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Acceleration Records in Recent Earthquakes and Structural Response Values

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ABSTRACT :

Recent years, many countries set up the accelerometers and get their records in earthquakes. According to these records, some maximum acceleration on ground surface can be over 980 (cm/sec^2) (=ｇ: gravity acceleration) near epicenters, but these maximum values are not always corresponded to the damages of building structures.
The acceleration records have their own properties in period or running time of big acceleration and the structure damages are considered to depend on these properties. That is because the dynamic response analysis about the structures is needed based on structural period.
Many countries use the base shear coefficient in seismic design codes. These values are usually 0.2ｇ, 0.1ｇ and others for static analysis. These values are very smaller than the acceleration records and that makes difficult the inelastic dynamic response analysis. This is because the inelastic dynamic response analysis is usually applied to the high-rise buildings considered to have the property of long structural period.
Therefore, this paper describes the results of inelastic dynamic response analyses using Bi-linear model of Single Degree of Freedom. The acceleration records used in analyses are ones recorded in Japan, U.S.A. and others. In Bi-linear model, the yielding strengths per mass are 0.2ｇ or 0.1ｇ, the damping ration is 5%, and the degrading ratio of the stiffness is 1/1000. According to the inelastic analyses, the response acceleration, velocity and displacement spectra are calculated in short structural period of 1.0 (sec) and less, too. In conclusion, the response displacement is corresponded to the seriousness in the building damages well, regardless of the peak acceleration. The response absolute acceleration and the response velocity are not always corresponded to the seriousness in the building damages.

KEYWORDS: Earthquake, Acceleration Records, Inelastic, Dynamic, Response Analysis

1. INTRODUCTION

Recent years, many countries set up the accelerometers and get their records in earthquakes. According to these records, some maximum acceleration on ground surface can be over 980 (cm/sec^2) (=ｇ: gravity acceleration) near epicenters, but these maximum values are not always corresponded to the damages of building structures.
The acceleration records are useful for the prediction of the behaviors in building structures during earthquakes, and the structural response values are able to be predicted by dynamic analyses using a model of Single Degree of Freedom. The dynamic analyses are sometime inelastic response analyses because the behaviors of building structures during earthquake are not always stable in elastic. However, because of the difficulties of the dynamic inelastic response analyses, the inelastic response analyses are usually adopted to the high rise buildings which their period are considered to be longer than other building structures.
So, this paper describes the comparison of the behaviors in low-rise houses by the dynamic inelastic response analyses using earthquake acceleration records in recent years.

2. Earthquake Acceleration Records

Earthquake acceleration records are shown in Table 1 and Table 2. These tables show Earthquake Names, Acceleration Records Names which mean the measured place names, Direction measured in horizontal which are North-South or East-West, Peak Acceleration which means the maximum of absolute value of strong motion data, Epicentral Distance, Damages Summary, and Remarks of Courtesy. The records of Table 1 are used for the

calculation with the hysteresis characteristic when the base shear coefficient *Co* is 0.2 (Qy=0.2ｇ. Qy is the yielding strength per mass referred to Chapter 3). This value of *Co* is adopted to the seismic design code in Japan. The records of Table 2 are used for the calculation about the cases of *Co*(=0.1) (Qy=0.1ｇ). This value of *Co* is nearly equal to the values of the seismic design codes in Mexico, USA and other countries.
The records of Table 1 are some of the strong motion data measured and provided by Japan Meteorological Agency and National Research Institute for Earth Science and Disaster Prevention, Japan. Some of the strong motion data means that the 5 records which Peak Acceleration values are almost over 600(cm/sec^2) in one horizontal direction, and the 2 data of the data of north-south direction of Imperial Valley Earthquake in 1940 provided by CESMD and the artificial earthquake acceleration data on bedrock provided by Building Center of Japan. Therefore, the number of the records in Table 1 about *Co*=0.2 is 7.
The records of Table 2 are the data provided by each Institutes showed in the Courtesy Remarks. These records are measured in earthquakes which took place huge damages regardless of the Peak Acceleration values. The record of 1985 Michoacan Mexico Earthquake is used because it is recorded on soft soil condition when the Magnitude is 8.0 even if the epicentral distance is long as 400 km, and the Peak Acceleration is small as 167.9(cm/sec^2). The number of the records in Table 2 about *Co*=0.1 is 5.
The records are not corrected in the response analyses.

