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Strengthening of Steel Buildings Against Earthquakes

Renforcement de constructions métalliques contre les séismes

Verstärkung von Stahlbauten hinsichtlich Erdbebenbeanspruchung

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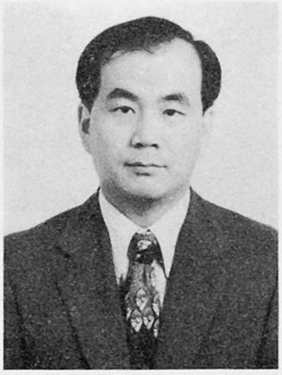

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SUMMARY

The basic concept of earthquake resistance capacity, upon which the seismic design regulations in the current Building Code and the Diagnosis and Therapy Standard were developed, is summarized. The principle of the methods for strengthening and repairing steel buildings in line with this concept is discussed.

RESUME

Le rapport décrit le principe fondamental de la capacité de la résistance aux séismes des constructions, sur lequel ont été developpés les règles de calcul dans les Réglements de construction récents et les recommandations pour le diagnostic et la thérapie. Le principe des méthodes de renforcement et de réparation des constructions métalliques, selon cette conception, est illustré.

ZUSAMMENFASSUNG

Der fundamentale Begriff der Erdbebensicherheit, auf welchem die Bestimmungen für erdbebensicheres Bauen in den gegenwärtigen Bauvorschriften, Diagnose- und Therapierichtlinien fussen, wird zusammengefasst. Der Grundsatz wie Stahlbauten verstärkt und wiederhergestellt werden können, wird diskutiert.

1. INTRODUCTION

In 1978 two strong earthquakes hit the middle part of Japan. Especially the second brought the considerable damage to the east of old Sendai City, the largest city of the northern Japan. Investigation on damaged steel buildings was carried out in the limited area (approximately 4 Km x 4 Km) where the damage was concentrated[1]. The numbers of houses in the investigation area are shown in Table 1. There were many steel office buildings and warehouses in this commercial and industrial region. Accordingly, considerable percentage of structures suffered by the earthquake.

Reinforced concrete buildings and wooden houses besides steel structures suffered considerably. This fact might accelerate the revision process of the seismic design regulations in Building Code which were already revised in 1981. The revised regulations had been developed on the basis of the new design concept that the earthquake resistance capacity is proportional to the plastic strain energy absorption. At almost the same time a diagnosis and therapy standard was prepared by us and other researchers under the sponsorship of the Japanese Government. The standard was developed on the same concept. This paper describes firstly the basic concept of the seismic design regulations and the diagnosis standard, and then the principle of the therapy, namely, the methods for strengthening and repairing existing buildings.

Total	Steel houses			R.concrete	Wooden	Others
	1-story	2-story and higher	LGS*			
2432	861	412	150	299	697	13
100%	35.4	16.9	6.2	12.3	28.7	0.5

* Light gaged steel

Table 1 The numbers of buildings and houses in the investigted area

2. SEISMIC DESIGN CODE AND DIAGNOSIS-THERAPY STANDARD

2.1 Basic Concept of Earthquake Resistance Capacity

In the earthquake resistant building design, there must be two classes of limit states: Namely, (a) the serviceability limit state for a moderate intensity earthquake and (b) the ultimate limit state for a major earthquake. Precise discussions have been done on these two classes of limit states by one of the authors[2]. Here, only a brief discussion on the ultimate limit state is repeated, because the establishment of such a limit state is necessarily required to set a standpoint for strengthening and repairing the existing building structures.

In a major earthquake which is unlikely to occur within the life time of a structure, the structure may be permitted to undergo considerable structural damage. However, the collapse of the structure must be avoided for safety of human life. The following design criterion can be accepted to evaluate the structural safety of a steel building.

