Tests on hinged connections for non-sway continuous frames

Tests on Hinged Connections for Non-Sway Continuous Frames

Essai de noeuds articulés pour ossatures contreventées

Versuche an gelenkigen Knoten für verstrebte Skelette

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1. INTRODUCTION.

Most of multi-story frames built at present have either a rigid central concrete core, or a diagonal bracing to resist wind forces, so that these structures may be designed as non-sway structures. They usually have rigid connections between beams and columns.

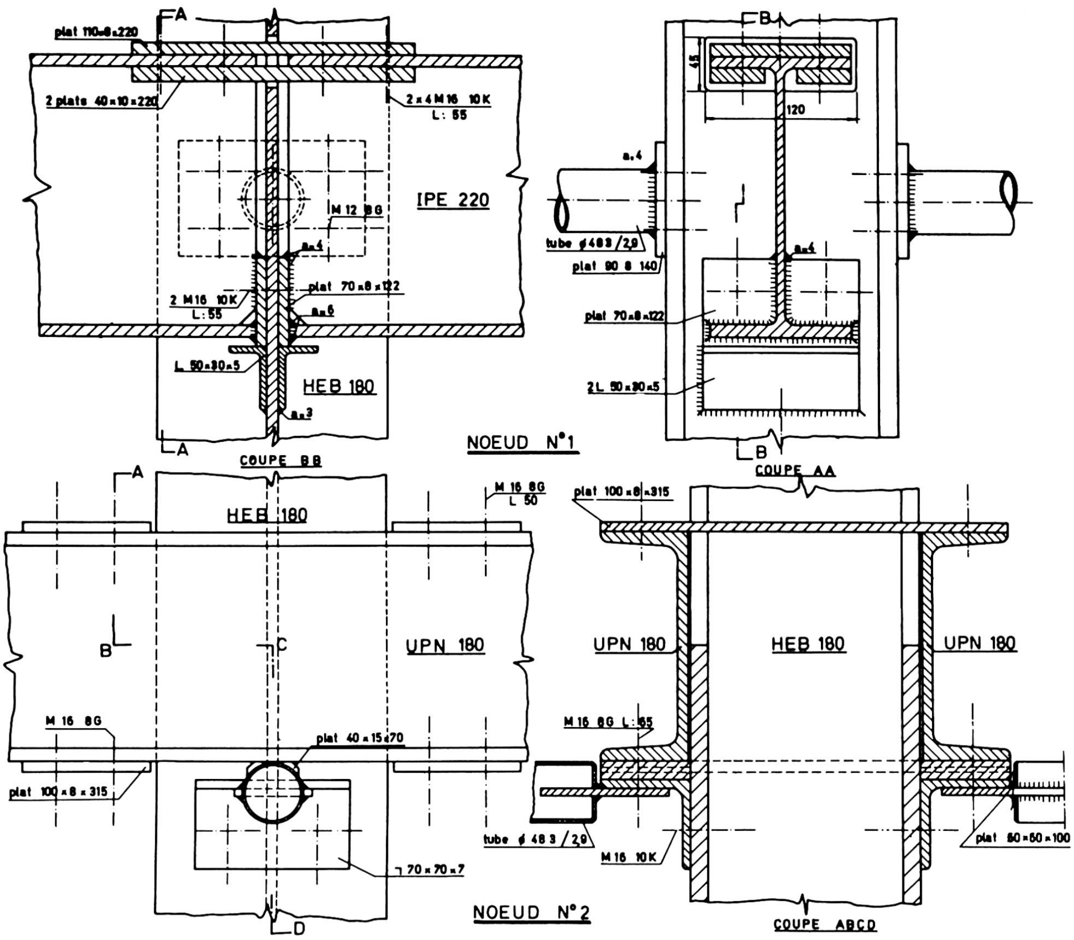
Research conducted in the United States, in Great Britain, and elsewhere have made possible the plastic analysis of such structures on a practical basis. In particular, the Lehigh team has developed a method solving the problem of verifying the rotational capacity of joints, and analysing the buckling of columns, by means of "column deflection curves", obtained theoretically for current shapes.