Table 1 Earthquake Acceleration Records for Qy* (= 0.2ｇ)

Earthquake Names [Acceleration Records Names in Notes of Fig. 3 and Fig. 4]	Direction (NS: North and South, EW:East and West)	Peak Acceleration (cm/sec^2)	Epicentral distance (km)	Damages Summary	Remarks of Courtesy for Strong Motion Data
2007 Niigata-ken Chuetsu-Oki Earthquake [2007 Kashiwazaki NS, K-net]	NS	667.9	21.3	14 people deaths. 1,244 buildings destroyed.	Japan Meteorological Agency
2004 Niigata-ken Chuetsu Earthquake [2004 Ojiya EW, JMA]	EW	897.6	7.0	67 people deaths. 3,175 buildings destroyed.	JMA
2004 Niigata-ken Chuetsu Earthquake [2004 Ojiya EW, K-net]	EW	1313.5	7.0		K-NET, NIED*, Japan
1995 Hyogo-ken Nanbu Earthquake [1995 Kobe NS, JMA]	NS	818.0	16.5	At least 6,430 people deaths. 111,942 buildings destroyed.	JMA
1993 Kushiro-Oki Earthquake [1993 Kushiro EW, JMA]	EW	919.3	8.2	2 people deaths. 53 buildings destroyed.	JMA
1940 Imperial Valley Earthquake [1940 Elcentro NS, CESMD]	NS	351.8	7～15 (predicted)	9 people deaths. At Imperial, 80 percent buildings damaged.	CESMD USA
BCJ Level2 <Artificial Earthquake Acceleration Data on Bedrock > [BCJ Level2 (Bedrock)]	－	355.7	---	---	Building Center of Japan

Notes) 1)Qy* : Yielding strength per mass referred to Chapter 3.
2)NIED* : National Research Institute for Earth Science and Disaster Prevention

Table 2 Earthquake Acceleration Records for Qy* (= 0.1ｇ)

Earthquake Names [Acceleration Records Names in Notes of Fig. 5 and Fig. 6]	Direction (NS: North and South, EW:East and West)	Peak Acceleration (cm/sec^2)	Epicentral distance (km)	Damages Summary	Remarks of Courtesy for Strong Motion Data
2005 Pakistan Earthquake [2005 Abbottabad EW, MSSP-PAEC]	EW	226.4	50	At least 86,000 people deaths, and 32,000 buildings destroyed.	MSSP-PAEC Pakistan
2003 Algeria Earthquake [2003 Dar El Beida EW, CGS Algeria]	EW	537	25	At least 2,200 people deaths, and 1,200 buildings damaged or destroyed.	CGS Algeria
2001 El Salvador Earthquake [2001 La Libertad NS, UCA El Salvador]	NS	1154.6	75	At least 844 people deaths, and 108,000 houses destroyed caused by landslides mainly.	UCA San Salvador El Salvador
1994 Northridge Earthquake [1994 Tarzana EW, Northridge COSMOS]	EW	1744.5	16.7	60 people deaths. More than 40,000 buildings damaged.	COSMOS USA
1985 Michoacan Mexico Earthquake [1985 Mexico City EW, SCT]	EW	167.9	400	At least 9,500 people deaths, and 3,500 buildings destroyed or seriously damaged. Tsunami also happened.	SCT Mexico

Note) Qy* : Yielding strength per mass referred to Chapter 3.

In Table 1, the highest Peak Acceleration is 1313.5 (cm/sec^2) recorded in 2004 Niigata-ken Chuetsu Earthquake and the heaviest damages took place in 1995 Hyogo-ken Nanbu Earthquake. In Table 2, the highest Peak Acceleration is 1744.5 (cm/sec^2) recorded in 1994 Northridge Earthquake and the heaviest damages took place in 2005 Pakistan Earthquake. That means the Peak Acceleration is not always corresponded to the damages and the response analyses are needed in order to estimate the seismic safety of structures. That is why this paper describes the inelastic response analyses. Moreover, the inelastic response analyses can calculate the hysteresis characteristics closer to the structures than the elastic response analyses, for example the yielding strength or the degraded stiffness.

