EVALUATION OF DESTRUCTIVENESS OF EARTHQUAKE MOTIONS BY COLLAPSE BASE SHEAR COEFFICIENT SPECTRA

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SUMMARY

The evaluation of destructiveness of earthquake ground motions is one of the most important subjects in the field of earthquake engineering. Peak ground acceleration (PGA) or velocity (PGV) has been used to evaluate the intensity of ground motions. Response spectra have also been used to get more information about the relationship between structural response and ground motions. However, they often do not correlate well with the structural damage. Therefore other methods taking into account the inelastic behavior of structures are proposed, e.g. elasto-plastic response spectra, evaluation taking into account of energy concept. An effective and general method, however, has not yet been found. In this paper, we propose a collapse base shear coefficient *Cc* and a *Cc* spectrum (*Cc* vs. *T*). In order to estimate *Cc*, the structure is idealized as a SDOF system considering P-Δ effect. The *Cc* is the maximum yield base shear coefficient when the model structure collapses because of the P-Δ effect and the collapse is assumed to happen when the rotation angle reaches $\pi/2$. The *Cc* spectrum shows a good accordance with the damage extent of structures caused by many previous earthquakes.

INTRODUCTION

Because of the development of earthquake ground motion observation system and the improvement of strong motion seismographs, many earthquake ground motions have been recorded in the world recently. Sometimes the peak acceleration of earthquake records exceeds the acceleration of gravity. However, it has been indicated that the peak value of acceleration or velocity does not correlate to the damage of structures around the observation site. The response spectrum has more information than a single index-value on earthquake motions. However, it is difficult to evaluate the destructiveness of earthquake motions to structures by elastic response spectrum. Therefore, it is demanded that the appropriate method which can evaluate the destructiveness of earthquake motions considering inelastic behavior of structures.

Many methods or indices have been proposed in order to respond the above demand. Benioff proposed the integration of the displacement response spectrum as a destructiveness index of earthquake motions [Benioff, 1934]. This was inherited to Housner's spectrum intensity [Housner, 1952] which was defined as the integration of the velocity response spectrum for a destructive index. Arias [Arias, 1970] proposed that the index which was obtained as an integration of the square of the ground acceleration for the earthquake motion strength. However, Araya and Saragoni [Araya and Saragoni, 1984] showed that Arias intensity predicts the destructiveness capacity in a suitable way only when the frequency content of different earthquakes is similar. They introduced a certain normalization of Arias intensity, defined as a destructiveness factor, which can be expressed as a function of the duration, peak ground acceleration and frequency content of earthquake motions.

Uang and Bertero [Uang and Bertero, 1990], and Bertero [Bertero, 1992] indicated that it is not possible to evaluate the destructiveness of large earthquake motions appropriately by the index calculated from ground motion time history itself or from elastic analysis.

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In Japan, Nagahashi and Kobayashi showed a good correlation between the building damage and the index calculated from the displacement response spectrum of SDOF elasto-plastic system. One of the indices which is widely used is a required strength spectrum. The required strength means the strength with which the structural response becomes a certain ductility factor (displacement ductility ratio) in a SDOF elasto-plastic system. There are other proposals for the indices, such as an index derived from the concept of total input energy [Berg and Thomaides, 1960] or energy input rate (momentary input energy) [Kuwamura et. al., 1997]. However an adequate index is not found yet.

In this paper, it is proposed that the method to evaluate the destructiveness of earthquake motions, using the minimum strength level when the building does not collapse (the collapse base shear coefficient). In this method, it is possible to clearly define collapse by using the finite rotation model.

ANALYTICAL MODEL AND PROCEDURE

The analytical model is shown in Figure 1, where hinges indicate rotation springs. It is a finite rotation model of SDOF systems considering the P-Δ effect which has been proposed by the authors [Ishiyama et. al., 1999]. A MDOF system can be idealized as an equivalent SDOF system (Figure 1) if the structure behaves a weak-beam strong-column multistory building. The equation of motion for this model is as follows;

$$\ddot{\phi} + \frac{C}{I}\dot{\phi} + \frac{M(\phi)}{I} = -\frac{\ddot{X}}{R}\cos\phi + \frac{g+\ddot{Y}}{R}\sin\phi \tag{1}$$

where ϕ is a rotation angle, C is a damping coefficient, $M(\phi)$ is a restoring moment, $\ddot{X}$ and $\ddot{Y}$ are respectively horizontal and vertical acceleration of the ground, and g is the gravitational acceleration. I is the moment of inertia of the building, R is the effective height of the building, and they are given as follows;

$$I = \sum_1^n \left(m_i \ R_i^2 \right) \tag{2}$$

$$R = \frac{I}{\sum_1^n \left(m_i \ R_i \right)} \tag{3}$$

where R_i is given by

$$R_i = \sum_i^n r_i \tag{4}$$

where r_i and m_i are respectively story height and mass of each story, n is the number of stories. R converges to 2/3 of the height of the building, when the story height and mass of each story is equal.