The energy E_d which contributes to the structural damage is the sum of the cumulative plastic strain energy (the dissipated strain energy) W_p and the elastic energy W_e;

$$E_d = W_e + W_p \quad (1)$$

and can be approximately expressed as follows after Housner[3],

$$E_d = \frac{1}{2} M S_v^2 \qquad (2)$$

in which S_v = spectral pseudo-velocity response for undamped system and M = total mass of the structure. As a simple example, further derivation of the above equations will be done on a portal frame as shown in Fig. 1. In the case that the frame is yielded and undergoes considerable plastic deformation up to δm as shown in Fig.2, Eqs. 1 and 2 can be expressed as follows;

$$\frac{1}{2} M g^2 \omega^2 C^2 (1+4\eta) = \frac{1}{2} M S_v^2 \qquad (3)$$

in which g = the acceleration of gravity, ω = the natural circular frequency, C = the yield shear coefficient and η = the cumulative ductility ratio defined as $\eta = \delta m/\delta y - 1$. After the vibration theory,

$$C = \frac{1}{\sqrt{1+4\eta}} \left(\frac{S_a}{g}\right) \qquad (4)$$

where S_a = the spectral acceleration response. Eq. 4 gives the relation between the yield shear coefficient C and the associate ductility ratio η.

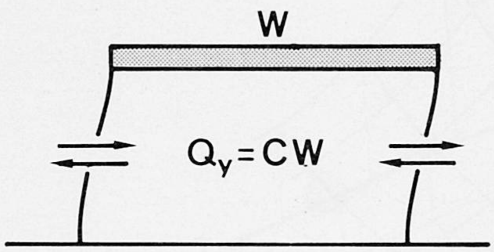


Fig.1 A portal frame

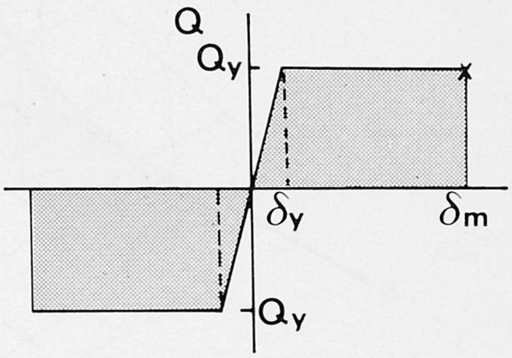


Fig.2 Story shear vs. sidesway relation

2.2 Seismic Design Regulations in Current Code

The seismic design regulations revised recently were established on the same basic concept as discussed above. From Eq. 4, the required yield shear coefficient C_1 is expressed as

$$C_1 \geq D_s (S_a/g)_D \,,\; D_s = \frac{1}{\sqrt{1+\alpha\eta_1}} \qquad (5)$$

in which $(S_a/g)_D$ = the design spectral acceleration index. The term D_s represents the structural characteristics of the frame and controlled by the required ductility ratio η_1. The coefficient a depends on the distribution of the stiffness and the story shear strength along the height in multi-story frames. The design spectral accumulation index $(S_a/g)_D$, a function of the natural period T, is expressed in the Code as

$$(S_a/g)_D = 1 \qquad ; \qquad T < T_c$$
$$(S_a/g)_D = 1 - 0.2(T/T_c - 1)^2 \quad ; \quad T_c \leq T < 2T_c \qquad (6)$$
$$(S_a/g)_D = 1.6T_c/T \qquad ; 2T_c \leq T$$

in which T_c = the specific period depending on the soil condition at the site. In a graphical way, the index can be represented by the curve in the C-T plane of Fig. 3. The required ductility ratio η_1 is dependent on the shapes of skeleton members. The possible ductility ratios must be evaluated for the width-to-thickness ratios of section shapes, the slenderness ratios of laterally unsupported members and the slenderness ratios of brace members in the braced frames. In the regulations of the Code the maximum value of the required yield shear coefficient C_1 is determined 0.5 for steel structure. In other words it is concluded that steel frames are expected to have some ductility. C_1 in Eq. 5 can be considered eventually a function of T and η_1 . Then the value of C_1 moves on a surface of the spreading roof shown in Fig. 3. According to the revised regulation of the Building Code, the design yield shear coefficient must be on or above the surface.