3. Inelastic Dynamic Response Analysis Method

The following is the overview of the inelastic dynamic response analysis method.
The equation of motion about Single Degree of Freedom (Fig.1) at time *t* (sec) is the following Equation (3.1).

$$m\ddot{x} + c\dot{x} + kx = -m\ddot{y} \tag{3.1}$$

where

- m : mass(N/(cm/sec^2))
- c : damping factor(N/(cm/sec))
- k : stiffness(N/cm)
- x : relative displacement(cm)
- $\dot{x}$: relative velocity(cm/sec)
  $(\dot{x} = \frac{d}{dt}x)$
  (Relative velocity is the time rate of change of relative displacement)
- $\ddot{x}$: relative acceleration(cm/sec^2)
  $(\ddot{x} = \frac{d}{dt}\dot{x})$
  (Relative acceleration is the time rate of change of relative velocity)
- $\ddot{y}$: horizontal acceleration record on the ground surface (cm/sec^2)
- t : time (sec)

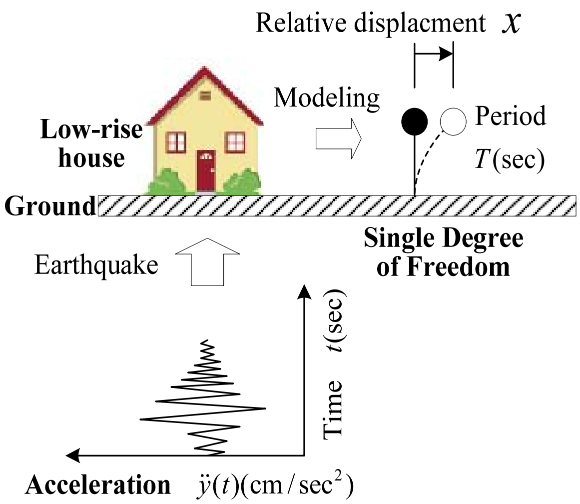


Fig. 1 Modeling a low-rise house to SDOF and the response analysis method

When the Equation (3.1) is divided by mass m, and both of $\sqrt{\frac{k}{m}} \equiv \omega$ and $\frac{c}{m} \equiv 2h\omega$ are introduced, the Equation (3.1) is transferred to the following Equation (3.2).

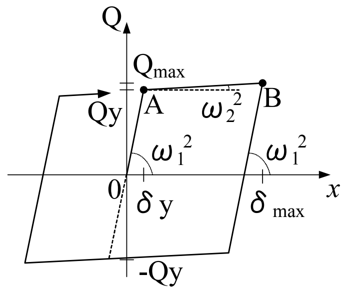
$$\ddot{x} + 2h\omega\dot{x} + \omega^2 x = -\ddot{y} \tag{3.2}$$

When the acceleration records in Table 1 or Table 2 are input to $\ddot{y}$ of the Equation (3.2), it gives the response values x, $\dot{x}$, $\ddot{x}$ of SDOF. The motions of these response values are the forced vibrations with damping under the angular frequency ω (radian/sec), and the damping coefficient h (1/rad.). The period *T* of SDOF on ground is given by the following Equation (3.3).

$$T = \frac{2\pi}{\omega} = 2\pi\sqrt{\frac{m}{k}} \tag{3.3}$$

The solution x in the Equation (3.2) is the summation in the Equation (3.4) of the solution x_c in the Equation (3.5) of the free vibration with damping, and the solution x_p in the Equation (3.6) of the forced vibration with damping.

$$\begin{cases} x = x_c + x_p & (3.4) \\ \ddot{x}_c + 2h\omega\,\dot{x}_c + \omega^2 x_c = 0 & (3.5) \\ \ddot{x}_p + 2h\omega\,\dot{x}_p + \omega^2 x_p = -\ddot{y} & (3.6) \end{cases}$$



