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Autor: Bernuzzi, Claudio / Zandonini, Riccardo

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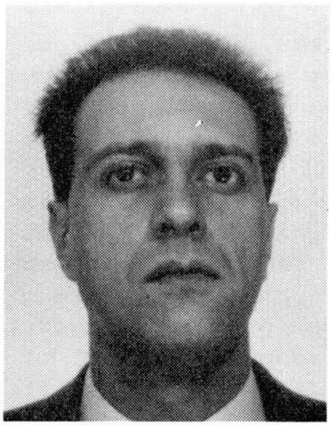
Serviceability and Analysis Models of Steel Buildings

Aptitude au service et modèles d'analyse en construction métallique

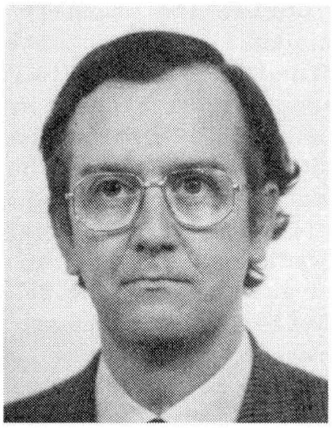
Gebrauchstauglichkeit und Berechnungsmodelle im Stahlbau

Claudio BERNUZZI
Research Associate
Univ. of Trento
Trento, Italy

Riccardo ZANDONINI
Professor
Univ. of Trento
Trento, Italy



Claudio Bernuzzi, born in 1962, received his degree at the University of Pavia, Italy. Since 1987 he has been at the University of Trento. His research work is focussed on the behaviour of beam-to-column joints and on the stability of semi-continuous frames both for steel and steel-concrete composite structures.



Riccardo Zandonini, born in 1948, received his engineering degree at the Technical University of Milan, where he served as a faculty member from 1972 to 1986. He has been on the faculty of Engineering in Trento since 1986. His research work is devoted to the study of the response of steel and steel-concrete composite structures.

SUMMARY

Recent trends of modern construction are likely to make in-service performance the governing design parameter. More and more refined models are used in structural analysis, allowing the response to be approximated quite closely. However, code serviceability checks still refer to traditional criteria, based on the elastic analysis of rather simple models of the bare structure. A review of such a philosophy seems necessary. This paper intends to emphasize the role of a link between analysis models and in service drift criteria.

RESUME

La tendance actuelle en construction métallique est de faire des conditions de service le paramètre prédominant lors du dimensionnement des cadres. Pour les analyses structurelles, on a recours à des modèles de plus en plus sophistiqués qui permettent d'approcher de manière très précise le comportement réel de la structure. Les codes pourtant continuent à s'appuyer, pour les vérifications en service, sur des critères traditionnels qui se fondent sur l'analyse élastique de systèmes idéalisés. Il semble nécessaire de revoir cette philosophie. Le présent article se propose de souligner le rôle de la liaison entre le modèle d'analyse et les limites des déplacements sous charges de service.

ZUSAMMENFASSUNG

Die neuesten Entwicklungen im Stahlbau zeigen, dass die Gebrauchstauglichkeit ein entscheidendes Kriterium im Entwurf ist. Immer präzisere Berechnungsmodelle erlauben es, dem tatsächlichen Verhalten des Tragwerkes näher zu kommen. Trotzdem stützen sich die Baunormen weiterhin auf althergebrachte Kriterien, welche auf dem elastischen Verhalten von vereinfachten Modellen gründen. Dieses muss kritisch betrachtet werden. Der Artikel bezweckt, die Wichtigkeit der Relation zwischen Berechnungsmodellen und den Kriterien für die Begrenzung der Verschiebungen unter Dienstlasten hervorzuheben.

