Single-Bolt Semi-Rigid Connections in Space Frames

Assemblages semi-rigides à un boulon dans les structures spatiales

Flexible Einschraubenanschlüsse beim Bau räumlicher Stabwerke

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SUMMARY
Analysis of space structures normally assumes ideal spherical hinges at the intersections of member axes. However, no building system provides these ideal joints. Offset-connections must actually have some bending stiffness in order to prevent node rotations, especially if members are transversally loaded. The design of single-bolt semi-rigid joints for directly loaded rectangular hollow members and some physical tests are the topic of discussion in this paper.

RÉSUMÉ
L'analyse des structures spatiales présuppose normalement des articulations sphériques idéales à l'intersection des axes des membrures. Il n'existe cependant aucun système offrant ces assemblages idéaux. En fait, ces liaisons doivent présenter une certaine rigidité à la flexion afin d'éviter la rotation des noeuds, particulièrement si les membrures sont chargées transversalement. L'article présente le projet d'assemblages semi-rigides à un boulon pour des membrures rectangulaires creuses sous charges axiales. Des essais sont rapportés.

ZUSAMMENFASSUNG
Die Berechnung von Raumfachwerken beruht auf der Annahme idealer Kugelgelenke in den Schnittpunkten der Stabachsen. Tatsächlich besitzt kein reales Raumfachwerksystem diese idealen Verbinder. Zur Vermeidung von Knotenrotationen dürfen die exzentrischen Anschlüsse an die Knoten, insbesondere bei querbelasteten Stäben, keine Gelenke sein. In diesem Beitrag wird ein Bemessungskonzept für flexible Anschlüsse von direkt belasteten Rechteck-Hohlquerschnitten und die versuchstechnische Prüfung des Konzeptes diskutiert.

1. BASIC CONCEPTS

The analysis of general space frames can be distinguished by the stability criterion according to Föppl [1]. Those frames in accordance with Föppl's stability requirements are space trusses, i.e. they require no bending-resistant connections for the analysis of (axial) member forces.

The normal procedure of analysis is to assume ideal spherical hinges at the intersections of the member axes. In reality no building system provides a true spherical hinge at the intersection of member axes, i.e. the real offset-connections have to provide bending stiffness even in the ideal analytical model.

Because the magnitude of the connections rigidity remains unknown, no bending moments can be calculated for the connections by a full frame analysis. The only possible solution is found by the ultimate load concept, e.g. the upper and lower bound criteria as described by Feinberg (1948), Greenberg and Prager (1949) and Horne (1950).

Neal [2] states the lower bound criterion: "For any distribution of bending moments for a given frame the loading is less than the ultimate load, provided that the distribution is statically allowable and nowhere the members and connectors ultimate capacities are exceeded".

A typical MERO top chord member is represented by a model as shown in fig. 1. The bending moment at position a for the given loads γP (lateral) and γD (axial) is to be checked with respect to the ultimate bending capacity of the connectors (c)

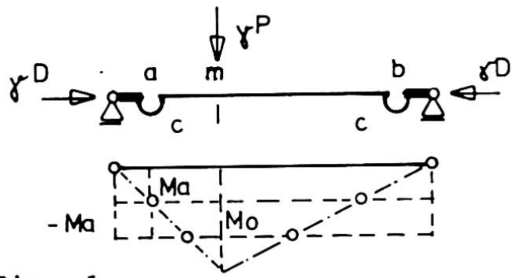

Fig. 1

$$M_a < M_{ult,c} \qquad (1)$$

The ultimate bending capacities and the spring rigidity c itself are related to the axial force. The check (1) is a straight forward application of the lower bound criteria, but does not explain the magnitude of nodal bending moments in the "real" structure: it only indicates that they cannot be greater than twice the ultimate bending capacity of the connection (resp. the sum of reverse bending capacities for non symmetric joints, see below).

2. DERIVATION OF A RATIONAL DESIGN CONCEPT BASED ON THE LIMIT STATE

Rectangular hollow sections (RHS) are most appropriate for the design of transversely loaded members in space frames. To interconnect them in a physical connector, a single bolt arrangement is used with the MERO-PLUS system. It offers the most advantages for fabrication and erection, including replacement of single members (fig. 2).