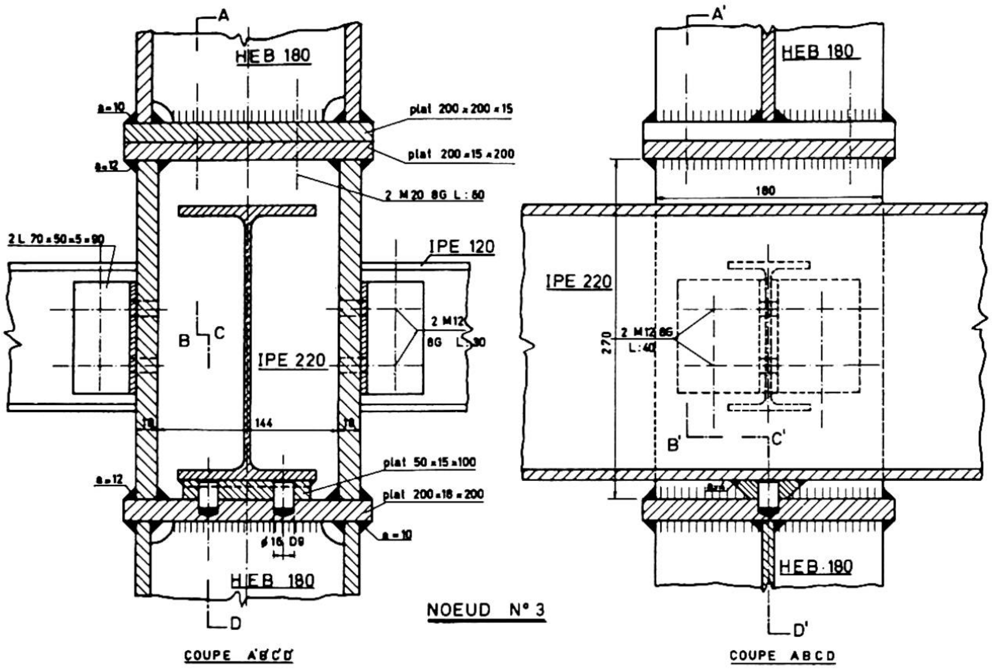
A simplification of this type of structure can be obtained by hinging continuous beams on continuous columns. The work of the designer is greatly reduced: beams are treated as continuous beams, either in plastic or elastic design; and columns become axially loaded, without bending. Such structures are also interesting from an economical point of view as substantial savings can be made on the columns, relatively to rigid-joint design. Of course, this advantage is partly compensated by an increase of the size of the beams. Two students working on different structures at Liege University have shown that the point of equilibrium between the two conceptions lies around 7 stories; the hinged solution shows better under this figure. When the number of stories increases, the bending effect in columns of rigid-joint structures become less and less significant against axial loading and this last effect governs the design, as in the hinged solution, so that the savings made on beams make the rigid solution more economical.

Of course, a prior requirement for the hinged-joints design is that these hinges may be effectively fabricated. Another requirement is that their cost must not off-set the savings made on the columns.