Notes) (1)Qy=0.2g, or 0.1g

(2)$\omega_2^2 = \dfrac{1}{1000}\omega_1^2$

(3)h =0.05

Fig.2 Hysteresis characteristics of Bi-linear model

In the response analysis, the response relative velocity at the time t (sec) is assumed to be $\dot{x}(t)$, and the response relative displacement at the time t (sec) is assumed to be $x(t)$. After the data sampling time Δt (sec), the response values $\dot{x}(t+\Delta t)$, $x(t+\Delta t)$ at the time $t+\Delta t$ (sec) are calculated by the Dummy Variable Method. The initial values when time t =0(sec) are as follows; $x(0)=0$, $\dot{x}(0)=-\ddot{y}(0)\cdot\Delta t$, $\ddot{x}(0)=2h\omega\,\ddot{y}(0)\cdot\Delta t-\ddot{y}(0)$.
The hysteresis characteristics is the Bi-linear model in Fig.2 which shows the variety of stiffness per mass $\dfrac{k}{m}(=\omega^2)$. In the Bi-linear model, the yielding strength per mass Qy is assumed to be 0.2g or 0.1g, the initial ω^2 is assumed to be ω_1^2, and the ω^2 after yielding is assumed to be ω_2^2(= $\dfrac{1}{1000}\times\omega_1^2$) which is the degraded stiffness. The value of damping coefficient h is 0.05.
When the sign (±) of incremental response displacement changes after yielding, the ω^2 is assumed to be ω_1^2 until the amount of incremental response displacement, after the ω^2 is changed to be ω_1^2, is over 2・δy (= 2・Qy/ω_1^2). The each response values x, $\dot{x}$, $\ddot{x}$ are calculated based on the acceleration records at the time t, but when the response value x is just over the point of the change in the value ω^2, another response values x at the time $t+\delta t$ ($\delta t<\Delta t$) between the sampling time Δt (=0.01 or 0.02 sec) are also calculated and the nearest equal value x to the displacement at the change of the value ω^2 is chosen. After this choice, the calculation by the changed value ω^2 is continued. Generally, in this response analysis, the 5 response values of x, $\dot{x}$, $\ddot{x}$, the strength per mass Q(=$\omega^2 x$), and the damping force per mass V(=$2h\omega\dot{x}$) are continuous timely, and these response values always satisfy the Equation (3.2). The analyses are executed for the period from 0.03 or 0.05 (sec) to 5 (sec), while the number of periods is 33 or 32, and in each period the duration of earthquake records is about 60 (sec).

4. Results of Analyses

The results of analyses about Co=0.2 are shown in several response spectra of Fig. 3 and Fig. 4.
The range of the horizontal axis in Fig. 3 is from 0 to 1(sec) because the period T (=2π/ω_1) of low-rise houses which are wooden, steel or reinforced concrete structures, is calculated in the range of less than about 0.5(sec), according to the structural design examples in the reference.
The reference describes the examples of structural design about 3 storeys houses which structure is Combined like a steel structure on 1st floor and a wooden structure on 2nd – 3rd floor, or a reinforced concrete structure on 1st floor and a wooden structure on 2nd – 3rd floor. These examples show the gravity in each floor mg (=Dead Load + Live Load), and the horizontal displacement of each floor x_{02} (cm) when the each floor is loaded by the seismic force Q_{02}, which is nearly 0.2mg, calculated according to the base shear coefficient which is 0.2. These values give the secant elastic modulus and the period T'(sec) of each floors by the flowing Equation (4.1).

$$T' = \frac{2\pi}{\omega} = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\frac{m}{Q_{02}}x_{02}} \qquad (4.1)$$

Table 3 shows that T' is 0.54(sec) in the wooden structure floor, 0.50(sec) in the steel frame structure floor, 0.26(sec) in the reinforced concrete frame structure floor, and about 0.089(sec) in the reinforced concrete frame structure floor with shear wall.