Natural periods of buildings T (s) are assumed to be given by $T = 0.1n$. The mass of each story is equal, and each story height is r = 4 (m). The analysis is carried out for T = 0.1 5.0(s). The damping coefficient is chosen so that the fraction of critical damping is 0.05 for elastic range and proportional to tangent stiffness for inelastic range. The restoring moment is perfect elasto-plastic (Figure 2). It is assumed that there is an infinite ductility, but the model comes to collapse because of the P-Δ effect. In the analysis, the yield level of the restoring moment was gradually decreased until the model collapses. And the base shear coefficient, when the model collapses (ϕ reaches $\pi/2$), is defined as a collapse base shear coefficient Cc.

In many cases, the perfect elasto-plastic restoring moment is used for steel buildings, and the degrading-tri-linear restoring moment is often used for RC buildings. However, it is indicated that the restoring force characteristics slightly affect the in-elastic response [Mahin and Bertero, 1981, Moss et. al., 1986]. And the effect of restoring force characteristics was negligible for a collapse base shear coefficient Cc. Therefore, perfect elasto-plastic restoring moment is used in this analysis.

The earthquake motions used for the analysis are 20 horizontal motions at 10 sites as shown in Table 1. El Centro and Taft were chosen as earthquake motions which have been used for many analysis, and other motions were chosen because the large acceleration was recorded, the building damage was severe, etc. The vertical motions at the same site were also used with the horizontal component in the analysis. Therefore, the effect of vertical motions is included in the analysis, but it should be mentioned that the effect of vertical motions was negligible.

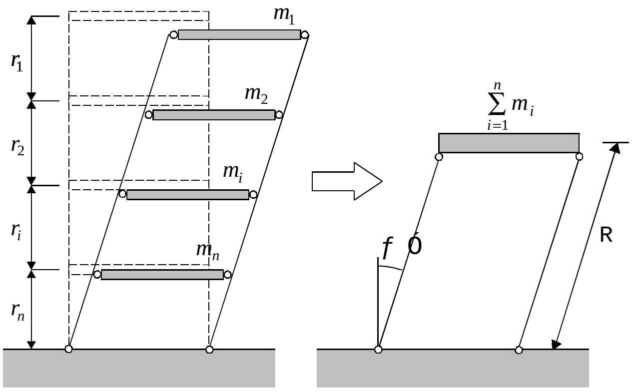

Figure 1: Analytical Model

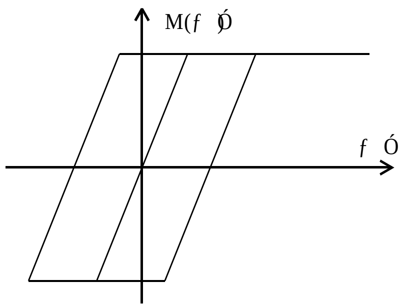

Figure 2: Restoring Moment

Table 1: Earthquake Motions for the Analysis

Earthquake Record (year)	Component	ID	PGA (gal)	PGV (kine)
El Centro (1940)	NS	elc-n	341.7	33.5
	EW	elc-e	210.1	36.9
Taft (1952)	NS	tft-n	152.7	15.7
	EW	tft-e	175.9	17.7
Hachinohe (1968)	NS	hcn-n	225.0	34.1
	EW	hcn-e	182.9	35.8
Mexico-SCT (1985)	NS	sct-n	98.0	38.7
	EW	sct-e	167.9	60.5
Kushiro-BRI (1993)	N063E	ksb-e	711.4	33.5
	N153E	ksb-s	637.2	42.0
Otobe (1993)	NS	otb-n	392.6	12.2
	EW	otb-e	1568.8	57.1
Sylmar (1994)	NS	syl-n	826.8	128.9
	EW	syl-e	592.6	76.9
Tarzana (1994)	NS	trz-n	970.7	77.2
	EW	trz-e	1744.5	110.2
Kobe-JMA (1995)	NS	kbj-n	818.0	90.2
	EW	kbj-e	617.3	74.2
Fukiai (1995)	N240E	fki-w	686.5	57.4
	N330E	fki-n	802.0	122.8