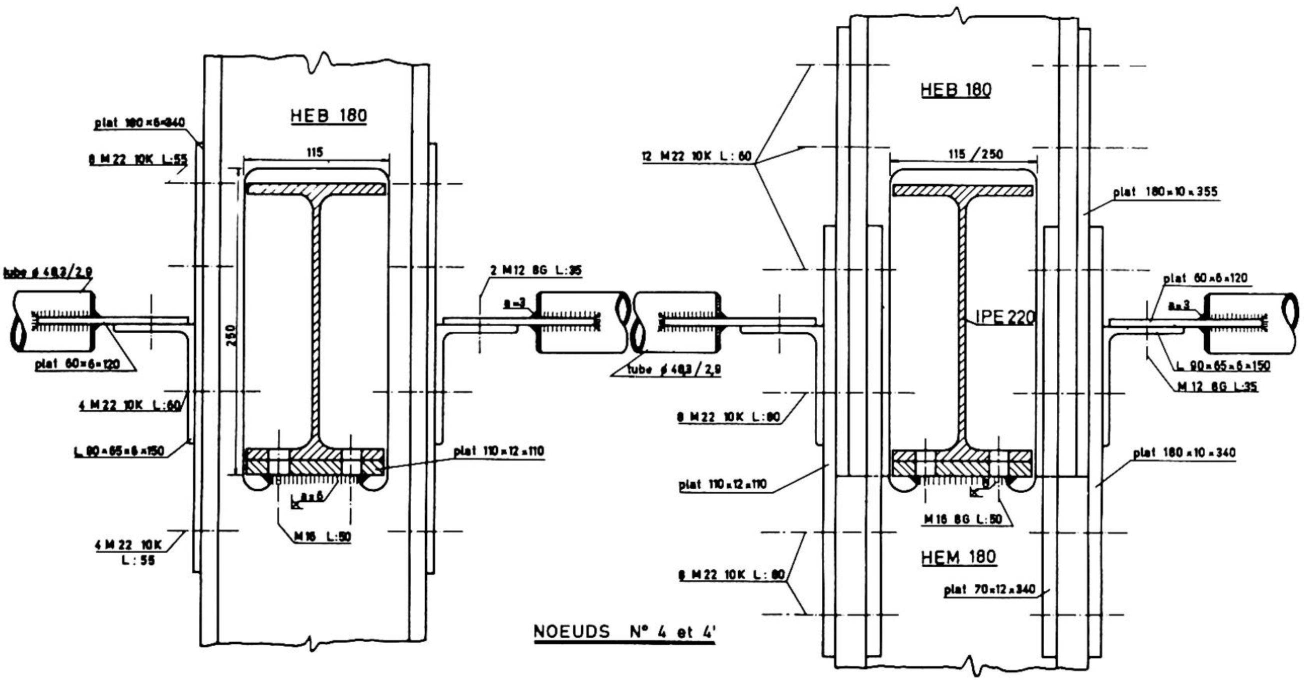
2. DESCRIPTION OF THE CONNECTIONS.

In order to check these two points, a research sponsored by the CECA (Commission Européenne du Charbon et de l'Acier) was undertaken by SERCoM (Station d'Essais et de Recherches de la Construction Métallique) in the laboratories of Liege University. Four types of joints were designed (Fig. 1 - 2 - 3 - 4) and one sample of each tested.


plat 110×8×220
2 plats 40×10×220
2×4 M16 10K
L: 55
IPE 220
M 12 8G
a=4
2 M16 10K
L: 55
plat 70×8×122
a=6
L 50×30×5
HEB 180
a=3
COUPE BB
NOEUD N°1
120
a=4
tube ϕ48.3/2.9
plat 90 8 140
plat 70×8×122
2L 50×30×5
COUPE AA
M 16 8G
L 50
HEB 180
UPN 180
M 16 8G
plat 40×15×70
plat 100×8×315
70×70×7
NOEUD N°2
plat 100×8×315
UPN 180
HEB 180
UPN 180
M 16 8G L: 65
tube ϕ 48.3/2.9
M 16 10K
plat 80×50×100
COUPE ABCD



NOEUD N° 3



NOEUDS N° 4 et 4'

N° 1 was developed from a scheme suggested by Romanian searchers working at Liège. The beam is cut at the joint and the two parts are placed on both sides of the web of the column, resting on small shoes. These are welded or bolted on the web of the column, and theoretically, they serve only during the assembly phase, for positioning the beams. On each half-beam, a small steel plate is welded on the lower flange and the bottom part of the web. They are joined together through the column by two high-strength bolts which insure transmission of the shear forces to the column.
Compression efforts in the lower flange are transmitted directly by contact, while tensile forces in the upper flanges are transmitted through coverplates going through a hole purposely made in the web of the column.
It is important to note that the plates on the webs of the beams must be as small as possible in height and the bolts as near as possible to the lower flange, in order to obtain a good rotational capacity of the beam. The design may be completed by shear stiffeners if needed, but they were not necessary in the tested model.

In N° 2 design, the beams are made of two U shaped profiles which pass on each side of the column. Each is supported by a small console, bolted to the flanges of the column. A small rocking skid is welded under the lower flange, to provide hinging. A bolt avoids translations. This system is quite simple, but the double-U beams are generally heavier than an equivalent I-beam, and they must be braced together against torsional effects proper to U-sections.

