Inelastic behaviour and earthquake-resistance design method for thin-walled metal structures

Inelastic Behavior and Earthquake-Resistance Design Method for Thin-Walled Metal Structures

Comportement inélastique et méthode de calcul des structures métalliques en profilés minces résistant aux séismes

Unelastisches Verhalten und erdbebensichere Entwurfsmethode für dünnwandige Metallkonstruktionen

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SUMMARY

This paper discusses the behaviour of cold-formed light-gauge steel, in particular the yield strength and the deformation capacity of frames by means of a horizontal loading test on the frame, and presents the methods for evaluating these properties and the ideas on which earthquake-resistant design is based.

RÉSUMÉ

Cette contribution, relative aux structures en profilés minces en acier formé à froid, concerne l'étude de la résistance plastique et de la capacité de déformation des cadres au moyen d'essais sous charges horizontales. Des méthodes d'évaluation de ces propriétés sont présentées ainsi qu'un concept de dimensionnement de telles structures résistant aux séismes.

ZUSAMMENFASSUNG

Die vorliegende Arbeit behandelt kaltgewalzte Stahlleichtkonstruktionen und diskutiert Fliesslasten und Verformungsvermögen eines Rahmens unter Horizontalbelastungen. Methoden zur Bewertung der Eigenschaften insbesondere im Hinblick auf erdbebensichere Bemessung werden vorgelegt.

1. INTRODUCTION

In Japan, four years have passed since the new design codes for earthquake resistance were put into effect. The design methods stipulated in the new design codes aim at securing safety for structures against the strong ground motion during earthquakes on the basis of energy absorption capacity of the frame. Therefore, it is absolutely necessary in application to accurately grasp and evaluate the ultimate strength and the deformation capacity of the frame and its component members. The various prescriptions in the new design code for earthquake resistance currently being applied were provided so as to comply with large amounts of experimental data on heavy steel structures built up of hot-rolled members. For light-gauge steel structures, which are mainly composed of members of large width-thickness ratios, represented by light-gauge steel and lightweight wide-flange members, not much data is available on their ultimate strength and deformation capacity and the evaluation methods for these properties have not been established. In fact, there is a tendency that enough considerations are not given to those properties in designing steel structures. One notable result of this problem is the damage of warehouses and houses caused at the 1978 Miyagi-ken-oki earthquake.

With this reality in mind, this research was conducted to obtain fundamental data for the evaluation of the earthquake resistance of light-gauge steel structures which are used in buildings of three stories or less and, on the basis of research data, to improve earthquake-resistance design. Taking up cold-formed light-gauge steel, this paper discusses the yield strength and the deformation capacity of and presents the evaluation methods of these properties and the ideas of earthquake-resistance design.

2. PLAN OF EXPERIMENT

This test was conducted for light-gauge steel structures for low-rise buildings and with the aim of finding inelastic deformation characteristics in horizontal loading of the frames composed of light-gauge steel members with sectional dimensions of rank III or IV under the new design codes for earthquake resistance. To prevent the frame being subjected to torsional deflection, the test specimen was formed in a 1-story, 1-span portal space frame with 4 columns,