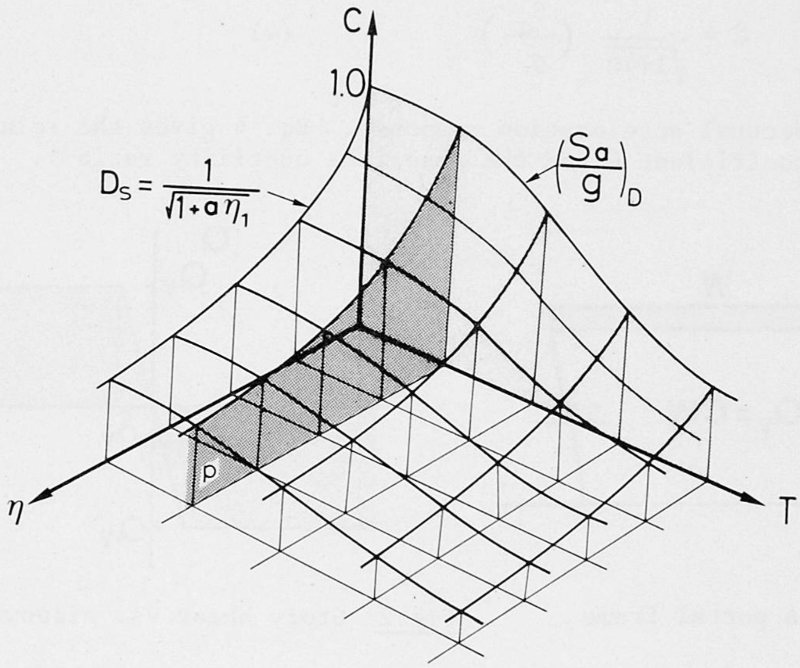


Fig.3 The same energy absorption level in the C-η-T space

2.3 Diagnosis and Therapy Standards of Existing Buildings

2.3.1 Diagnosis Standard

In parallel to the revision of the Building Code, a diagnosis and therapy standard for existing building was prepared. The existing buildings which were designed according to the old Building Code must be re-examined from the new standpoint of seismic design. The Diagnosis Standard consists of the regulations to calculate the existing yield shear coefficient and the regulations to evaluate the ductility ratio from the member sizes and shapes of inspected frames. In some existing building frames there can be seen the considerable shortage in strength and ductility. It is caused by a lack of consideration about the plastic behavior of structural members. Especially, the strength of joints is often insufficient to provide the required plastic deformation of connected members.

2.3.2 A Result of Damage Investigations

From the investigation results on the damaged steel buildings, the estimated maximum story shear coefficient α_u (the ultimate strength/the sustained weight) is plotted against the corresponding natural period of the building as shown in Fig. 4. The damage grades classified in Fig. 4 are:

(a) 0 = No damage, (b) I = No structural damage,
(c) II = Lightly damaged, (d) III = Moderately damaged,
(e) IV = Heavily damaged, (f) V = Collapse or Seriously damaged.

The details of the damage analysis are reported in the literature[1]. Hence, further discussion is eliminated here. However, it must be stressed that in the shorter natural period range, higher shear coefficients are required to reduce the structural damage.

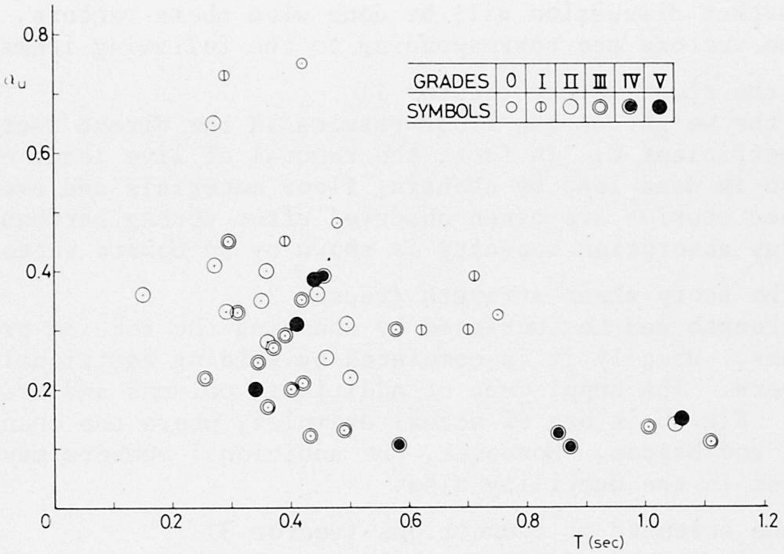


Fig.4 α_u-T relaton of damaged buldings

2.3.3. Strengthening-Therapy Standard

In the case that the insufficiency in the strength and the ductility exists, the story shear coefficient C is plotted under the roof in Fig. 3. By the energy absorption concept similar to that described in the preceding article, there can be considered another roof on which C, η and T are related at the same energy absorption level.