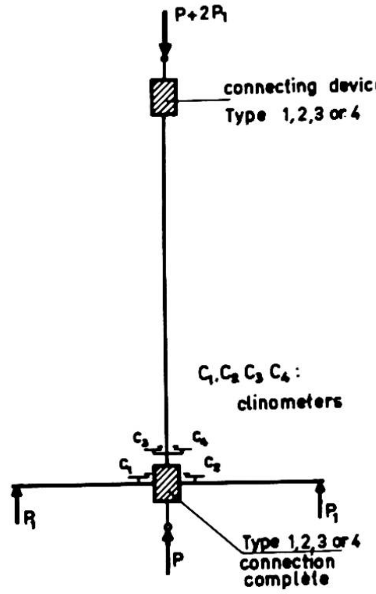
In type 3 design, the column is interrupted at joint level and provided with a hollow box, the walls of which must be able to resist the vertical forces coming from above. The beam goes uninterrupted through the box, hinged on a rocking skid and secured against translation by pins. The box is welded on top of the column section below, and the bottom of the above column section is bolted on top of the box. Of course, columns must not have a joint at each level, and they might come from workshop in lengths involving three or four boxes, in all-welded execution. It can be noticed here that, as the column buckles centrically, the orientation of the column does not matter, so it can be placed with its weak axis of inertia in the plane of the frame. In this way, the walls of the box are in the same plane as the flanges of the column, and the horizontal plates of the box may be relatively thin. If the column were placed with its strong axis in the plane of the frame, the walls and flanges would be orthogonal to each other and the horizontal plates would have to be very thick to provide a good transmission of forces. Anyway, troubles would occur at the corners of the flanges.
This design, though quite simple in its principle, involves some disagreements which grow with the size of the column: milling of thick plates, much welding, etc... and the erection of a large structure with these joints could lead to some difficulties because of the necessity of running the beams through the boxes.

Type 4 is in some way a digest of types 1 and 3: a large hole is cut in the web of the column and the beam passes through it.
The flanges are reinforced by cover plates to compensate for the hole. The beam rests directly on the edge of the web which is locally reinforced.
The small tongue of web so formed constitutes the hinge. As in type 3, the beam-column connection can easily be combined with a joint in the column.

Each of the above connections is completed by connections between the column and the secondary beams. This does not lead to any difficulty.

3. TESTING LAYOUT.

The testing layout was arranged according to the scheme described at fig. 5 : (Specimen is turned upside down from normal position):



a column section supports two connections; the one at the bottom is complete, with main and secondary beam; the one at the top has only the connecting device and the beams are omitted.
The loading is composed of :