Table 3 Period *T'* of each floor in low-rise houses

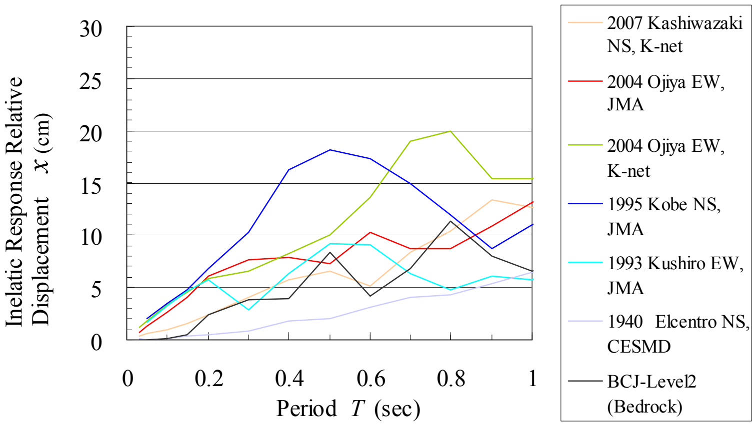
Structure type (Number of floor, Direction X or Y)	Gravity mg (kN)	Seismic forces coefficient along height Ci (-)	Seismic force Q_{02} (kN)	Horizontal displacement loaded by Q_{02} x_{02} (cm)	Secant ealstic modulus k (kN/cm)	Period T' (sec)
Wooden(2nd floor, Y)	379	0.25	94.78	1.80	52.8	0.538
Steel frame(1st floor, Y)	743	0.20	148.6	1.22	122	0.496
Reinforced concrete frame (1st floor, X)	1252	0.20	250.4	0.334	750	0.259
Reinforced concrete frame with shear wall (1st floor, Y)	1252	0.20	250.4	0.0394	6360	0.0890

Notes 1) mg : Summation of Dead Load and Live Load
2) Ci : Ci=Ai×Co
3) Ai : Distribution of seismic forces along the height of a building
4) Co : Base shear coefficient (=0.2)
5) Q_{02} : $Q_{02} = Ci \times mg$
6) k : $k = Q_{02} / x_{02}$

In Fig. 4 (1), the inelastic response base shear coefficient *Cd* is the maximum of the absolute value of the strength per mass Qmax (Referred to Fig. 2) divided by g during each analysis in a period T (=$2\pi/\omega_1$). The relationship between *Cd* and Qmax is the Equation (4.2).

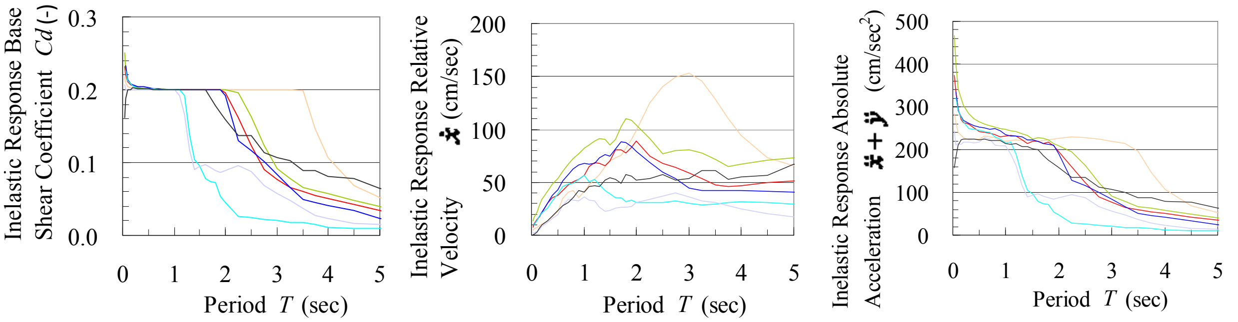
$$Cd = \frac{\mathrm{Qmax}}{g} \tag{4.2}$$