1. INTRODUCTION

Limit state design is nowadays accepted by the vast majority of National and International Codes. This design philosophy stresses the link between structural reliability and performance of the system with respect to both service and ultimate loading conditions. Although the theoretical background of the method is well established, some of the problems related to its implementation in design practice have not yet been fully solved. Among these, an important question, requiring further studies, arises from the existing imbalance in the state of knowledge of the performance of the structure in different loading conditions. Research activity has been focussed mainly on the ultimate resistance of the structure, despite the fact that the performance under service loads is the critical requirement for many structural forms and materials. As a result of this limited research interest, a comprehensive information on in service responses of buildings is lacking, even for the most popular forms of construction. Furthermore, no criteria are available for an effective selection of the parameters to be used in serviceability checks and for a realistic definition of their limit values. Finally, the performance of the construction is usually investigated via numerical analyses, which make use of more or less refined models to simulate the response of the structural system as well as its interaction with the non structural components. Figure 1 provides a schematic representation of typical components with reference to a building with a steel braced framework as main structural system. Traditionally, the complexity of the many interactions determining the performance of the construction is substantially simplified in design analyses. However, the sophistication of the computing tools nowadays available to practising engineers permits increasing refinement of design analyses, which is more and more exploited, due to the strong competition among different structural materials. A relationship exists between the degree of refinement of the model adopted and the performance "required" of the model. This relation is clearly recognized for ultimate limit states, whilst even guidelines for a practical appraisal of this relationship with reference to serviceability are lacking. This condition is clearly reflected by recent structural Codes, even advanced ones, such as the Eurocodes [1,2]: they are still based on the traditional philosophy. This approach contrasts with the remarkable refinement of the prescriptions related to the ultimate limit state.

This situation represents a heavy burden for the design, in particular, of steel and composite steel-concrete buildings, for which the current trend towards larger spans and lighter systems makes serviceability increasingly important. Recognition of this significant imbalance in the design quality for the ultimate condition and for the serviceability condition has lead the European Coal and Steel Community (ECSC) to fund a research programme focussing on the static deflection of steel framed buildings. The research project, which was started in late 1990, comprised: (1) Investigation of the service performance of buildings (TNO-Bouw), (2) Review of existing Code requirements and their basis (University of Nottingham) and (3) Numerical studies and consideration of design models (University of Trento). A report giving the findings of each aspect of the work has been presented to ECSC. The content of this paper is based on the section on numerical studies and design models and is complemented at this Conference with three other papers which deal with the other topics.

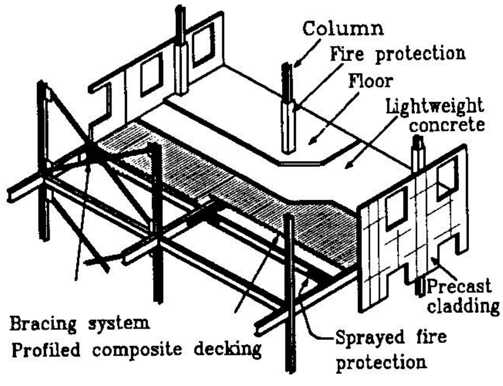


FIGURE 1

2. THE NUMERICAL STUDIES

The influence of the design model adopted for assessing the performance under service loads has been appraised through numerical studies on the two frame configurations shown in fig. 2. The figure reports as well the member sizes and the reference loads q and F. The main parameters considered were: (1) the joint action, (2) the cladding action and (3) the ratio ß between the vertical and horizontal loads at each storey. The M-ϕ curves in figure 3, from available test data on end plate and cleated connections [3,4,5], were selected to represent typical responses of flexible and semi-rigid joints. Besides these curves, the upper and lower bounds of the semi-rigid range defined by the Eurocode 3 [1] were also used as moment-rotation relations. The cladding stiffness and strength vary within a fairly wide range due to the very different cladding forms (ranging from glass to reinforced concrete panels) and cladding-to-structure interface connections [6,7]. It was then decided to make reference, in this first phase of the study, to the simple metal deck panel in figure 4, for which the shear response was determined experimentally for an aspect ratio very close to the bay span-to-height ratio of the frames considered [8]. An additional series of analyses permitted determination of the minimum value of the cladding stiffness required to meet the serviceability drift limits. The ratio ß was varied from 0.0125 (low to medium wind load) to 0.10 (high wind load or low seismic forces). The analyses incorporated both geometric and material nonlinearities; the limited aim of the study permitted simulation of cladding action through an equivalent diagonal bracing member [7]. All loads were increased proportionally through a common load multiplier α up to collapse. First order analyses were also conducted in selected cases in order to investigate the importance of the P-Δ effect. The serviceability limits specified by Eurocode 3 were assumed in the evaluation of the results (i.e. H/500 and h/300).