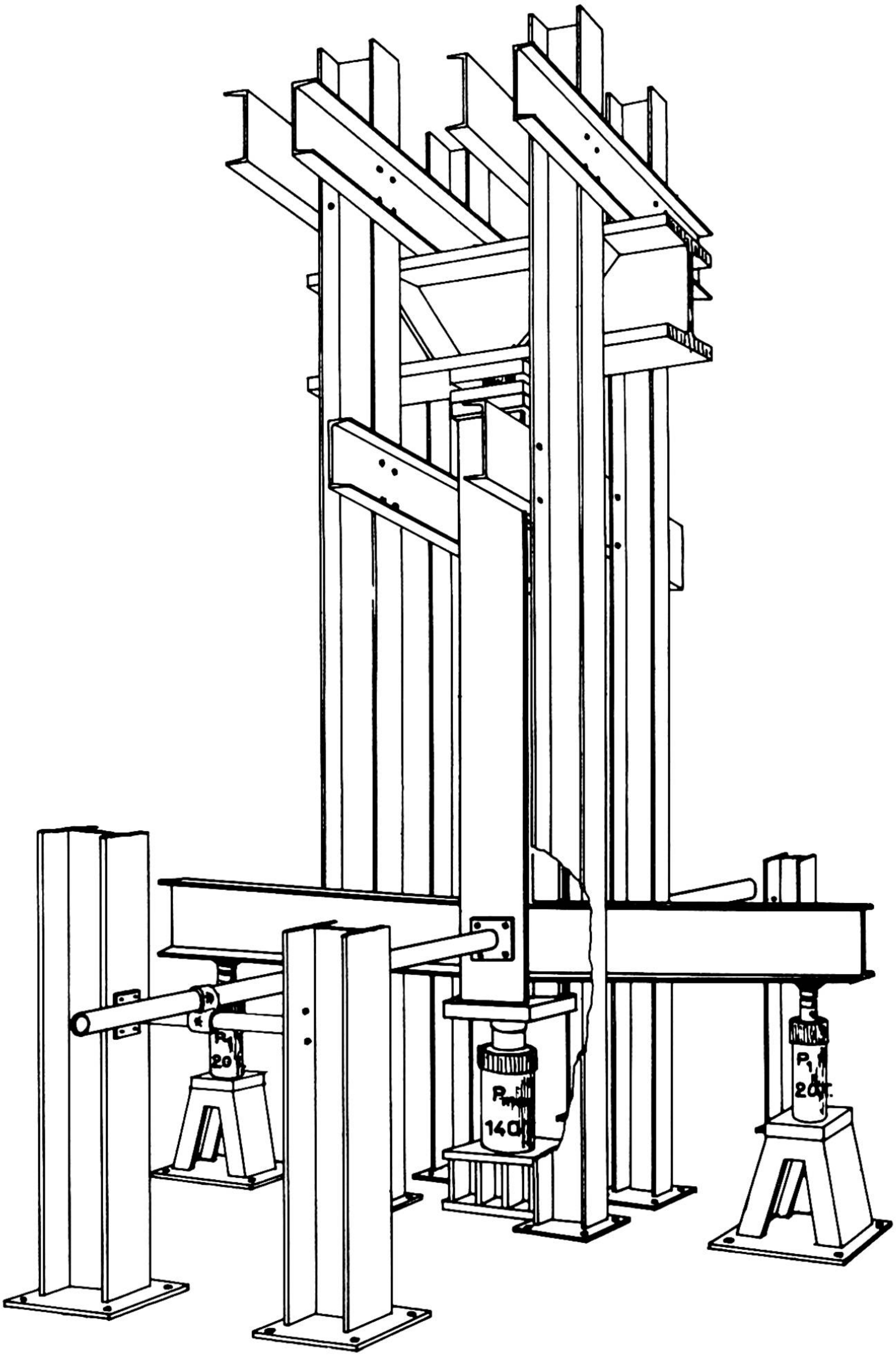
a) vertical loading on the column, to simulate the effect of the higher stories ;

b) vertical loading on the end of the main beam, to produce a negative moment in the beam at the right of the connection.

The column section is hinged at its two ends, with hinges placed as near as possible to the connection levels, in order to respect the buckling length.

The maximum load that could be applied with the testing equipment was 160 tons (metric), so it was decided to make the tests at a 6/10 scale.
The profile of the column was HEB 180, and for the beam: IPE 220 (and UPN180 for type 2). The distance between two levels was 2,10 m. The specimen so obtained represents a current column in a building of 8 x 4 m module, supporting loads of 750 kg/m^2.

There is little to say about the testing apparatus :
The specimens are place in a frame structure bolted to the concrete slab equipping the laboratory, and loads are applied by means of hydraulic jacks, placed under the column for the column load, and under the ends of the beam. Loads are controlled by strain-gages load cells. The apparatus is shown at fig. 6.


P_1
20
P_m
140
P_1
20T.

Testing for each specimen was conducted in three phases :

a) In phase 1, a small load is applied to the column and two equal forces are applied to the beam at different distances from the column. In this way, the rotational capacity of the "hinge" can be tested.
b) In phase 2, the loads on the beam, acting symetrically, are increased until the full plastic moment is developed in the beam. A small load is applied on the column.
c) In phase 3, the column loading is increased up to the collapse of the column or the maximum loading, whichever occurs.

During phase 1, measurement is made of the rotations of the beam and the column sections near the connection, by means of clinometers (see fig. 5).

During phases 2 and 3, only loads and the plastic rotation in the beam are measured.

4. EXPERIMENTAL RESULTS.

4.1. Phase 1 - Control of hinging action.

The table below gives the mean value of the rotations of the beam and the column during the phase 1 tests on the four types of connections.

The rotation was obtained by placing the beam jacks at 0.7 and 1.0 meter apart from the center of the joint.