COMPARISON BETWEEN COLLAPSE BASE SHEAR COEFFICIENT SPECTRUM AND REQUIRED STRENGTH SPECTRUM

In order to evaluate the destructiveness of earthquake motions to buildings, one of the methods that is widely used and has good correspondence with the actual damage is the required strength spectrum. For the calculation of the required strength spectrum, a certain ductility factor which corresponds to the level of building damage must be assumed in advance (for example the collapse happens at μ=5), and there are many unclear points such as whether the assumed ductility is appropriate, whether the equal ductility indicates the same damage level for buildings with different natural periods, etc.

Here, the relationships between collapse of buildings and related parameters, such as ductility factor, natural period of building and characteristics of earthquake motions are discussed, comparing the required strength spectrum with the collapse base shear coefficient spectrum (*Cc* spectrum) which authors propose.

Figures 3 and 4 show elastic base shear coefficient spectrum (*Ce* spectrum), three required strength spectra ($C_{\mu=2,5,10}$ spectrum) and *Cc* spectrum for ElCentro-1940-NS and for KobeJMA-1995-NS, respectively.

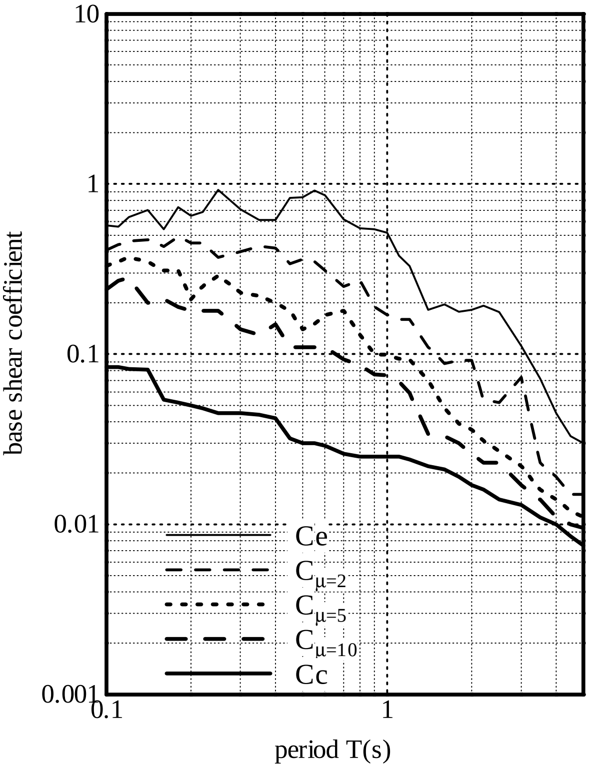

Figure 3: *Cc* spectrum and other spectra

(ElCentro-1940-NS)

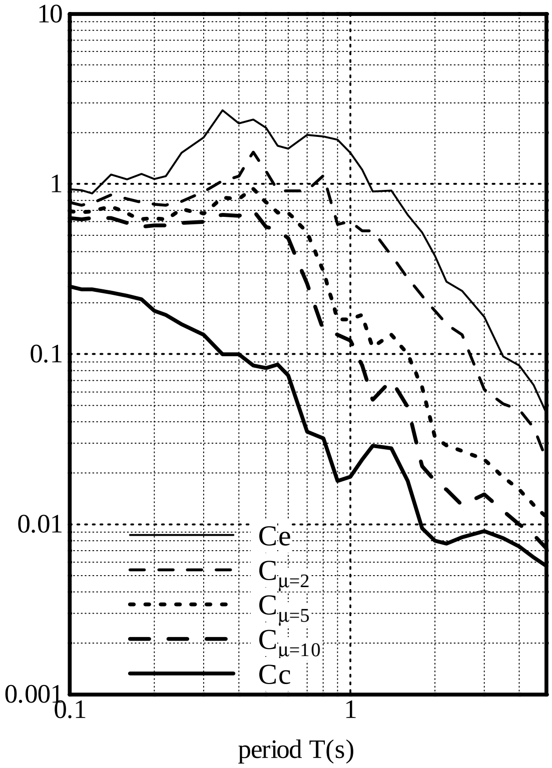

Figure 4: *Cc* spectra and other Spectra

(KobeJMA-1995-NS)

