
(1)	(2) kN	(3) m	(4) kNm	(5) kN	(6) m	(7) kNm	(8) kN	(9) m	(10) kNm	(11) kN	(12) kN	(13) m	(14) kNm	(15) kNm	(16) kNm	(17) kNm
1-15	350	1.75	612	84	1.98	166	225	1.75	393		598	0.41	245	245	166	245
1-20	294	1.75	514	92	1.98	182	225	1.75	393		598	0.41	245	245	182	245
2-13	350	0.90	316	60	1.07	64	323	0.90	291		598	0.30	179	179	64	179
2-20	294	0.90	264	92	1.07	98	291	0.90	263		598	0.30	179	179	98	179
3-18	470	1.48	696	92	1.71	157	278	1.48	412		1020	0.41	418	418	157	412
3-21	456	1.48	674	92	1.71	157	278	1.48	412		1020	0.41	418	418	157	412
4-18	470	1.48	696	92	1.71	157	208	1.48	308		1020	0.41	418	418	157	308
4-21	456	1.48	674	92	1.71	157	208	1.48	308		1020	0.41	418	418	157	308
5-18	464	1.52	706	118	1.76	206	304	1.52	461		1020	0.42	428	428	206	428
5-21	464	1.52	706	138	1.76	242	304	1.52	461		1020	0.42	428	428	242	428
1	294	1.75	514	92	1.98	182	222	1.75	388	411	598	0.41	169	169	169	169
2	294	1.75	514	92	1.98	182	243	1.75	425	617	598	0.41	245	245	182	245
3	456	1.48	674	92	1.71	157	278	1.48	412	411	598	0.41	169	169	157	169
4	456	1.48	674	92	1.71	157	278	1.48	412	617	598	0.41	245	245	157	245
5	456	1.80	820	92	2.04	188	278	1.80	461	617	1020	0.47	290	290	188	290

Table 4: Summary of the design moments computed with three different versions of the old design method.

It is evident that the old design method gives too high values when the mechanisms due to shear and compression of the beam web are neglected. However, when shear of the column web is the governing failure mechanism, the new design method give better values as is shown by the results of tests 1, 2, 3 and 5 and in comparison with the moment rotation curves.
Thus the assumptions mentioned at the start of this section appears to be true.

The assumption that the first boltrow in the unstiffened part of the flange causes a complete yield line mechanism may only be proved by comparison of the computed and measured boltforces.
This comparison is possible in the moment - boltforce curves in figures A 2.41 to A 2.60 inclusive.
In these figures the computed boltforces are indicated with solid lines drawn through the origin of the moment boltforce curve and the value at the limit state design moment .
Despite the pretension, the origin is taken zero.
The assumed prying action is also indicated.
Generally it may be concluded that the pretension is hardly exceeded in the cases that small boltforces are computed whereas an increase of the forces occur in the boltrows which are assumed to cause a complete yieldline mechanism(row 1 of test 1 and the second rows of tests 2 to 5 inclusive).
The latter is especially true for tests 1,2 and 5.

7.2 The axial force in beam and column.

In the computations, the existence of a compressive force in beam and column has been neglected. The results show that this is possible.

7.3 The web doubler.

The results of the elastic finite element computation(appendix A 1 test 2) is confirmed by the testresults.

Figure 15 shows the yielding patterns in the column-web of test 1, 3 and 4.
Test 4 is provided with a web doubler.
Evidently the shear stresses cause yielding in the web at the edge of the web doubler (see test 4)
The shear force is built up by the upper two bolt rows as can be seen in test 4 too.

The rotations are only caused by the deformations of the webs. These rotations are computed with the transducers 7, 10, 14 and 17 as shown in fig. 7 .
Tests 2, 4, and 5 have web doublers whereas tests 1 and 3 have the original web section area. The influence of the web doubler is evident and may be expressed in a factor of about 1.5 between the moments at which large rotations begin to form.
This factor 1.5 is also used in the computations. However the actual plastic moment of the column has influenced the result.

The conclusion is that a web doubler which covers the central part of the web may be sufficient to improve the shear force capacity.
More tests and computations are necessary to develop design rules which are generally valid.

7.4 Stiffness.

In appendix A 3 a comparison is made between the rotations computed with stiffness formula (26) and the actual rotations. The conclusion is that formula (26) give too large rotations. This implies that the actual moments in the connections of a structure become larger than assumed. It is shown in appendix A 3 that this phenomenon is not disastrous because the increase of the moment is smaller than 16% for the connections concerned.

An approximation of the moment-rotation relationship assuming a rigid connection and neglecting the haunch appears to give good results for tests 2 to 5 inclusive.
This is shown by the bilinear curves indicated in the figures A 2.1 to A 2.15.
The deviation of test 1 is caused by a gap of 1.5 mm between the edge of the end-plate and the column flange prior to testing.

That the flange deformations are also influenced by the behaviour of the haunced beam is shown by the moment-rotation curve of the flange of test 4 (see figure 17)

7.5 Interaction in the column web.

The limit state design load for yielding or buckling of the column web is computed with the Graham formula without any reduction for all tests.
The main conclusion in |8| is that shear stresses in the column web do not reduce the limit design load for buckling whereas a reduction is necessary when the bending stresses are greater than half the yield-stress.
The testresults confirm this conclusion to be on the safe side.

7.6 Lateral bracing.

A general requirement is that plastic hinge locations associated with all but the last failure mechanism shall be adequately braced to resist lateral and torsional displacement.
In the tests the bracing was fixed on the connection in the points B, C and D as indicated in figure 4, thus on the tension sides of beam and column. Generally the fear exists that the compressed parts will fail prematurely due to lateral buckling. These tests did not confirm this fear.

Lateral buckling of the column flanges of tests 2, 4 and 5 only occurred when the plastic moment of the column was reached.
(see figure 19). However, this phenomenon is prevented if the requirement stated in the first sentence of this section is fulfilled.
These testresults confirm our expectation,that it is not necessary to brace laterally the compressed parts of a connection when the tension side is fixed and either the compressed beam flange or column flange continues.

7.7. Weld sizes

7.7.1. Welds between end plates and beam flanges

Formula (24) and (25) give weld sizes, which are larger than the actual sizes when it is assumed that large rotational capacity is required (see computation).

7.7.2. Between the beam and the haunch without a flange

Because the required sizes of the fillet welds between beam and haunch determines finally the attractiveness of the haunch without a flange, the fillet welds were given a minimum size of 4 mm. In comparison with the results of formula (37) (a_{wc} = 1.98 mm) this throat size is far too large.
Formula (37) is based on the assumption that the fillet welds may only be dimensioned on the shear force between haunch and beam.

It is assumed that the compression between haunch and beam does not load the welds.
This is only possible when a gap between haunch and beam is avoided. The chance of this possibility is rather good due to weld shrinkage, but it is a condition for the use of formula(37).
Other tests are necessary to prove the adequacy of the formula.
Test 5-21 of report 6-18-15 had also fillet welds with a throat size a_{wc} = 4 mm.
In that case the result of formula (37) becomes:

$$a_{wc} \geqslant 1 \ \frac{12,2 * 180 * 256}{1,65 * 330 * 240} = 4,3 \text{ mm} \qquad (37)$$

These welds did not fail. However, the ratio between actual and limit state load was: 0,92, thus this latter testresult is hardly sufficient to show the adequacy.

8.3. Haunch

8.3.1. In the new design method the bending moment capacity of the end plate is generally smaller than with the old design method. This may cause failure due to shear of the beam web over the length of the haunch.
Formula (27) give adequately the limit state design moment of this failure mechanism.

8.3.2. More tests are necessary to prove the adequacy of the formulae which determine the weld sizes and dimensions of the haunch without a flange.

8.4. Web doubler

The shear force capacity of a web panel is improved considerably with a web doubler which is welded between the toes of the fillets.

8.5. Influence of an axial force in the beam

The influence of an axial compression force in the beam is negligible.

8.6. Interaction in the column-web

The testresults confirm the conclusion in |8| that the load carrying capacity of the column-web for compression is not influenced by shear stresses.

8.7. Lateral bracing

These testresults confirm the expectation that lateral bracing of the compressed part of the connections is not necessary when the column-flanges continues and the flanges in tension are supported laterally.

9. REFERENCES

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4 Zoetemeijer, P. (1974)
A design method for the tension side of statically loaded beam-to-column connections.
Heron, Vol. 20, 1974, no. 1.

5 Doornbos, L.M. (1979)
Design method for the stiffened column flange, developed with yield line theory and checked with experimental results (in Dutch)
Thesis, Delft University of Technology.

6 Zoetemeijer, P. (1981)
Semi-rigid beam-to-column connections with stiffened column flanges and flush end plates.
Proceedings of the International Conference held at Teesside Polytechnic Middlesbrough, Cleveland, 6-9 April 1981.

Appendix A1 : Computation of tests 1 to 5 inclusive.

The computation is executed in accordance with the design method described in report 6-82-7 of which the formulae are given in appendix A-5.

Test 1

Column flange at first boltrow

The column has not an end plate thus the formulae of the unstiffened column flange are valid. These formulae are identical to formulae (11) and (12) used for the second boltrow. The effective length of the first boltrow is:

$$b_m = e_1 + 2m + 0{,}625n'$$

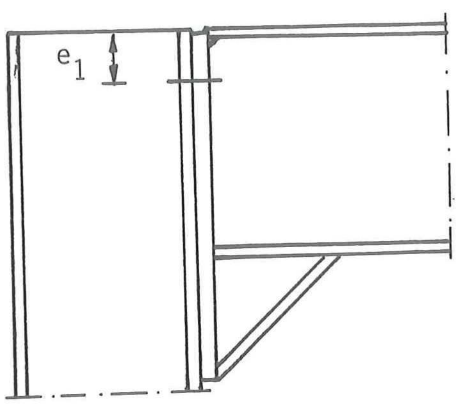
where:

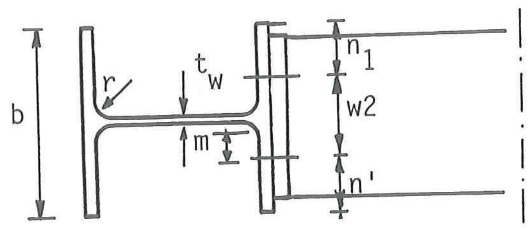
e_1 = the vertical distance between the end of the column and the first boltrow.

In this case: $e_1 = 60$ mm

$$m = \frac{w2 - t_w - 1.6r}{2}$$

$$n' = \frac{b - w2}{2}$$





w2 = 110 mm

t_w = 9 mm; r = 27 mm (see figure 9 in 6-81-15)

b = 300 mm

$$m = \frac{110 - 9 - 1.6 * 27}{2} = \frac{57.8}{2} = 28.9 \text{ mm}$$

$$n' = \frac{300 - 110}{2} = 95 \text{ mm}$$

$$b_m = 60 + 2 * 28.9 + 0.625 * 95 = 177 \text{ mm}$$

Now $\hat{F}_{1f} = \dfrac{b_m\, t_f^2\, \sigma_y}{m}$ (see formula (11))

or $\hat{F}_{1f} = \dfrac{b_m\, t_f^2\, \sigma_y + 4\hat{B}_t \cdot n}{2(m+n)}$ (see formula (12)).

t_f = 13 mm (see figure 9 in 6-81-15)

σ_y = 262 N/mm (see table 1 in 6-81-15)

$\hat{B}_t$ = 0,7 * 210 = 147 kN (see appendix 1, 6-81-15, curve b)

n = 35 mm < 1,25 * m = 1,25 * 28,9 = 36 mm

Test 1

Column flange at fourth and fifth boltrow

The same situation as with boltrow 4 because:

$b_m = 117$ mm $< \frac{1}{2}h_5 = \frac{1}{2} * 290 = 145$, thus $\hat{F}_{5f} = 92$ kN

Boltrow 6 does not contribute because the distance h_6 is too small.

Column web at first boltrow

The effective length is similar to that of the column-flange, thus:

$\hat{F}_{1w} = b_m * t_w * \sigma_y = 177 * 9 * 298$ (see table 1, 6-81-15)

$\hat{F}_{1w} = 474$ kN (formula (19) adjusted).

or:

$\hat{F}_{1w} = (2d_n + 2t_f \sqrt{\frac{b}{t_w}})\ t_w\ \sigma_y$ (formula (15) adjusted)

$\hat{F}_{1w} = (2 * 20 + 2 + 13 \sqrt{\frac{300}{9}}\ )\ 9 * 298 = 509$ kN

or:

$\hat{F}_{1w} = (e_1 + d_n + t_f \sqrt{\frac{b}{t_w}}\ )\ t_w\ \sigma_y$ (formula (18) adjusted)

$\hat{F}_{1w} = (60 + 20 + 13 \sqrt{\frac{300}{9}})\ 9 * 298 = 416$ kN

Conclusion: $\hat{F}_{1w} = 416$ kN.

Column web at other boltrows

$\hat{F}_{pw} = 60 * 9 * 298 = 161$ kN

Formula (20): $\Sigma\hat{F}_i < \hat{F}_{1w} + (n-1)\hat{F}_{pw}$

$\Sigma\hat{F}_i = 222 + 92 + 92 + 92 = 498$ kN $\leqslant 416 + 3 * 161 = 899$ kN is O.K.

Test 1

$m_p = \frac{1}{4} \cdot 20^2 \cdot 266 = 26.6$ kN/mm'

$\hat{F}_{1e} = 2 * 9.5 * 26.6 = 505.4$ kN

$2\hat{B}_t = 2 * 0.7 * 210 = 294$ kN

$\frac{\hat{F}_{1e}}{2\hat{B}_t} = \frac{505.4}{294} = 1.71$ Because $\lambda_1 > 0.5$; $f_{pr} = \frac{4}{2 + \frac{\hat{F}_{1k}}{2\hat{B}_t}}$ (2b)

$f_{pr} = \frac{4}{2 + 1.71} = 1.08$

thus bolt failure governs the limit state design situation of the end plate as far as the first boltrow is concerned and: $\hat{F}_1 = \frac{2\hat{B}_t}{f_{pr}} = \frac{294}{1.08} = 273$ kN (2)
But $\hat{F}_{1f} = 222$ kN, thus $\hat{F}_1 = 222$ kN.

Endplate at second boltrow

Formulae (11) and (12)

$m = m_1 + 0.2a_w\sqrt{2} = 44.6 + 0.2 * 4\sqrt{2} = 45.7$ mm

$m_{1,2} = 44.6$ mm

$$\hat{F}_{2e} = \frac{(p - m_{1,2} + 2m + 0.625n')\, t_e^2\, \sigma_y}{m} \qquad (11)$$

$2m + 0.625n' = 2 * 45.7 + 0.625 * 35 = 113$ mm

$p - m_{1,2} = 60 - 44.6 = 15.4$ mm

$$\hat{F}_{2e} = \frac{(15.4 + 113) \cdot 20^2 \cdot 266}{45.7} = 300 \text{ kN}$$

$$\hat{F}_{2e} = \frac{(p - m_{1,2} + 2m + 0.625n')\, t_e^2\, \sigma_y + 4\hat{B}_t \cdot n}{2(m+n)} \qquad (12)$$

$$\hat{F}_{2e} = \frac{(15.4 + 113)\, 20^2 \cdot 266 + 4 * 147000 * 35}{2(45.7+35)} = 212 \text{ kN}$$

Despite the situation that bolt failure governs the limit state design of boltrow 1, formula (13) is not valid, because the column-flange gives sufficient deformation to reach $\hat{F}_{2e}$ in accordance with formula (12)
Conclusion: $\hat{F}_{2e} = 212$ kN but on the column side $\hat{F}_{2f}=92$kN, thus $\hat{F}_{2f}=92$ kN.

Test 1

Formula (20)

$$\Sigma \hat{F}_i \leqslant \hat{F}_{1w} + (n-1)\hat{F}_{pw}$$

$\Sigma \hat{F}_i$ = 411 kN (see formula (21) for the column web).

$$\hat{F}_{pw} = 60 * 9{,}5 * 290 = 165 \text{ kN}$$

$$\hat{F}_{1w} + 3\hat{F}_{pw} = 501 + 3 * 165 = 996 \text{ kN}$$

Thus: $\Sigma\hat{F}_i \ll \hat{F}_{1w} + (n-1)\hat{F}_{pw}$

Check of the weld sizes

It is concluded that $\hat{F}_1$ = 222 kN, determined by bending of the column-flange.

Formula (25)

For a statically determinated situation $f_c = 1$

$f_v = 1$ because $\lambda_1 > 0{,}5$

Thus: $a_f \geqslant 0{,}7 * 1 * 1 * \dfrac{220000}{2(40{,}7+35)240} = 4{,}23$ mm

$a_f = 5$ mm thus O.K.

However for a statically indeterminated structure $f_c = 1{,}4$ and

$a_f \geqslant 5{,}93$ mm. In that situation the weld size would be insufficient. But the actual situation shows that a weld size of 5 mm is sufficient to reach a rotation of the connection of $60 * 10^{-3}$ radians. Thus here formula (25) appeared to be conservative.

