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Seismic Design of Steel Buildings in Japan

Conception antisismique des bâtiments en acier au Japon

Seismischer Entwurf von Stahlbauten in Japan

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SUMMARY

The basic concept of the new Japanese seismic code is introduced. Then the theoretical and experimental backgrounds of this code are discussed focusing attention on the design of steel structures.

RÉSUMÉ

Les concepts de base des normes antisismiques japonaises sont présentés. Leur provenance, basée à la fois sur l'expérience et la théorie, est analysée du point de vue de la conception des charpentes métalliques.

ZUSAMMENFASSUNG

Das Grundkonzept des neuen japanischen seismischen Codes wird vorgestellt. Die theoretischen und experimentellen Grundlagen dieses Codes, im Hinblick auf den Entwurf von Stahlbauten, werden besprochen.

1. INTRODUCTION

The Japanese national building code for seismic design was revised in June 1981. It took five years for drafting and furthermore took another three years to put the draft into the practical final code, after getting consensus of administrators, practical engineers and researchers.

This paper introduces the basic concept of the new code firstly. Then, the theoretical and experimental backgrounds are discussed focussing on the design of steel structures.

2. BASIC CONCEPT OF THE NEW SEISMIC CODE

Basic concept and structure of the new code are introduced herein, more detailed description is given in reference.1.

2.1 Design criteria

Similar to the design against other loading conditions, two classes of limit states are pertinent to earthquake-resistant building design. They are; (1) the serviceability limit state for a moderate intensity earthquake; and (2) the ultimate limit state for a major earthquake.

-(1) Serviceability limit state design
The structure should be proportioned to resist the moderate earthquake elastically and without excessive lateral deflection so as the building can remain in serviceable condition as soon as the earthquake is over. Moderate earthquakes are expected to occur with a reasonably high probability during the life of a structure. The maximum design spectral acceleration of short-period structures against a moderate earthquake is 0.2g in Japan, where g= the acceleration of gravity.

-(2) Ultimate limit state design
The structure may be permitted to undergo considerable structural damage when it is subjected to a major earthquake. The collapse of the structure and resulting loss of human life, however, must be avoided. A major earthquake is unlikely to occur within the life of a structure, but is used in the design to examine the ultimate structural safety. The maximum design spectral acceleration of a short-period structure in the case of a major earthquake is 1.0g in Japan.

Since the earthquake loading is unique, the definition of load intensity for serviceability limit state is somewhat different from other types of loadings.

2.2 Serviceability limit state design

The lateral seismic shear, ${}_eQ_i$, of the i-th story above the ground level is given as

$$ {}_eQ_i = {}_eC_i \cdot W_i ; \quad {}_eC_i = Z\, R_t\, A_i\, {}_eC_o \tag{1} $$

in which ${}_eC_i$= the lateral seismic shear coefficient of the i-th story for serviceability limit state design; ${}_eC_o$= standard base shear coefficient for serviceability limit state design; W_i=weight of the building above i-th story; Z=seismic hazard zoning coefficient(1.0-0.7); R_t=nondimensional response spectrum(design spectral coefficient) which is determined by the type of subsoil conditions(hard, medium and soft) and fundamental period of the building(T, sec) as illustrated in Fig.1; A_i=lateral shear distribution factor as shown bellow,

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T} \qquad (2)$$

$\alpha_i = W_i/W$, where W_i is the weight above i-th story and W is the total weight of the building above the ground level.

A structure should be proportioned to be elastic against the lateral forces ${}_eQ_i$ given by Eq.1,and the drift of each story must be less than 1/200 of story height,the value of which can be increased up to 1/120,if non-structural elements are flexible enough to follow-up this magnitude of deformation.

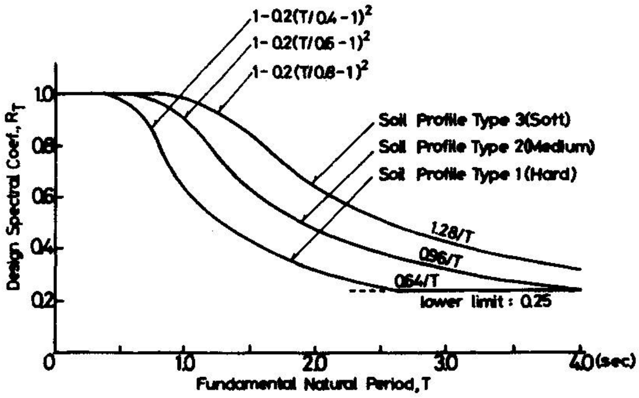

FIG.1 Design spectral coefficient,R_t

2.3 Ultimate limit state design

The lateral seismic shear,${}_uQ_i$,of the i-th story above the ground level is given as

$${}_uQ_i = D_s\, F_{es}\, {}_uC_i\, W_i; \qquad {}_uC_i = Z\, R_t\, A_i\, {}_uC_0 \qquad (3)$$

in which D_s=structural characteristics factor which represents energy dissipating capacity of the building structure related with ductility for each story; F_{es}= shape factor which reflects the adverse effects of eccentricity of stiffness and a drastic change of stiffness along the height; and ${}_uC_0$ = 1.0 = standard base shear coefficient for ultimate limit state design.