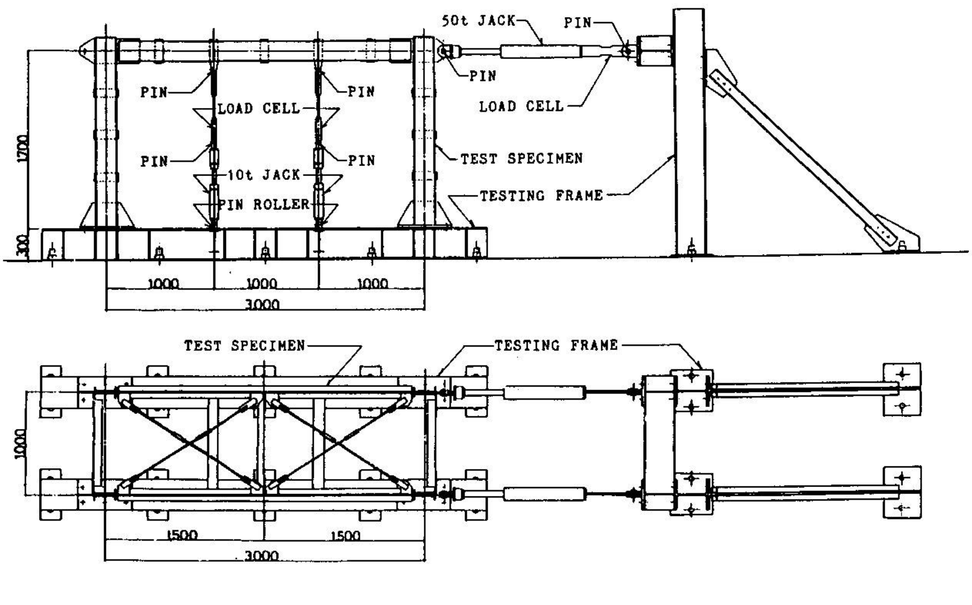

Fig.1 Set-Up of Frame Test

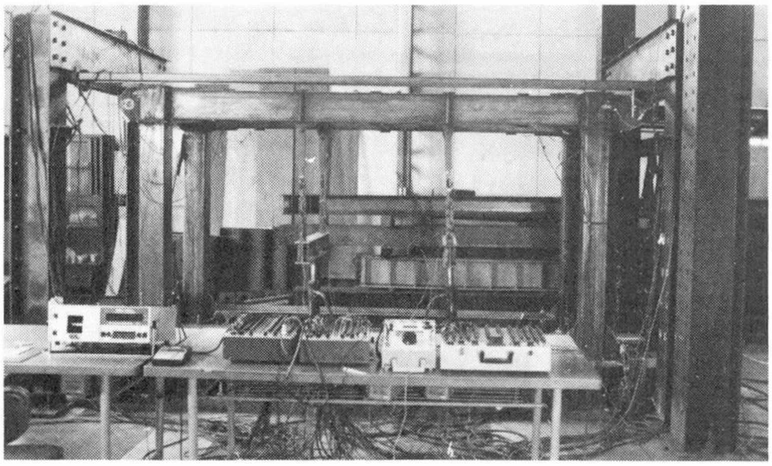

Photo.1 Test Set-up

consisting of two portal frames of the same type, arranged in parallel. The test parameters taken up were the shape of column cross section, rigidity of the connections and the degree of column base fixation. Fig. 1 shows the general view of the testing equipment. As shown in Fig. 1, a horizontal force was applied gradually in horizontal loading to the column capital of the test specimen. In addition, as gravity load for design, gravity loads were applied in two-point concentrated loading on the trisecting points of a beam so that it was $0.2M_y$ at the ends of the beam (M_y = yield moment of a beam member). The test specimens are outlined in Table 1.

3. INELASTIC DEFORMATION CHARACTERISTICS OF FRAMES

3.1 Elasto-Plastic Behavior of Frames

Figs. 2 to 8 show the test results of the frames. Fig. 2 indicates the hinge points and the order of the test pieces. Photo.2 shows the local buckling deformation of LG-1. Figs. 3 to 8 show the load-deflection curves of the frames obtained by the test. The vertical scale indicates the horizontal load Q (ton) of the column capital and the horizontal scale represents its horizontal displacement δ (cm). Fig. 3 presents the result of repeated loading to LG-1. The broken lines show the load-deflection curve in unidirectional loading converted from the curve in repeated loading. This indicates that the idea regarding the accumulated plastic deformation of heavy steel frames is applicable to this type