Before the bending moment determined by the haunch dimensions is computed, a summary of the computation will be given here.

It appears that the limit state design moment is determined by shear of the column-web.

Bending moment determined by the haunch dimensions

Tension side

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{0{,}58t_w\ (l_c+2t_e) + \dfrac{bt_e^{\ 2}}{y}}{A_f + t_w\ y} \tag{27}$$

In this formula the actual yield stresses should be substituted to check the specimen behaviour.

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{0{,}58*9{,}5\ (200*290+2*20*266) + \dfrac{180*20^2*266}{4(60-6{,}1)}}{180*12{,}2*256+9{,}5*(60-6{,}1)\ 256} = 0{,}67$$

$\hat{M}_e = W_e\,\sigma_y = 1160 * 10^4 * 256 = 297$ kNm

Thus:

$\hat{M}_c \geqslant 0{,}67 * 297 = 200$ kNm

In the test $\dfrac{M_v}{M_c} = \dfrac{1{,}95}{1{,}95-0{,}20} = 1{,}11$

Thus $\quad M_v = 1{,}11 * 200 = 222$ kNm

Compression side

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{1{,}25\ \text{ctg}\ \alpha}{\dfrac{A_f}{A_d} + 0{,}5\ \text{ctg}\ \alpha} \tag{28}$$

ctg α = 1

$A_f = 180 * 12{,}2 \qquad A_d = \{20+5\ (12{,}2+21)\}\ 9{,}5 = 1767\ \text{mm}^2$

The actual yield stresses are: $\sigma_y = 290$ N/mm^2 for the web

$\sigma_y = 256$ N/mm^2 for the flange

Test 1

Check of the stiffness

Formula (26)

$$\alpha_1 = \left(\frac{\hat{F}_1}{\hat{F}_{1w}}\right)^2 \frac{1}{0{,}8t_w} = \left(\frac{222}{416}\right)^2 \cdot \frac{1}{0{,}8 * 9} = 0{,}030 \text{ mm}^{-1}$$

$$\alpha_2 = \left(\frac{\hat{F}_1}{\hat{F}_{1f}}\right)^2 \frac{4m_k^2}{t_{fk}^3} = \left(\frac{222}{222}\right)^2 \frac{4 * 28{,}9^2}{13^3} = 1{,}52 \text{ mm}^{-1}$$

$$\alpha_3 = \left(\frac{\hat{F}_1}{2B_t}\right)^2 \frac{l_b}{2A_s} = \left(\frac{222}{2*147}\right)^2 \left(\frac{33+4+0{,}5*4/3*20}{2*245}\right) = 0{,}058 \text{ mm}^{-1}$$

$$\alpha_4 = \left(\frac{\hat{F}_1}{\hat{F}_{1e}}\right)^2 \frac{12.\lambda_{2e} m_{1e}^2}{t_e^3} = \left(\frac{222}{505}\right)^2 \frac{12*0{,}51*44{,}6^2}{20^3} = 0{,}294 \text{ mm}^{-1}$$

$$\alpha_5 = \left(\frac{\Sigma\hat{F}_i}{\hat{F}_d}\right)^2 \frac{1}{0{,}8\ t_{wk}} = \left(\frac{411}{598}\right)^2 \frac{1}{0{,}8*9} = 0{,}065 \text{ mm}^{-1}$$

$$\alpha_6 = \left(\frac{\Sigma\hat{F}_i}{\hat{F}_s}\right)^2 \frac{1}{0{,}24\ t_{wk}} = \left(\frac{411}{411}\right)^2 \frac{1}{0{,}24*9} = \underline{0{,}462 \text{ mm}^{-1}}$$

$$= 2{,}431 \text{ mm}^{-1}$$

$$f_i = \frac{\Sigma(\hat{F}_i h_i)}{\hat{F}_1 h_1} \quad \frac{222*0{,}53+92*0{,}47+92*0{,}41}{222*0{,}53} = 1{,}69$$

$$k = \frac{2{,}431}{1{,}69*210*530^2} = 2{,}438 * 10^{-8} \text{ radians/kNmm}$$

$k = 2{,}438 * 10^{-5}$ radians/kNm at $\hat{M}_v = 200$ kNm

$\hat{M}_v = 200$ kNm $\quad \phi = 4{,}87 * 10^{-3}$ radians

Test 2

According to the design method described in report 6-82-7, table 4, the cooperation of the other boltrows should be neglected because the deformation at the first boltrow would be insufficient in order that the yield line mechanism at the other rows can develop.
However, the testresult shows that this theory does not apply here.
It may be possible that sufficient deformation capacity exists when the value: $\frac{\hat{F}_{1k}}{2\hat{B}_t} \leqslant 1$

But more research is necessary to prove that here it will be accepted that the other boltrows cooperate in the moment capacity of the connection.
The same situation is observed in test 2-20 of report 6-81-15.

Test 2

Thus the forces on the tensionside of the connection are restricted until

$\hat{F}_s = 617$ kN and $\hat{F}_d = 598$ kN (see test 1).

For the end plate the same results as in test 1 are valid.

Conclusion

$$
\begin{array}{llll}
\hat{F}_{1f} = 243 & \hat{F}_{1e} = 294 & \hat{F}_1 \cdot h_1 = 243 * 0{,}53 = & 129 \text{ kN} \\
\hat{F}_{2f} = 214 & \hat{F}_{2e} = 212 & \hat{F}_2 \cdot h_2 = 212 * 0{,}47 = & 100 \text{ kN} \\
\hat{F}_{3f} = 92 & \hat{F}_{3e} = 140 & \hat{F}_3 \cdot h_3 = 92 * 0{,}41 = & 38 \text{ kN} \\
\hat{F}_{4f} = 92 & \hat{F}_{4e} = 140 & \hat{F}_4 \cdot h_4 = \underline{51} * 0{,}35 = & \underline{18 \text{ kN}} \\
 & & \qquad\quad 598 & 285 \text{ kN}
\end{array}
$$

Bending moment determined by the haunch dimensions of test 2

Tension side

$y = 60 - 6{,}1 = 53{,}9$ mm

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{0{,}58 * 9{,}5\ (400 * 290 + 2 * 20 * 266) + \dfrac{180 * 20^2 * 266}{4 * 53{,}9}}{180 * 12{,}2 * 256 + 9{,}5 * 53{,}9 * 256} = 1{,}13$$

Compression side

$A_d = 5\ \{20 + (12{,}2 + 21)\}\ 9{,}5 = 1767\ \text{mm}^2 \qquad \text{ctg}\ \alpha = 2$

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{1{,}25\ \text{ctg}\ \alpha}{\dfrac{A_f}{A_d} + 0{,}5\ \text{ctg}\ \alpha} = \frac{1{,}25*2}{\dfrac{180*12{,}2+256}{1767*290} + 0{,}5 * 2} = \frac{2{,}5}{2{,}09} = 1{,}19$$

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{\text{ctg}\ \alpha}{\dfrac{A_f}{A_d}} = \frac{2}{\dfrac{180*12.2*256}{1767*290}} = \frac{2}{1.09} = 1.83$$

Test 2

$f_c = 1$ (statically determinated structure)

$m_2 = 40,7$ mm

$n' = 35$ mm

$$a_f \geq 0,7 * 1 * 1 * \frac{243000}{2(40,7+35)240} = 4.67 \text{ mm}$$

$a_f = 5$ mm thus o.k.

However, in a side-sway frame $f_c = 1,7$; then $a_f = 7.94$ mm

In that situation the weld size would be insufficient. The moment rotation curve shows a large rotational capacity.

Thus formula (25) is good as far as this test is concerned.

Check of the haunch-flange

$$t_c \geq \frac{1*180*12,2*256}{180*0,8944*266} = 13,12 \text{ mm} \tag{29}$$

$$t_c \geq 1 * \frac{0,7+180*12,2*256}{(10*\ 12,2*9,5)*0,8944*290} = 10,76 \text{ mm} \tag{30}$$

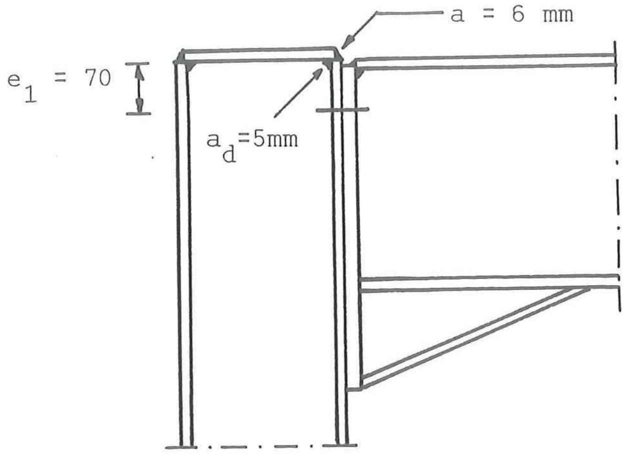
$$t_c \geq \frac{180}{17} = 10,58 \text{ mm} \tag{31}$$

$$a_c \geq 1 * \frac{0,7*180*12,2*256}{(10*12,2+2*9,5)*256} \sqrt{3+0,25} = 19,65 \text{ mm} \qquad (32) \text{ o.k.}$$

The conclusion is that the limit state moment of test 2 is determined by failure of the column flange in combination with failure of the column web on the compression side at $\hat{M}_v = 285$ kNm.

Test 3

Columnflange at first boltrow



e_1 = 70 mm

w2 = 120 mm (see figure 1)

t_w = 9 mm   r = 27 mm

b = 300 mm

b_e = 220 mm

$m_1 = \frac{120-9-54}{2} = 28.5$ mm

n' = 95 mm

$m_2 = 70 - 5\sqrt{2} = 63$ mm

$\lambda_1 = \frac{28.5}{28.5+95} = 0.23$ $\lambda_2 = \frac{63}{28.5+95} = 0.51$

$\alpha = 4\pi$

$\hat{F}_{1f} = 2\alpha m_p(1) \rightarrow \hat{F}_{1f} = 8\pi.\frac{1}{4}.13^2.262 = 278$ kN

$\hat{B}_t = 0.7 * 326 = 228$ kN (see appendix 1, 6-81-15, curve c).

$\frac{\hat{F}_{1f}}{2\hat{B}_t} = \frac{278}{456} = 0.60$

Conclusion: $\hat{F}_1 = 278$ kN because $\frac{\hat{F}_{1f}}{2\hat{B}_t} < \frac{1}{f_{pr}} = 0.67$

thus the limit state load of boltrow 1 is determined

by yielding of the column flange.

Test 3

Column web at first boltrow

Formula (14)

$$z = 13 \sqrt{\frac{300}{9}} = 75.06 \text{ mm}$$

$m_2 = 63$ mm $\qquad y = 70 + 7.3 = 77.3$ mm

Formulae (16) to (19) inclusive

$t_w = 9$ mm $\quad \sigma_y = 298$ N/mm^2

$t_f = 13$ mm $\quad \sigma_y = 262$ N/mm^2

$t_d = 14{,}6$mm $\quad \sigma_y = 268$ N/mm^2

$$\hat{F}_{1w} = (24 + 75 + \frac{63}{2})\ 9 * 298 + \frac{300 * 13^2}{2 * 63} * 262 \qquad (16)$$

$$\hat{F}_{1w} = 350 + 105 = 455 \text{ kN}$$

$$\hat{F}_{1w} = (127 + \frac{63}{2})\ 9 * 298 + \frac{300 * 13^2}{2 * 63} * 262 \qquad (17)$$

$$\hat{F}_{1w} = 425 + 105 = 530 \text{ kN}$$

$$\hat{F}_{1w} = (24 + 75 + \frac{77.3}{2})\ 9 * 298 + \frac{300*13^2*262+300*14.6^2*268}{4 * 77.3} \qquad (18)$$

$$\hat{F}_{1w} = 369 + 98 = 467 \text{ kN}$$

$$\hat{F}_{1w} = (127 + \frac{77.3}{2})\ 9 * 298 + \frac{300*13^2*262+300*14.6^2*268}{4 * 77.3} \qquad (19)$$

$$\hat{F}_{1w} = 444 + 98 = 542 \text{ kN}$$

Test 3

$$m_1 = \frac{120 - 9.5 - 8\sqrt{2}}{2} = 49.6 \text{ mm}$$

$$n' = \frac{220 - 120}{2} = 50 \text{ mm}$$

$$\lambda_1 = \frac{49.6}{49.6+50} = 0,50$$

$$m_2 = 70 - 12.2 - 5\sqrt{2} = 50.7 \text{ mm} \qquad \lambda_2 = 0.51$$

From chart of formula (4a) in table A5.2. $\alpha = 9.8$

$$\hat{F}_{1e} = 2\alpha\, m_p \tag{1}$$

$$m_p = \tfrac{1}{4} \cdot 21^2 \quad 310 = 34.1 \text{ kN/mm}^2$$

$$\hat{F}_{1e} = 19.6 * 34.1 = 670 \quad \text{kN}$$

$$2\hat{B}_t = 2 * 0.7 * 326 = 456 \text{ kN}$$

$$\frac{\hat{F}_{1e}}{2\hat{B}_t} = \frac{670}{456} = 1.47 \quad \text{Because } \lambda_2 = 0.51;\ f_{pr} = \frac{4}{2 + 1.47} = 1.15 \tag{2b}$$

Thus $\hat{F}_{1e} = \frac{670}{115} = 582$ kN, but $\hat{F}_{1f} = 278$ kN thus

$$\hat{F}_1 = 278 \text{ kN}$$

End plate at second boltrow

Formulae (11), (12) and (13).

$m = m_1 + 0.2\ a_w \sqrt{2} = 49.6 + 0.2 * 4\sqrt{2} = 50.7$ mm

$m_{1,2} = 50.7$ mm because $\alpha < 4\pi$ and $m_1 < m_2$ at boltrow 1.

$p = 70$ mm

$2m + 0.625n' = 2 * 50.7 + 0.625 * 50 = 132.7$ mm

Test 3

$$\hat{F}_{1w} = (24 + 101 + \frac{70-6.1}{2})\ 9.5*290 + \frac{220*21^2*310+180*12.2^2*256}{4 * (70-6.1)} \tag{19}$$

$$\hat{F}_{1w} = 453 + 145 = 599\ \text{kN}$$

$$\hat{F}_{pw} = 70 * 9.5 * 290 = 193\ \text{kN}$$

$$\Sigma\hat{F}_i \leqslant 577 + 3 * 193 = 1156\ \text{kN} \tag{20}$$

Conclusion test 3

$\hat{F}_{1f} = 278$	$\hat{F}_{1e} = 582$	$\hat{F}_1 . h_1$	$= 278 * 0.52 = 144$ kNm
$\hat{F}_{2f} = 175$	$\hat{F}_{2e} = 330$	$\hat{F}_2 . h_2$	$= 133 * 0.45 = 60$ kNm
		$\hat{F}_s$	$= 411$ kN $\quad = 204$ kNm

Test 3

Beam side

$$f_v = \frac{m_1}{m_1 + m_2} \qquad \text{because} \quad \lambda_1 = 0.5$$

$$f_v = \frac{49.6}{49.6+50.7} = 0.49$$

$$a_f \geqslant 0.7\ f_v . f_c . \ \frac{\hat{F}_1}{2(m_2+n')\sigma_y} \qquad (25)$$

$$a_f \geqslant 0.7\ .\ 0.49\quad 1\quad \frac{278000}{2(50.7+50)240} = 3.95\ \text{mm}$$

for a statically determinated structure.

a_f = 5.52 mm for a braced frame

a_f = 6.71 mm for an unbraced frame (where the rotational capacity should be larger than 0.04 radians)

The rotation of test 3 was ultimately 0.065 radians whereas the weld a_f = 5 mm.

This result shows the adequacy of formula (25) in this case.