Ce, $C_{\mu=2}$, $C_{\mu=5}$, $C_{\mu=10}$ and *Cc* are almost parallel in Figure 3. This implies that it is able to appropriately evaluate the destructiveness of earthquake motion by any spectra. However, *Cc* approaches to $C_{\mu=5}$ and $C_{\mu=10}$ in the long period range over 1.0(s), so it means that the condition when the ductility factor μ exceeds about 5 is considerably close to collapse for long period buildings subjected to ElCentro-1940-NS.

That is to say, in case of ElCentro-1940-NS, the required strength spectrum which was obtained to satisfy the fixed ductility factor such as $C_{\mu=5}$ or $C_{\mu=10}$, expresses a damage level with the considerable margin to collapse for short-period buildings and with little margin to collapse for long-period buildings. Therefore the damage level of the same ductility factor is considerably different among buildings of different natural periods.

The required strength spectrum (for example, $C_{\mu=10)}$ and *Cc* spectrum approach each other as the period becomes longer not only for ElCentro-1940-NS in Figure 3 but also for KobeJMA-1995-NS in Figure 4. However, probably because of period characteristics of Kobe record, Figure 4 shows different tendency that is not observed in Figure 3 as follows. The difference between $C_{\mu=10}$ and *Cc* spectra for the period range of 0.3 1.0(s) is larger than that for very short period range (0.1 0.2(s)). And the ductility factor μ=10 expresses the damage level which is considerably far from collapse for the period range (0.3 1.0(s)). This means that the ductility factor μ=10, which is assumed as a level of severe damage or collapse of buildings, may not be the assumed damage level. Therefore, the difference between required strength spectrum and *Cc* spectrum is considerably affected not only by the natural period of buildings but also by period characteristics of earthquake motions.

From the above discussion, *Cc* spectrum that can clearly define collapse seems to be more adequate for evaluating the destructiveness of earthquake motions than the required strength spectrum.

BUILDING DAMAGE OBSERVED AT EACH RECORD SITE

For evaluating the destructiveness of earthquake motion, building damage at each record site (or vicinity) are summarized as follows.

For El Centro and Taft, it was reported that no significant damage was observed.

Hachinohe is a record of the Hachinohe harbor during 1968 Tokachi-Oki Earthquake. By this earthquake, some RC buildings were damaged, and especially, low rise buildings of 3 4 story or less suffered severe damage.

The building damage of 1985 Mexico earthquake was severe, and especially buildings over 6, 7 story received severe damage. Mexico-SCT was recorded at the vicinity where the damage was severe.

During 1993 Kushiro earthquake, there was almost no direct damage regardless of structural types and the natural period of buildings, but there was some indirect damage of buildings caused by damage of ground. It is reported that the site of Kushiro Local Meteorological Observatory, where Kushiro-BRI was recorded, responses larger than other places.

Otobe is a record at Meiwa elementary school during the largest aftershock of 1993 Hokkaido-Nansei-Oki Earthquake, and all buildings including this school building had almost no damage.

After 1994 Northridge earthquake, about 3000 buildings were evaluated as unsafe (Red tag). The site of Sylmar is about 14km from the epicenter to the north-northwest, and the seismic intensity was estimated about 6 of the Japan Meteorological Agency (JMA) intensity. The site of Tarzana is about 5km from the epicenter to the south, though it was very close to the epicenter, building damage was little in the vicinity.

During 1995 Hyogo-ken Nanbu Earthquake, about 55000 buildings (most of them were 1 2 story wooden houses) were collapsed in Kobe City. Except wooden houses, most buildings of severe damage and collapse were 3 5 story buildings of steel structure and 5 7 story RC buildings. Kobe-JMA (Kobe marine meteorological observatory) is nearby the epicenter, but it is not in area of JMA intensity of 7. Fukiai (OSAKA GAS Fukiai conjunction supply center) is in the region of JMA intensity of 7, but the damage is comparatively moderate around the site.

It can be said that Mexico (1985), Hyogo-ken Nanbu (1995) and Northridge (1994) earthquakes caused severe damage, but during other earthquakes the damage was rather minor.