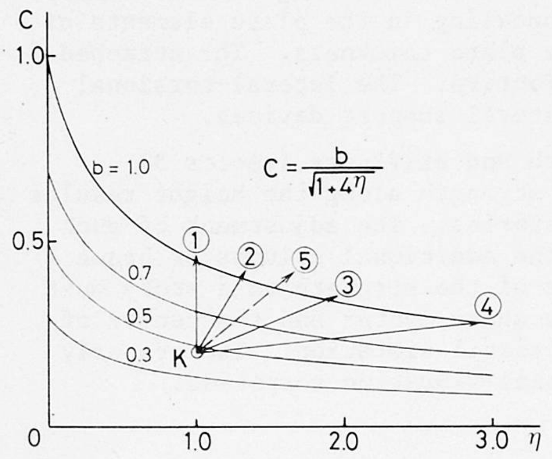


Fig.5 The same energy absorption levels in the C-η plane

Also the laminated roofs can exist for various levels of the energy absorption. Fig. 5 shows the section of such a structure made by the cutting plane denoted as p in Fig. 3. The solid curve represents the section of the roof required by the Building Code.

By help of the methods suggested in the Diagnosis Standard, the existing C and η of an inspected frame can be estimated. Such a set of (η, C) represents a point like K in Fig. 5. The curve of b = 0.5, passed near point K in the figure, shows energy absorption level considerably lower than the design level. Therefore, the point K must be moved into the region above the solid curve by means of several methods for strengthening. The details of the proposed methods will be discussed in the next.

3. METHODS FOR STRENGTHENING AND REPAIRING

There are various methods for strengthening and repairing. Some of them are efficient in increasing the strength, namely, the yield shear coefficient C. Some are efficient in increasing the cumulative ductility ratio η or both of C and η. The amount of increase in C and η can be graphically expressed by a vector in the C-η-T space. The vectors in Fig. 5 represent the projections of such vectors. Regardless of the change in the natural periods of remedied structures, the further discussion will be done with these vectors. The numerals beside the vectors are corresponding to the following items.

(1) Reduction in the floor weight (vector 1)
The reduction in the weight on the floor results in the direct increase in the yield shear coefficient C. In fact, the removal of live loads on the floor, the decrease in dead load by changing floor materials and even the elimination of upper stories are often observed after strong earthquakes. The change of the energy absorption capacity is shown by an upward vector 1.

(2) Increase in the story shear strength (vector 2)
The story shear strength can be increased by changing the section properties of columns and beams. Usually it is completed by welding additional cover plates to the members. The supplement of additional columns and brace members is most effective. Fig. 6 is one of actual examples, where the channel members are added to steel rod braces. Moreover, the additional members may sometimes produce the increase in the ductility also.

(3) Increase in the strength of connections (vector 3)
The sufficient plastic deformation of members can be achieved, provided that the connections between the members are strong enough to maintain the member forces without fracture even in the plastic range. During the earthquake excitation the collapse of a structure is often initiated by the fracture of the connections due to careless design procedure and/or poor workmanship. Replacing and revising the weak connections result in the increase in the strength and the ductility.

(4) Improvement of member ductility (vector 4)
The member ductility is improved to avoid the local buckling and the lateral-torsional buckling in members. The local buckling in the plate elements of sections can be prevented by increasing the plate thickness. The attached coverplates as shown in Fig. 7 are also effective. The lateral-torsional buckling is prevented by properly placed lateral support devices.

(5) Adjustment of unbalance in the strength and stiffness (vector 5)
The excessive unbalance in the story shear strength along the height results in the damage concentration to the weaker stories. The adjustment of such an unbalance can be attained by supplying the additional columns or brace members. However, the excessive supplement of the strength to a story must be avoided. Disagreement between the story shear center and the center of sidesway stiffness in the plan causes a torsional vibration. The properly placed brace members can reduce such torsional vibration responses.

