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STRUCTURAL CHARACTERISTIC FACTOR TO REDUCE SEISMIC FORCES DUE TO ENERGY ABSORBING CAPACITY

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ABSTRACT

Many seismic codes in the world adopt a factor that is to reduce seismic design forces considering an energy absorbing capacity of a structure, *e.g.* R in U.S., Ds in Japan, and Q in Mexico. Though the concept of the factor is well recognized, the value varies much. This is one of the largest differences between the seismic codes in the world. This paper discusses the background of the factor and to what extent the seismic forces can be reduced due to energy absorbing capacity.

KEYWORDS

seismic design; seismic code; seismic design force; structural characteristic factor; P-delta effect; energy absorbing capacity; vertical component

INTRODUCTION

Many seismic codes in the world adopt a factor that is to reduce seismic design forces considering an energy absorbing capacity of a structure. The concept of the factor appeared in the SEAOC code as K factor. Now similar factors are included in many codes, e.g. R in U.S., Ds in Japan, and Q in Mexico. Though the concept of the factor is well recognized, the value varies much. This is one of the largest differences between the seismic codes in the world. For example, for the most ductile structures, U.S. code reduces the design forces by the factor of 1/8=0.125, whereas Japanese code and Mexican code reduce the forces by the factor of 1/4=0.25. This paper discusses the background of the factor and to what extent the seismic forces can be reduced due to energy absorbing capacity.

STRUCTURAL CHARACTERISTIC FACTOR IN SEISMIC CODES

Japanese Factor and Its Background

The 1923 Great Kanto Earthquake (M=7.9) occurred near Tokyo and killed more than one hundred forty thousand people. Most of the deaths occurred because of the fires which spread after the earthquake. Next year (1924) one article was supplemented into the Urban Building Law which had been already enforced at that time. The article says, "Horizontal seismic coefficient shall be more than 0.1." This value was determined taking into account the fact that the expected horizontal coefficient (acceleration) at the ground surface could be $0.3(g)$ (where g is the acceleration due to gravity) and the safety factor of structural materials should be 3. At that time, there had been only one allowable stress for each material which corresponded to allowable stress for permanent load or about one third of the yield level of the material.

During World War II, because of the shortage of construction materials, a dual allowable stress system was introduced into Japan. In 1950, after World War II, the Building Standard Law and its enforcement order replaced the old regulations. The horizontal seismic coefficient became 0.2 and the allowable stress for temporary loads became twice the allowable stress which had been used.

After the San Fernando Earthquake (M=6.4) in 1971, a five year national research project for establishing a new seismic design method was carried out from 1972 to 1977 by the Ministry of Construction, with the cooperation of the Building Research Institute, Public Works Research Institute, Universities, private companies and many other organizations. In 1978 Miyagiken-oki Earthquake (M=7.5) hit the Sendai area and its occurrence accelerated adoption of the new seismic design method. The new seismic design method, which had already been proposed, was reviewed and evaluated for use as a practical design method.

The new seismic design method has been included in the Building Standard Law Enforcement Order, Notifications by the Minister of Construction, etc. which constitute the current seismic design code since 1981. In the current seismic code, the expected horizontal ground acceleration could be from 0.33 to $0.4g$, and the response of the short period buildings could be $1g$ in case the buildings behave elastically. The elastic response of $1g$ force was too large to design the buildings economically. Then the structural characteristic factor Ds to reduce the $1g$ force was introduced, taking into account the energy absorbing capacity of the structure. The minimum value of Ds was decided to be 0.25 or 1/4, since many buildings which had been designed using old codes survived the severe ground motion and the old seismic coefficient for allowable stress design was 0.2 which correspond to 0.25 for ultimate design.

U.S. Factor and Its Background

The U.S. first mandatory seismic codes were published in California after 1933 Long Beach Earthquake. Its design seismic coefficient was 0.025. Since each municipality has its own seismic regulation even within the State of California, in 1957 the Structural Engineers Association of California (SEAOC) organized the Seismology Committee to develop a uniform seismic code which would resolve the differences in seismic codes, *i.e.* the base shear seismic coefficient varied from 0.133 to 0.025 in California. After two years activity, the committee proposed the formula to give the base shear V as follows (Binder and Wheeler, 1960):

$$V = KCW \tag{1}$$

where K is the factor modifying influence of the response performance of different types of construction with varying degree of damping, stiffness, ductility and energy absorption which varies 1.33, 1.0, 0.80, and 0.67. C is the base shear coefficient with a maximum of 0.1. W is the dead load of the structure. Since the Committee did not explicitly mention the expected maximum ground acceleration, the K

factor only reflects the relative differences to adjust the base shear coefficient for different types of construction.