2.4 Special provisions

2.4.1 Exemption of ultimate limit state design

In steel buildings not exceeding 31m in height and satisfying the following requirements,ultimate limit state design as specified in 2.3 is not required.

-1.Eccentricity of stiffness and change of stiffness along the height should be negligible and thus F_{es}=1 should be met.

-2.For braced frames,the following increased design seismic shear should be used

$$Q_{bi} = (1 + 0.7\beta)\ {}_eQ_i \qquad (4)$$

in which β = the ratio of lateral shear capacity of diagonal bracings to the total lateral shear capacity of the story.

-3.Joint strength of diagonal bracings should meet the following condition,

$${}_jT_u \geq 1.2\ T_y \qquad (5)$$

in which ${}_jT_u$= ultimate strength of joint of a diagonal bracing and T_y= yield strength of the bracing member.

-4.Width-to-thickness ratios of plate elements of beam-columns and beams shall meet the ductility class I of Table.2 given in 3.2.2.

-5.Strength of beam-to-column connections shall meet the following condition,

$$ {}_jM_u \geq 1.3\ M_y \tag{6}$$

in which ${}_jM_u$=maximum bending strength of beam-to-column connection and M_y=yield moment of the pertinent beam or column.

2.4.2 Highrise buildings

Design of buildings whose height exceeds 60 meters should be carried out on the basis of time history dynamic analysis for two levels of input earthquake ground motions and the design procedure must be reviewed by the special committee appointed by Minister of Construction.

3. COMMENTARY

The response spectra provide the meaningful measure of the intensity of an earthquake motion. They are expressed on the basis of the following characteristic responses,

Spectral pseudo-velocity response

$$S_v = [\ {}_0\!\int^t \ddot{v}_g(\tau)\ \exp[-\xi\ \omega(t-\tau)]\sin\omega(t-\tau)d\tau]_{max.} \tag{7}$$

in which v_g=ground displacement; ξ=damping ratio and ω=undamped natural circular frequency.

Spectral displacement

$$S_d = \frac{S_v}{\omega} \tag{8}$$

Spectral acceleration

$$S_a = \omega\ S_v \tag{9}$$

These responses can be applied to the linear elastic structures,and the design criteria for serviceability limit state(elastic limit state) as prescribed in 2.2 can be formulated on the basis of the concept of the response spectra. In fact,$R_t\ {}_eC_o$ in Eq.1 is the nondimensional spectral acceleration response, and related to S_a as $R_t\ {}_eC_o = S_a/g$, in which g is the acceleration of gravity. And the basic structure of this criterion is much the same as other ones specified in many seismically active countries.

On the other hand,the response spectra cannot apply directly to the ultimate limit state design since it involves inelastic deformations. To overcome this difficulty,the design criterion for ultimate limit state is based on energy concept making use the fact that the input energy E into a structure during an earthquake is given as,whether it behaves elastically or not[2,3,4]

$$E = \frac{1}{2} M\ S_v^2 \tag{10}$$

in which M=total mass of the structure. Average velocity response spectrum can be approximated by two straight lines as shown in Fig.2. This means that the value of S_v is independent of the fundamental period T for its medium range. Since the fundamental period of a struct-

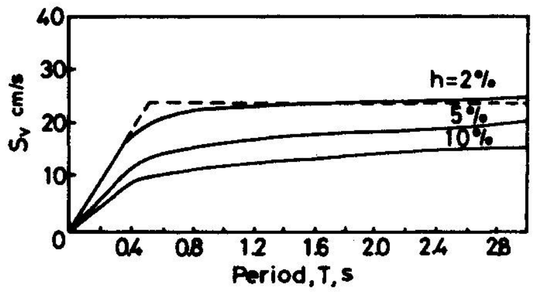

FIG.2 Average velocity response spectrum

ure changes when it is plastified during vibration,above characteristic is very convenient for practical application. On the other hand,S_v changes linearly with T in short period region and the characteristic is uncertain for long period region,therefore,the application of Eq.10 to the structures with very short or very long fundamental periods leaves some questions. This is one reason why the special design procedure is required for high-rise buildings in 2.4.2. Another reasons are to check the damage concentration into a particular story and to check the P-Δ effect.
The followings are theoretical and experimental backgrounds of the formulation of ultimate limit state design criteria.