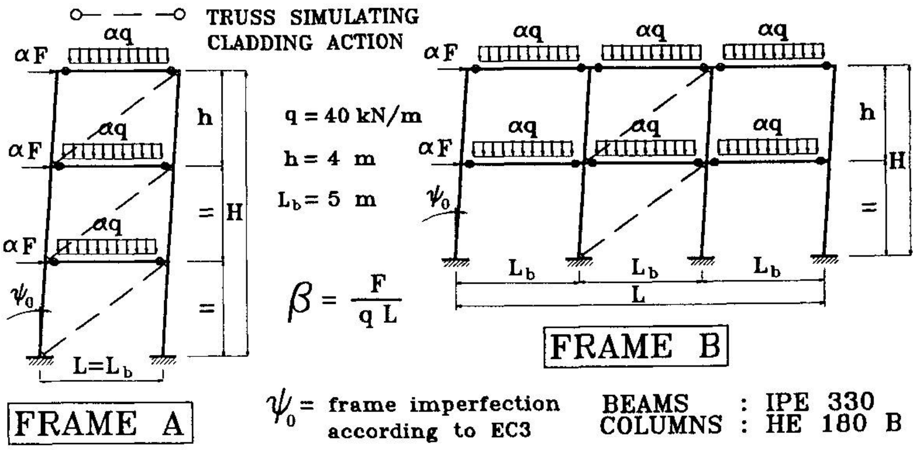


FIGURE 2

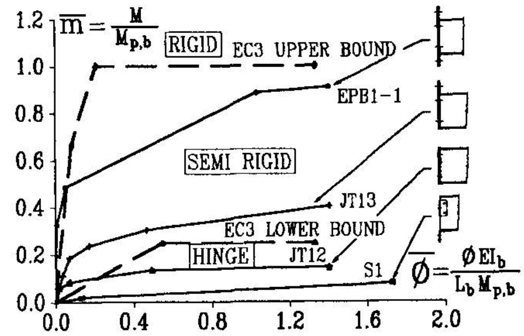


FIGURE 3

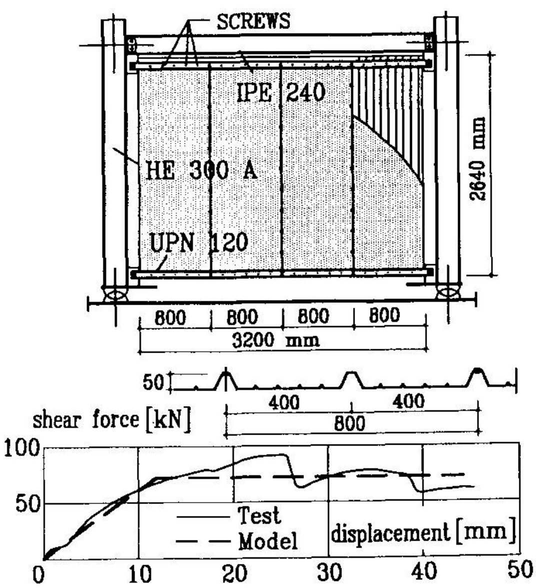


FIGURE 4

3. INFLUENCE OF JOINT ACTION

Traditional design of steel frames assumes that connections behave according to the ideal models of hinge and rigid joints. The present knowledge of joint response enables use of models closer to the actual behaviour, i.e. it permits designers to incorporate joint action in a fairly accurate way. A first series of analyses aimed at assessing the "effect" of this finer numerical simulation on the evaluation of the frame performance in service. To this purpose, the frames were first analysed for the limit cases of hinged and rigid joints. The service loads (i.e., $\alpha_{s,h}$ and $\alpha_{s,r}$ respectively) were defined by dividing the ultimate load multiplier α_u by an "average" safety factor equal to 1.43; sway indexes H/V were then determined at these load levels. The responses of the frames were subsequently determined incorporating the "actual" joint behaviour, and the sway indexes were evaluated for the service loads defined for the corresponding ideal case (i.e., the rigid frame for frames with EPB1-1 and EC3 upper bound joints and the "hinged" frame for frames with S1, JT12, JT13 and EC3 lower bound joints). The significant influence of joint action is apparent from figure 5 and table 1. The degree of continuity provided even by the most flexible connections substantially increases the stiffness of the frame (up to 30 times for frame A and ß=0.0125). The ultimate frame strength also improves remarkably, hence higher service loads might be admissible ($\alpha_{s,j}$). The frame stiffness, however, is not sufficient to allow this potential increase