The Applied Technology Council published the "Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC 3)" in 1978 (Applied Technology Council, 1978). The horizontal ground acceleration expected is $0.4g$ in the most active area of seismicity. The elastic response of short period structures is magnified 2.5 times of the ground which results in $1g$ elastic response of the structure. This value coincides with the expected elastic response in the Japanese seismic code. The design seismic coefficients in the two countries, however, differ significantly. This is mainly due to the factor to reduce seismic force considering the energy absorbing capacity of the structure, *i.e.* for the most ductile structures, U.S. code reduces the design forces by the factor of $1/R = 1/8 = 0.125$, whereas Japanese code and Mexican code (International Association for Earthquake Engineering, 1992) reduce the forces by the factor of $Ds = 1/Q = 1/4 = 0.25$. ATC 3 was revised into NEHRP (Federal Emergency Management Agency, 1986), however, the basic concept remains the same.

Comparison of Japanese and U.S. Factors

Most factors involved in seismic codes in Japan and U.S. are very similar and those values are approximately the same, except the structural characteristic factors, whose difference is twice (Ishiyama and Rainer, 1987). The minimum value of $Ds = 0.25$ in Japan corresponds to the ductility factor $\mu = 8.5$, using the following Newmark's formula (Veletsos and Newmark, 1960) ;

$$\mu = \frac{1}{\sqrt{2\mu - 1}} \tag{2}$$

The maximum R factor in U.S. is 8 which corresponds to $\mu = 32.5$, using the same formula. Therefore, the possibility is pointed out that structural yielding may occur during moderate earthquake motions if the structural overstrength is not sufficient (Uang, 1991).

The difference of the factor does not necessarily cause the difference of the structure. Therefore, in order to survey the actual difference, comparative studies were carried out designing the same building to be constructed in Tokyo and in California. As for a reinforced concrete building, Japanese seismic code requires that the amount of concrete is 1.3 times and the amount of reinforcing bars is 2.4 times more than that the U.S. code requires (Okoshi *et al.*, 1989). As for a steel frame building, the amount of steel is almost the same in the two countries. This is due to the fact that only exterior frames have moment resistant connection in U.S. practice, whereas all connections are moment resistant in Japanese practice (Hisatoku and Tahara, 1989).

MINIMUM STRUCTURAL CHARACTERISTIC FACTOR

Analytical Model and Procedure

Since the structural characteristic factors cause great differences in the structure, it is very important to study the minimum value of the factor (or the maximum of R or Q factors). Therefore the analytical study has been carried out as follows:

The analytical model is a single degree of freedom (SDOF) system as shown in Fig. 1 which takes into account P-delta effect. The equation of motion is:

$$\frac{d^2}{dt^2}\phi + 2\xi\frac{2\pi}{T}\frac{t}{dt}\phi + \frac{M(\phi)}{mr^2} = -\frac{\ddot{X}}{r}\cos\phi + \frac{g+\ddot{Y}}{r}\sin\phi \tag{3}$$

where ϕ is the rotation angle, ξ is the fraction of critical damping, T is the natural period, $M(\phi)$ is the restoring moment at the base, r is the height to the mass, $\ddot{X}$ and $\ddot{Y}$ are the horizontal and vertical acceleration of the ground motion, m is the mass and g is the acceleration of gravity. The fraction of critical damping is 0.05 for the elastic analysis and the elastic range of inelastic analysis.

Since one of the objectives of the analysis is to study to what extent the seismic forces can be reduced due to energy absorbing capacity, the analytical model is chosen so that it has infinite ductility. For the inelastic analysis the restoring moment is perfect elasto-plastic (Fig. 1) which means that the ductility of the structure is assumed to be infinite. But the structure can collapse because of P-delta effect. The yield level is gradually decreased until the model collapses. The collapse is assumed to happen when the rotation angle ϕ reaches $\pi/2$.

Multi-story buildings should have been treated as multi-degree-of-freedom (MDOF) systems. But the collapse mechanism of the most ductile multi-story building is the one that yield hinges are formed at the end of beams. Therefore, the multi-story buildings are treated as SDOF systems as shown in Fig. 2. The period $T(s)$ is taken as $T = 0.1N$ where N is the number of stories and the story height is 4 meters. The story number analyzed is from one to twenty ($T = 0.1 \sim 2.0s$).

The input ground motions are listed in Table 1. All inputs are scaled multiplying the factor so that the maximum horizontal velocity becomes 100 $kine(cm/s)$. The vertical component, in case it is available, is considered simultaneously and the same factor as the horizontal component is also multiplied to the vertical component.

