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Development of the Damping Coefficients for Weak and Moderate Earthquake Ground Motions

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Abstract

Most of seismic design code provisions provide the design response spectra for defining design earthquake ground motions. The design spectra in the code provisions generally come under the 5% of critical damping value, which corresponds to the responses of common structure under the design earthquake. Energy dissipation devices and seismic isolation systems became more popular and the design response spectra at higher damping levels are required. Damping coefficients can be effectively used in conversion of 5%-damped design spectra into other damping levels. These coefficients in the current seismic design code provisions are based on the strong ground motion records. Since the weak and moderate earthquake data have different characteristics from those of strong earthquake data, the application of these coefficients should be investigated in the weak and moderate earthquakes zones. In this study, damping coefficients based on the weak and moderate ground motions were developed and compared to those of current seismic design code provisions.

***Key Words*:** Response spectra, Design spectra, Ground motions, Damping coefficient

요 지

대부분의 내진설계기준에서는 설계지반운동을 정의하기 위해서 설계스펙트럼을 제시하고 있다. 기준에서 제시되는 설계스펙트럼은 일반적으로 5% 임계감쇠비에 대한 것이며, 이것은 일반적인 건축구조물에 적용할 수 있는 것이다. 에너지 소산장치나 면진 시스템의 적용이 점차 증가하고 있으며, 이러한 장치를 적용한 건축구조물의 내진해석을 위해서는 5% 임계감쇠비를 초과하는 설계스펙트럼이 필요하다. 5% 임계감쇠비에 대한 설계스펙트럼을 다른 임계감쇠비에 대한 설계스펙트럼으로 변환하기 위해서는 감쇠계수가 효과적으로 이용될 수 있다. 현재의 내진설계기준에서 제시하고 있는 감쇠계수는 강진자료를 바탕으로 제시된 것이다. 중진 및 약진은 강진과는 다른 특성을 가지므로, 이러한 감쇠계수가 중진 및 약진 지역에 적용하는 것은 충분한 검토가 필요할 것이다. 이 논문에서는 중진 및 약진자료를 이용한 감쇠계수를 제시하고, 현재 설계기준에서 제시하고 있는 감쇠계수와 비교하였다.

핵심용어 : 반응스펙트럼, 설계스펙트럼, 지반진동, 감쇠계수

1. Introduction

Since the concepts of the response spectrum and design spectrum were introduced in earthquake engineering, they have been widely used to estimate the force and deformation demands for structures subject to earthquake ground motions. Currently, design spectra form the basis of design seismic forces and design ground motions in most seismic design code provisions.

The design spectra in the code provisions generally come under the 5% of critical damping value, which corresponds to the responses of common structure under the design ground motions. Energy dissipation devices and seismic isolation systems became increasingly popular, and performance-based design approaches have been developed, such as capacity spectrum method, that require response spectra of higher damping level than 5% of critical value.

Damping coefficients can be effectively used in conversion of the 5%-damped design spectra into other damping levels. The coefficients used in the current seismic design code provisions are based on the ratios of median spectrum amplification factors of Newmark and Hall (1982). Their amplification factors were developed from the 28 acceleration records. These accelerograms represented a fairly complete set of strong ground motion records at the time of their study.

The weak and moderate earthquake ground motions have different characteristics from those of strong earthquakes. Therefore, the application of damping coefficients embodied in the contemporary code provisions should be investigated in the regions where weak and moderate earthquakes are expected to occur. In this study, damping coefficients based on the weak and moderate ground motions are developed and compared to those of Newmark and Hall, and current seismic design code provisions.

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The coefficients based on the ratios of the response spectra from 101 weak and moderate ground motions are statically evaluated.

2. Damping Coefficients in Current Practice

Damping coefficients can be represented by the following basic formula:

$$R_x = \frac{R_5}{B} \qquad (1)$$

where R_x and R_5 are the spectral ordinates of the x% and 5% damped response spectrum at a given period, respectively, and B is the corresponding damping coefficient. Damping coefficients of current seismic design code provisions are based on the spectrum amplification factors developed by Newmark and Hall (1982).

In the monograph Newmark and Hall proposed the spectrum amplification factors for various damping levels, which can be applied for the systematic construction of elastic design spectra. These factors are distinguished among the domains of constant acceleration, constant velocity and constant displacement. Table 1 shows the damping coefficients derived from the median spectrum amplification factors of Newmark and Hall. The coefficients are represented separately for the displacement, velocity and acceleration domains.