Table 1 Dimensions of Frame Test Specimen

SPECIMEN	SECTION (mm)		FLANGE b/t_f	WEB b/t_w	σ_y (t/cm^2)
LG-1	BEAM	2[-200×50×3.2	19.4	58.5	3.65
	COLUMN	2[-200×75×20×3.2	13.6	58.5	3.65
LG-2	BEAM	2[-200×50×3.2	19.4	58.5	3.65
	COLUMN	2[-200×50×3.2	19.4	58.5	3.65
LG-3	BEAM	2[-200×50×4.5	9.1	40.4	3.70
	COLUMN	□-175×175×6.0	25.2		3.65
LG-4	BEAM	2[-200×75×20×3.2	13.6	58.5	3.65
	COLUMN	2[-200×75×20×3.2	13.6	58.5	3.65
	BRACE	18φ			3.50
LG-5	BEAM	2[-200×50×3.2	19.4	58.5	2.70
	COLUMN	2[-200×75×20×3.2	13.6	58.5	3.70
LG-6	BEAM	2[-200×50×4.5	9.1	58.5	3.65
	COLUMN	□-175×175×6.0	25.2		4.00
LG-7	BEAM	2[-200×50×3.2	19.4	58.5	2.70
	COLUMN	2[-200×50×3.2	19.4	58.5	2.70
LG-8	BEAM	2[-200×50×3.2	19.4	58.5	2.70
	COLUMN	2[-200×50×3.2	19.4	58.5	2.70

SEPARATER PL 6 $\sigma_y = 2.64 t/cm^2$ PL 9 $\sigma_y = 2.62 t/cm^2$
PL 13 $\sigma_y = 2.54 t/cm^2$ PL 16 $\sigma_y = 2.60 t/cm^2$

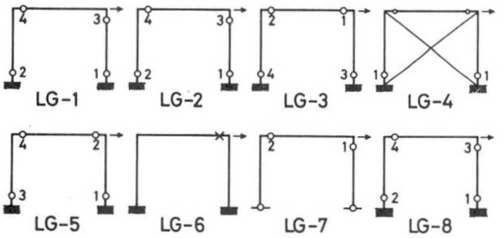

Fig.2 Location of Plastic Hinge

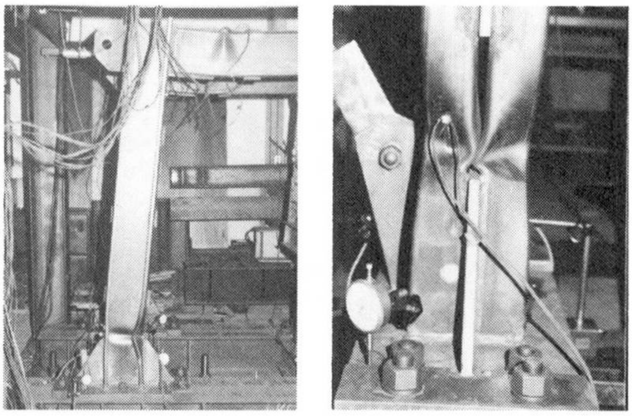

Photo.2 Local Buckling of LG-1

of frame. Fig. 4 shows the test result of LG-5, the connections of which were designed on the basis of ultimate strength, though the cross section of its member is the same as in LG-1. Fig. 5 shows a combined test result of LG-2, LG-7 and LG-8 with the boundary condition of the column bases varied. Comparing the load-deflection curves of LG-2 made of 32mm-thick plate and LG-8 made of 13mm-thick plate, we find hardly any difference despite the difference in yield stress of the members. LG-7 with hinged column bases decreased in the redundant strength by the reduction of the degree of redundancy. Hence, it was confirmed that the first hinge was formed at the column capital in the neighborhood of 3.2 tons. Thereafter, the load hardly rose. Fig. 6 shows the test result of LG-3 which used box-sections for the column members. Here the load-deflection curve shows stable features, including the lowering gradient. Shown in Fig. 7 is the test result of LG-6 which is of the same section as LG-3 and its connections are designed on the basis of maximum stress. When the load is 22.0 tons, the rigidity decreases to approximately 1/30 of the elastic rigidity and a relatively distinct bi-linear load deflection behavior is witnessed. At this time, clearly recognizable hinges were not found in the column and beam members. Later, while the rigidity was about 1/30 of the elastic rigidity, the load was gradually increased. After reaching the maximum strength of 23.8 tons, the strength started to decrease rapidly and when it dropped to about 23.0 tons, a failure was confirmed visually in the splice plates at the beam-column connection on the load-applied side. From the observation of the hinge formation and also from the comparison with the test result of LG-3, the decrease in rigidity and strength are considered to have resulted from the failure of the splice plate. The failure of the splice plate, which was in the final collapse mode, is considered attributable to the fact that the yield stress became about 1.54 times the basic