Cd of the dynamic response analysis is considered to be the base shear coefficient *Co*(≥0.1, ≥0.2, and so on in each country) of static analysis.
In an elastic dynamic response analysis, a seismic force is considered to be the maximum strength of a building structure $kx_{\max}$ which is the multiplication of the stiffness k and the maximum of absolute value of elastic response displacement $x_{\max}$. So, $kx_{\max}$ per gravity, which is $\frac{kx_{\max}}{mg} = \frac{\omega^2}{g} x_{\max} = \frac{(2\pi)^2}{T^2 g} x_{\max}$, is considered to be *Cd* in the inelastic dynamic response analysis.
The relative acceleration $\ddot{x}$ is an inertial force per mass, and the negative of the horizontal acceleration record $(-\ddot{y})$ is the external force per mass. The absolute value of the subtraction between these values, which is $\ddot{x}+\ddot{y}$, is considered to be the shear force per mass F(=$\ddot{x}+\ddot{y}$) in structural vibrations without damping.
According to Fig. 3, in the range of less than 0.5(sec) for the period *T'* in each floor of low-rise houses, the maximum of inelastic response relative displacement is analyzed in the record of 1995 Hyogo-ken Nanbu Earthquake. The maximum Peak Acceleration in Table 1 is 2004 Niigata-ken Chuetsu Earthquake which inelastic response relative displacement is not the highest value. That means the inelastic response relative displacement does not depend on the Peak Acceleration but it is corresponded to the damage.
Fig. 4 (1) shows that *Cd* is from 0.16 to 0.23 when the period T (=$2\pi/\omega_1$) is less than 1.0(sec), and *Cd* decreases when the period T is over the critical point more than 1.0(sec). Fig. 4 (2) and (3) show that the response values $\ddot{x}$ and $\ddot{x}+\ddot{y}$ in the range of the period T less than 1.0(sec) are highest in 2004 Niigata-ken Chuetsu Earthquake. $\ddot{x}+\ddot{y}$ is from 158 to 467 (cm/sec^2) when T is less than 1.0(sec).
The results of analyses about *Co*=0.1 are shown in several response spectra of Fig. 5 and Fig. 6.
Fig.5 and Fig.6 show the results of earthquake records in Table 2 and 1995 Hyogo-ken Nanbu Earthquake in order to compare with the results of the analyses when Qy = 0.2g.
The range of the horizontal axis in Fig. 5 is from 0 to 5(sec).
According to Fig. 5, in the range of less than 0.5(sec) of the period, the maximum of inelastic response relative displacement is analyzed in the record of 1995 Hyogo-ken Nanbu Earthquake and the next one is 1994



Note) Qy=0.2g, $\omega_2^2 = \frac{1}{1000}\omega_1^2$, h=0.05

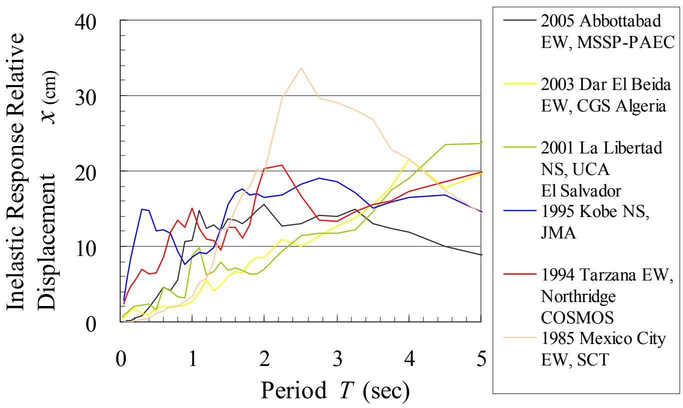
Fig. 3 Inelastic Response Relative Displacement Spectrum (x)



(1)Base Shear Coefficient(Cd) (2)Relative Velocity($\dot{x}$) (3)Absolute Acceleration($\ddot{x}+\ddot{y}$)