Bending moment determined by the haunch dimensions

Tension side

Formula (27)

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{0.58*9.5\ (400*290+2*21*310) + \dfrac{220*21^2*310}{4(70-6.1)}}{180 * 12.2 * 256 + 9.5\ (70-6.1)\ 256} = 1.15$$

Test 3

Check of the stiffness

Formula (26)

$$\alpha_1 = \left(\frac{\hat{F}_1}{\hat{F}_{1w}}\right)^2 \cdot \frac{1}{0.8t_w} = \left(\frac{278}{455}\right)^2 \cdot \frac{1}{0.8*9} = \quad 0.05 \quad mm^{-1}$$

$$\alpha_2 = \left(\frac{\hat{F}_1}{\hat{F}_{1f}}\right)^2 \cdot \frac{12\lambda_2\, m_{1k}^2}{t_{fk}^3} = \left(\frac{278}{278}\right)^2 \frac{12*0.51*28.5^2}{13^3} = \quad 2.26 \quad mm^{-1}$$

$$\alpha_3 = \left(\frac{\hat{F}_1}{2B_t}\right)^2 \cdot \frac{l_b}{2A_s} = \left(\frac{278}{456}\right)^2 \cdot \frac{34+4+0.5*4/3*24}{2*353} = \quad 0.03 \quad mm^{-1}$$

$$\alpha_4 = \left(\frac{\hat{F}_1}{\hat{F}_{1e}}\right)^2 \quad \frac{12\lambda_2\, {}_e m_{1e}^2}{t_e^3} = \left(\frac{278}{670}\right)^2 \frac{12*0.51*49.6^2}{21^3} = \quad 0.28 \quad mm^{-1}$$

$$\alpha_5 = \left(\frac{\Sigma\hat{F}_i}{\hat{F}_d}\right)^2 \quad \frac{1}{0.8t_{wk}} = \left(\frac{411}{598}\right)^2 \quad \frac{1}{0.8*9} \quad = 0.07 \quad mm^{-1}$$

$$\alpha_6 = \left(\frac{\Sigma\hat{F}_i}{\hat{F}_s}\right)^2 \quad \frac{1}{0.21t_{wk}} = \left(\frac{411}{411}\right)^2 \quad \frac{1}{0.24*9} \quad = \underline{0.46 \quad mm^{-1}}$$

$$3.15 \quad mm^{-1}$$

$$f_1 = \frac{278 * 0.52 + 133 * 0.45}{278 * 0.52} = 1.41$$

$$k = \frac{3.15}{1.41*210*520^2} = 3.91 * 10^{-8} \text{ radians / kNmm}$$

$$k = 3.91 * 10^{-5} \text{ radians/kNm at } \hat{M}_v = 204 \text{ kNm}$$

$$\phi = 7.97 * 10^{-3} \text{ radians} \quad \text{at } \hat{M}_v = 204 \text{ kNm}$$

Test 4

Compression side

The same situation as in test 1, thus: $\hat{M}_c$ = 232 kNm and $\hat{M}_v$ = 258 kNm

Hence:

Before strengthening, the limit state moment: $\hat{M}_v$ = 234 kNm determined by shear of the beam web.

After strengthening, the limit state moment $\hat{M}_v$ = 276 kNm determined by shear of the column web together with yielding of the column flange on the tension side of the connection.

Check of the stiffness at a moment $\hat{M}_v$ = 276 kNm

α_1 = (see test 3) = 0.05 mm^{-1}

α_2 = (see test 3) = 2.26 mm^{-1}

α_3 = (see test 3) = 0.03 mm^{-1}

α_4 = (see test 3) = 0.28 mm^{-1}

$$\alpha_5 = \left(\frac{598}{598}\right)^2 \frac{1}{0.8*9} = 0.14 \text{ mm}^{-1}$$

$$\alpha_6 = \left(\frac{598}{617}\right)^2 \frac{1}{0.24*19} = \underline{0.20} \text{ mm}^{-1}$$

$$2.96 \text{ mm}^{-1}$$

$$f_i = \frac{276}{78*0.52} = 1.91$$

$$k = \frac{2.96}{1.91*210*520^2} = 2.73 * 10^{-8} \text{ radians/kNmm}$$

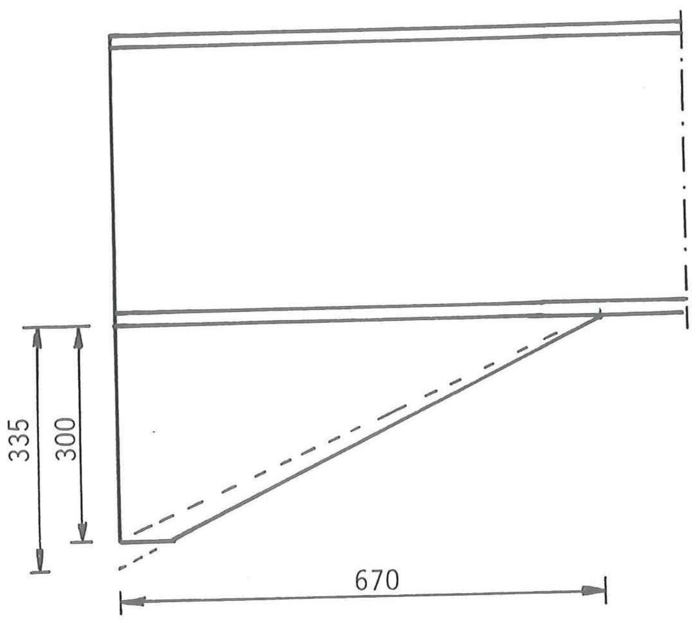
$k = 2.73 * 10^{-5}$ radians/kNm at $\hat{M}_v$ = 276 kNm

$\phi = 7.53 * 10^{-3}$ radians at $\hat{M}_v$ = 276 kNm

$\phi = 5.41 * 10^{-3}$ radians at $\hat{M}_v$ = 234 kNm

Test 5

Bending moment determined by the haunch dimensions



$tg\,\alpha = \frac{300}{670} = 0.45$ $\cos\alpha = 0.91$ $\cos\alpha = 0.83$

$ctg\,\alpha = 2.23$

$\sin^2\alpha = 0.17$

Tension side

Formula (27)

The result of test 3 shows that $\frac{\hat{M}_c}{\hat{M}_e} > 1$ when $l_c = 400$ mm

hence:

$$\frac{\hat{M}_c}{\hat{M}_e} > 1 \text{ with } l_c = 670 \text{ mm}$$

Test 5

Check of the stiffness

α_1 = (see test 3) $= 0.05 \ mm^{-1}$

α_2 = (see test 3) $= 2.26 \ mm^{-1}$

α_3 = (see test 3) $= 0.03 \ mm^{-1}$

α_4 = (see test 3) $= 0.28 \ mm^{-1}$

$$\alpha_5 = \left(\frac{617}{617}\right)^2 \frac{1}{0.8 * 9} = 0.11 \ mm^{-1}$$

$$\alpha_6 = \left(\frac{617}{617}\right)^2 \frac{1}{0.24 * (9+10)} = \underline{0.22} \ mm^{-1}$$