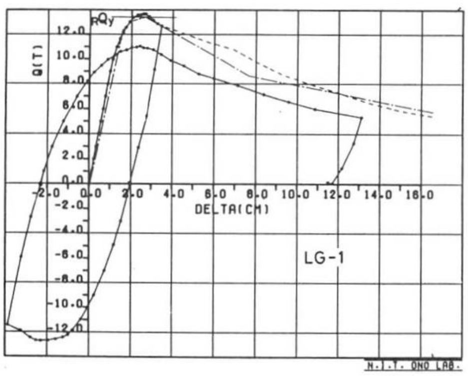

Fig.3 Load-Deflection Curve of LG-1

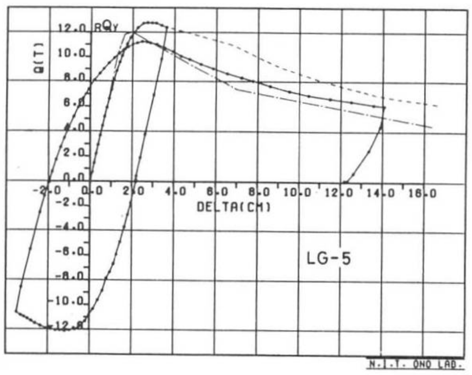

Fig.4 Load-Deflection Curve of LG-5

value of F used in design due to the effect of cold working. The connections of LG-6 are designed on the basis of the maximum stress of the members to be jointed and the shape of the splice plate was determined by the following formula.

$$ {}_fM_u = 1.2M_p = 1.2 \cdot F \cdot Z_p \qquad (1)$$

where ${}_fM_u$ = maximum bending moment of the splice plate
F = basic value of steel
M_p = full plastic moment of the beam

In LG-6, the full plastic moment of the beam M_p is $1.54 \cdot F \cdot Z_p$ because of the raised yield stress. This exceeds $1.2M_p$ (actually, $1.31M_p$ if the rise in the yield stress of the separators is taken into consideration) on the right-hand side of formula (1) even if the rigid zone is not taken into account. If, from the test result of the members, the strength of the members is considered not larger than the yield moment M_y, the right-hand side of formula (1) should be $1.54 \cdot (M_p/1.3) = 1.18M_p$, since the shape factor of the beam members is 1.3. Yet if the rise in the moment at the connections due to the extension of the rigid zone by the separators is taken into account, the moment will be $1.5M_p$ at the ends of the beam, which exceeds the $1.31M_p$ calculated by taking account of the rise in the yield stress. From the foregoing, when the frame is designed on the basis of ultimate strength and is made up of cold-formed light-gauge members, the rise in the yield stress by the effect of cold working is so large that it is considered problematical to design beams according to the nominal yield stress without discrimination. Fig. 8 shows the test result of LG-4, a braced frame.