The response of the model structure is not proportional to input ground motion, because of the nonlinearity of the model. Therefore inelastic analysis for different levels of input motions was also carried out, i.e. 50 and 25 $kine(cm/s)$ of the maximum horizontal velocity. In this range of ground motion (100, 50, 25 kine), the inelastic response was almost proportional to input motions. This means that structural characteristic factor, which will be discussed in the next subsection, is not affected by the level of input ground motions.

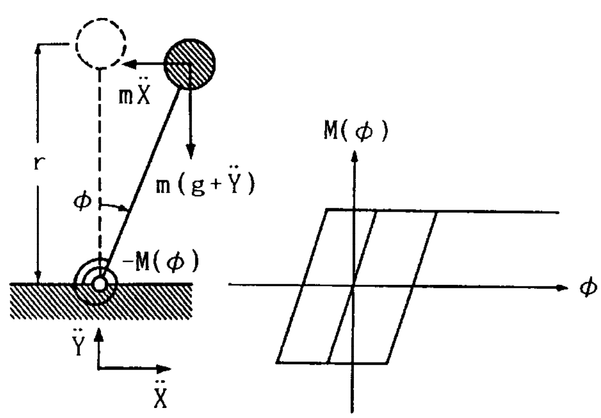

Fig. 1 SDOF analytical model

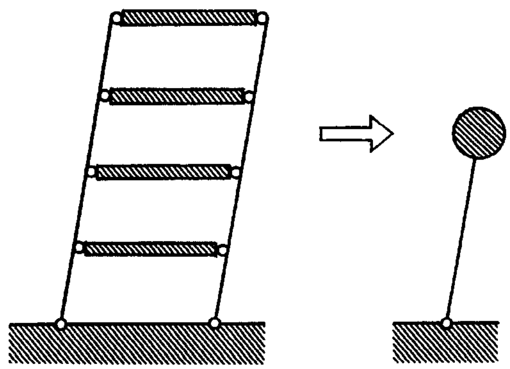

Fig. 2 Equivalent SDOF for MDOF

Table 1 Input ground motions

Earthquake Record	Year	Comp.	Max. Accel. ($gal = cm/s^2$)	Max Vel. ($kine = cm/s$)
		NS	341.7	33.4
El Centro	1940	EW	210.1	36.9
		UD	206.3	8.2
Taft	1952	NS	152.7	15.7
		EW	175.9	17.7
Tokyo 101	1956	NS	74.0	7.6
Sendai 501	1962	NS	57.5	3.5
Osaka 205	1963	EW	25.0	5.1
Hachinohe	1968	NS	225.0	34.1
		EW	182.9	35.8
		NS	258.2	36.2
Miyagiken-oki	1978	EW	202.6	27.6
		UD	152.8	12.5
		N063E	711.4	34.2
Kushiro	1993	N153E	637.2	40.4
		UD	363.4	14.8
		NS	817.2	90.2
Kobe	1995	EW	616.6	74.2
		UD	332.8	39.9