For reasons of clarity, the values of the angular rotations ϕ_c and ϕ_b of the column and the beam, respectively, are given in decimal division of the degree .

The reference value is obtained by applying some load on the column (usually one ton) to get rid of errors due to the initial self-positioning of the different devices.

		Type 1		Type 2		Type 3		Type 4	
F_c Tons	F_b Tons	ϕ_b	ϕ_c	ϕ_b	ϕ_c	ϕ_b	ϕ_c	ϕ_b	ϕ_c
1.0	0			0	0	0		0	0
15.0	0	0	0	0.033	0.036	0.028	less than 0.01	0.06	0.06
15.0	0.5	0.13	0.015	0.46	0.038	1.79		0.54	0.06
15.0	1.0	0.36	0.018	0.98	0.051	3.00		2.50	0.02
15.0	2.0	0.62	0.056	2.26	0.085	***		▼	
15.0	3.0	0.89	0.086						
15.0	4.0	1.45	0.120						
15.0	5.0	* 2.60	0.206						
15.0	0	1.45 **	0.016	0.28	0.037				

* When this value was reached, web of beam was abuting against web of column.
** Actually, F_c for this tests was not maintained constant, but gradually increased from 15 tons (F_b = 0) to 22 Tons (F_b = 5) and back to 15 tons.

*** The freedom of rotation was such that no state of equilibrium could be obtained beyond 0.5 T. The 3.00 deg. value was reached before 1.0 T was acting as F_b.

▼ Test was stopped at this point because farthest jack was out of stroke.

As can be seen from above results, all four types of connections behave satisfactorily, and have a good rotational ability. As could be predicted, type one is the stiffest of the four.
The rotation of the beam occurs without inducing significant rotation in the column, excepted for the "stiff" type one. But even for the latter, the rotation of the column does not exceed ten percent of that of the beam. Furthermore, some part of the column's rotation may be a result of the increasing of the load on the column, as the whole specimen is hinged at its two ends.
The lower hinge is constituted by the jacks themselves, and for kinematical reasons its center of rotation is the center of the lower face of the corresponding bearing plate.

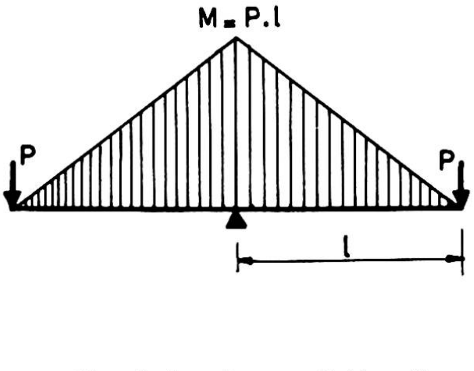
So, it may be concluded that the four proposed types can be considered as real "hinged" connections. The relative rotation of more than 2 degrees between beam and column is far beyond service values.

4.2. Testing of resistance.

4.2.1. Phase 2 - Moment transmission capacity.

The steel of the IPE 220 beams, theoretically of A 37 grade, has its yield-point at 28 kg/mm^2, which leads to a theoretical plastic moment of 8.400 kgm. For the UPN 180, these values were 30 kg/mm^2 and 5.125 kgm, respectively, giving 10.250 kgm for the tested double beam.

The beams were submitted to the moment distribution of fig. 7, obtained by placing the two jacks at one meter from the column.



This distance was chosen in order to respect the shear-moment relationship existing at the support of a uniformly loaded continuous beam. A 15 T load acted on the column.

Lateral buckling of the beam was prevented by appropriate guisding devices or local reinforcement of the tips.

The whole specimens were coated with whitewash to allow visualization of the development of plastic yielding.

The behaviour of the four specimens was as follows :

Type 1 : At 9 Tm, a gliding in the joint of the flanges, and yielding of both upper flanges occured, thus forming a plastic hinge.
The relative rotation between the two halves of the beam was 4.3 deg.

Type 2 : At 9 tm, some yielding appeared in the webs, near the support, combined with some bending of the consoles.
Yielding developed regularly when load was increased, until it appeared on the flanges at 11 Tm. The formation of the plastic hinges continued until the test was stopped at 12.0 Tm. The relative rotation of the two halves was 5.7 deg.