4. CONCLUDING REMARKS

In Japan a great number of steel buildings have been built for last several decades. The number increased suddenly in 1970's. It is obvious by the

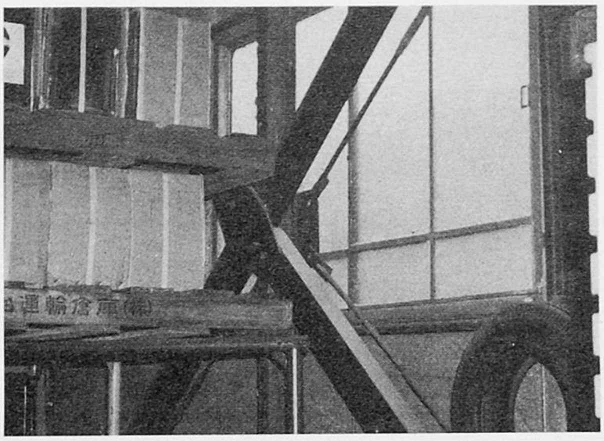


Fig.6 Channel braces added

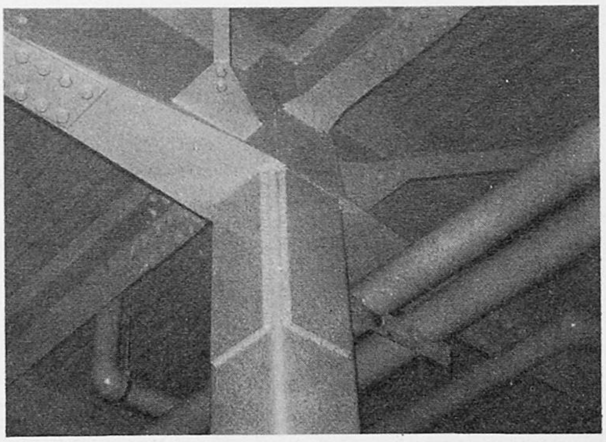
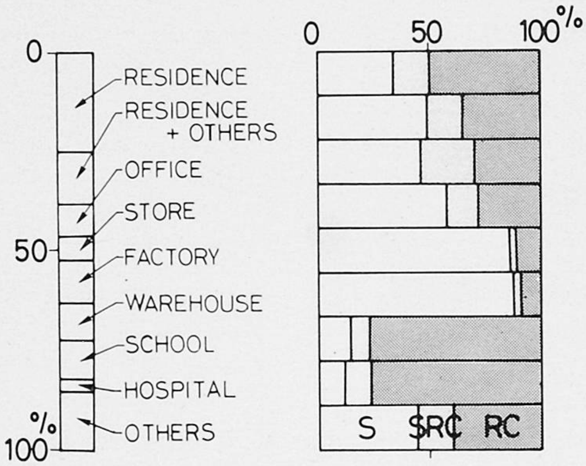
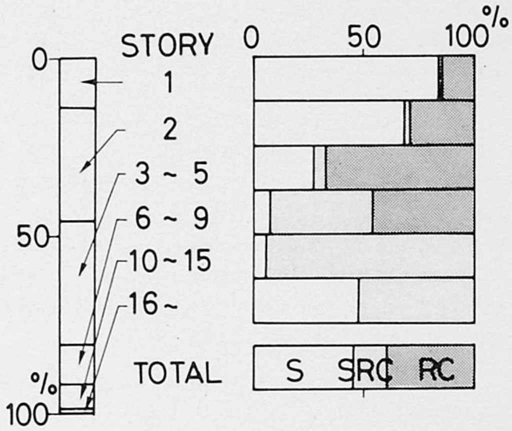


Fig.7 Coverplates welded

statistics of the annual total of newly constructed floor area in 1979 that steel structures are widely used for factories, warehouses and stores as shown in Fig. 8. Moreover, low rise buildings are often made of steel frames as shown in Fig. 9. Therefore, the inspection of earthquake resistance capacity of existing buildings and the strengthening become very important. In fact a municipal government set force their plans for strengthening houses along the Standard described in this paper. Some of them have been already examined and will be strengthened in the suitable methods.





S ; steel, SRC ; mixed, RC ; reinforced concrete

Fig.8 Usage of three types of structures (right) in different types of buildings (left)

Fig.9 Usage of three types of structures (right) in different story buildings (left)

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