Fig. 4 Inelastic Response Spectra

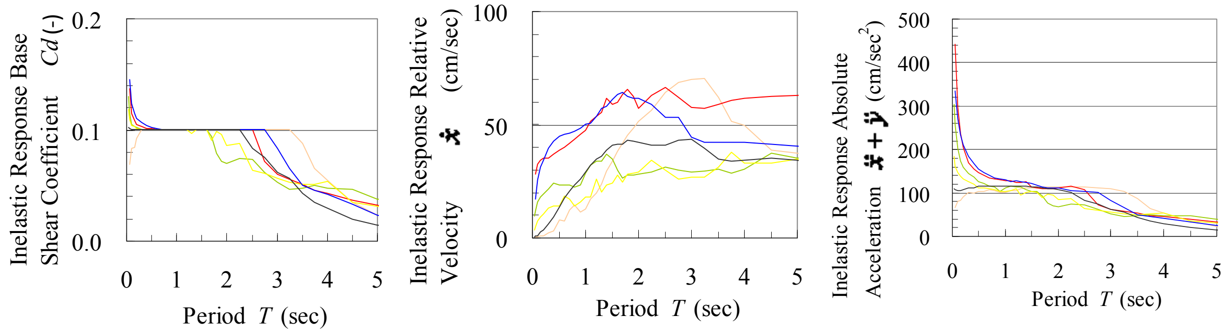
Notes) (1)Qy=0.2g, $\omega_2^2 = \frac{1}{1000}\omega_1^2$, h=0.05
(2)The colors of lines and earthquakes are same as Fig. 3



Note) Qy=0.1g, $\omega_2^2 = \frac{1}{1000}\omega_1^2$, h=0.05

Fig. 5 Inelastic Response Relative Displacement Spectrum (x)

Northridge Earthquake. When the period T (=2 π/ω_1) is from 2(sec) to 4(sec), the maximum displacement is replaced to 1985 Michoacan Mexico Earthquake.
The range of the period T (=2 π/ω_1) from 2(sec) to 4(sec) is longer than the period T' from 0.089(sec) to 0.54(sec) in each floor of low-rise houses shown in Table 3. In 1985 Michoacan Mexico Earthquake, some of



(1)Base Shear Coefficient(*Cd*) (2)Relative Velocity($\dot{x}$) (3)Absolute Acceleration($\ddot{x}+\ddot{y}$)

Fig. 6 Inelastic Response Spectra

Notes) (1)Qy=0.1g、 $\omega_2^2 = \frac{1}{1000}\,\omega_1^2$ 、 h=0.05
(2)The colors of lines and earthquakes are same as Fig. 5

middle-rise or high-rise buildings were damaged, and the analyzed large response displacement has some possibility to be a reason of this damages.
The maximum Peak Acceleration in Table 2 is 1994 Northridge Earthquake and the inelastic response relative displacement is higher value. That means the inelastic response relative displacement some time depend on the Peak Acceleration and it is corresponded to the damage.
Fig. 6 (1) shows that *Cd* is from 0.07 to 0.15 when the period *T* is less than 1.0(sec), and *Cd* decreases to 0.0(-) when the period *T* is over the critical point more than 1.0(sec).
Fig. 6 (2) and (3) show that the response values $\dot{x}$ and $\ddot{x}+\ddot{y}$ are highest in 1994 Northridge Earthquake, 1995 Hyogo-ken Nanbu Earthquake, or 1985 Michoacan Mexico Earthquake. $\ddot{x}+\ddot{y}$ are resolved from 66 to 443 (cm/sec^2) when *T* is less than 1.0(sec).
To compare the results of 1995 Hyogo-ken Nanbu Earthquake in Fig. 3 and Fig. 5, the values of x and $\dot{x}$ when Qy = 0.1g are not always larger than the values when Qy = 0.2g because the yielding displacement δy (= Qy/ω_1^2) when Qy = 0.1g is a half of the one when Qy = 0.2g.