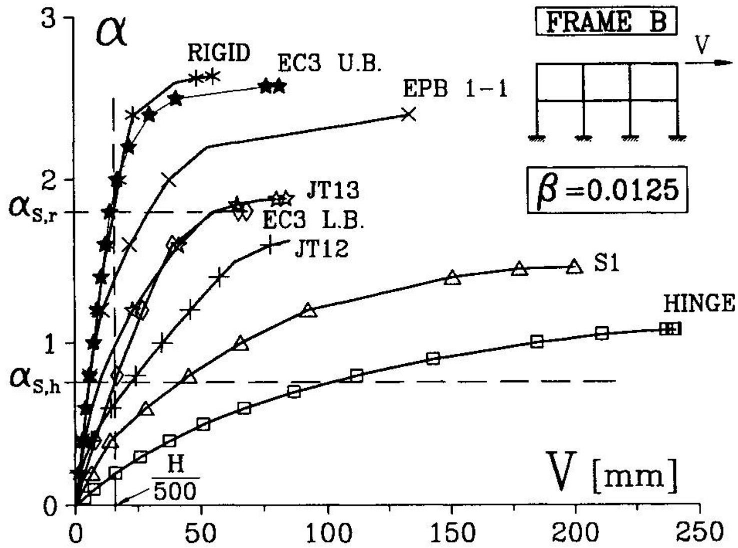


FIGURE 5

of resistance to be fully exploited (see the values of sway index determined at $\alpha_{s,j}$). Frame response is far less sensitive to a variation of joint flexibility with respect to the rigid joint model. This applies to both ultimate strength and stiffness in service. However, the recognition of the semi-rigid behaviour of joint EPB1-1 implies a remarkable increase in the flexibility of the frame model. Appraisal of frame performance in service based on this model would lead to regard the frame as inadequate also for ß = 0.0125. Since extended end plate joints are traditionally considered "rigid", frames accepted in the past on the basis of a rigid frame analysis would be rejected now if joint action is incorporated in the design analysis.

TABLE 1 - LATERAL DRIFT: FRAME A WITHOUT CLADDING

JOINT	ß	S.I. at $\alpha_{s,h}$	S.I.$_h$ / S.I.$_j$	$\alpha_{s,j}$ / $\alpha_{s,h}$	S.I. at $\alpha_{s,j}$
Hinge	1/80	60	1.00	1.00	60
S1		203	0.30	1.43	118
JT12		655	0.09	2.33	163
EC3 L.B.		583	0.10	2.58	188
JT13		1784	0.03	2.69	265
Hinge	1/20	39	1.00	1.00	39
S1		56	0.70	1.36	41
JT12		347	0.11	2.09	136
EC3 L.B.		266	0.15	2.72	89
JT13		645	0.06	3.09	141
Hinge	1/10	34	1.00	1.00	34
S1		71	0.48	1.33	44
JT12		387	0.09	1.93	106
EC3 L.B.		214	0.16	2.59	77
JT13		599	0.06	2.94	135
JOINT	ß	S.I. at $\alpha_{s,r}$	S.I.$_r$ / S.I.$_j$	$\alpha_{s,j}$ / $\alpha_{s,r}$	S.I. at $\alpha_{s,j}$
RIGID	1/80	677	1.00	1.00	677
EC3 U.B.		662	1.03	1.01	656
EPB1-1		308	2.19	0.99	295
RIGID	1/20	259	1.00	1.00	259
EC3 U.B.		251	1.03	1.02	217
EPB1-1		197	1.32	0.98	204
RIGID	1/10	192	1.00	1.00	192
EC3 U.B.		185	1.04	1.00	183
EPB1-1		154	1.25	0.94	175

S.I. = Sway Index (= H/V)

4. INFLUENCE OF CLADDING ACTION

Interaction between the frame skeleton and the cladding elements is fairly complex, as it depends on the responses of these two systems as well as on the type of interface connection. For the limited scope of this study a very simple model was adopted, which uses an equivalent concentric diagonal strut (Fig. 2). The cross section area of this strut can be easily computed when the elastic shear stiffness of the panel is known [8]. The first series of analyses considered that all storeys were "stiffened" by the metal sheeting panel in fig.4. Comparing fig. 6 and fig. 5, it is readily apparent that cladding action is the prime factor affecting the