$$2.95 \ mm^{-1}$$

$$f_i = \frac{324}{278*0.59} = 1.97$$

$$k = \frac{2,95}{1.97*210*590^2} = 2.04 * 10^{-8} \text{ radians/kNmm}$$

$$k = 2.04 * 10^{-5} \text{ radians/kNm at } \hat{M}_v = 324 \text{ kNm}$$

$$\phi = 6.62 * 10^{-3} \text{ radians at } \hat{M}_v = 324 \text{ kNm}$$

Appendix A2

TEST RESULTS

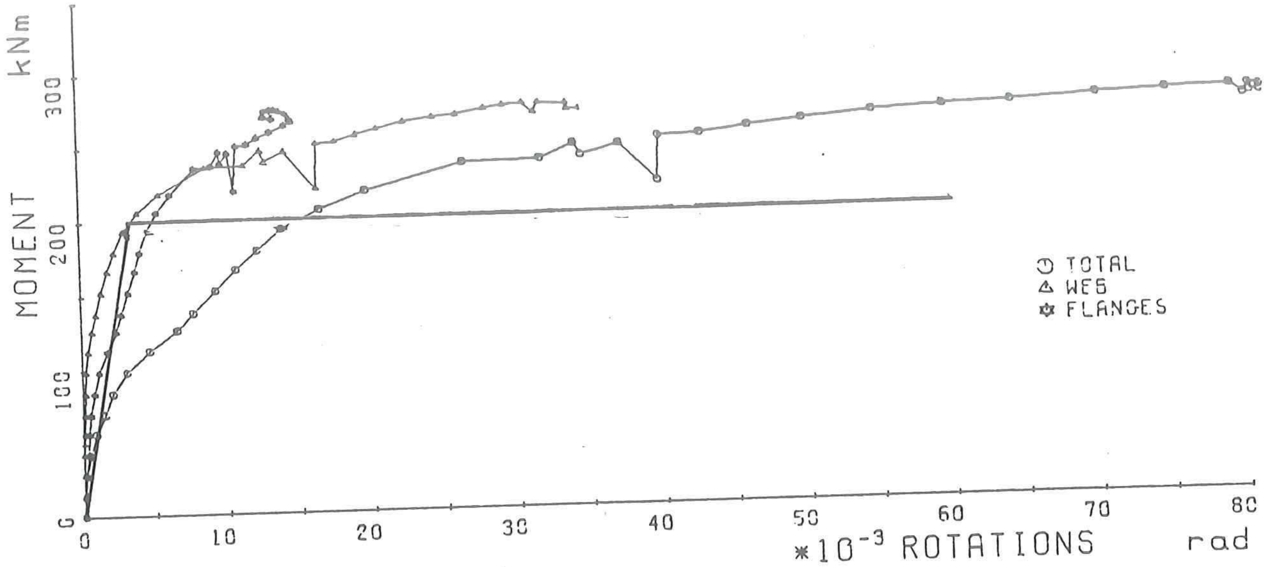


Fig. A2.1: Test 1

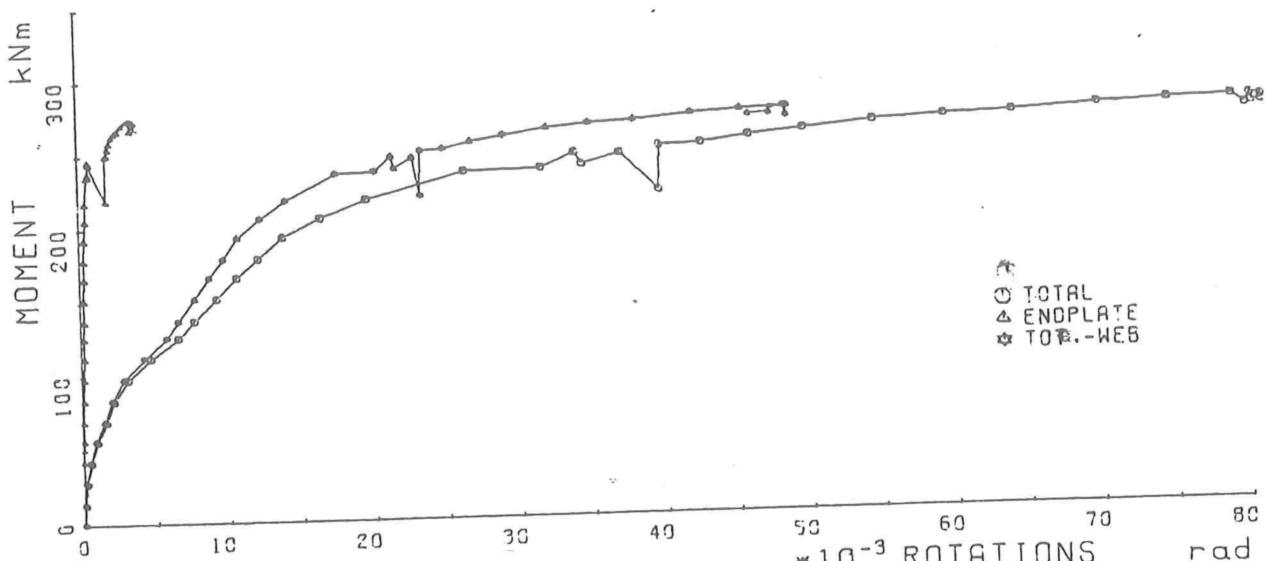


Fig. A2.2: Test 1

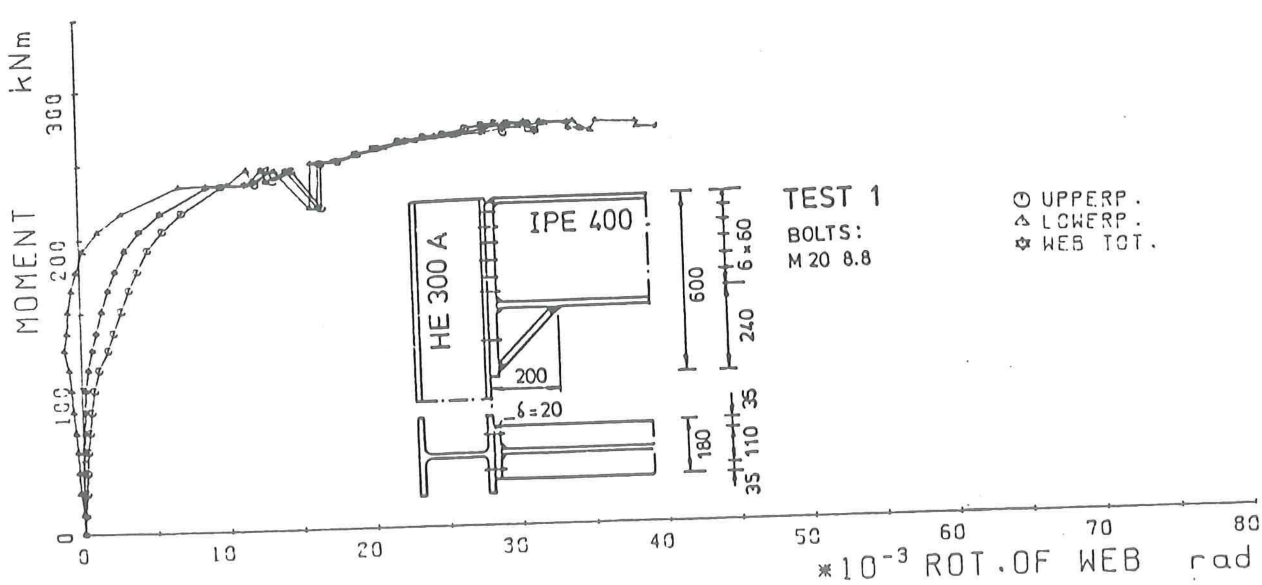


Fig. A2.3: Test 1

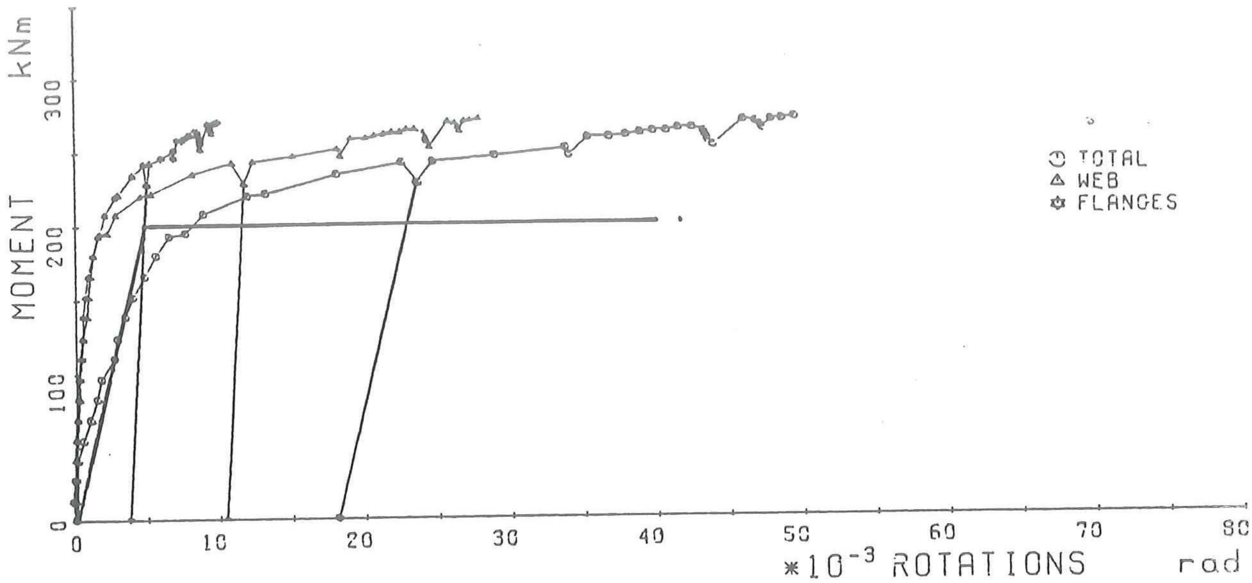


Fig. A2.7: Test 3

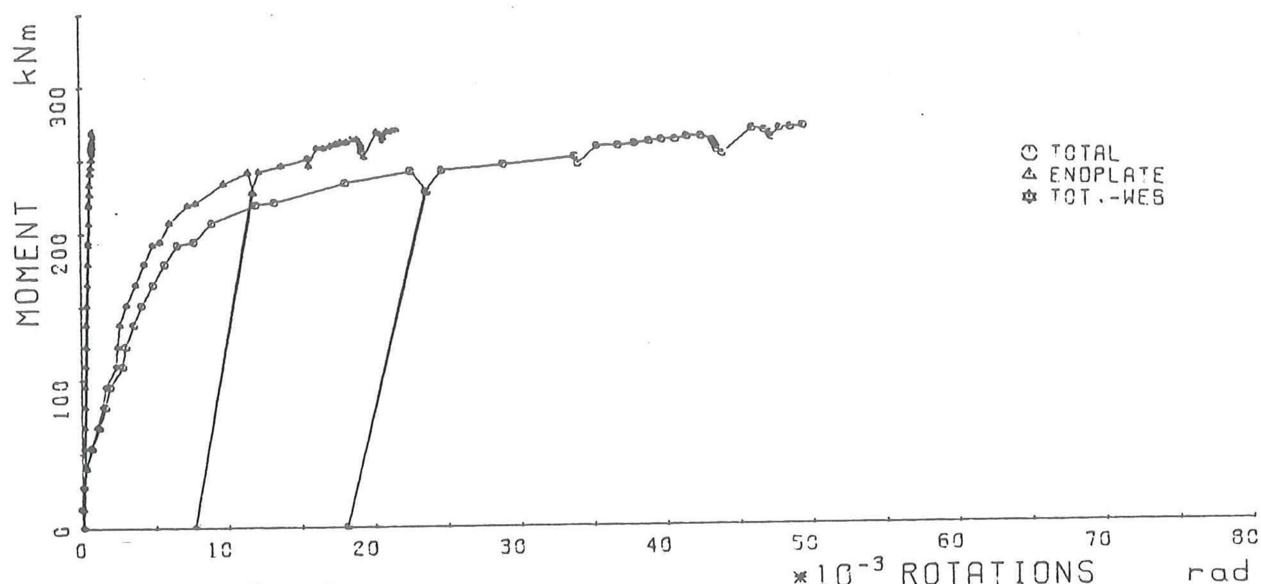


Fig. A2.8: Test 3

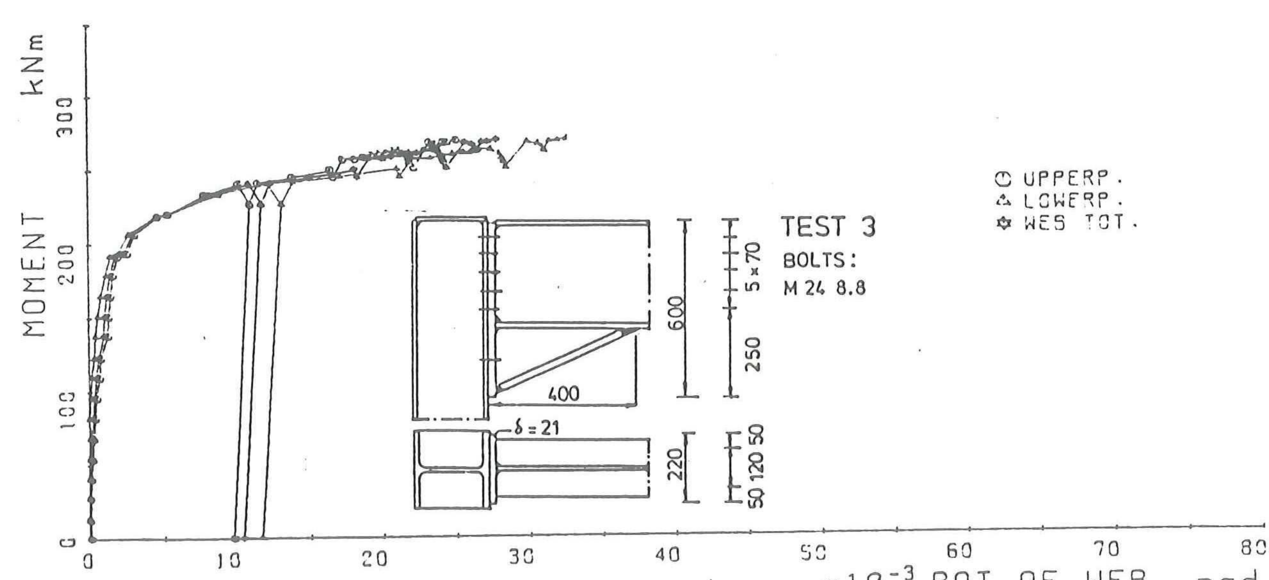


Fig. A2.9: Test 3

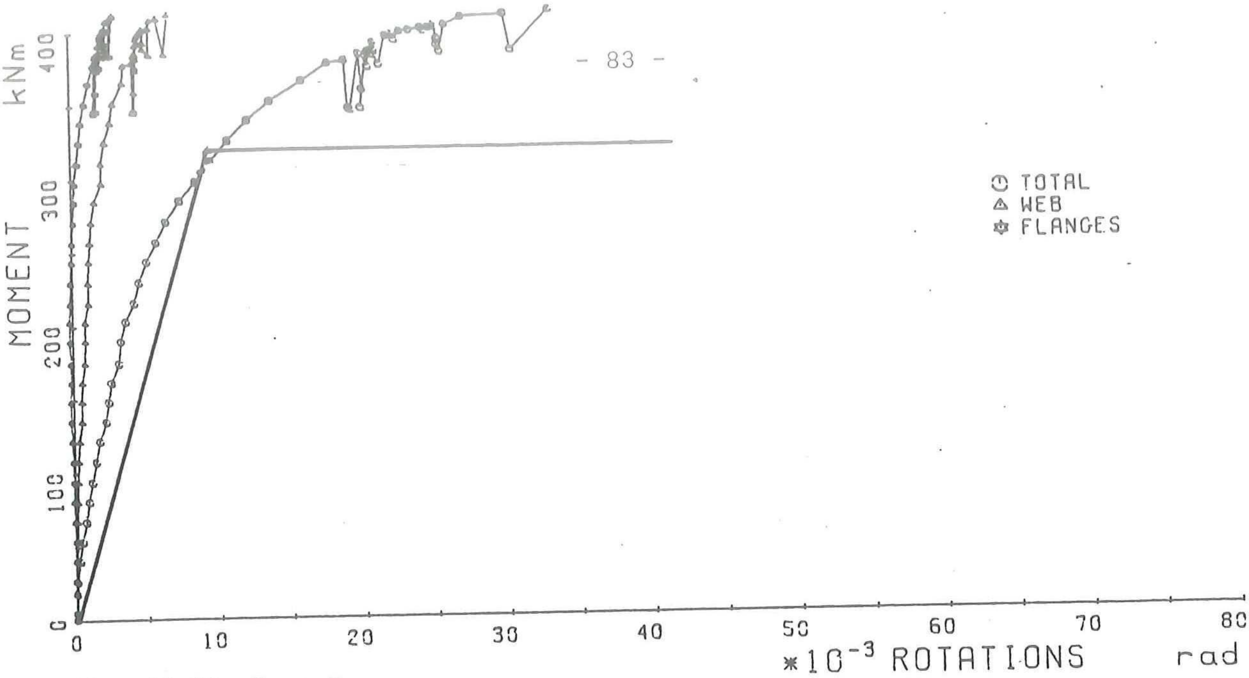


Fig. A2.13: Test 5

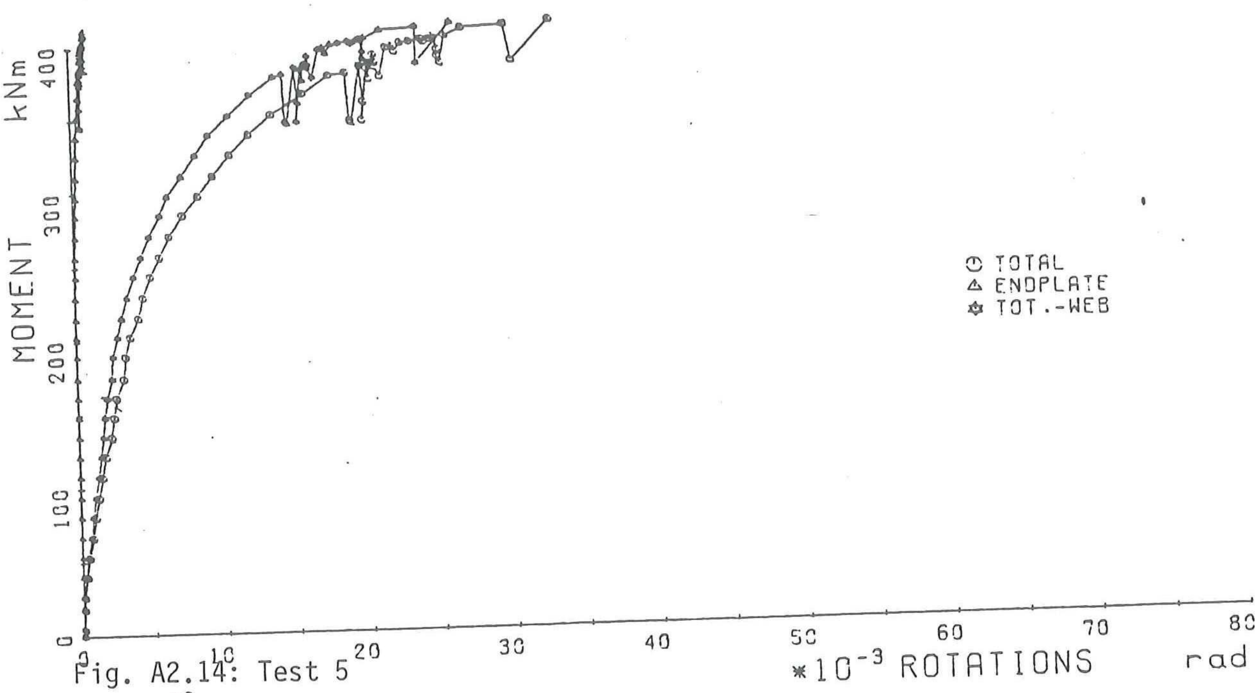