From the values listed in Table 1, it is clear that the effect of damping increases as the period decreases. That is, the damping coefficients increase as the period shifts from the displacement domain to the velocity domain and from the velocity domain to the acceleration domain. Since damping is generally more effective as the number of cycles increase, the increase in damping coefficients with the decreases in period seems reasonable (Naeim and Kircher, 1991).

The first code provisions for damping coefficients were appeared in the 1991 UBC (ICBO, 1991). The coefficients were provided only for the application to the base isolation systems. Since the base isolated structures have long periods, the coefficients were based on the spectrum amplification factors of in velocity domain. Table 2 represents the damping coefficients in the 1991 UBC.

The damping coefficients of 1991 UBC are almost identical to those of Newmark and Hall for damping levels up to 20%. At higher levels of damping, the coefficients are somewhat less than those of Newmark and Hall. These conservative coefficients were adopted for the design of buildings with very highly damped isolation system.

Table 2. Damping Coefficients of 1991 UBC

Damping Value (Percentage of Critical)	Damping Coefficient
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
= 50	2.0

The following seismic design code provisions use the same damping coefficients for the design of seismically isolated structures as those of 1991 UBC:

- 1994 UBC (ICBO, 1994),
- 1994 NEHRP Provisions (FEMA, 1995),
- 1997 UBC (ICBO, 1997),
- 1997 NEHRP Provisions (FEMA, 1998),
- 2000 IBC (ICC, 2000).

The NEHRP Guidelines for Seismic Rehabilitation of Buildings (FEMA, 1997) adopted the damping coefficients for both velocity domain and acceleration domain. Table 3 shows the damping coefficients of 1997 NEHRP Guidelines.

In the table B_1 represents the coefficients of 1-s period, that is, the coefficients for velocity domain. The coefficients for acceleration domain are expressed as B_s, which means the damping coefficients for short period. The short-period damping coefficients are required for the design of short-period buildings with damper systems and for nonlinear push over analysis of buildings using the capacity -spectrum method (ATC,1996).

Table 3. Damping Coefficients of 1997 NEHRP Guidelines

Damping Value (Percentage of Critical)	Damping Coefficient	
	B_1	B_s
5	1.0	1.0
10	1.2	1.3
20	1.5	1.8
30	1.7	2.3
40	1.9	2.7
= 50	2.0	3.0

Table 1. Damping Coefficients based on the Median Spectrum Amplification Factors of Newmark and Hall (1982)

Damping Value (Percentage of Critical)	Damping Coefficient		
	Displacement Domain	Velocity Domain	Acceleration Domain
5	1.00	1.00	1.00
10	1.16	1.21	1.29
20	1.37	1.53	1.80
30	1.54	1.80	2.36
40	1.68	2.07	3.02
50	1.81	2.34	3.85

The damping coefficients in velocity domain, B_1, are identical to those in 1991 UBC. The damping coefficients in acceleration domain, B_s, were decided in the same manner as the damping coefficients in velocity domain. That is, the damping coefficients of 1997 NEHRP Guidelines for short period are almost identical to those of Newmark and Hall for damping levels up to 20%. At higher levels of damping, the coefficients are somewhat less and more conservative than those of Newmark and Hall.

3. Weak and moderate earthquake ground motions

For weak and moderate ground motions, 101 records were selected from a database compiled by the U.S. National Geographic Data Center (NGDC). The conditions for the selected records were: (1) the peak ground accelerations ranged between 30 and 150 cm/sec^2; (2) the epicentral distance was less than 50km; (3) ground acceleration records of horizontal component were adopted; (4) it passed correction process; (5) it was recorded in freefields or ground level of buildings; (6) it had precise information about the soil condition of recording station; and (7) the data were not collected during pre-shock or after-shock. Table 3 presents the selected ground motions.