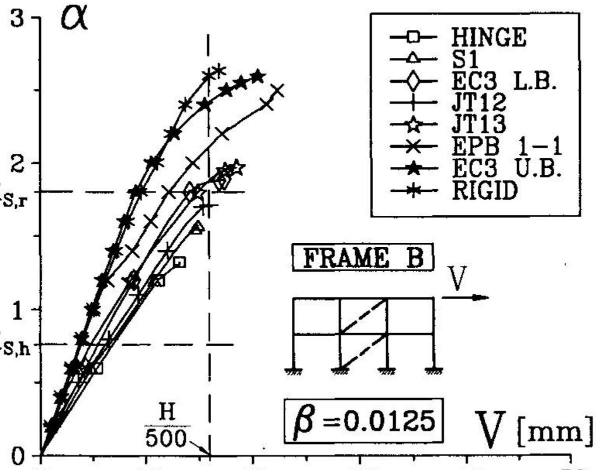


FIGURE 6

frame stiffness. Drifts at the service load levels $\alpha_{s,h}$ and $\alpha_{s,r}$, determined for the hinged and for the rigid frame neglecting cladding action, vary only moderately with the type of joint (less than 30% difference between the frames with hinge and semi-rigid (JT13) joints). Moreover, the simple panel considered provides in many cases sufficient stiffness to make the ultimate limit state govern design. In other terms the potential strength of the framework can be fully utilized, at least in the presence of low to moderate horizontal forces. Fairly high horizontal forces (ß = 0.10) still make serviceability limits critical. Shear forces in the panel "in service" are well within the elastic range of its response. It should be also considered that the stiffening action of cladding substantially reduces the geometrical (P - Δ) effects, enabling the designer to use a first order analysis, at least for serviceability checks: lateral drifts determined via first and second order analysis differed less than 8%.

It is interesting to note that for joint EPB1-1 second order analysis should be required if the analysis were conducted on the frame skeleton incorporating the joint response.

A further series of analyses considered the different conditions raising when panels were not present at all storeys. The results indicated that a substantial improvement of the frame model performance is associated even with the presence of the cladding in only one storey. For frame A the maximum influence of a single panel is achieved when this is located at the third storey.

Finally, the minimum shear stiffness K_{cl} required of the cladding to make the frame

TABLE 2 - LATERAL DRIFT: FRAME A WITH CLADDING

JOINT	ß	S.I. at $\alpha_{s,h}$	$\frac{S.I._h}{S.I._j}$	$\frac{\alpha_{s,j}}{\alpha_{s,h}}$	S.I. at $\alpha_{s,j}$
Hinge S1 JT12 EC3 L.B. JT13	1/80	3659 3703 4013 4013 4761	1.00 0.99 0.91 0.91 0.77	2.05 2.25 2.56 2.83 2.95	2483 1617 1440 1378 1333
Hinge S1 JT12 EC3 L.B. JT13	1/20	1980 2000 2091 2091 2490	1.00 0.99 0.95 0.95 0.79	3.11 3.53 3.87 4.28 4.47	610 558 494 471 461
Hinge S1 JT12 EC3 L.B. JT13	1/10	1509 1548 1633 1617 1881	1.00 0.95 0.92 0.93 0.80	4.52 5.12 5.60 6.19 6.45	326 291 265 256 253
JOINT	ß	S.I. at $\alpha_{s,r}$	$\frac{S.I._r}{S.I._j}$	$\frac{\alpha_{s,j}}{\alpha_{s,r}}$	S.I. at $\alpha_{s,j}$
RIGID EC3 U.B. EPB1-1	1/80	1536 1490 1488	1.00 1.03 1.03	1.00 1.01 0.98	1528 1477 1460
RIGID EC3 U.B. EPB1-1	1/20	674 630 561	1.00 1.07 1.20	1.26 1.26 1.26	533 508 501
RIGID EC3 U.B. EPB1-1	1/10	506 490 480	1.00 1.03 1.05	1.74 1.74 1.74	284 271 259