Fig. A2.14: Test 5

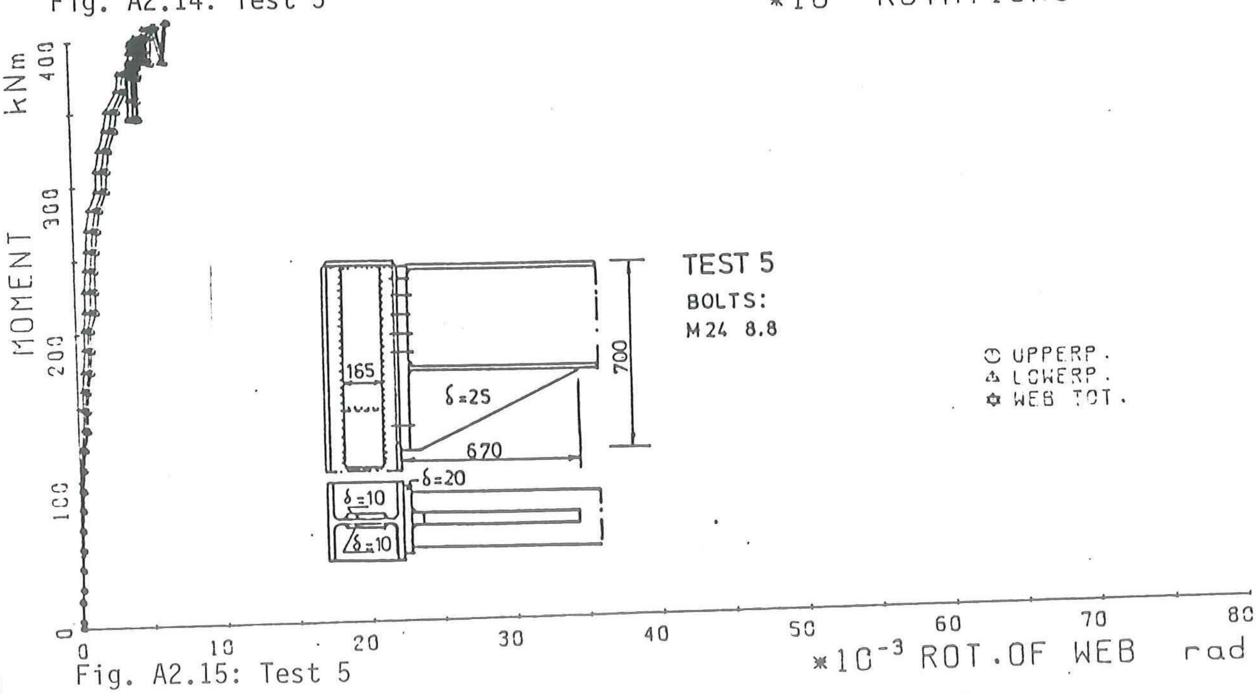


Fig. A2.15: Test 5

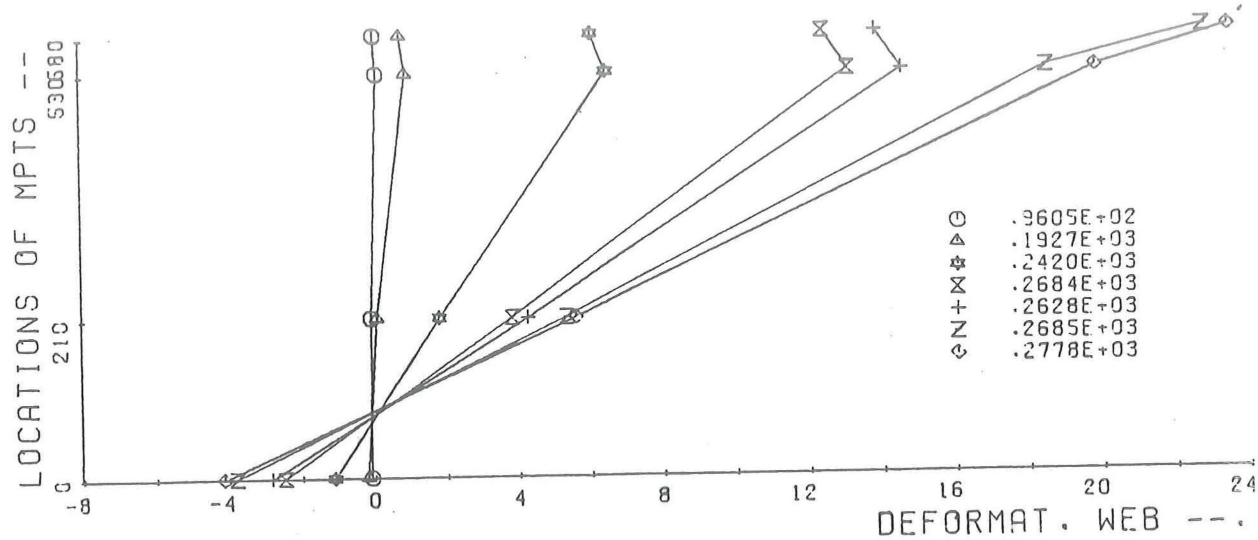


Fig. A2.18: Test 3

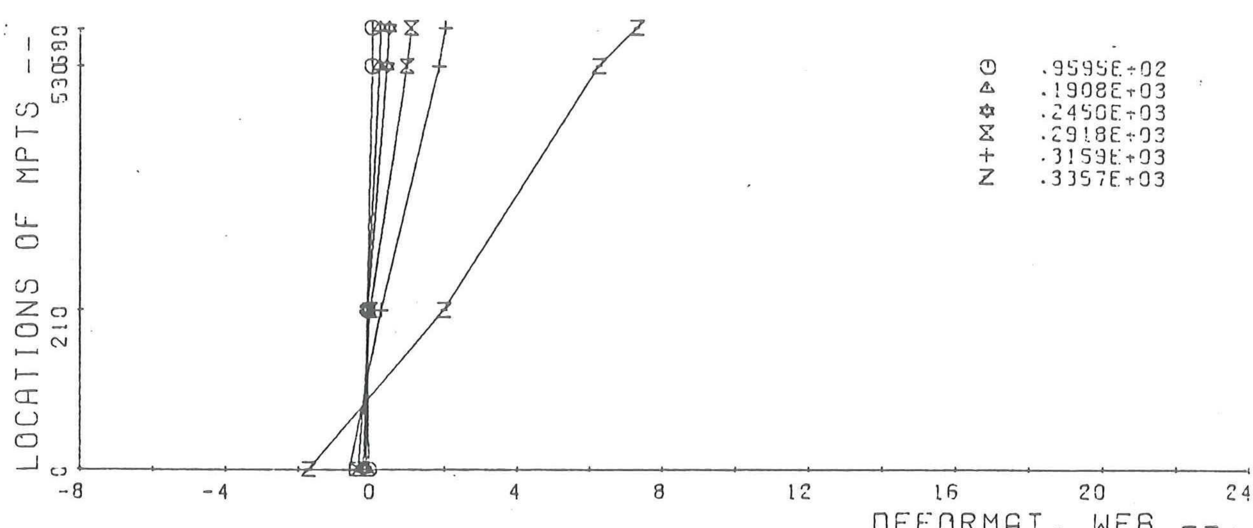


Fig. A2.19: Test 4

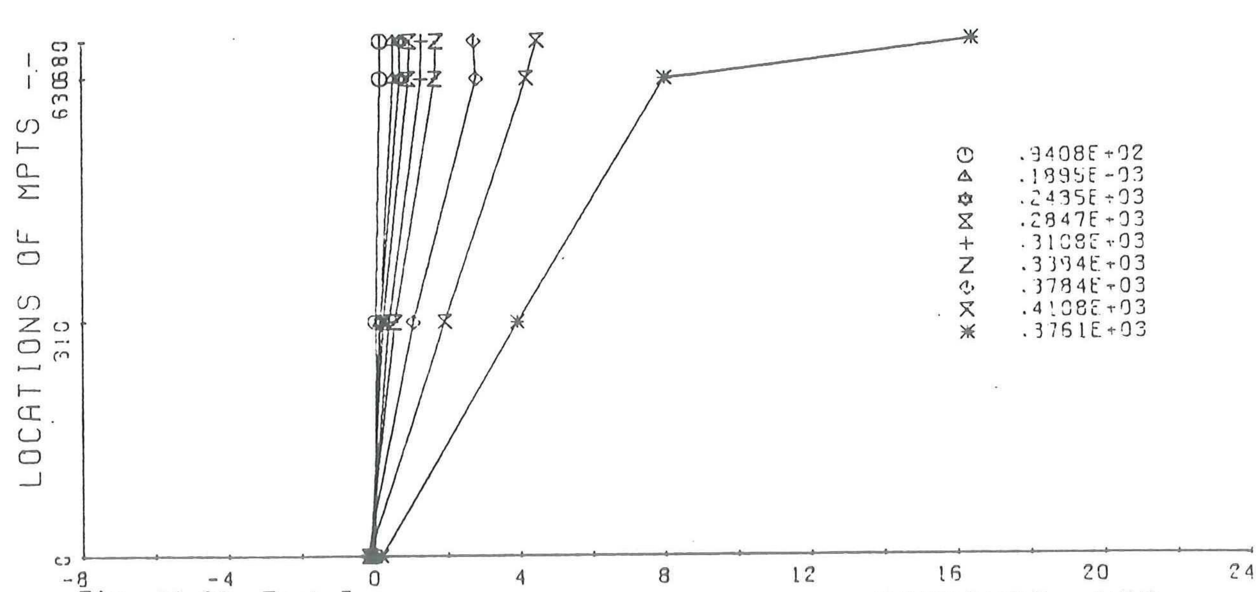


Fig. A2.20: Test 5

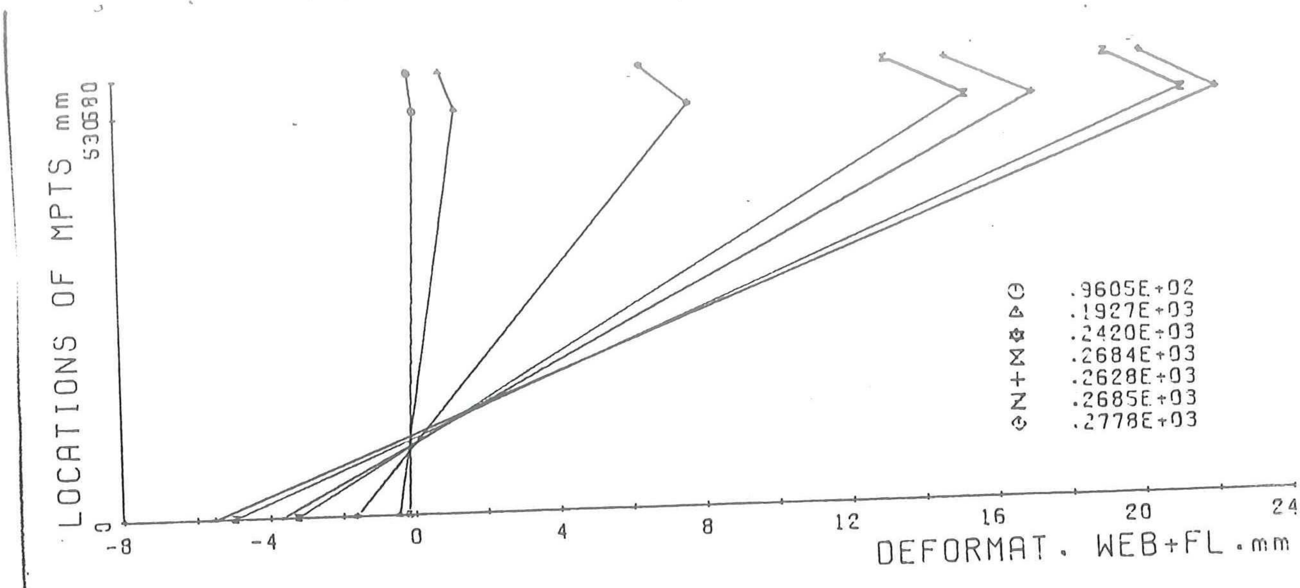


Fig. A2.23: Test 3

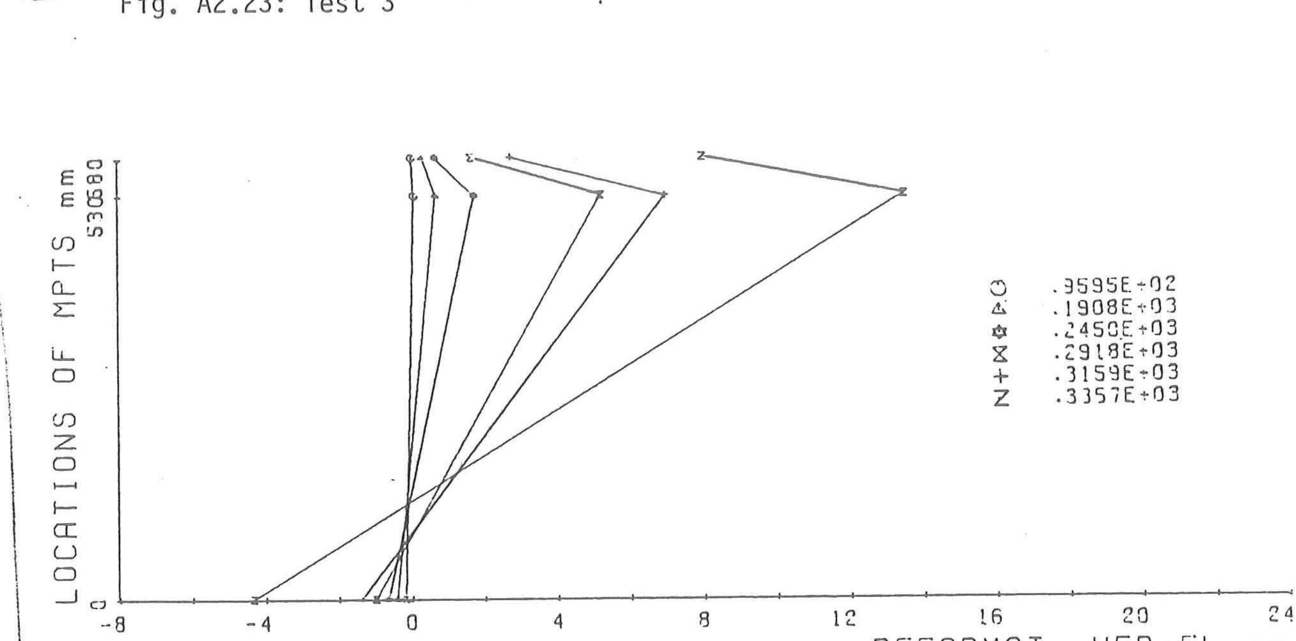


Fig. A2.24: Test 4

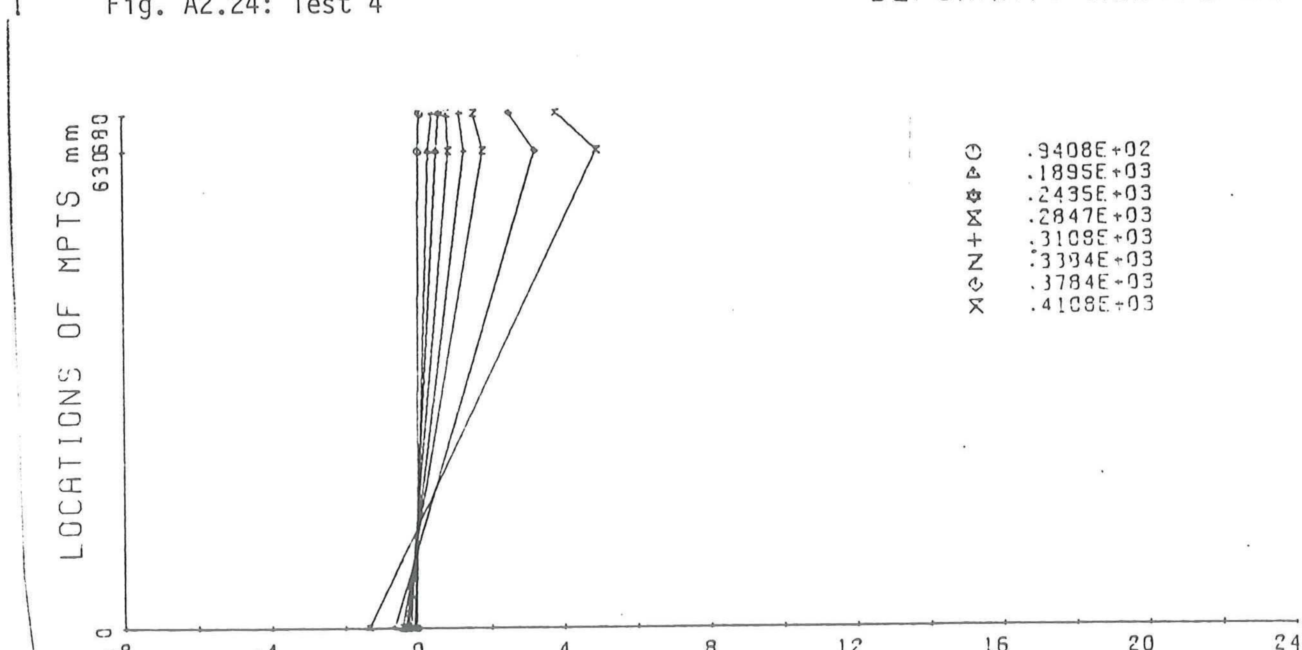


Fig. A2.25: Test 5

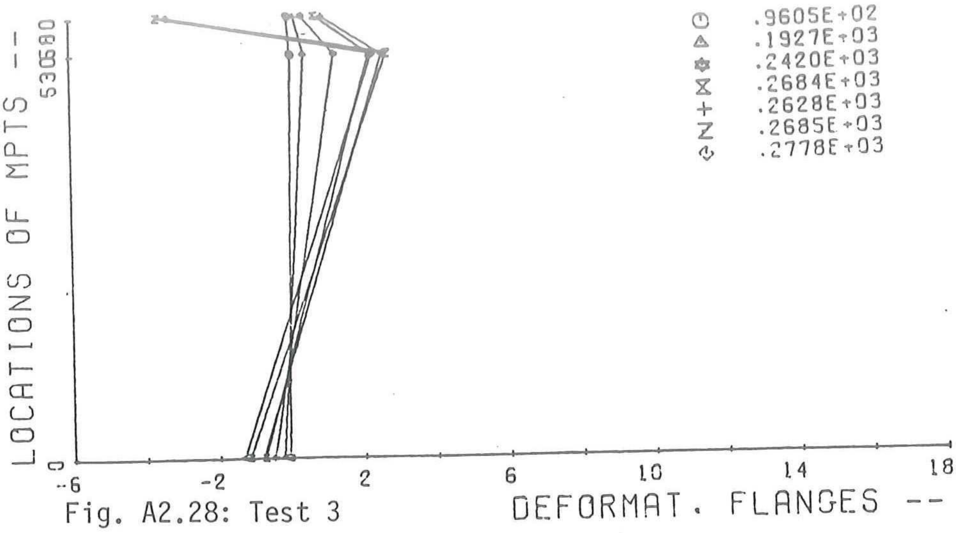


Fig. A2.28: Test 3

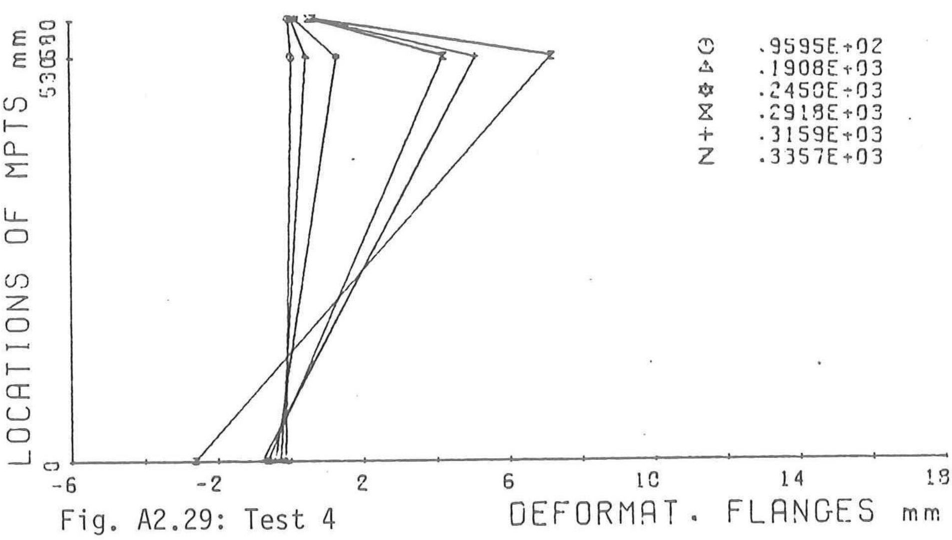


Fig. A2.29: Test 4

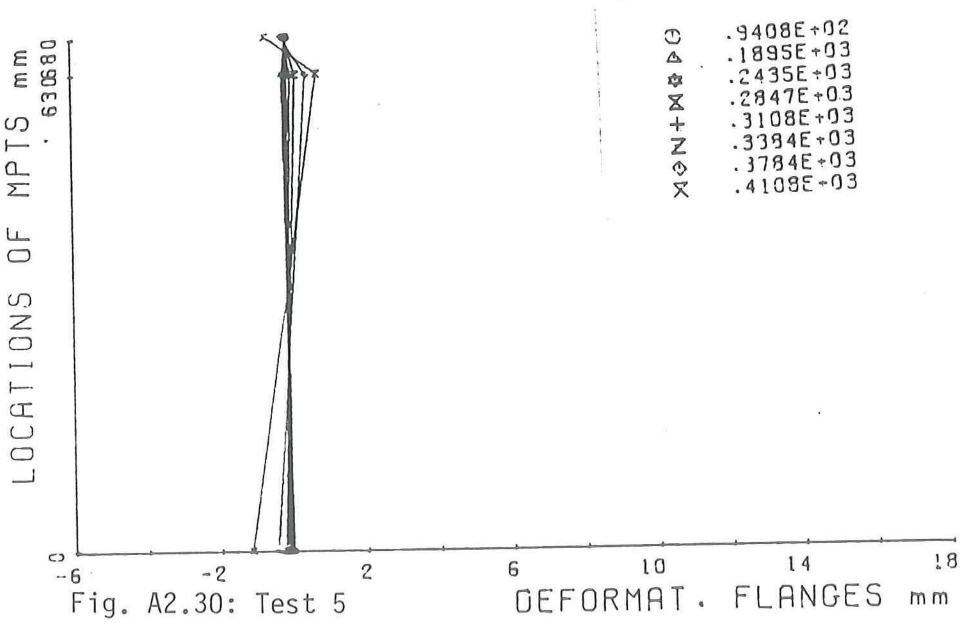


Fig. A2.30: Test 5

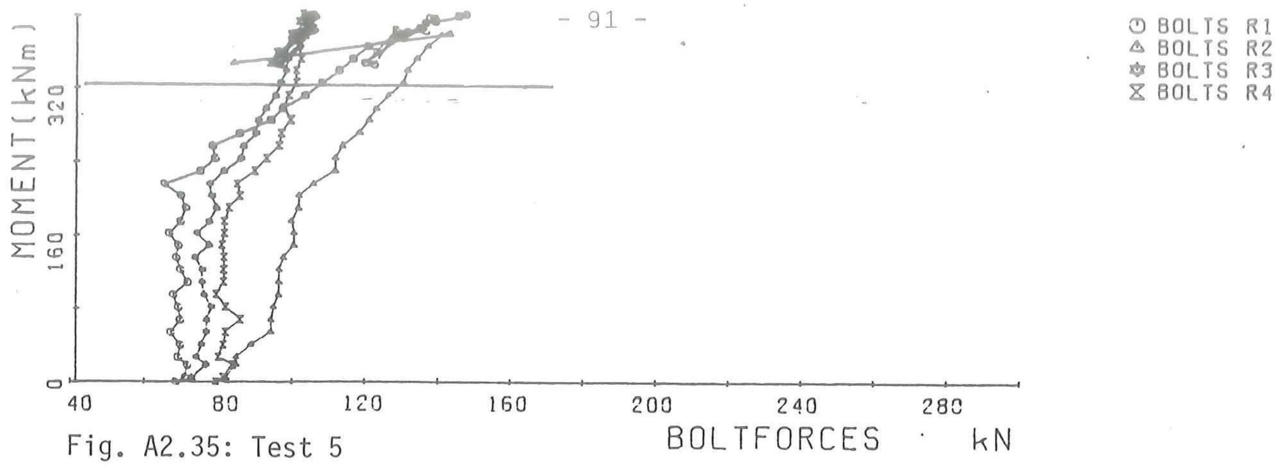
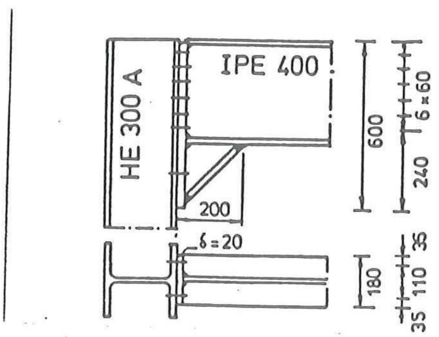
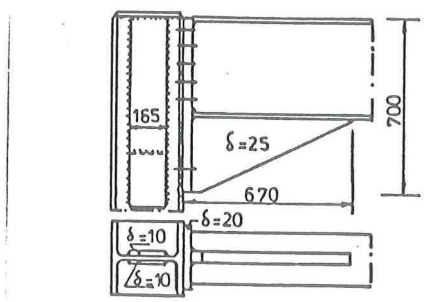