Among the conditions discussed, (1) and (2) are directly

Table 3. Selected Weak and Moderate Ground Motions

Eq. no.	NGDC id.	Event name	Year	PGA (gal)
001	Alg01-127	Izmar	1997	136.4
002	ind01-070	Koyna	1968	75.5
003	ind01-072	Koyna	1968	45.3
004	ind01-082	Koyna	1970	53.7
005	ind01-084	Koyna	1970	68.5
006	mex03-064	Mexico city	1985	138.5
007	mex03-065	Mexico city	1985	137.8
008	mex03-068	Mexico city	1985	44.3
009	mex03-070	Mexico city	1985	121.0
010	mex03-071	Mexico city	1985	85.9
011	usaak01-037	Sitca	1972	70.1
012	usaak01-039	Sitca	1972	91.3
013	usaak02-040	Alaska	1964	34.2
014	usaak02-068	Sitca	1972	76.5
015	usaak02-070	Sitca	1972	89.4
016	usaak02-074	Alaska	1974	98.3
017	usaak02-076	Alaska	1974	117.8
018	usaak02-100	Alaska	1976	65.7
019	usaak02-124	Alaska	1983	46.9
020	usaak02-126	Alaska	1983	40.5
021	usaca02-025	San Fernando	1971	86.8
022	usaca02-026	San Fernando	1971	138.0
023	usaca13-001	San Fernando	1971	87.5
024	usaca13-104	San Fernando	1971	143.5
025	usaca13-106	San Fernando	1971	119.3
026	usaca13-107	San Fernando	1971	109.5
027	usaca13-143	San Fernando	1971	147.7
028	usaca30-08a	Westmorland	1981	102.5
029	usaca30-08b	Westmorland	1981	78.3
030	usaca36-26a	Morgan hill	1984	85.9
031	usaca36-26b	Morgan hill	1984	95.0
032	usaca38-037	Hollister	1974	134.7
033	usaca38-039	Hollister	1974	94.1
034	usaca38-049	Cape Mendocino	1975	92.1
035	usaca38-051	Cape Mendocino	1975	72.4
036	usaca38-063	Humboldt	1975	103.0
037	usaca38-140	Oroville	1975	82.5
038	usaca38-141	Oroville	1975	90.6
039	usaca39-019	Whittier narrows	1987	121.3
040	usaca42-007	Whittier narrows	1987	121.4
041	usaca42-009	Whittier narrows	1987	133.8
042	usaca66-025	Loma Prieta	1989	78.2
043	usaca66-027	Loma Prieta	1989	74.8
044	alg01-132	Horasan	1983	148.5
045	jap03-001	River Ebo	1956	75.6
046	jap03-002	River Ebo	1956	57.8
047	jap03-004	River Ebo	1956	69.1
048	jap03-005	River Ebo	1956	52.6
049	jap03-015	Chiba	1963	92.6
050	jap03-016	Chiba	1963	79.2
051	jap03-018	Ibaraki	1964	56.3
052	jap03-019	Ibaraki	1964	39.7
053	jap03-021	Ibaraki	1964	136.7
054	jap03-022	Ibaraki	1964	41.4
055	jap03-023	Ibaraki	1964	31.7
056	jap03-031	Suruga	1965	108.6
057	jap03-046	Japanese	1966	118.8
058	jap03-058	Japanese	1966	128.5
059	jap03-059	Japanese	1966	99.7
060	jap03-115	Japanese	1968	49.0
061	jap03-116	Japanese	1968	86.0
062	jap03-117	Chiba	1968	72.9
063	jap03-118	Chiba	1968	33.5
064	jap03-142	Niigata	1971	148.1
065	jap03-147	Chiba	1971	47.6
066	jap03-158	Chiba	1974	37.7
067	jap03-159	Chiba	1974	111.6
068	jap03-169	Oita	1975	96.2
069	jap03-170	Oita	1975	137.5
070	usaak02-123	Alaska	1983	55.5
071	usaak02-135	Alaska	1983	40.5
072	usaca13-046	San Fernando	1971	119.4