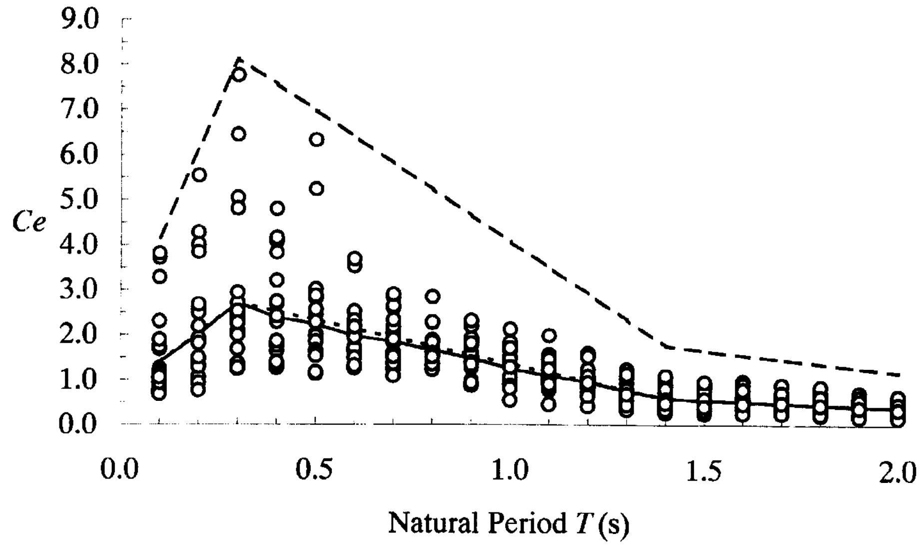

Fig. 3 Elastic base shear coefficient *Ce*

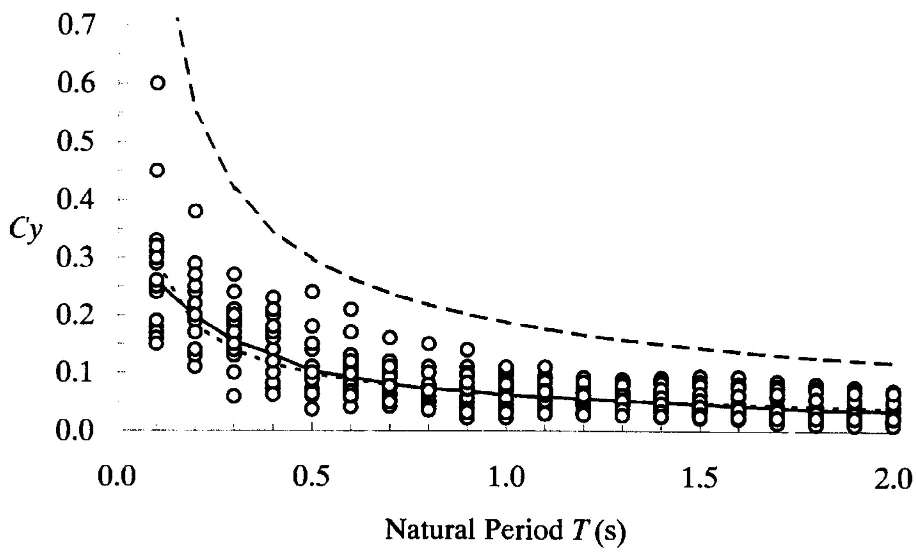

Fig. 4 Yield base shear coefficient *Cy*

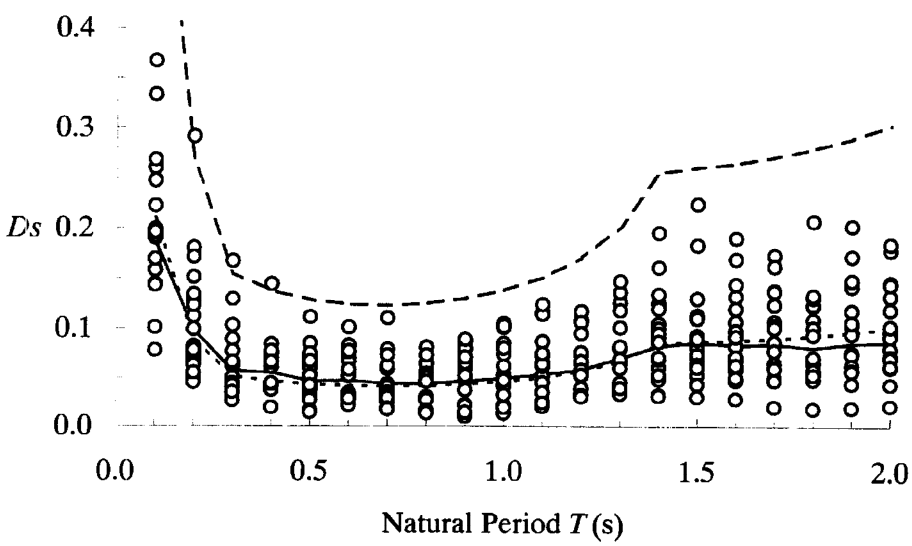

Fig. 5 Structural Characteristic Factor $Ds=Cy/Ce$

Analytical Results and Discussion

The maximum response of the elastic base shear coefficient Ce is shown in Fig. 3. In the figure each circle is the maximum of Ce for each input ground motion and the solid curve shows the average of Ce, assuming the logarithmic normal distribution. The dotted line in the figure shows the approximation of the solid curve by three linear lines, i.e.

$$Ce = \begin{cases} 6.5T + 0.75 & for \quad 0.1 \leq T \leq 0.3 \\ -1.9T + 3.27 & for \quad 0.3 \leq T \leq 1.4 \\ -0.3T + 1.03 & for \quad 1.4 \leq T \leq 2.0 \end{cases} \tag{4}$$

Fig. 4 shows the maximum yield base shear coefficient Cy that leads a structure to collapse. Each circle in the figure is Cy for each input ground motion and the solid curve is the average of Cy, assuming the logarithmic normal distribution. The dotted curve is the approximation of the solid curve by a simple expression, i.e.

$$Cy = \frac{1}{16T^{2/3}} \tag{5}$$

Fig. 5 shows the Cy/Ce which indicates the structural characteristic factor Ds. In the figure, each circle is Cy/Ce for each input ground motion and the solid curve is the average of Cy/Ce, assuming the logarithmic normal distribution. The dotted curve is the average of Cy (Eq. (5) or dotted curve of Fig. 4) divided by the average of Ce (Eq. (4) or dotted curve of Fig. 3).