EVALUATION OF DESTRUCTIVENESS OF EARTHQUAKE MOTION RECORDS

Here, the destructiveness is evaluated on each earthquake motion record shown in Table 1. The elastic base shear coefficient *Ce* spectrum (damping 5%) of each earthquake record is shown in Figures 5 and 6, and *Cc* spectrum calculated by the method which has been described in section 2 is shown in Figures 7 and 8. *Cc* spectrum is shown, because it is still used as a simple evaluation method of earthquake motions. The analysis was carried out for all earthquake motions in Table 1, but the analytical results for 10 records are shown in Figures 5, 6, 7 and 8.

Elastic Response Base Shear Coefficient *Ce* spectra

Ce spectra of Figure 5 show that *Ce* of ksb-e (ID in Table 1 is used to indicate the record) is quite large in the period range shorter than 0.4(s). Though hcn-n and sct-e are the motions which dominate long period content, *Ce* of hcn-n is almost the same as elc-n in the long period range over 1(s), and *Ce* of sct-e is only large in the period range around 2(s). Figure 6 indicates that *Ce's* of otb-e and trz-e that recorded the large acceleration are very big in the short period range of less than 0.4(s). Though *Ce's* of syl-n, kbj-n and fki-n are big in short period range, they are not as big as otb-e or trz-e, and they are smaller than ksb-e. *Ce's* of syl-n, trz-e, kbj-n and fki-n are almost equivalent in the period range over 0.5(s), and *Ce* of fki-n is the biggest in the long period range over

1(s). *Ce's* of these 4 records are much bigger than other 6 records in the middle and long period range. The decrease of *Ce* of otb-e is remarkable for the long period range, i.e. *Ce* is very big in the short period range, whereas in the long period range over 1(s), it is the smallest among 10 records.

Table 2 shows a ranking of destructiveness for all 20 records at periods of 0.2, 0.5, 1.0, 2.0, and 4.0(s) in the order of *Ce's* which are shown in parentheses.

Comparing the observed building damage described in section 4 and the analytical results shown in Figures 5, 6 and Table 2, it is noted that the elastic response spectrum may not be suitable to evaluate the destructiveness in most cases, though the spectrum gives some information related to the destructiveness and period characteristics of earthquake motions.

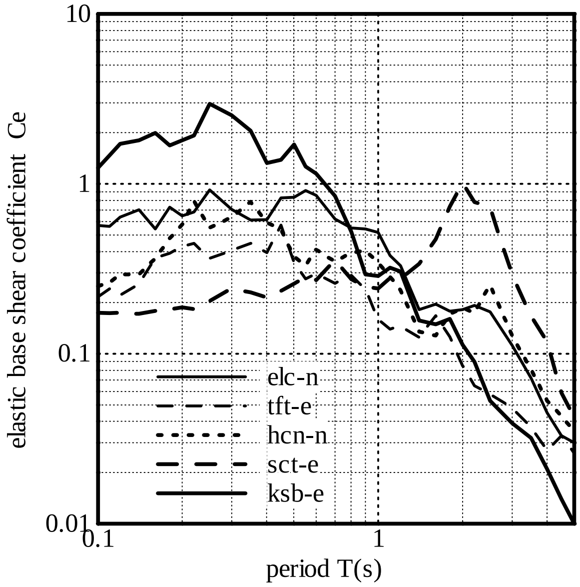

Figure 5: *Ce* Spectra (part 1)

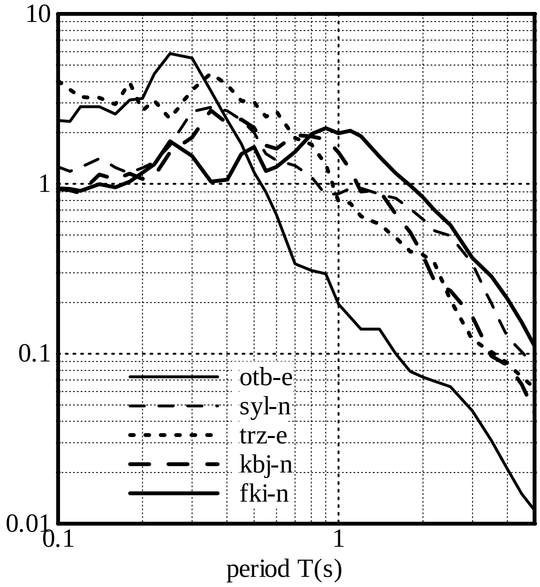

Figure 6: *Ce* Spectra (part 2)