073	usaca13-047	San Fernando	1971	112.3
074	jap03-057	Japanese	1966	114.2
075	jap03-105	Saitama	1968	80.3
076	jap03-106	Saitama	1968	127.4
077	usaca01-028	San Jose	1955	100.2
078	usaca01-143	San Fernando	1971	131.7
079	usaca02-202	Nothern CA	1941	118.6
080	usaca02-203	Nothern CA	1941	113.6
081	usaca02-206	Nothern CA	1949	119.4
082	usaca02-238	Ferndale	1967	103.1
083	usaca13-029	San Fernando	1971	146.0
084	usaca21-034	Mt. Diablo	1980	121.0
085	usaca24-042	Imperial Valley	1979	113.4
086	usaca24-046	Imperial Valley	1979	113.4
087	usaca24-048	Imperial Valley	1979	138.7
088	usaca24-052	Imperial Valley	1979	136.2
089	usaca24-054	Imperial Valley	1979	114.6
090	usaca24-055	Imperial Valley	1979	136.2
091	usaca24-057	Imperial Valley	1979	139.3
092	usaca30-005	Westmorland	1981	145.8
093	usaca38-041	Hollister	1974	89.2
094	usaca38-043	Hollister	1974	112.1
095	usaca38-066	Humboldt	1975	128.1
096	usaca39-021	Whittier Narrow	1987	103.3
097	usaca39-023	Whittier Narrow	1987	143.8
098	usaca40-057	Whittier Narrow	1987	104.6
099	usaca40-072	Whittier Narrow	1987	89.3
100	usaca42-044	Whittier Narrow	1987	82.9
101	usaca66-022	Loma Prieta	1989	113.3

related to the selection of weak and moderate ground motions. The ground motions with attenuated small PGA at distances far from the fault were not the focus of this study. It was therefore necessary to determine not only the range of PGA but also the limit of epicentral distance. The attenuation of PGA with distance to the source of energy release was studied by many researchers (Donovan and Bornstein, 1978; Campbell, 1981; Joiner and Boore, 1981; Seed and Idriss, 1982). According to these studies, PGA at 50 km from the fault is much less than the half of the PGA at 10 km from the fault. Based on this approximate estimation, the upper limit of epicentral distance was set at 50 km.

4. Evaluation of damping coefficients

Pseudo-acceleration response spectra of each ground motion were constructed for 5%, 10%, 20%, 30%, 40% and 50% of critical damping. The structural period range of response spectra was selected at 0.01 second interval between 0.01-s and 4.0-s. For each ground motion, the damping coefficients were calculated at every structural period from the spectral amplitudes of different damping level. Then, the calculated damping coefficients of each ground motion were assembled for statistical studies.

The mean values of damping coefficients, which were determined from 101 weak and moderate ground motions, are shown in figures 1 through 5. In addition to mean values, coefficients of variation (COV) are also shown in the figures for statistical reference.