3.1 Safety criterion

The input earthquake energy into a structure given by Eq.10 is absorbed and dissipated by the elastic strain energy W_e and the cumulative plastic strain energy W_p. For the survival of a structure,the structure's capacity of cumulative plastic energy dissipation ${}_uW_p$ must be greater than the cumulative plastic energy demand,and thus

$$ {}_uW_p \geq W_p = \frac{1}{2} M S_v^2 - W_e \tag{11} $$

This is the criterion to evaluate the safety of a steel structure in the major earthquake.

The elastic strain energy W_e is approximately given as[5]

$$ W_e = \frac{1}{2} M \left(\frac{T}{2\pi} \alpha_1 g\right)^2 \tag{12} $$

in which α_1= yield base shear coefficient.

The earthquake input energy of a multistory building is distributed to each story. If a structure is poorly proportioned,the input energy will concentrate on a particular story. In this sense,it is important to determine the distribution of design shear coefficient along the height so as to develop uniform cumulative plastic deformation at each story. The lateral shear distribution factor A_i given by Eq.2 was found to be suitable one to satisfy this requirement by a series of parametric study[6,7]. Through this study,the information on the distribution of plastic works done by each story was also obtained,therefore, the safety of a structure can be examined at any one story. From the viewpoint of practical design,however,it is convenient to determine the required yield base shear coefficient by carrying out the safety check by Eq.11 at the first story and then to determine the yield shear coefficient for upper stories in accordance with Eq.2.
The ratio,a_1,of the plastic work by the whole structure to that by the first story obtained from above study is

$$ a_1 = \frac{{}_uW_p}{{}_uW_{p1}} = \sum_{i=1}^{N} s_i \, d_i^{-12} \tag{13} $$

in which ${}_uW_{p1}$=plastic work done by the first story; s_i=energy distribution ratio at i-th story relating to the distribution of mass,stiffness and yield shear coefficient of structure and d_i=coefficient at i-th story reflecting an inevitable discrepancy between the optimum and actual yield shear coefficient distribution.

The hysteretic shear force-deflection relationship of a story is related to the monotonic loading curve;thus,the cumulative plastic work is also related to the plastic work under monotonic loading. If this equivalent monotonic loading curve is depicted by Fig.3,in which Q_{y1}=yield base shear force;δ_{y1}=first story yield deformation;δ_{m1}=critical deformation,and $\eta_1=(\delta_{m1}-\delta_{y1})/\delta_{y1}$=critical cumulative ductility ratio,the capacity of cumulative plastic work by the first story in the two directions is

$$_uW_{p1}=2Q_{y1}\,\eta_1\,\delta_{y1}=4W_e c_1\eta_1 \qquad (14)$$

in which $c_1=k_{eq}/k_1$; $k_{eq}=4\pi^2M/T^2$=equivalent spring constant of the whole structure,and k_1=spring constant of the first story.

Combining Eqs.11,12,13 and 14,the required yield shear coefficient of the first story, α_1,is defined by the plastic deformation capacity,η_1,and the intensity of the earthquake,$S_a=(2\pi/T)S_v$,

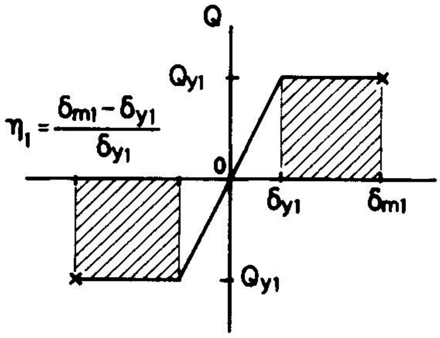

FIG.3 Cumulative ductility ratio

$$\alpha_1 g \geq \frac{1}{\sqrt{1+4c_1a_1\eta_1}} \qquad (15)$$

Eq.15 can be rewritten as

$$Q_{y1}\geq D_s\,Q_e;\qquad D_s=\frac{1}{\sqrt{1+4c_1a_1\eta_1}} \qquad (16)$$

in which $Q_{y1}=\alpha_1 gM$=required yield base shear strength,and $Q_e=S_aM$= elastic maximum shear force corresponding to the spectral acceleration response S_a. Thus the basic skeleton of the ultimate limit state design given by Eq.3 was derived.

3.2 Plastic deformation capacity of steel frame

The evaluation of critical cumulative ductility ratio,η,is necessary to determine the structural characteristic factor D_s. η can be determined by evaluating the plastic deformation capacity of steel frames. Failure of the steel frame under load reversals occurs when the cumulative plastic deformation in one direction reaches the capacity of plastic deformation under monotonic loading. And the plastic deformation capacity of a frame under monotonic loading is governed by the local buckling,flexural torsional buckling and breaking of its member elements.