Table 2: Ranking of Destructiveness and Ce values in parentheses

Ranking	Natural Period				
	0.2(s)	0.5(s)	1.0(s)	2.0(s)	4.0(s)
1	otb-e(3.19)	trz-e(3.02)	fki-n(1.99)	sct-e(1.01)	fki-n(0.21)
2	trz-e(2.70)	kbj-n(2.14)	kbj-n(1.53)	fki-n(0.84)	syl-n(0.13)
3	trz-n(2.36)	syl-n(2.01)	kbj-e(1.22)	sct-n(0.64)	sct-e(0.12)
4	ksb-s(1.82)	trz-n(1.81)	syl-n(0.88)	syl-n(0.62)	trz-n(0.10)
5	ksb-e(1.81)	ksb-e(1.71)	trz-e(0.79)	syl-e(0.46)	elc-e(0.09)
6	fki-w(1.61)	fki-n(1.65)	fki-w(0.77)	trz-e(0.38)	trz-e(0.09)
7	syl-n(1.25)	syl-e(1.35)	ksb-s(0.75)	kbj-n(0.38)	syl-e(0.09)
8	fki-n(1.16)	ksb-s(1.29)	syl-e(0.61)	trz-n(0.32)	fki-w(0.09)
9	kbj-n(1.07)	otb-e(1.17)	hcn-e(0.56)	fki-w(0.25)	kbj-n(0.09)
10	kbj-e(1.06)	kbj-e(1.09)	elc-n(0.52)	kbj-e(0.25)	kbj-e(0.09)
11	syl-e(0.92)	elc-n(0.84)	trz-n(0.50)	elc-e(0.22)	sct-n(0.06)
12	otb-n(0.71)	fki-w(0.74)	hcn-n(0.34)	ksb-s(0.20)	hcn-e(0.06)
13	elc-n(0.65)	elc-e(0.64)	ksb-e(0.29)	hcn-n(0.19)	hcn-n(0.05)
14	hcn-n(0.58)	hcn-e(0.63)	elc-e(0.28)	hcn-e(0.19)	elc-n(0.05)
15	elc-e(0.51)	otb-n(0.47)	sct-e(0.24)	elc-n(0.18)	ksb-s(0.04)
16	tft-e(0.43)	tft-n(0.38)	otb-e(0.20)	ksb-e(0.11)	tft-n(0.03)
17	tft-n(0.35)	hcn-n(0.37)	sct-n(0.19)	tft-e(0.09)	tft-e(0.03)
18	hcn-e(0.31)	tft-e(0.35)	tft-n(0.18)	otb-e(0.07)	ksb-e(0.02)
19	sct-e(0.19)	sct-e(0.26)	tft-e(0.16)	tft-n(0.06)	otb-e(0.02)
20	sct-n(0.12)	sct-n(0.14)	otb-n(0.06)	otb-n(0.03)	otb-n(0.01)

Collapse Base Shear Coefficient *Cc* Spectra

Figure 7 shows that *Cc* of ksb-e is very big in the very short period range less than 0.2(s). *Ce* of ksb-e (Figure 5) was very big in the short period range of less than 0.4(s). Though *Cc* of sct-e in the period range shorter than 0.3(s) is not so big, in the period range longer than 0.3(s), it is the largest among 10 records, and it is remarkably bigger than any other records in the range longer than 1.0(s). *Cc* of hcn-n is not big in the short period range, but it is comparatively constant regardless of the period (similar behavior of sct-e), because of long period content of the record. Therefore, other records are surpassed in the long period range, and in the long period range over 1(s), *Cc* of hcn-n becomes the second biggest to sct-e. Figure 8 shows that as to otb-e which showed the largest elastic response in the short period range, its *Cc* is small at any period and the smallest among 10 records. In the very short period range shorter than 0.2(s), *Cc's* of trz-e, kbj-n, fki-n and ksb-e are almost the same and the biggest among 10 records. *Cc* of fki-n is remarkably bigger than other records in the period range longer than over 0.2(s), but it is not big in the long period range over 1(s). Though *Cc* of syl-n is not big in the short period range, it does not decrease even if the period become longer because of the long period content, and therefore, it becomes as big as trz-e at about 1.0(s), and it becomes the second biggest to sct-e in the period range longer than 1.5(s).