5. Damping coefficient

In these analyses, the damping coefficient h is assumed to be constant and it is 0.05(1/rad.). That means the damping factor c in the Equation (3.1) is constant for one ω in one analysis, but c is different number for other ω by the following Equation (5.1). It is already introduced between the Equation (3.1) and (3.2).

$$\frac{c}{m} = 2h\omega = 2h\cdot\frac{2\pi}{T} = 4\pi\cdot\frac{h}{T} \tag{5.1}$$

Fig.7 shows the relationship between the period T (=2π/ω_1) (sec) and the damping factor per mass (*c/m*) (1/sec) when the damping coefficient h is 0.05 (1/rad.). According to this figure, when T is less than 0.6 (sec), the value of (*c/m*) is increasing to more than 1.0 (1/sec). The value of (*c/m*) is a coefficient of a relative velocity $\dot{x}$ in the Equation (3.2) and has some effects to the results of the response values. When the value of (*c/m*) is large, the relative velocity $\dot{x}$ has to become small.
Fig. 4 and Fig.6 show that the response values of ($\ddot{x}+\ddot{y}$) and *Cd* are overestimated when the period *T* is less than 1.0 (sec), compared with the statistic values which are nearly equal to 200 (cm/sec^2), 0.2 (-), and others.
If the value of (*c/m*) is assumed to be constant and 1.0 (1/sec) when *T* is less than about 0.6 (sec), the h is various value according to each *T* and decreases to 0.0 (1/rad.). See the Equation (5.2), (5.3) and Fig. 8.
This various h can give other results of response values x, $\dot{x}$, $\ddot{x}+\ddot{y}$, and *Cd*. For example, an absolute

$$\begin{cases} h = \dfrac{c}{m}\cdot\dfrac{T}{4\pi} = \dfrac{T}{4\pi} & (T \le 0.2\cdot\pi) \quad (5.2) \\ h = 0.05 & (T > 0.2\cdot\pi) \quad (5.3) \end{cases}$$

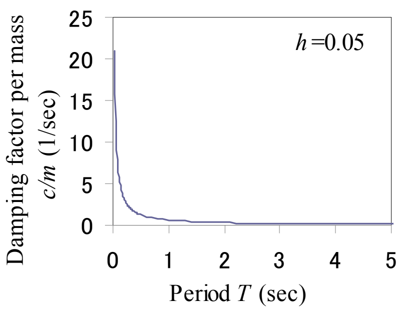


Fig. 7 Damping factor per mass c/m under constant h (=0.05)

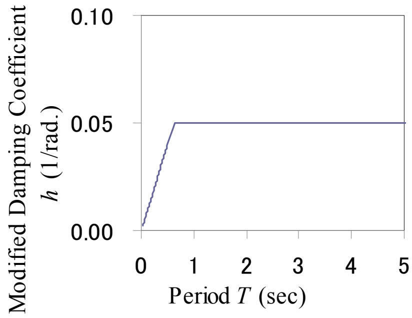


Fig. 8 Modified Damping Coefficient h in each period T (=2 π / ω_1)

acceleration ($\ddot{x}+\ddot{y}$) is usually decreasing when the h decreases.
Therefore, it has some possibility that the Modified Damping Coefficient h is not always 0.05 (1/rad.) and the values of h are various numbers proportional to the period T ($\leq 0.2 \cdot \pi$) (sec). See the Equation (5.2) and (5.3).

6. Conclusions

The inelastic dynamic response analyses about low-rise houses are executed, using 11 Earthquake Acceleration Records. The analyses method is that the hysteresis characteristics is Bi-linear Model, the yielding strength per mass is 0.2 g or 0.1 g, the degrading ratio of stiffness is 1/1000, and the damping coefficient is 0.05 (1/rad.).
The response value corresponded to earthquakes damages is the inelastic response relative displacement.
The period of each floor of structural deign examples in the reference is about less than 0.5 (sec).
According to the results of analyses using 7 Acceleration Records when the yielding strength per mass is 0.2 g, and the period is less than 0.5 (sec), the maximum inelastic response relative displacement is resolved in 1995 Hyogo-ken Nanbu Earthquake. The inelastic response base shear coefficient is nearly equal to 0.2 (-).
According to the results of analyses using 5 Acceleration Records when the yielding strength per mass is 0.1 g, the maximum inelastic response relative displacement is resolved in 1995 Hyogo-ken Nanbu Earthquake and 1994 Northridge Earthquake. Also the record of 1985 Michoacan Mexico Earthquake gives the maximum response values in long period around from 2 to 4 (sec).
The response absolute acceleration is overestimated in the short period probably because the damping coefficient, which is usually considered to be 0.05 (1/rad.), is too large.

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