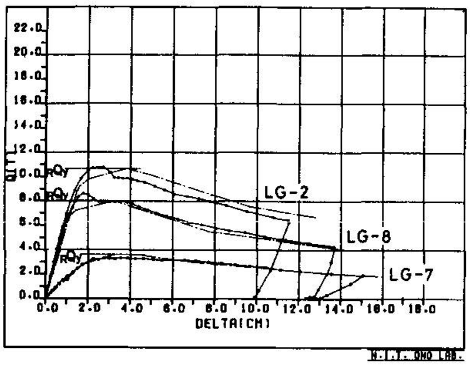

Fig.5 Load-Deflection Curves of LG-2, LG-7 and LG-8

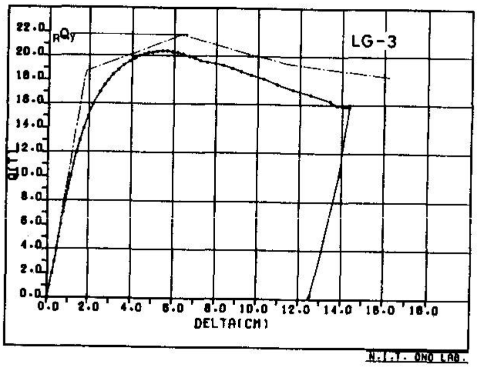

Fig.6 Load-Deflection Curve of LG-3

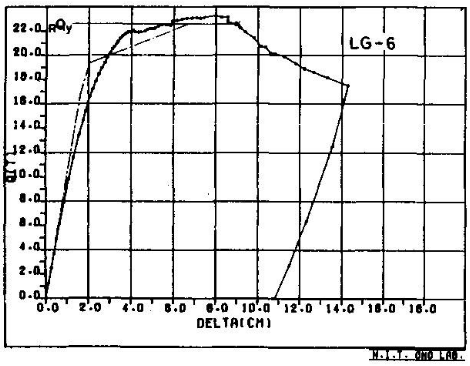

Fig.7 Load-Deflection Curve of LG-6

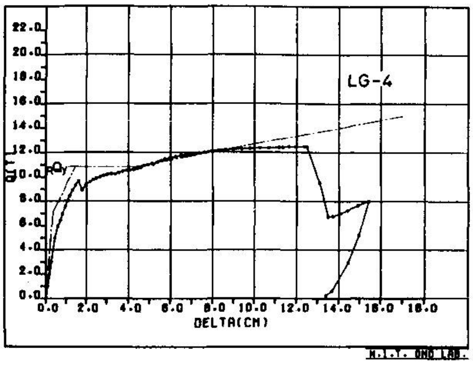

Fig.8 Load-Deflection Curve of LG-4

3.2 Ultimate Horizontal Strength and Load-Deflection Curves of Frames

This section, on the basis of the test results, presents the evaluation of the ultimate horizontal strength of light-gauge steel structures and the induction of the load-deflection curves of the frame. The ultimate horizontal strength is ${}_RQ_p$ when the limit moment of members is expressed by the full plastic moment M_p and the ultimate horizontal strength is ${}_RQ_y$ when the limit moment of members is expressed by the yield moment M_y. Incidentally, for LG-3 and 6, since their column members are of closed section and the width-thickness ratios are small (rank I), in the calculation of ${}_RQ_y$ of the column members the limit moment is expressed by the full plastic moment. As for LG-6, the limit moment of the end of the beam is determined by the maximum bending strength of the splice plate in accordance with the test results. The ultimate horizontal strength Q thus obtained are compared with the test results in table 2. Except for LG-6, the beam-end limit moment of which is determined by the maximum bending moment of the splice plates, none of the test specimens have the maximum strength which measures up to the ultimate strength by the evaluation of the full plastic moment. On the average, maximum strength is no more than 80 percent of the ultimate horizontal strength ${}_RQ_y$. On the other hand, the ultimate strength ${}_RQ_y$ by the evaluation of the yield strength shows good agreement with the maximum strength measured in the test. In the table, ${}_RQ_E$ is the horizontal load when some part of the frame reaches the yield strength. From the above results, it is considered adequate to evaluate the ultimate strength of frames using the maximum strength ${}_RQ_y$ by the evaluation of the yield strength. The dotted broken lines show the analysis results by using the plastic hinge method based on the evaluation of yield moment.