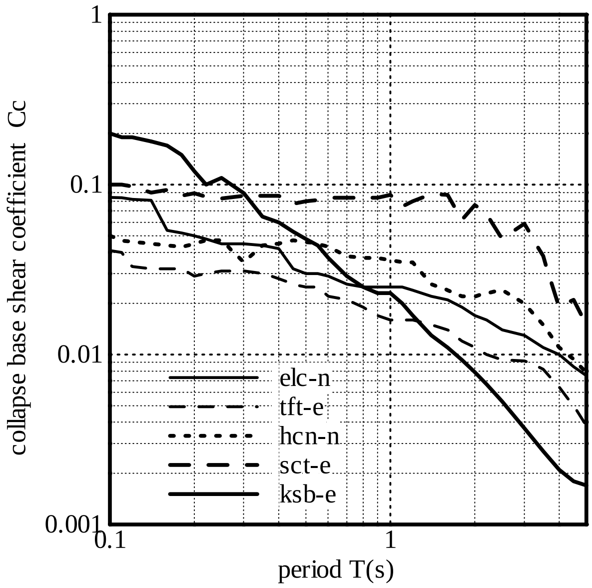

Figure 7: *Cc* Spectra (part 1)

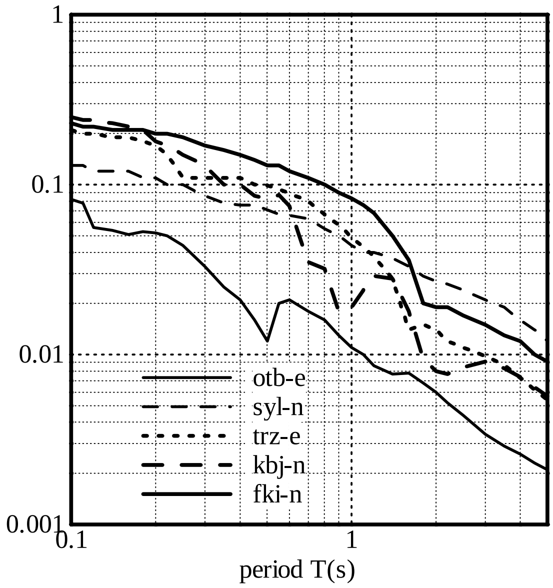

Figure 8: *Cc* Spectra (part 2)

Table 3: Ranking of Destructiveness and Cc values in parentheses

Ranking	Natural Period				
	0.2(s)	0.5(s)	1.0(s)	2.0(s)	4.0(s)
1	fki-n(0.200)	fki-n(0.130)	sct-e(0.087)	sct-e(0.076)	sct-e(0.019)
2	trz-n(0.180)	trz-e(0.099)	fki-n(0.083)	sct-n(0.050)	sct-n(0.016)
3	kbj-n(0.180)	kbj-n(0.083)	sct-n(0.063)	syl-n(0.027)	syl-n(0.016)
4	trz-e(0.170)	sct-e(0.080)	trz-e(0.049)	syl-e(0.023)	elc-e(0.013)
5	kbj-e(0.140)	syl-n(0.071)	syl-n(0.044)	hcn-n(0.022)	fki-n(0.012)
6	ksb-e(0.120)	trz-n(0.062)	trz-n(0.039)	fki-n(0.019)	trz-n(0.012)
7	ksb-s(0.110)	ksb-s(0.062)	syl-e(0.039)	trz-n(0.018)	syl-e(0.011)
8	syl-n(0.110)	sct-n(0.061)	hcn-n(0.036)	elc-e(0.018)	hcn-n(0.011)
9	sct-e(0.089)	fki-w(0.057)	hcn-e(0.028)	elc-n(0.017)	elc-n(0.010)
10	fki-w(0.080)	syl-e(0.051)	elc-n(0.025)	fki-w(0.017)	fki-w(0.010)
11	sct-n(0.076)	ksb-e(0.048)	ksb-s(0.024)	kbj-e(0.016)	kbj-e(0.010)
12	syl-e(0.070)	hcn-n(0.046)	ksb-e(0.023)	trz-e(0.014)	kbj-n(0.007)
13	elc-e(0.066)	kbj-e(0.045)	elc-e(0.023)	hcn-e(0.011)	trz-e(0.007)
14	otb-e(0.052)	hcn-e(0.038)	fki-w(0.022)	tft-e(0.011)	hcn-e(0.007)
15	elc-n(0.050)	elc-e(0.034)	kbj-e(0.022)	tft-n(0.009)	tft-e(0.006)
16	hcn-e(0.049)	tft-n(0.033)	kbj-n(0.019)	ksb-s(0.009)	tft-n(0.006)
17	hcn-n(0.045)	elc-n(0.030)	tft-e(0.016)	kbj-n(0.008)	ksb-s(0.004)
18	otb-n(0.044)	tft-e(0.025)	tft-n(0.014)	ksb-e(0.008)	otb-n(0.004)
19	tft-n(0.040)	otb-n(0.021)	otb-n(0.011)	otb-n(0.006)	otb-e(0.003)
20	tft-e(0.029)	otb-e(0.012)	otb-e(0.011)	otb-e(0.006)	ksb-e(0.002)