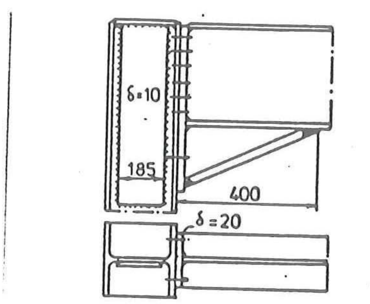
Fig. A2.35: Test 5



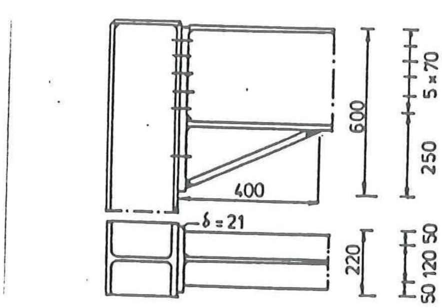
TEST 1
BOLTS:
M 20 8.8



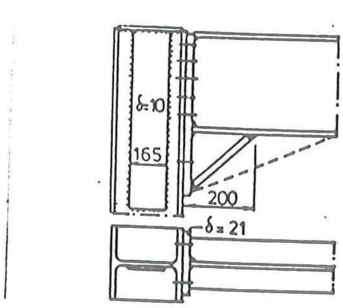
TEST 5
BOLTS:
M 24 8.8



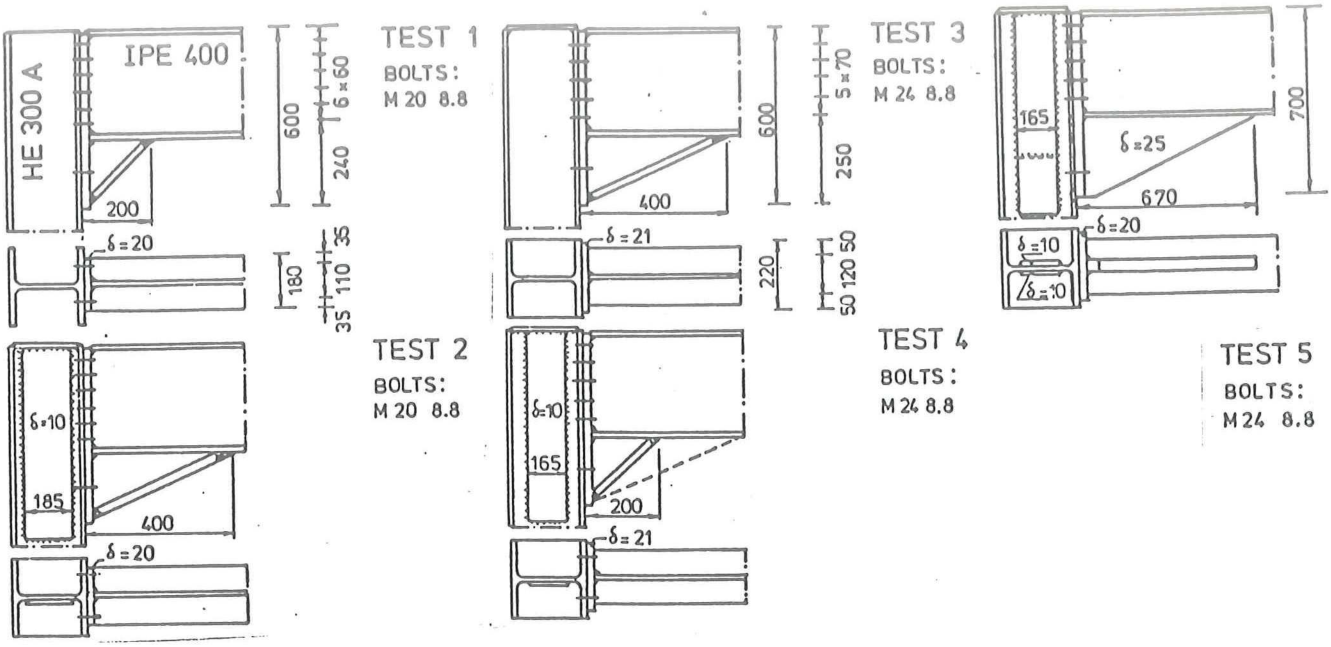
TEST 2
BOLTS:
M 20 8.8



TEST 3
BOLTS:
M 24 8.8



TEST 4
BOLTS:
M 24 8.8



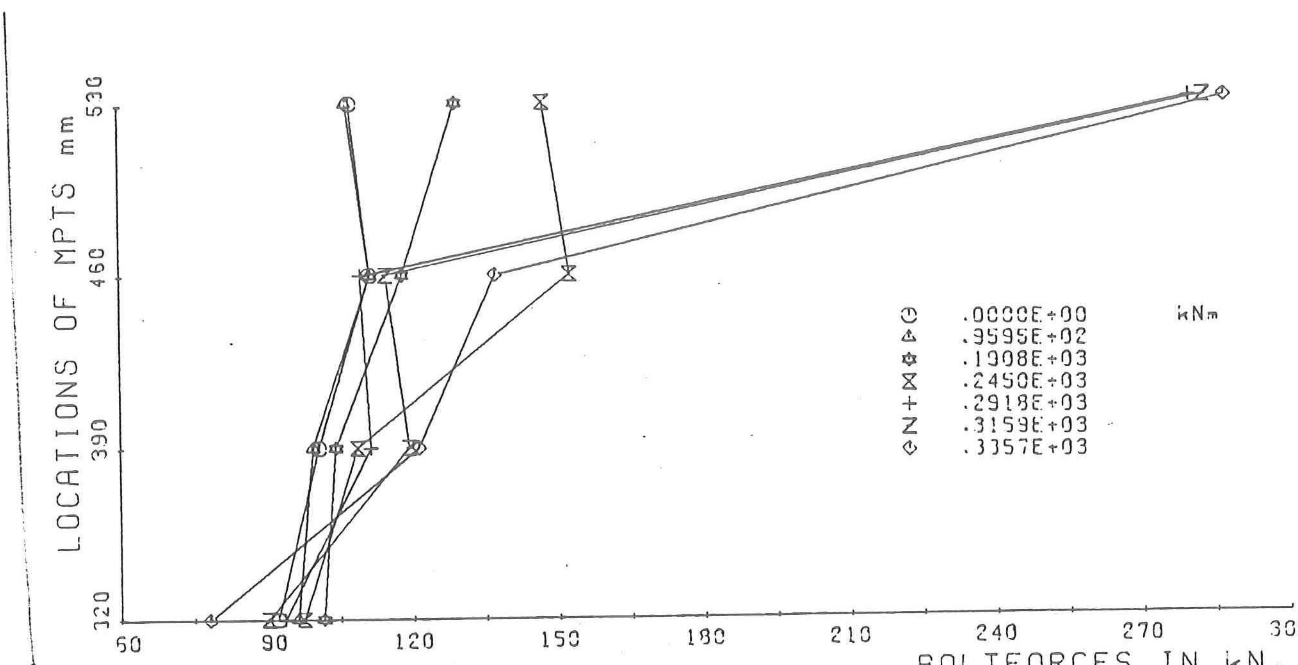


Fig. A2.39: Test 4

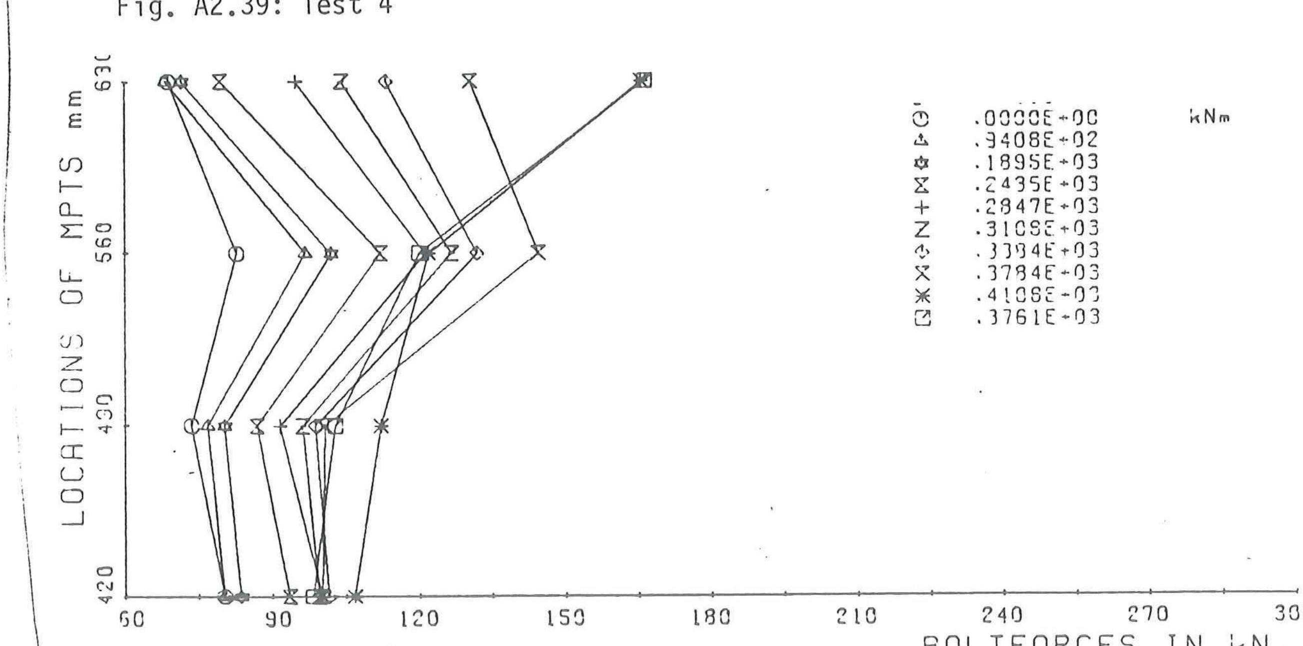


Fig. A2.40: Test 5

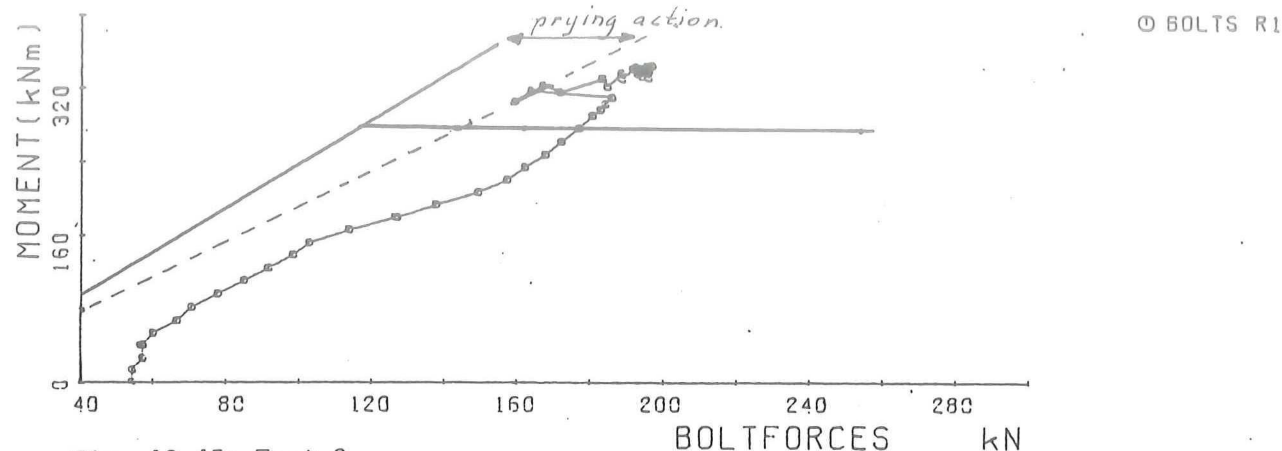


Fig. A2.45: Test 2

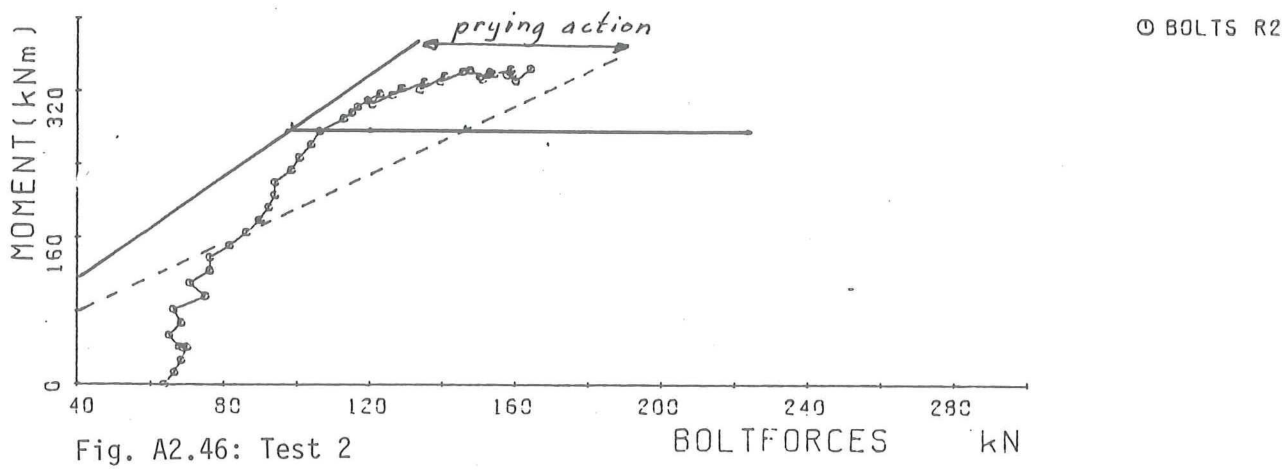


Fig. A2.46: Test 2

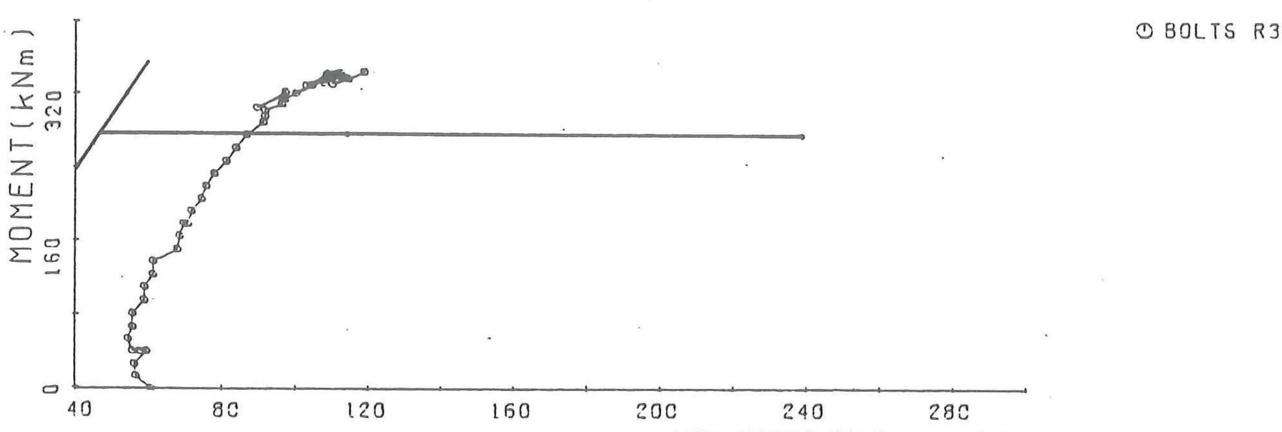


Fig. A2.47: Test 2

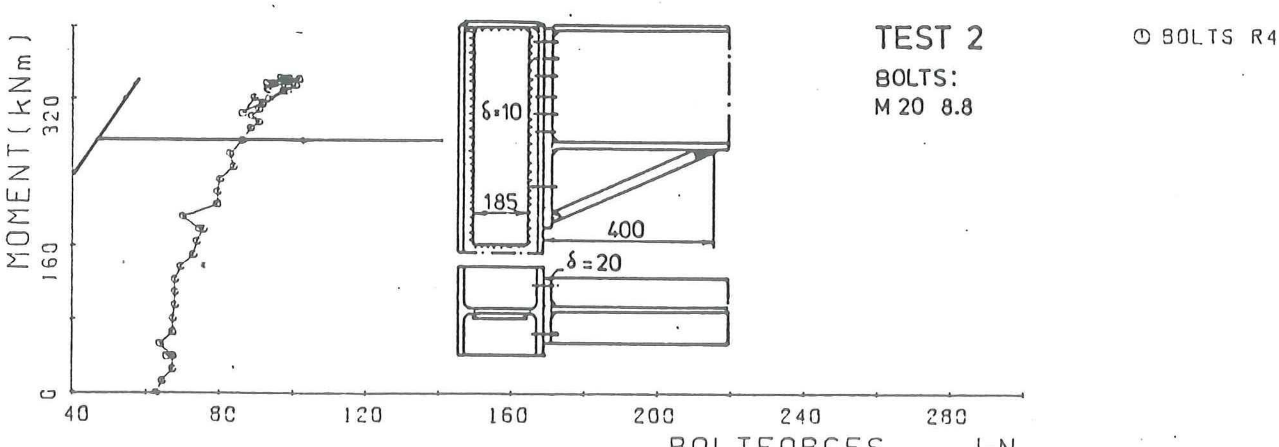


Fig. A2.48: Test 2

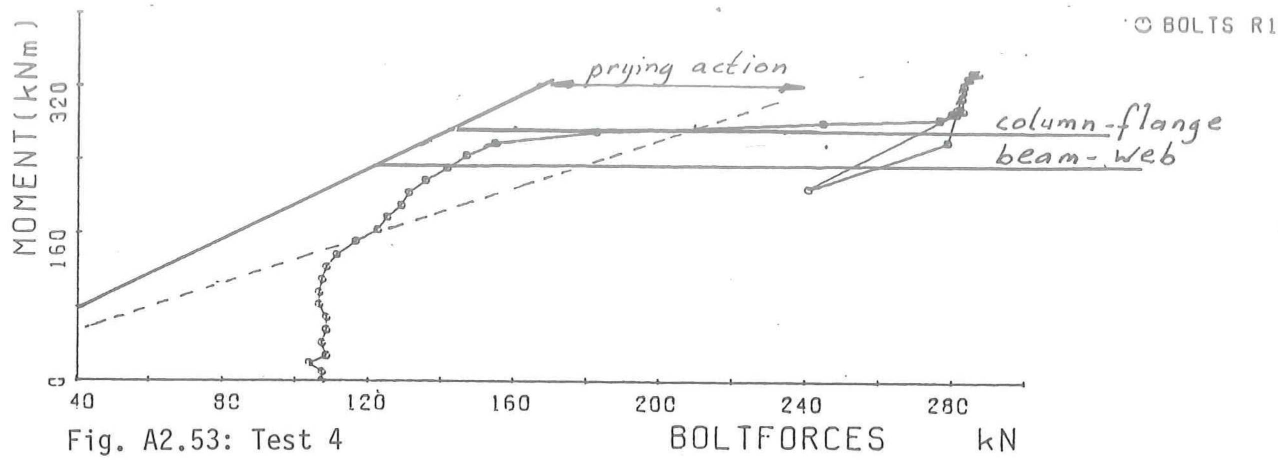


Fig. A2.53: Test 4

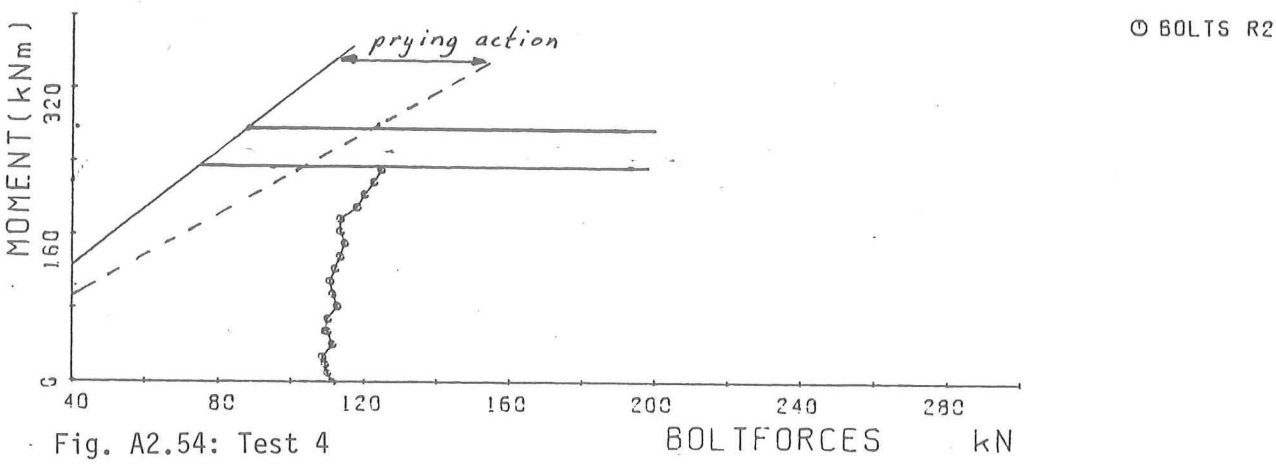


Fig. A2.54: Test 4

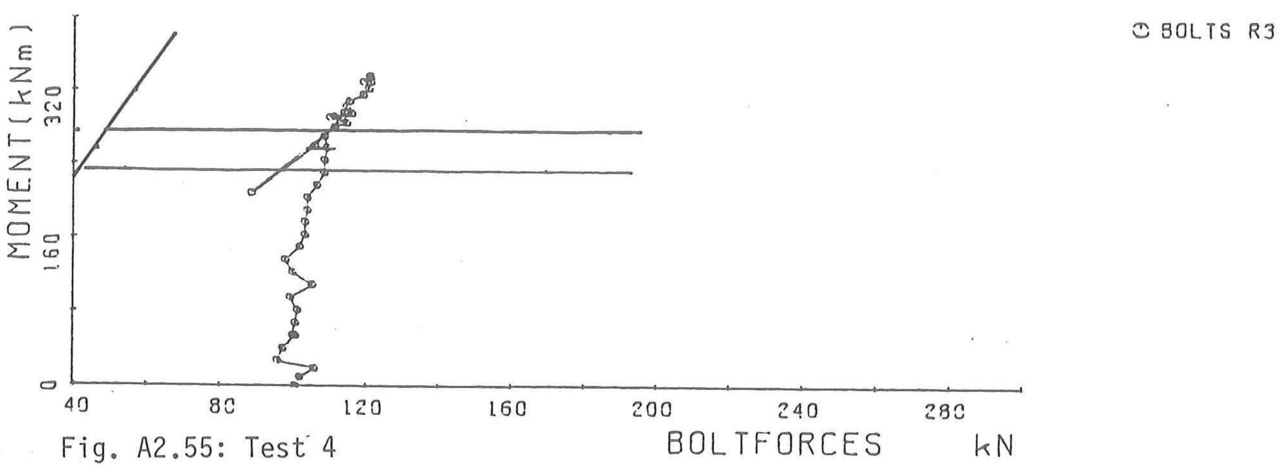


Fig. A2.55: Test 4

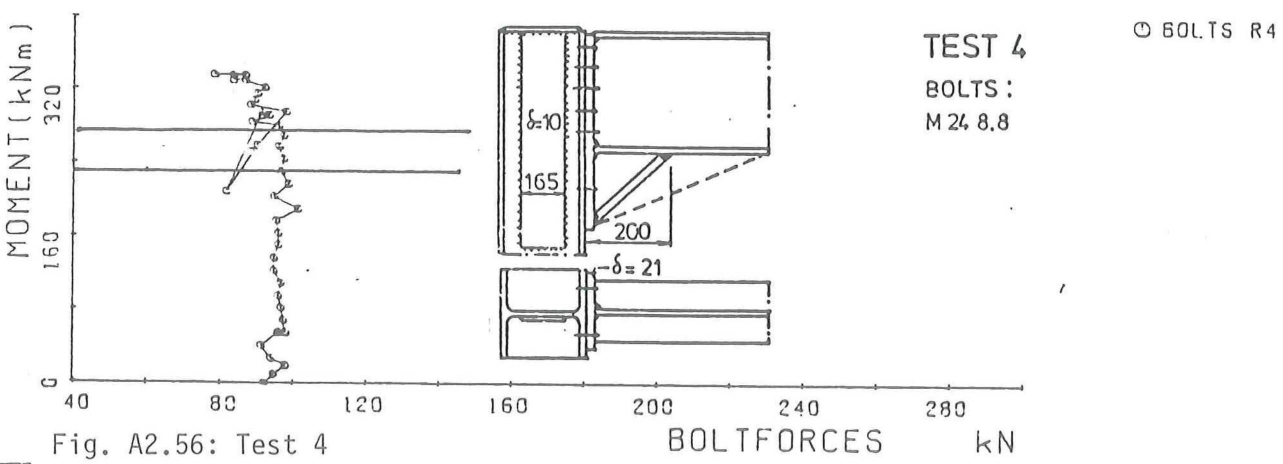


Fig. A2.56: Test 4

Appendix A3:

Comparison of the measured rotations with computed rotations.

The rotation between the measuring points is composed of the following rotations (see figure A 3.1.):

- over the part of the column ϕ_k
- over the connection ϕ_v
- over the haunched part of the beam ϕ_c
- over the part of the beam ϕ_b

The rotation over the connection ϕ_v is computed with formula (26) in appendix A 1.

Here a comparison of the total rotations will be made to check the adequacy of the stiffness formula (26).

It is assumed that the rotations over the parts of column and unhaunched beam may be computed with the theory of elasticity, thus:

$$\phi_x = \frac{M_x dx}{EI} \qquad \text{(A 3.1)}$$

where: I is moment of inertia of column or beam.

Formula (A 3.1) is also used for the computation of the rotation over the haunched part of the beam, however, with a variable value I_x.

The variable I_x is approached with:

$$I_x = \left(\frac{h_x}{h}\right)^2 I$$

where: h_x, x and h are the parameters as shown in figure A 3.1.

The rotations are computed over the distances as shown in the same figure. It is assumed that the rotation over the part $\frac{1}{2}h_k$ is incorporated in rotation ϕ_v.

The locations of the measuring points are given in figure 6.

The computational results are tabulated . The first table A 3.1. gives the factors with which M_v should be multiplied to get the rotations over the parts which behaves elastically, whereas the factor of ϕ_v should be multiplied with the quadratic value of M_v to get the rotation over the connection (The dimension of M_v = kNm). Thus:

$$\phi_{sum} = (c_k + c_c + c_b)\frac{M_v}{10^6} + c_v \frac{M_v^2}{10^6 \hat{M}_v} \qquad \text{or}$$

Test	C_{rigid}	C_k	C_c	C_b		C_v	
1	17,99	9,76	2,59	0,83		24,38	
2	22,66	11,93	4,86	0,74		13,19	
3	22,66	11,93	4,86	0,74	$* \dfrac{M_v}{10^6}$	39,10	$* \dfrac{M_v^2}{10^6 \hat{M}_v}$
4	19,53	11,93	2,59	0,83		27,30	
5	28,49	11,52	3,18	1,39		20,40	

Table A 3.1.: Factors with which the rotations have been computed.

M_v kNm	ϕ_{rigid} 10^{-3}rad	$\phi_k + \phi_c + \phi_b$ 10^{-3} rad.	ϕ_v 10^{-3} rad	ϕ_{sum} 10^{-3}rad	ϕ_{actual} 10^{-3}rad	$\dfrac{\phi_{sum}}{\phi_{actual}}$	$\dfrac{\phi_{rigid}}{\phi_{actual}}$
100	1,80	1,32	1,22	2,54	3,2	0,79	0,56
120	2,15	1,58	1,75	3,33	5,9	0,56	0,36
140	2,52	1,84	2,39	4,23	7,90	0,53	0,32
160	2,88	2,11	3,12	5,23	10,00	0,52	0,29
180	3,24	2,37	3,94	6,31	12,00	0,53	0,27
200	3,60	2,64	4,87	7,51	14,75	0,51	0,24