3.3 Evaluation of Inelastic Deformation Capacity of Frames

When discussing the safety of structures against the strong ground motion during an earthquake, it is essential to grasp their deformation capacity. This section presents the evaluation of the inelastic deformation capacity of frames by the method indicated in Fig. 9. The η corresponds to the equivalent multiplying factor of accumulate plastic deformation[1]. As shown in the figure, the η is calculated using the yield strength ${}_EQ_y$ which is determined by the intersection of the tangent when the quadratic gradient is averaged as 0.05 (k = elastic gradient) with the elastic line. The R is calculated with reference to the deformed amount δ_E, used as the basic value, which corresponds to the horizontal load ${}_RQ_E$ when some part of the frame yields. The point of plastic deformation when the strength drops again to ${}_RQ_E$, after reaching the maximum strength, is

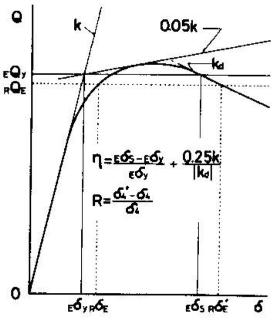

Fig.9 Evaluating Method of Deformation Capacity

Table 2 Experimental Results and Evaluated Value of Frame Test

SPECIMEN	$Q_{max}/{}_RQ_P$	$Q_{max}/{}_RQ_y$	$Q_{max}/{}_RQ_E$	η	R	RANK OF STRUCTURE
LG-1	0.83	1.02	1.14	2.88	2.03	III
LG-2	0.80	1.00	1.23	4.36	3.58	II
LG-3	0.86	0.94	1.14	7.27	2.62	I
LG-4	0.92	1.16	1.80	15.90	15.75	I
LG-5	0.87	1.07	1.24	4.07	3.69	II
LG-6	1.03	1.03	1.39	5.35	6.38	II
LG-7	0.73	0.92	1.15	5.77	3.89	II
LG-8	0.86	1.07	1.37	5.59	5.19	II

considered as the limit of plastic deformation. The R and η are compared in Table 2. In LG-3 in which the rigidity decreased considerably early and in LG-7 of which the lowering gradient is small, the value of η became fairly larger than the value of R, but the value is almost the same as with other test specimens. The structural ranking in the table is set by the value of η based on the new design code for earthquake resistance in Japan. Also in LG-1 which is the minimum in the amount of plastic deformation, the inelastic deformation capacity of rank III could be secured, while for the other test pieces the plastic deformation capacity of rank I or II could be confirmed. As for LG-6, though it belongs to rank I in terms of sectional dimensions, the plastic deformation capacity of rank I could not be confirmed, the reason for this is ascribed to the failure the splice plate. As described above, it was found that the plastic deformation capacity of rank I to III can be secured even for the frames which fall under rank III or IV in terms of sectional dimensions (width-thickness ratio of plate element). This suggests the possibility of applying the earthquake resistance design method, which takes into account the plastic deformation capacity of the frame to light-gauge steel structures.

3.4 Evaluation of Earthquake Resistance of Light-Gauge Steel Frames

The Japanese design code for earthquake resistance provide a formula for the judgement of the safety against earthquake as follows.

$$Q_{un}' \geq D_s \cdot F_{es} \cdot Q_{ud} \tag{2}$$

where Q_{un}' = required ultimate horizontal strength of each layer of frame
D_s = reduction factor depending on the deformation capacity of frame
F_{es} = shape factor
Q_{ud} = maximum horizontal force that each layer of frame would be subjected to if it responds elastically to a strong earthquake