Damping coefficients calculated in this study show similar

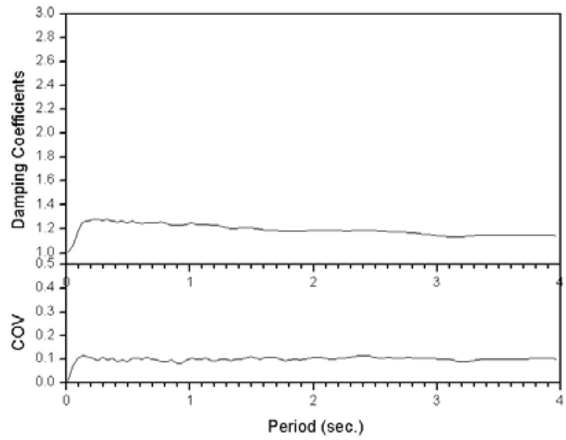

Fig. 1. Damping Coefficients for 10% of Critical Damping

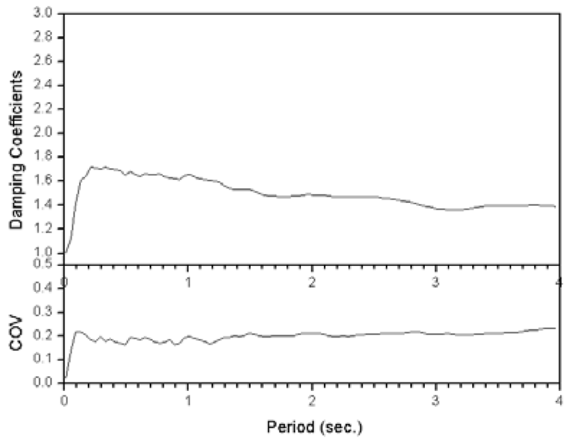

Fig. 2. Damping Coefficients for 20% of Critical Damping

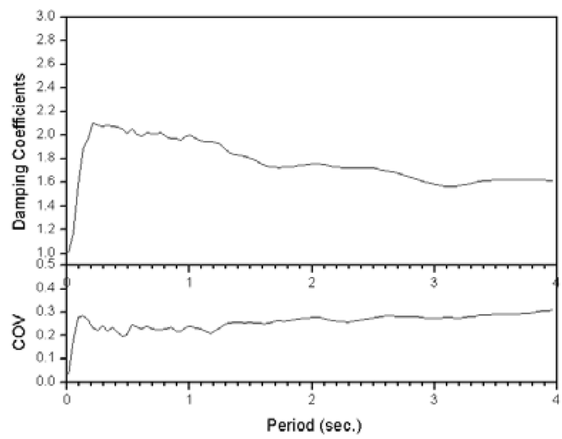

Fig. 3. Damping Coefficients for 30% of Critical Damping

trend with those of Newmark and Hall, and seismic design code provisions such as 1997 NEHRP Guidelines. That is, the damping coefficients increase as the period shifts from the long-period domain to the short-period domain. And then, the differences between the short-period and long-period damping coefficients increase as the damping values increase. However, the coefficients for short-period domain of this study represent considerably smaller values than those of Newmark and Hall, and 1997 NEHRP Guidelines at higher levels of damping. For example, damping coefficient of Newmark and Hall is 3.85 for acceleration domain at the 50% of critical damping. As a conservative value, 3.0 was suggested for that case in the 1997 NEHRP Guidelines. However, the maximum coefficient calculated is not greater than 2.8 at corresponding damping level.

The relatively small coefficients of this study in the short-period domain indicate weak dependency of damping coefficients on the period of vibration.

In the previous study of Naeim and Kircher (2001), the results of statistical analysis to evaluate mean values and standard deviation of damping coefficient did not represent significantly different damping coefficients in the acceleration and velocity domains. The trend of mean values was a very weak function of period. However, their study was limited to the damping values of 10% and 20% of critical.

To evaluate the period dependency of damping coefficients more quantitatively, regression analysis was performed. The results of linear regression analysis are shown in figure from 6 to 10.

The slope of regression curve in the above figures indicates the degree of period dependency on the damping coefficients. It was found that the period dependency becomes stronger as