Table 3 shows a ranking of destructiveness for all 20 records at periods of 0.2, 0.5, 1.0, 2.0, and 4.0(s) in the order of *Cc's* which are shown in parentheses.

Figures 7, 8 and Table 3 indicate that fki-n is the most destructive earthquake motion for structures with long natural period, and sct-e is the most destructive for structures with short natural period. Comparing the observed building damage which has been described in section 4 and the analytical results of this section, the correlation of damage level and *Cc* spectrum is considerably good. Therefore, the *Cc* spectrum seems to be effective as a measure to evaluate the destructiveness of earthquake motions.

CONCLUSION

In order to evaluate the destructiveness of earthquake motions, it is proposed that the method based on elasto-plastic earthquake response analysis using finite rotation model considering the P-Δ effect.

Considering the fact that if the strength is not enough the finite rotation model collapses because of the P-Δ effect even if the ductility is infinite, authors propose a collapse base shear coefficient *Cc* that is defined as a strength (the base shear coefficient) which is a boundary value to collapse and the *Cc* spectrum.

Comparing the *Cc* spectrum with the required strength C_μ spectrum, it is shown that the margin of buildings to collapse estimate by C_μ spectrum of the same ductility factor differs greatly by period characteristics of the earthquake motions.

Good correlation is observed between building damage caused by earthquakes and *Cc* spectrum, therefore *Cc* spectrum can be used as an index to evaluate the destructiveness of earthquake motions against buildings.

REFERENCES

Araya, R. and Saragoni, G.R. (1984), "Earthquake accelerogram destructiveness potential factor", *Proc. 8th world conf. earthquake eng.*, pp438-469.

Arias, a. (1970), "A measure of earthquake intensity", *Seismic Design for Nuclear Power Plants*, MIT Press, Cambridge, MA, pp438-469.

Benioff, H. (1934), "The physical evaluation of seismic destructiveness", *Bull. seismol. soc. Amer*, Vol.24, pp398-403.

Berg, G.V. and Thomaides, S.S. (1960), "Energy consumption by structures in strong-motion earthquakes", *Proc. 2nd world conf. earthquake eng.*, Vol.2, pp681-697.

Bertero, V.V. (1992), "Lessons learned from recent catastrophic earthquakes and associated research", *Proc. 1st Torroja int. conf.*, pp410-411.

Housner, G.W. (1952), "Spectrum intensities of strong-motion earthquakes", *Proc. symp. Earthquake and Blast effects on structures*, Earthquake Engineering Research Institute.

Ishiyama, Y. et. al. (1999), "Extreme of Structural Characteristic Factor – Analysis of SDOF model considering P-Δ effect", *J. Struct. Constr. Eng.*, AIJ, No.520, pp29-35.

Kuwamura, H. et. al. (1997), "Energy input rate in earthquake destructiveness – Comparison between epicentral and oceanic earthquake", *J. Struct. Constr. Eng.*, AIJ, No.491, pp29-36.

Mahin, S.A. and Bertero, V.V. (1981), "An evaluation of inelastic seismic design spectra", *Journal of Structural Division*, American Society of Civil Engineers, Vol.107, ST9, pp1777-1795.

Moss, P.J. et. al. (1986), "Seismic response of low-rise buildings", *Bull. New Zealand national soc. earthquake eng.*, Vol.19, pp180-198.

Uang, C.M. and Bertero, V.V. (1990), "Evaluation of seismic energy in structures", *Earthquake eng. struct. dyn.*, Vol.19, pp77-90.