When formula (2) is applied to rigid-frame light-gauge steel structures, the problem is what reduction factor (D_s) should be set and what maximum strength Q_{un}' should be given. In the light-gauge steel structures composed of members of relatively large width-thickness ratios such as are dealt with as the object of this paper, it is difficult to form stable hinges at the full plastic moment described up to the preceding section. Therefore, it is difficult to apply to them the above formula intended for heavy steel structures. Because the ultimate strength $_RQ_y$ of the frame by the evaluation of the yield moment M_y better exceeds the maximum strength obtained by the test by replacing Q_{un}' with $_RQ_y$ in Eq. (2), we have:

$$_RQ_y \geq D_s \cdot F_{es} \cdot Q_{ud} \tag{3}$$

However, in view of the simplicity when relatively small-scale steel structures are designed, it is more practical to consider the allowable strength for temporary loading $_RQ_E$ to be basic value for frame design. Assuming that $_RQ_y = \alpha\ _RQ_E$, formula (3) will be as follows.

$$_RQ_E \geq \frac{1}{\alpha} \cdot D_s \cdot F_{es} \cdot Q_{ud} \tag{4}$$

The ratio of $_RQ_y$ to $_RQ_p$ is about 0.8 to 1.0 as obtained by the test, but 0.8 for ensuring safety. Assuming that the ratio of $_RQ_p$ to $_RQ_E$ is 1.5, the value of α is given by

$$\alpha = (_RQ_y / _RQ_p) \cdot (_RQ_p / _RQ_E) = 0.8 \cdot 1.5 = 1.2 \tag{5}$$

On the other hand, the plastic deformation capacity of the frames, if evaluated by η, falls into rank I, II or III even if the component members are of rank IV. Therefore, the design of frames with sufficient aseismic properties is con-

sidered possible by using the allowable strength for temporary loading ${}_{R}Q_{E}$ as the basic value if a value corresponding to rank III for higher safety is set for the reduction factor D_s and formula (4) is applied using the value of given by formula (5).
Since the diagonal bracing is ${}_{B}Q_{p} \fallingdotseq {}_{B}Q_{E}$ in the case of a frame, the left-hand member of formula (4) is $1/\alpha \cdot {}_{B}Q_{E}$.

4. CONCLUSION

To summarize the results obtained by this research,

1) The maximum strength of frames composed of members of relatively large width-thickness ratios such as light-gauge steel can be nearly determined by the ultimate horizontal strength in the evaluation of the yield moment.

2) When cold-formed steel products such as light-gauge steel are used and connected on the basis of the ultimate strength, a considerable rise occurs in the yield stress by the effect of cold working and it is problematic to design structures based on the basic strength F indiscriminately.

3) To secure, aseismic properties at the same level as is provided by the new earthquake-resistance design method in Japan, following two design methods are presented. One is limit load design method by estimating the plastic hinge by the evaluation of the yield moment M_y. Otherwise, frames should be designed without calculating the ultimate horizontal strength and by applying formula (6) taking as the basic value a short-term allowable strength based on the allowable stress design method.

REFERENCES

1. Akiyama H., "Aseismic Limit Design of Building Structure", Tokyo Univ. Publisher, 1980.
2. Architectural Institute of Japan, "Recomendations for the Design and Fabrication of Light Weight Steel Structures" 1985
3. Kato B., Akiyama H. and Kitazawa S., "Deformation Characteristic of Box-Shaped Steel Members Influenced by Local Buckling", Trans, of AIJ, NO. 268, 1978.
4. Ono T., "Deformation Capacity and Ultimate Strength of Cold Formed Steel Members and Frames", Proc. of 7th ECEE, 1982.
5. Suzuki T. and Ono T., "Experimental Study of Inelastic Steel Beams" (1),(2), Trans. of AIJ, NO. 168, NO. 171, 1970.
6. Suzuki T., Ono T. and Kanabako Y., "Local Buckling and Inelastic Deformation Capacity of steel Beams under Shear Bending", Trans. of AIJ, NO. 260, 1977.