Type 3 : Plastic hinge began to form at 9 Tm and processed while loads was increased up to 11 Tm, where the beam collapsed by buckling of the compressed flange. Rotation was 7.8 deg.

Type 4 : At 9.0 Tm, 45° shear yield-lines appeared in the web of the beam, at the support.
At 9.45 Tm, some yielding began in the web of the column under the beam.
At 10.9 Tm, the stretched flange of the beam began to yield while the compressed flange collapsed by local buckling on both sides of support.
Total relative rotation was 6.6 deg.

It is clear from above results that the four connections are perfectly able to perform the transmission of the full plastic moment of the beam. None showed premature failure or instability in the beam, the column or the binding parts before reaching the plastic moment.

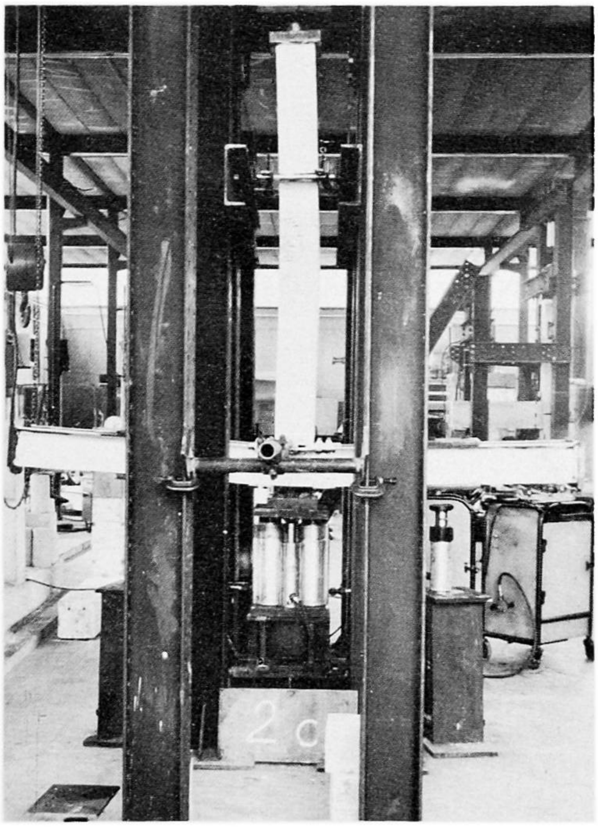
4.2.2. Phase 3 - Vertical load transmission capacity.

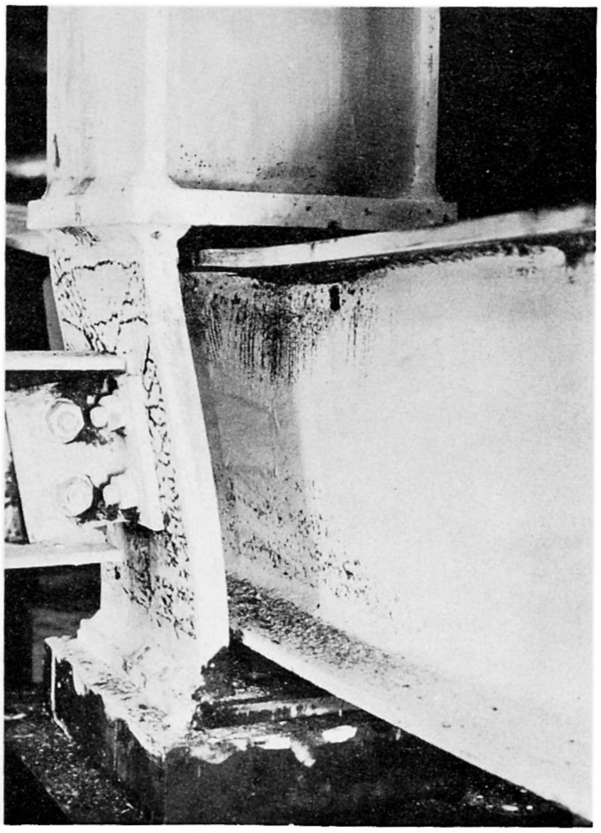
For this test, the load acting on the column was increased as high as possible, while a load of 10 Tons was maintained in the two beam-jacks.