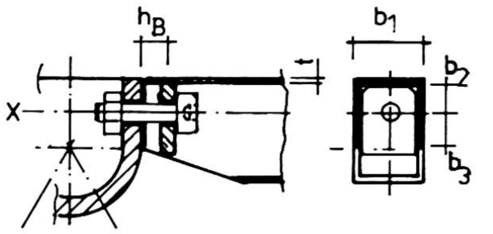

Fig. 2 Mero-Plus connector

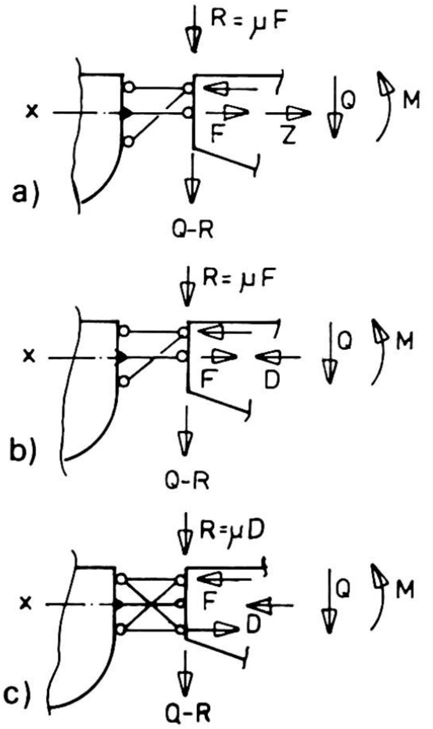

Fig. 3 Limit state models

Three different limit states can be distinguished (fig. 3). For the derivation of the design method some basic limit values are in common:

- the bolt tension capacity

$$Z_{Tr,B} = \beta_{Tr,B}\ A_{Bs}$$

where $\beta_{Tr,B}$ is the bolt yield stress and A_{Bs} the bolt stress area;

- the bolt bending capacity

$$M_{Tr,B} = 0{,}24\ \beta_{Tr,B}\ A_B^{3/2}$$

where A_B is the gross section area;

- the bolt shear capacity

$$Q_{Tr,B} = M_{Tr,B}/h_B,$$

h_B after fig. 2;

- the member contact surface compression capacities

$$D_{Tr,i} = \beta_{Tr,St}\ b_i\ t$$

where $\beta_{Tr,St}$ is the steel yield stress and b_i (i=1,2,3) and t are according to fig. 2.

Using the above definitions, the ultimate loads can be derived for varying loading conditions, considering the limit state models in fig. 3. For each model further case separations have to be observed, as the sign of bending moments ($M > 0$, $M < 0$), the bolt strength ($Z_{Tr}-Z$, $Z_{Tr}+D$) and surface compression capacities ($D_{Tr,i}$) are governing the failure criterion. The derivation shall be shown as an example for case a in fig. 3 for a negative moment ($M < 0$) and the surface compression capacity limiting the connectors capacity (representing the obviously most critical situation): $Z_{Tr,B}-Z > D_{Tr,3}$ (fig. 4).

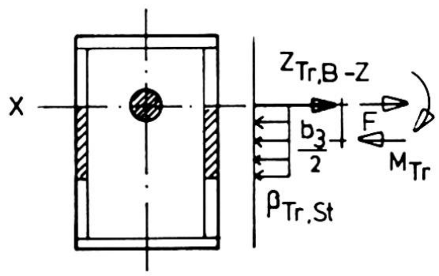

Fig. 4 Ultimate load stress distribution

The equilibrium equation for axis x yields with $F = D_{Tr,3}$ (fig. 2):

$$M^*_{Tr} = D_{Tr,3}\ b_3/2 + M_{Tr,B}\left(1 - \frac{Z+D_{Tr,3}}{Z_{Tr}}\right) \quad (2)$$

The related ultimate shear load is found according to (2) conservatively

$$Q^*_{Tr} = Q_{Tr}\ (1 - Z/Z_{Tr}) \quad (3)$$

where $Q_{Tr} = \mathrm{Min}(0.9\ Q_{Tr,B},\ \mu\ Z_{Tr})$, i.e. 0.9 $Q_{Tr,B}$ is the limit shear force that can be taken simultaneously with $M_{Tr,B}$ and vice versa.

In the same manner all possible combinations of loads and capacities as discussed above can be treated, yielding eventually a set of design formulae.

Still the question remains, whether the single bolt connections are providing enough ductility (rotational capacity) to allow for an application of the limit state concept. Tests under axial and transverse load have been performed to study the rotation of the connections.

3. TESTS CONFIRMING THE RATIONAL ANALYSIS CONCEPTS

Four tests were conducted at the Royal Institute of Technology, Stockholm, two in compression and two in tension. The tested construction simulated a top cord of the Stockholm Globe Arena and consisted of one rectangular hollow section top chord member bolted to two cup-shaped connectors. Details of a connector are shown in fig. 5a. The main purpose of the tests was to study the U-shaped connection area between the connectors and the space frame members. See fig. 5b.