3.2.1 Frame ductility and member ductilities

As a feasible approach,multibay,multistory frame was reduced into a linkage of unit frames,and the deformation capacity of the unit frame for each story was evaluated on the basis of member ductilities. The deformation of a story unit-frame consists of deformations of columns,beams and joint panels. In general, it is likely that all these elements be plastified at the ultimate state of the frame. However,to develop a simple design rule,it was assumed that one member element of the unit frame(columns or beams) contribute to the plastic deformation of the frame. Furthermore,the effects of plastic shear deformation of joint-panels and the apparent increase of deformability due to Baushinger's effect were considered on the basis of experimental results,and finally an empirical formula that relates the frame ductility on average to the ductility of individual members was obtained as

$$\eta = \frac{2}{3}\,{}_{I}\eta + 2.0 \tag{17}$$

in which ${}_{I}\eta$ = ductility ratio of columns or beams,whichever is smaller.

3.2.2 Ductility ratios of individual members

The slenderness of beams and columns is limited as follows;
For columns: $\lambda_y \leq 70$ (for grade SS41 steel and SM50 steel)
For beams: grade SS41 steel, $\lambda_y \leq 150 + 20n$
grade SM50 steel, $\lambda_y \leq 130 + 20n$

in which λ_y=slenderness ratio of columns and beams with respect to weak axis; and n=number of equally spaced stiffening members. Nominal yield stress of SS41 is 235 MPa and that of SM50 is 324 MPa.
Under these limitations,steel members fail by the local buckling of plate elements of their sections.Based on a large number of laboratory tests,the rotational ductility ratio of members with H-sections,box-sections,and circular hollow sections was evaluated in terms of the width-to-thickness ratio(diameter -to-thickness ratio) and the axial stress[8].

The allowable rotational ductilities of members, ${}_{I}\eta$, are categorized into three classes considering the convenience of the common design practice,and the corresponding ductility ratios of story frame, η, are calculated by Eq.17,as shown in Table 1. And on the basis of the mentioned study,the limiting width-to-thickness ratios corresponding to each ductility class were determined for various shapes with different dimensions and steel grades,as shown in Table 2. Detailed discussions of 3.1 and 3.2 are given in references [9] and [8] respectively.

Table 1 Member and frame ductility ratio

Ductility ratio	Ductility class		
	I	II	III
${}_{I}\eta$	6.0	1.5	0
η	6.0	3.0	2.0

Table 2 d/t(D/t) ratio limitation for each ductility class

Member	Section	Nominal yield stress,MPa	Width-to-thickness ratio		
			Ductility class		
			I	II	III
Column	H-shaped flange	235	10	11	16
		324	8	10	13
Column	H-shaped web	235	43	43	48
		324	37	37	41
Column	Box-shaped	235	33	37	48
		324	27	31	41
Column	Circular tube	235	50	70	100
		324	36	50	73
Beam	H-shaped flange	235	9	11	16
		324	8	9	13
Beam	H-shaped web	235	60	65	71
		324	50	55	61

Note; b=width; D=diameter;and t= wall thickness

3.3 Special provision

In 2.4.1,it is stated that,if the height of a steel building does not exceed 31m and if the structural elements satisfy the prescribed requirements,ultimate limit state design is exempted. The prescribed requirements are enough to guarantee the structure for exhibiting the class I ductility(η=6) in Table 1. And introducing this value of η into Eq.16, the D_s-values are obtained to be 0.25-0.3 depending to a_1 and c_1 values.

And if D_s=0.3 is introduced into Eq.3 assuming that F_{es}=1.0,

$$ {}_uQ_i = 0.3\ Z\ R_t\ A_i\ W_i \tag{18}$$

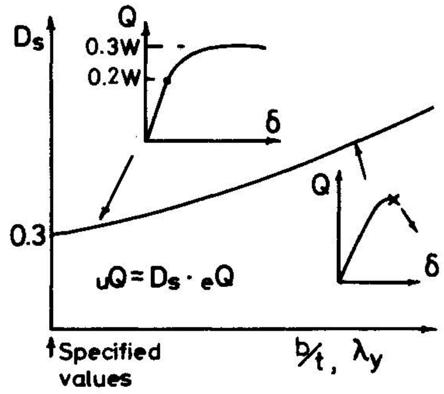

FIG.4 (b/t,λ_y)-D_s relation

On the other hand, Eq.1 for serviceability limit state is rewritten as

$$ {}_eQ_i = 0.2\ Z\ R_t\ A_i\ W_i \tag{19}$$

Comparing Eq.18 and Eq.19,it can be seen that the ultimate limit state design becomes unnecessary if ${}_uQ_i/{}_eQ_i \geq 1.5$. In usual rigid frames,the ultimate strength,${}_uQ_i$,is larger than 1.5 times the elastic limit strength,${}_eQ_i$,due to the effects of moment redistributions and of the increase of bending moment of individual members. The situation is illustrated in Fig.4. This is the rationale of this provision.

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