The four specimens behave as follows :

Type 1 : The column buckled in the weak plane under a load of 130 T.
No special phenomenon occured in the hole in the web.

Type 2 : The column buckled in the weak plane under a load of 158 T.
Fig. 8 gives a general view of the testing apparatus, with buckling column of type 2 specimen.


2c



Type 3 : The upper box, with no primary nor secondary beam, began to show signs of yielding in the walls at 95 T.
The upper box collapsed by buckling of the walls at 117 T.
That box was then removed and the load increased again. The test was interrupted at 156 T, when the lower box, in turn, collapsed by local buckling (see fig. 9). That difference in the resistance of the two boxes is due to the fact that the lower box is reinforced locally by the connecting parts for the secondary beams, and braced by those beams, while the upper box is not.

Type 4 : Load was increased up to 160 T without exhausting the resistance of the specimen.

We do not intend here to make a complete discussion of the buckling of the column, as this was only a secondary aspect of the research. It will suffice to observe that, excepted for type 3, the collapse of the column, when achieved, obviously proceeded from the overall conditions of testing and not from local weakenings due to the connections. In case of type 3, it can be seen that the boxes are weak points of the structure. The walls of the box should be reinforced in order to prevent buckling, either by thickening the walls or by welding stiffeners thereon.

5. CONCLUSIONS.

From the above results, we may conclude that at least three of the four types of tested connections completely fulfill the required strength conditions : they are able to transmit the full plastic moment, they have a sufficient hinging action and they do not involve early local failure or collapse.
The fourth (type 3), though satisfactory as far as hinging and bending behaviour are concerned, should be improved with respect to its behaviour under vertical loading. One improvement could be the use of T-shaped walls, instead of plates, the webs being placed vertically and outside, thus serving accessorily as binding part with the secondary beam.

Up to now, we do not have enough information to make a numerical estimation of the cost of a plastically designed "hinged" structure, and compare it with the cost of the same structure with rigid connections. Such a study would be rather intricated as, to be complete, it should include not only the cost of steel and labour to manufacture the connections, but also a comparison of the ease or difficulty of assembly (local and global) in each conception and a prevision of the troubles that could appear in one and not in the other.

However, if as a first approximation, we limit the comparison to the connections themselves, it can be expected that the hinged connections will not be more expensive than rigid ones, excepted for type 3 (which had also the less interesting behaviour).
Type 2 and 4 which are quite simple, could even reveal cheaper.
There is thus reasonable matter to hope that the "hinged" solution could lead to actual savings.

The prior requirement for the application of this solution, that is feasibility is then certainly met, and it can be expected that the second, that is economy, will be, too.

ACKNOWLEDGMENTS.

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RÉSUMÉ

Cet article présente quelques essais effectués à l'Université de Liège sur des noeuds articulés destinés à des ossatures, du type "à noeuds fixés", conçues avec poutres et colonnes continues articulées entre elles à leurs points de jonction.

Quatre types de noeuds sont décrits, ainsi que les résultats d'essais sur la capacité de rotation des articulations, la résistance à la flexion, et la résistance aux charges verticales.

La discussion des résultats montre que le comportement des noeuds est très satisfaisant et l'article se termine par quelques considérations d'ordre économique.

SUMMARY

This paper presents some tests made at Liege University on hinged connections between continuous beams and columns of accordingly designed non-sway frames.

Four types of connections are described, so as the results of tests made on them, investigating rotational ability, bending moment transmission, and vertical load transmission.

A discussion of the results shows that the connections behave satisfactorily, and paper ends with some economical considerations.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt einige Versuche der Universität Lüttich über gelenkige, unverschiebliche Balken und Stützen, die gelenkig verbunden sind. Vier Knoten werden beschrieben, ebenso die Versuchsergebnisse über die Drehfähigkeit der Gelenke, die Biegefestigkeit sowie die Tragfähigkeit gegen lotrechte Lasten. Die Ergebnisse über das Verhalten der Knoten sind sehr zufriedenstellend. Betrachtungen über die Wirtschaftlichkeit beschliessen den Bericht.