Fig. 5c shows the four tests and table 1 gives some test results. The transverse load Q was in direct proportion to the axial load N. The load combination was chosen to be unfavourable for the contact region, not for the member as a whole. Therefore the compression members are acted upon by a transverse load corresponding to wind suction and the tension members to wind compression or snow load.

The material in the 150*100 hollow sections was St 52-3 with yield stress 387 N/mm^2 and ultimate stress 540 N/mm^2 according to coupon tension tests. The member thickness was 7,91 mm. The mean value of the ultimate load for the M27 10.9 bolts was 491 kN.

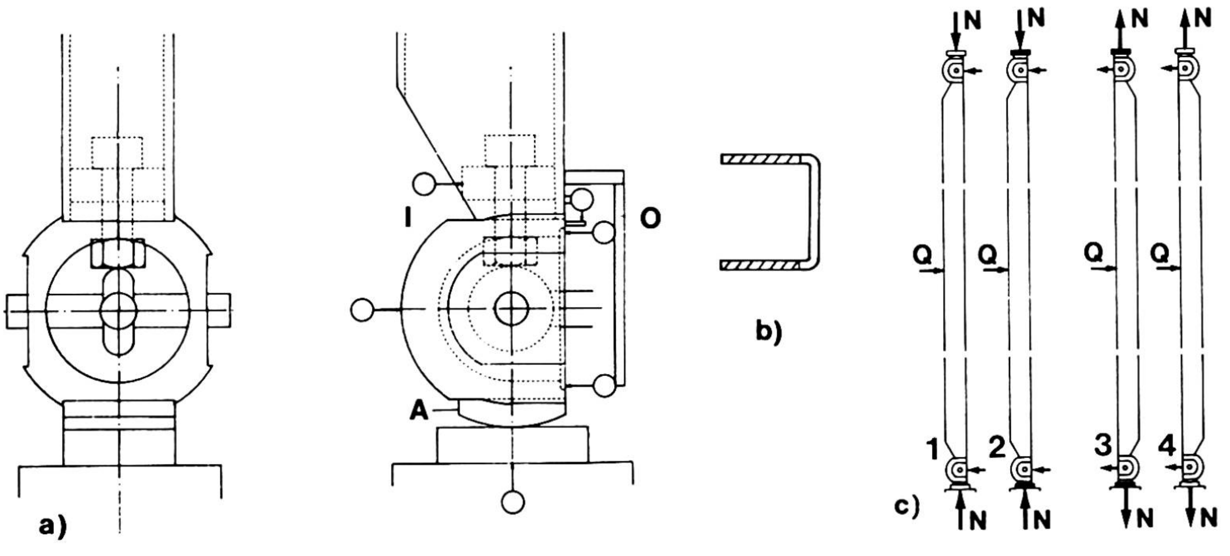

Fig. 5 a) Connector b) Contact area c) Tests

Table 1 Test results.

Test no	Load direction	$\frac{Q}{N}$	Support	Ultimate load N[kN]	Ultimate load Q[kN]	Failure mode
1	compression	0.045	simple	710	32	Yielding in contact region
2	compression	0.045	fixed	950	43	
3	tension	0.060	fixed	470	28	Failure of tension bolt
4	tension	0.12	simple	480	58	

3.1 Test no. 1

The cylindrically formed supporting plate (A in fig. 5a) was given a radius with the center coincident with the center of the connector. The normal force acts through the center of the supporting connector even when it is rotated. In the beginning of the tests both inner and outer sides (I and O, see fig. 5b) were in compression, see fig. 6a. At larger loads there becomes a gap at the outer side and a local compression of the inner side. Only the legs of the U-formed end of the hollow section member were carrying the axial and shear load. At the ultimate load, 710 kN, the calculated nominal stress on the contact area was 660 N/mm^2 which is 1.2 times the ultimate stress 540 N/mm^2 given by the coupon tension tests.

Most of the yielding took place close to the connecting surface. No yielding of the connector was observed. The transverse load seemed to have neglectable influence on the contact pressure.