Fig. 5 shows that Ds is a function of the natural period of the structure. Fig. 5 is similar to the idealized response modification factor which Riddell *et al.* reported, but does not coincides to it especially for long period range (Riddell *et al.*, 1989). As to the average of Ds, it becomes the largest $Ds = 0.2$ at $T = 0.1s$, then decreases $Ds = 0.05$ at $T = 0.3s$, keep constant up to $T = 1.0s$, increases $Ds = 0.1$ at $T = 1.5s$, then keep constant up to $T = 2.0s$.

Fig. 5 indicates that $Ds = 0.25$ in Japanese code is not sufficiently safe for short period structures of $T = 0.1 \sim 0.2s$, even if the structures are infinitely ductile. The figure also indicates that $R = 8$ in U.S. code, which correspond to $Ds = 0.125$ in Japanese code, means that the possibility of collapse of short period structures is very high and the collapse of long period structures is also possible.

Since the divergence of Ce, Cy and Ds is significantly large, the dashed line in each figure from. Fig. 3 to Fig. 5 indicates three times of the average.

SUMMARY AND CONCLUSIONS

In order to investigate to what extent seismic forces can be reduced due to energy absorbing capacity, a SDOF model structure was analysed. The model structure is infinitely ductility, but it can collapse because of P-delta effect.

The structural characteristic factor Ds which can be defined as the yield base shear coefficient Cy (Fig. 4) divided by the elastic base shear coefficient Ce (Fig. 3) was calculated for various input ground motions of 100 kine velocity. In case Ds is smaller than that was obtained by the analysis, the structure collapses even if it is infinitely ductile. Though the value of Ds has significant divergence, the average of Ds is obtained as indicated by a solid line in Fig. 5.

Fig. 5 shows that Ds is a function of the natural period of the structure. As to the average of Ds, it becomes the largest $Ds = 0.2$ at $T = 0.1s$, then decreases $Ds = 0.05$ at $T = 0.3s$, keep constant up to $T = 1.0s$, increases $Ds = 0.1$ at $T = 1.5s$, then keep constant up to $T = 2.0s$. Because of great divergence, the value of Ds can be three times of the average (dashed lines in Fig. 5).

Fig. 5 indicates that $Ds = 0.25$ in Japanese code is not sufficiently safe for short period structures of $T = 0.1 \sim 0.2s$, even if the structures are infinitely ductile. The figure also indicates that $R = 8$ in U.S. code, which correspond $Ds = 0.125$ in Japanese code, means that the possibility of collapse of short period structures is very high and the collapse of long period structures is also possible.

REFERENCES

Applied Technology Council (1978). Tentative provisions for the development of seismic regulations for buildings, *ATC 3-06*, 1-599.

Binder, R.W. and W.T. Wheeler (1960). Building code provisions for aseismic design, *Proc. 2WCEE*, **Vol.3**, 1843-1860.

Federal Emergency Management Agency (1986). NEHRP recommended provisions for the development of seismic regulations for new buildings, part 1 provisions, *Earthquake Hazards Reduction Series* **17**, 1-129.

Hisatoku, T. and T. Tahara (1989). Summary of two comparative case studies done by the second U.S. - Japan workshop -steel structure-, *ATC 15-2*, 183-192

International Association for Earthquake Engineering (1992). Earthquake resistant regulations A World List - 1992

Ishiyama, Y. and J.H. Rainer (1987). Comparison of seismic provisions of 1985 NBC of Canada, 1981 BSL of Japan and 1985 NEHRP of USA, *5th Canadian Conf. Earthq. Eng.*, 747-756.

Okoshi, T., T. Teramoto and K. Nakagawa (1989). Comparison of three reinforced concrete buildings designed using ATC3-06 and current Japanese code, *ATC 15-2*, 219-227.

Riddell, M.R., P. Hidalgo and E. Cruz (1989). Response modification factors for earthquake resistant design of short period buildings, *Earthquake Spectra*, **Vol.5, No.3**, 571-590.

Uang, C.M. (1991). Comparison of seismic force reduction factors used in U.S.A. and Japan, *EESD*, **Vol.20**, 389-397.

Veletsos, A.S. and J.H. Newmark (1960). Effect of inelastic behavior on response of simple systems to Earthquake Motions, *Proc. 2WCEE*, **Vol.2**, 895-912.