Table A 3.2 : Rotation of test 1 at various bending moments of the connection

M_v kNm	ϕ_{rigid} 10^{-3}rad.	$\phi_k + \phi_c + \phi_b$ 10^{-3} rad.	ϕ_v 10^{-3} rad.	ϕ_{sum} 10^{-3}rad	ϕ_{actual} 10^{-3}rad.	$\dfrac{\phi_{sum}}{\phi_{actual}}$	$\dfrac{\phi_{rigid}}{\phi_{actual}}$
142	3,29	2,54	0,92	3,46	3,12	1,11	1,05
170	3,94	3,05	1,33	4,38	4,25	1,03	0,93
198	4,60	3,56	1,81	5,37	5,25	1,02	0,88
226	5,26	4,07	2,36	6,43	6,50	0,99	0,81
255	5,91	4,58	2,99	7,57	8,50	0,89	0,70
283	6,57	5,08	3,73	8,77	11,00	0,80	0,60

Table A 3.3 : Rotations of test 2

This may be seen in the moment-rotation curve at a moment of about 120 kNm.
With the other tests special attention was given to this point and there no gaps existed.
The rotations ϕ_{rigid} are computed with the assumption that the beam and column are connected rigidly, neglecting the presence of the haunch and only taking into account the rotations due to bending along the system axes.
It appears that this way of computation gives a better approximation for test 5 than the use of formula (26).
Testspecimen 5 behaves as a rigid connection. The prediction of the rotation of the other tests is rather disappointing.
But this may be expected as already explained in report 6-81-15, because no special attention is given to the tightening of the bolts (no pre-tension) and the location of the contactforce.
The question may arise whether a deviation of the connection stiffness has a large influence on the prediction of the overal behaviour of the structure.
That is why this question is researched for a beam in a braced frame as shown in figure A 3.2. The beam is loaded by a distributed load q. The resistance of the connection is the moment M and the deflection in the mid-span is f. The rotation in the connection is kM.
It can be proved that:

$$M = \frac{q l^3}{12\ EI\left(\frac{l}{EI} + 2k\right)} \quad \text{and}$$

$$f = \frac{q\ l^4}{384\ EI}\ \left\{\ 5\ l^4 - \frac{4 l^5}{EI\ \left(\frac{l}{EI} + 2k\right)}\right\}$$

In the graphs in figure A 3.2. the factors $\frac{12\ M}{q l^2}$ and $\frac{384\ EI}{q l^4}\ f$ are given as a function of the factor $\frac{kEI}{l}$.

Where: $\frac{EI}{l}$ = the stiffness-rate of the beam and
k = the stiffness-rate of the connection.

Table A 3.8 gives the ratios between the actual and the assumed bending moments and deflections, when a mistake of a factor 2 exists in the prediction of k.

Generally it may be said that the mistake in the bending moment is less than 40% and in the deflection less than 30% if the deviation in the stiffness factor is 100%.

The mistake in the bending moment decreases with a decreasing value of $\frac{k\,E\,I}{l}$.

A decreasing value $\frac{k\,E\,I}{l}$ means an increasing connection stiffness with respect to the beam stiffness.

$\frac{k\,E\,I}{l}$ $k = k_{assumed}$	$\frac{k_{actual}}{k_{assumed}} = 2$ $\frac{M_{actual}}{M_{assumed}}$	 $\frac{f_{actual}}{f_{assumed}}$	$\frac{k_{actual}}{k_{assumed}} = \frac{1}{2}$ $\frac{M_{actual}}{M_{assumed}}$	 $\frac{f_{actual}}{f_{assumed}}$
0.01	0.98	1.06	1.01	0.97
0.02	0.96	1.12	1.02	0.94
0.04	0.93	1.20	1.03	0.88
0.08	0.88	1.28	1.07	0.84
0.10	0.86	1.32	1.09	0.81
0.20	0.77	1.30	1.16	0.76
0.3	0.72	1.27	1.23	0.76
0.4	0.68	1.25	1.29	0,77
0.5	0.66	1.22	1.32	0,78
0.6	0.64	1.21	1.38	0.79
0.7	0.63	1.19	1.40	0.80

Table A 3. 8 : Deviations of deflections and bending moments with a mistake of 100% in the connection stiffness.