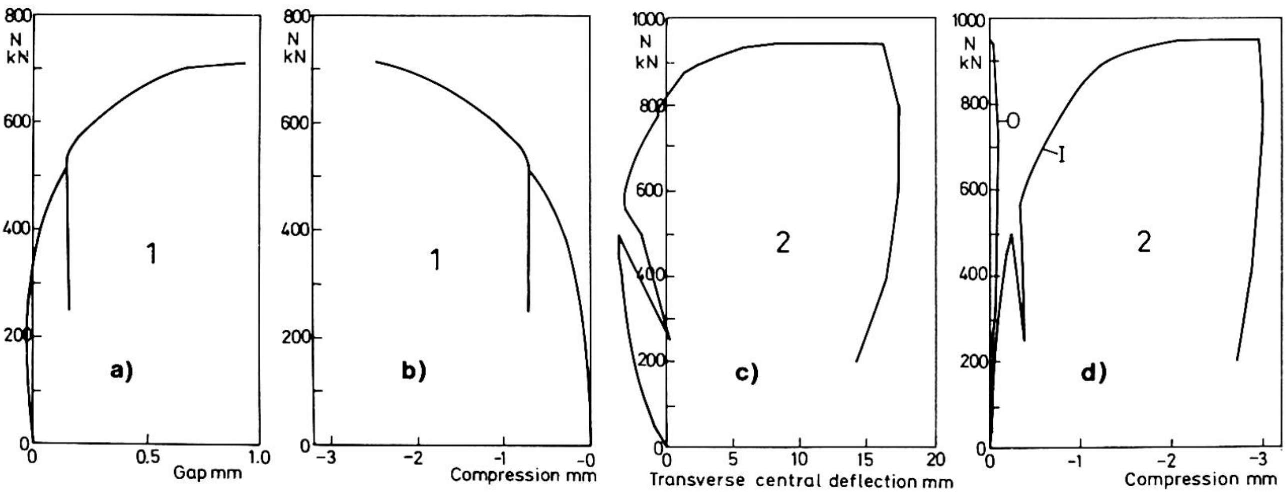

Fig. 6 a) Gap at outer side O and b) local compression at inner side I. Test no 1. c) Central deflection and d) local compression at inner side I and outer side O. Test no 2

3.2 Test no. 2

The connectors were fixed to the testing machine by bolts and rectangular plates. Both ends of the member were fixed. In the beginning, the direction of the transverse deflection coincided with the direction of the transverse load. At about 500 kN the transverse deflection changed sign and at the ultimate load the maximum deflection was about 17 mm in the opposite direction. See fig. 6c.

Both sides of the connection area were in compression during the testing but the compression on the outer side was very small, almost zero at the ultimate load. See fig. 6d. The local compression at the inner side was about 3 mm. If all the U-shaped area was supposed to be in compression then the average stress would be 490 N/mm^2. If only the legs where in compression then the calculated stress would be about 860 N/mm^2 compared to 660 in test no 1.

3.3 Tests no. 3 and 4

Both tests were fixed to the testing machine by the same type of bolts (M27 10.9) as between the connectors and the hollow section member.

At small loads there was a gap only at the outer side but at about 200 to 350 kN gaps also arose at the inner side. See fig. 7. The final mode of action was the same for the two tests although the transverse load was twice as much in test 4 as in test 3. The ultimate load was 473 and 483 kN respectively. In both tests the uppermost bolt failed by thread stripping at loads close to the bolt tension strength 491 kN.

The transverse shear load was 14 and 29 kN which apparently had a minor influence on the ultimate load.

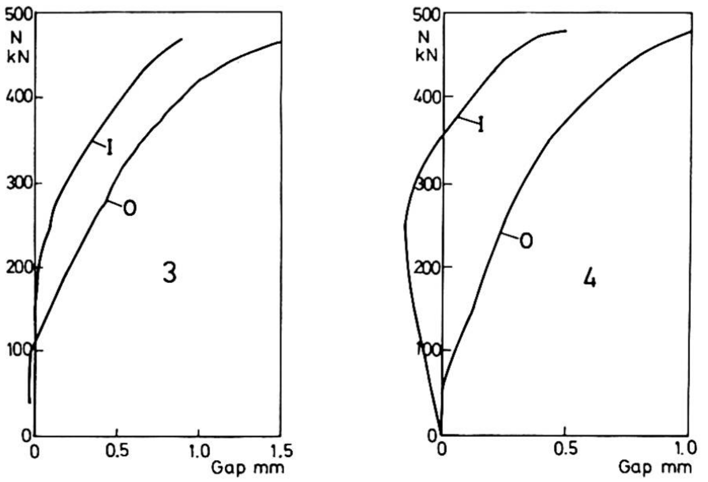

Fig. 7 Gap at the inner and outer side of the intersection between the connector and the member. Tests no. 3 and 4

4 CONCLUSION

The theory is in good agreement with the tests except that due to partially restrained lateral expansion of the member ends the ultimate compression capacity is larger than the theoretical capacity based on the yield stress. In a hyperstatic space frame there is a reserve in compression strength of the connector and possibilities of redistribution of forces after local yielding.

[1] Föppl A. Das Fachwerk im Raume, Teubner, Leipzig 1928.
[2] Neal B.G. Die Verfahren der plastischen Bemessung biegesteifer Stahlstabwerke, Springer, Berlin 1958.