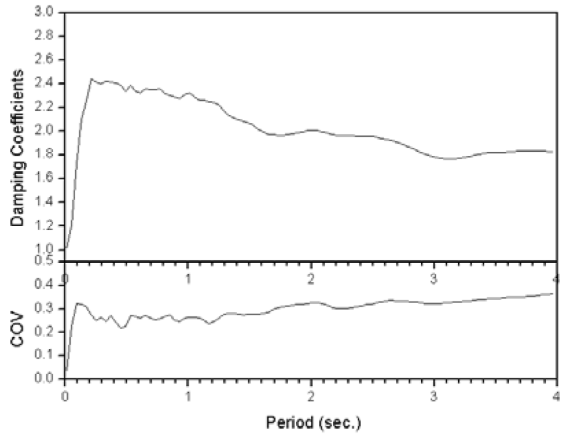

Fig. 4. Damping Coefficients for 40% of Critical Damping

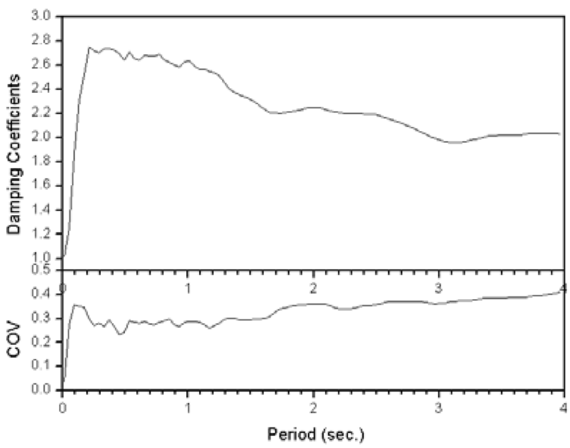

Fig. 5. Damping Coefficients for 50% of Critical Damping

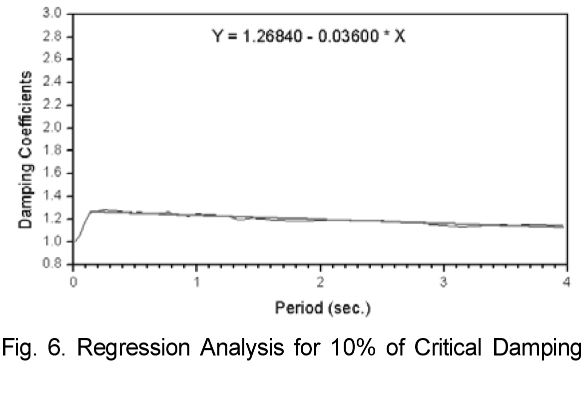

Fig. 6. Regression Analysis for 10% of Critical Damping

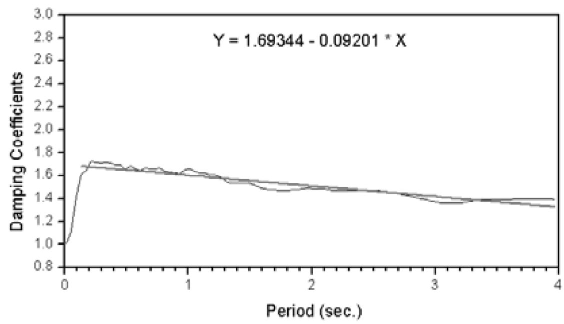

Fig. 7. Regression Analysis for 20% of Critical Damping

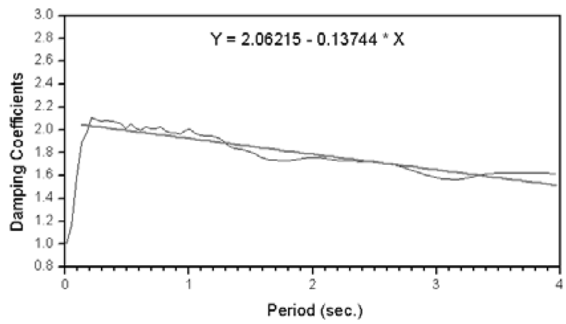

Fig. 8. Regression Analysis for 30% of Critical Damping

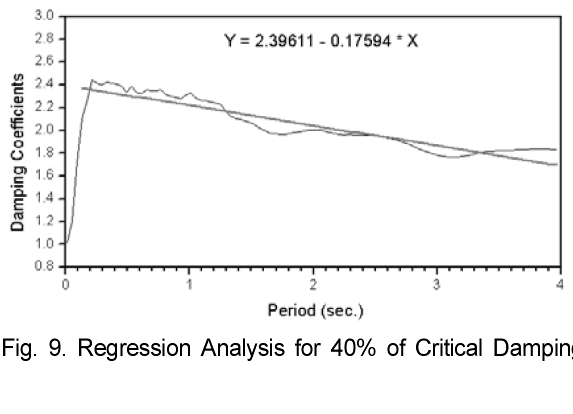

Fig. 9. Regression Analysis for 40% of Critical Damping

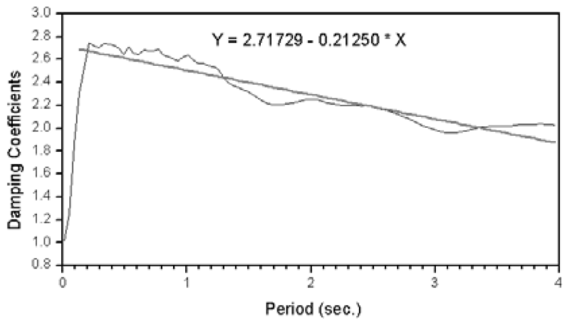

Fig. 10. Regression Analysis for 50% of Critical Damping

the damping values increase. However, the degrees of period dependency evaluated in this study are quite low as compared to those of Newmark and Hall, and those of the 1997 NEHRP Guidelines.

5. Conclusion

The damping coefficients based on the ratios of the response spectra from 101 weak and moderate ground motions are statistically evaluated, and compared to those of previous studies and current seismic design code provision. The main observations in this study are summarized as follows:

(1) The damping coefficients in short-period domain represent considerably smaller values than those of Newmark and Hall, and 1997 NEHRP Guidelines at higher levels of damping.
(2) The dependency on the vibration period of damping coefficients becomes stronger as the damping values increase. However, the degrees of period dependency evaluated were quite low as compared to those of Newmark and Hall, and those of the 1997 NEHRP Guidelines.

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