TABLE 3 - MINIMUM VALUES OF THE REQUIRED SHEAR STIFFNESS FOR CLADDING

		FRAME A		FRAME B	
JOINT	ß	K_{cl} at $\alpha_{s,h}$	K_{cl} at $\alpha_{s,j}$	K_{cl} at $\alpha_{s,h}$	K_{cl} at $\alpha_{s,j}$
Hinge S1 JT12 EC3 L.B. JT13	1/80	0.38 0.04 // // //	0.38 0.36 0.46 0.64 0.56	1.28 0.77 0.51 // //	1.28 1.54 1.64 1.79 1.68
Hinge S1 JT12 EC3 L.B. JT13	1/20	1.03 0.36 0.10 0.34 //	1.03 0.89 1.38 2.05 2.06	3.33 2.05 1.28 1.79 //	3.33 3.07 5.12 7.68 6.66
Hinge S1 JT12 EC3 L.B. JT13	1/10	1.33 0.87 0.20 0.82 //	1.33 1.23 1.54 2.66 2.46	4.61 3.69 1.83 2.46 //	4.61 4.35 7.17 11.78 9.73

K_{cl} in kN/mm;

// the frame meets serviceability limit without cladding model meet serviceability limits was determined. These values, reported in table 3 for the frames with flexible joints, range from 0.04 to 11.8 kN/mm. This range of stiffnesses can be easily provided by metal sheeting panels [6]; the panel in fig. 4 has a shear stiffness equal to 5.4 kN/mm. Masonry and concrete infills do present significantly higher stiffness, also in the presence of openings.

5. CONCLUDING REMARKS

The present approach to the checking of the structural performance in service has no scientific background, and intends to serve merely as an indication that the structure is likely to possess sufficient stiffness to prevent unsatisfactory behaviour. Deflection limits recommended by Codes were empirically established and by no means represent an indication of the actual in service performance. They were basically defined to be compared with frame deformations computed on an elastic model of the bare frame with ideal restraint conditions. The numerical analyses presented in the previous sections show clearly that joint and cladding action have a substantial influence on the response of the system. Incorporation of these actions in design analysis allows in many instances to make serviceability limits less critical and the ultimate strength of the structure to be fully exploited. In particular, the presence of light cladding seems sufficient to wash out the increase of frame flexibility associated to the use of semi-rigid joints in lieu of rigid joints. Noticeable advantages can in effect be achieved just by accounting for the presence of cladding solely for the service conditions. First order elastic analysis under working loads would be adequate in this case.
Knowledge of building behaviour is improving rapidly, and numerical models at hand to designers become more and more sophisticated. It seems hence important and beneficial to revise the present criteria for appraising structural serviceability. A first step in this direction might be represented by the definition of a link between the analysis model and the serviceability limits.

ACKNOWLEDGEMENTS

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REFERENCES

[1] European Committee for Standardisation (CEN), Eurocode 3: Design of Steel Structures - Part 1: General Rules and Rules for Buildings, February 1992.
[2] European Committee for Standardisation (CEN), Eurocode 4: Design of Composite Steel and Concrete Structures - Part 1: General Rules and Rules for Buildings, 1992.
[3] Davison J.B., Kirby P.A., Nethercot D.A., Rotational Stiffness Characteristics of Steel Beam-to-Column Connections, Joint Flexibility in Steel Frames, ed. W.F. Chen, Elsevier Applied Science, London, 1987.
[4] Davison J. B., Lam D., Nethercot D.A., Semi-rigid action of composite joints, The Structural Engineer, Vol. 68, No. 24, December 1990.
[5] Bernuzzi C. Zandonini R., Zanon P., Rotational behaviour of End Plate Connections, Costruzioni Metalliche, Vol. 2, 1991.
[6] Bryan E.R., The Stress Skin Design of Steel Buildings., Constrado Monographs, Crosby Lockwood Staples, London, 1972.
[7] Liauw T.C., Steel Frames with Concrete Infills, Stability and Strength series: Steel-Concrete Composite Structures, ed. R. Narayanan, Elsevier Applied Science, 1988.
[8] Mazzolani F.M., Sylos Labini F., Skin-frame interaction in seismic resistant steel structures, Costruzioni Metalliche, Vol. 4, 1984.