Appendix A4

Computation of tests 1 to 5 inclusive of report 6-81-15 in accordance with the design method described in report 6-82-7.

Test 1-15

Column flange at first boltrow

Because no stiffeners are used, the formulae of the unstiffened column flange are valid 6 .

$$b_m = p + 2m + 1{,}25n' = 60 + 4 * 28{,}9 + 1{,}25 * 95 = 294 \text{ mm}$$

$$\hat{F}_{1f} + \hat{F}_{2f} = \frac{294 * 13^2 * 262}{28{,}9} = 450 \text{ kN} \qquad (11)\ (11)$$

$$\hat{F}_{1f} + \hat{F}_{2f} = \frac{294 * 13^2 * 262 + 8 * 0{,}7 * 250000 * 1{,}25 * 28{,}9}{2 * 2{,}25 * 28{,}9} = 489 \text{ kN} \qquad (12)$$

Column flange at third boltrow

$p = 60$ mm

This boltrow is situated within the effective length of the second boltrow, because $p = 60$ mm and the effective length $b_m = 2m + 0{,}625n' = 117$ mm; thus the overlap is 60 mm.

Hence:

$$\hat{F}_{3f} = \frac{60 * 13^2 * 262}{28{,}9} = 92 \text{ kN}$$

provided that $2m + 0{,}625n' = 117 \quad \frac{1}{2}h_3 = \frac{1}{2} \cdot 410 = 205$ mm

Column flange at 4th and 5th boltrow

The same situation as is valid for boltrow 3, because:

$b_m = 117$ mm $\quad \frac{1}{2} \cdot h_4 = \frac{1}{2} * 350 = 175$ mm and

$b_m = 117$ mm $\quad \frac{1}{2} \cdot h_5 = \frac{1}{2} * 290 = 145$ mm

Column web on the tension side

The effective length behind 5 boltrows is

$t = b_m + 4 * p = 294 + 4 * 60 = 534$ mm.

Thus $F_i \quad 534 * 9 * 298 = 1432$ kN is o.k.

Appendix A4 Test 1-15 (6-81-15)

$$\hat{F}_{2e} = \frac{(15.4 + 113)\ 15.3^2 * 270 + 4 * 0.7 * 250000 * 35}{2(45.7 + 35)} = 202 \text{ kN} \qquad (12)$$

Thus: $\hat{F}_{2e}$ = 178 kN and this is smaller than

$\hat{F}_{2f}$ = 225 kN thus $\hat{F}_2$ = 178 kN

End plate at other boltrows

p = 60 mm and 2m + 0.625n' = 113 mm

The overlap is 60 mm, thus:

$$\hat{F}_{3f} = \frac{60 * 15.3 * 270}{45.7} = 83 \text{ kN} = \hat{F}_{4f} = \hat{F}_{5f}$$

From report 6-81-15 it appears that the beam web is not the determining factor, thus:

$\hat{F}_{1f} = 225$	$\hat{F}_{1e} = 250$ kN	→	$\hat{F}_1 h_1$	= 225 * 0.53 = 119
$\hat{F}_{2f} = 225$	$\hat{F}_{2e} = 178$ kN	→	$\hat{F}_2 h_2$	= 178 * 0.47 = 84
$\hat{F}_{3f} = 92$	$\hat{F}_{3e} = 83$ kN	→	$\hat{F}_3 h_3$	= 83 * 0.41 = 34
$\hat{F}_{4f} = 92$	$\hat{F}_{4e} = 83$ kN	→	$\hat{F}_4 h_4$	= 83 * 0.36 = 30
	$\hat{F}_{5e} = 83$ kN	→	$\hat{F}_5 h_5$	= 29 * 0.31 = 8
			ΣF_i	= 598 kN m_v = 274 kNm

Bending moment determined by the haunch dimensions of test 1-15

Tension side formula (27)

$$\frac{\hat{M}_c}{\hat{M}_e} = \frac{0.58*9.5(200*290+2*15.3*270) + \dfrac{180*15.3^2 * 270}{4(60-6.1)}}{180*12.2*256+9.5(60-6.1)\ 256} = 0.60 \qquad (27)$$

$$\hat{M}_e = W_e * \sigma_y = 1160 * 10^3 * 256 = 297 \text{ kNm}$$

$\hat{M}_c = 0.60 * 297 = 279$ kNm $\qquad \hat{M}_v = \dfrac{1.95}{1.95-0.2} * 179 = 199$ kNm

Appendix A4 Test 1-20 (6-81-15)

The limit state design moment determined by the column side is the same as in test 1-15,3.

End plate at first boltrow

$t_e = 20$ mm $\sigma_y = 266$ N/mm^2 (see table 1, 6-81-15)

m_1, m_2, λ_1 and λ_2 equal to test 1-15.3

$$\hat{F}_{1e} = \frac{9.5 * 20^2 * 266}{2} = 505 \text{ kN} \qquad (4a)$$

$$\frac{\hat{F}_{1e}}{2\hat{B}_t} = \frac{505000}{2 * 0.7 * 210000} = 1.71$$ (see curve b, on page 81 of report 6-81-15)

Because $\lambda_1 > 0.5$; $f_{pr} = \frac{4}{2 + 1.71} = 1.08$

$$\hat{F}_{1e} = \frac{2 * 0.7 * 210}{1.08} = \frac{294}{1.08} = 273 \text{ kN} \qquad (2)$$

The conclusion is that this end plate does not give rotational capacity.

End plate at second boltrow

The same configuration as for test 1-15, thus:

$$\hat{F}_{2e} = \frac{(15.4 + 113)\ 20^2 * 266}{45.7} = 299 \text{ kN} \qquad (11)$$

$$\hat{F}_{2e} = \frac{(15.4 + 113)\ 20^2 * 266 + 4 * 0.7 * 210000 * 35}{2\ (45.7 + 35)} = 212 \text{ kN} \qquad (12)$$

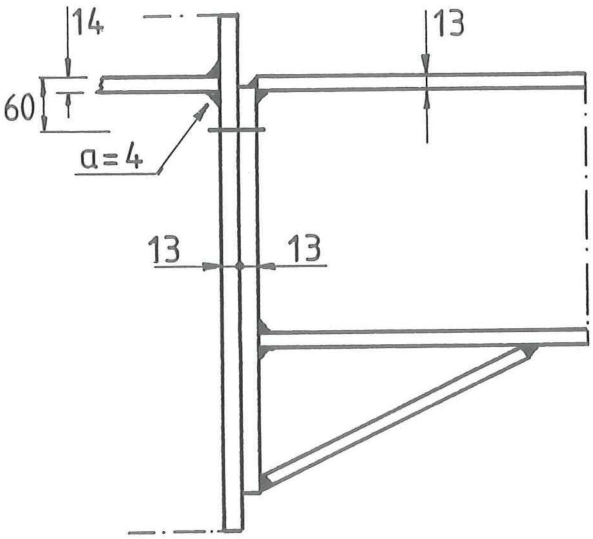
Because the column flange yields at $\hat{F}_{1f} = 225$ kN, the column flange gives sufficient deformation capacity and formula (13) is not required.

End plate at other boltrows

$$\hat{F}_{1f} = \frac{60 * 20^2 * 266}{45.7} = 140 \text{ kN}$$

Test 2-13

Column flange at first boltrow



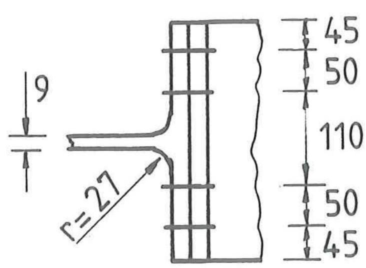
m_1 = 23.5 mm
m_2 = 40.3 mm
λ_1 = 0.2
λ_2 = 0.34

see 6-81-15, page 88

$\hat{F}_{1f} = 8\pi . \frac{1}{4} . 13^2 . 262 = 278$ kN

$\frac{\hat{F}_{1f}}{2\hat{B}_t} = \frac{278}{350} = 0.79; \lambda_1$ and $\lambda_2 < 0.5$

Formula (4a) governs because $\frac{\hat{F}_{1f}}{2\hat{B}_t} < 0.8$



$m_{1,2}$ = 23.5 mm

$b_m = b - w2 - 2m_{1,2} = 300\text{-}110\text{-}2*23.5 = 143$ mm

$$\hat{F}_{1ft} = \frac{2*143*13^2*262}{4*40.3} = 79 \text{ kN} \qquad (5)$$

$$\hat{F}_{1ft} = \frac{143 * 13^2 * 262 + 10 * 0.7 * 250000 * 40.3}{9 * 40.3} = 212 \text{ kN} \qquad (6)$$

$\hat{F}_{1som} = 278 + 79 = 356$ kN

Columnflange at second boltrow

$m_{1,2}$ = 23,5 because $\alpha = 4\pi$ and $m_1 < m_2$ (see formula (11))

$p - m_{1,2}$ = 60 - 23.5 = 36,5 mm

m = 23.5 + 0.2 * 27 = 28.9 mm

b_m = 2m + 0.625n' = 2 * 28.9 + 0.625 * 95 = 117 mm

$$\hat{F}_{2f} = \frac{(36.5 + 117)\ 13^2 * 262}{28.9} = 235 \text{ kN} \qquad (11)$$

$$\hat{F}_{2f} = \frac{36\text{-}5 + 117)\ 13^3\ 262 + 4 * 0.7 * 250000 * 1.25 * 28.9}{2\ (28.9 + 1.25 * 28.9)} = 246 \text{ kN} \qquad (12)$$

Test 2-13

End plate at second boltrow

$m_{1,2}$ = 41.3 mm
m = 44.6 + 0.2 * 4 2 = 45,7 mm
p = 60 mm
b_m = 60 - 41.3 + 2 * 45.7 + 0.625 * 95 = 169 mm

$$\hat{F}_{2e} = \frac{169 * 13^2 * 270}{45.7} = 169 \text{ kN} \qquad (11)$$

$$\hat{F}_{2e} = \frac{169 * 13^2 * 270 + 4 * 175000 * 1.25 * 45.7}{2(45.7 + 1.25 * 45.7)} = 232 \text{ kN} \qquad (12)$$

Formula (13) is not valid here, because the column flange gives sufficient deformation capacity at $\hat{F}_{1f}$ = 278 kN

End plate at other boltrows

p = 60 mm

$$\hat{F}_{ie} = \frac{60 * 13^2 * 270}{45.7} = 60 \text{ kN} \qquad (11)$$

However, only when $\frac{1}{2}h_i$ > 2 * 45.7 + 0.625 * 95 = 150 mm and this does not occur with these boltrows.

Conclusion:

$\hat{F}_{1som}$ = 278 + 79 kN $\hat{F}_{1som}$ = 280 + 45 kN (278 + 45) * 0.37 = 120 kNm

$\hat{F}_{2f}$ = 235 kN $\hat{F}_{2e}$ = 169 kN; 169 * 0.31 = 53 kNm

$\hat{F}_d$ = 598 kN   500 kN $\hat{M}_v$ = 175 kNm

Bending moment determined by the haunch dimensions of test 2-13

t_f = 13 mm σ_y = 266 N/mm^2 y = 60 - 6.5 = 53.5 mm
t_w = 9 mm σ_y = 295 N/mm^2 r = 27 mm
t_e = 13 mm σ_y = 270 N/mm^2

$\text{tg}\ \alpha = \frac{150}{270} = 0.555 \rightarrow \alpha = 29.05$ $\text{ctg}\ \alpha$ = 1,8

$\hat{M}_e = w_e * \sigma_e$ = 1260 * 10^3 * 266 = 335 kNm

End plate at first boltrow of test 2-20

$t_e = 20$ mm $\quad \sigma_y = 266$ N/mm^2

$\hat{F}_{1e} = 8 \ \frac{1}{4} \ 20^2 * 266 = 669$ kN

$\frac{\hat{F}_{1e}}{2\hat{B}_t} = \frac{669}{294} = 2.27 \quad 2$ formula (3) governs at the end plate side

$$\hat{F}_{1et} = \frac{107.4 * (13^2 * 266 + 20^2 * 266)}{4 * 53.5} = 76 \text{ kN} \tag{8}$$

$$\hat{F}_{1et} = \frac{2 * 107.4 * 13^2 * 266}{4 * 41.3} = 58 \text{ kN} \tag{5}$$

Thus: $\hat{F}_{1som} = 243 + 58 = 293$ kN, because

$\hat{F}_{1f} = 243$ kN and $\hat{F}_{1et} = 58$ kN.

End plate at second boltrow of test 2-20

$$\hat{F}_{2e} = \frac{169 * 20^2 * 266}{45.7} = 393 \text{ kN} \tag{11}$$

$$\hat{F}_{2e} = \frac{169 * 20^2 * 266 + 4 * 147000 * 1.25 * 45.7}{2(45.7 + 1.25 * 45.7)} = 251 \text{ kN} \tag{12}$$

$\hat{F}_{2e} = \frac{370 - 60}{370}$ 294 = 246 kN, but this is not valid, because $\hat{F}_{2f} = 243$ kN

Conclusion: Test 2-20 fails at a moment $\hat{M}_v = 184$ kNm due to failure of the corner bolt with prying action.

Appendix A4 test 3-18 (6-81-15)

Column web is o.k.

See table 2, page 21 report 6-81-15.

$\hat{F}_d = 1020$ kN

End plate at first bolt row of test 3-18

$t_e = 18$ mm $\quad \sigma_y = 266$ N/mm^2

$m_1 = 49.6$ mm

$m_2 = 50.3$ mm $\quad$ see 6-81-15, page 91

$\lambda_1 = 0.5 \quad \lambda_2 = 0.5$

$\alpha = 9.9$

$\hat{F}_{1e} = 2 * 9.9 * \frac{1}{4} * 18^2 * 266 = 426$ kN

$\frac{\hat{F}_{1e}}{2\hat{B}_t} = \frac{426}{470} = 0.906$, because and $= 0.5$

$f_{pr} = \frac{6}{4 + 0.906} = 1.21$

$\hat{F}_1 = \frac{470}{1.21} = 388$ kN

End plate at second bolt row of test 3-18

$m = 44.6 + \frac{1}{5} * 4\sqrt{2} = 45.7$ mm

$m_{1,2} = 50.3$ mm

$p - m_{1,2} = 70 - 50.3 = 19.7$ mm

$2m + 0.625n' = 2 * 45.7 + 0.625 * 50 = 122.6$ mm

$$\hat{F}_{2e} = \frac{(19.7 + 122.6) \quad 18^2 \quad 266}{45.7} = 268 \text{ kN} \qquad (11)$$

$$\hat{F}_{2e} = \frac{(19.7 + 122.6) \quad 18^2 * 266 + 4 * 235000 * 50}{2\,(45.7 + 50)} = 310 \text{ kN} \qquad (12)$$

Formula (13) is not valid because formula (2a) governs at the first boltrow but also because the column flange gives sufficient deformation.

Test 3-21

The only difference with test 3-18 is caused by the end plate thickness for the third and second boltrow.

End plate at second boltrow of test 3-21

$$\hat{F}_{2e} = \frac{(19.7 + 122.6)\ 21^2 * 310}{45.7} = 426\ \text{kN} \tag{11}$$

$$\hat{F}_{2e} = \frac{(19.7 + 122.6)\ 21^2 * 310 + 4 * 0.7 * 326000 * 50}{3(45.7 + 50)} = 340\ \text{kN} \tag{12}$$

Conclusion test 3-21

$\hat{F}_{1f}$	= 278	$\hat{F}_{1e}$	278	$\hat{F}_1 h_1$	= 278 * 0.52 = 145 kNm
$\hat{F}_{2f}$	= 278	$\hat{F}_{2e}$	= 340	$\hat{F}_2 h_2$	= 278 * 0.45 = 125 kNm
$\hat{F}_{3f}$	= 216	$\hat{F}_{3e}$	= 209	$\hat{F}_3 h_3$	= 209 * 0.38 = 79 kNm
$\hat{F}_{4f}$	= 91	$\hat{F}_{4e}$	= 209	$\hat{F}_4 h_4$	= 91 * 0.31 = 28 kNm
$\hat{F}_{5f}$	= 0	$\hat{F}_{5e}$	= 0	$\hat{F}_5 h_5$	= 0 * 0.24 = 0 kNm
		F_d	= 1020 kN	856	M_v = 377 kNm

Bending moment determined by the haunch dimensions

In corporation with test 3-18 it is evident that this part of the connection has sufficient strength capacity because the end plate is thicker than in test 3-18.

Column flange at the first and second boltrow

The column flange has no stiffeners, thus the design method of the unstiffened column flange is valid.

$$m = \frac{120 - 9.5 - 2 * 0.8 * 27}{2} = 33.9\ \text{mm}$$

$n' = 1.25 * 33\ 9 = 42.4$ mm

$b_m = 70 + 4 * 33.9 + 1.25 * 90 = 70 + 248 = 318$ mm

Appendix A5

DESIGN TABLES OF REPORT 6-82-7