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Ductility response of SDOF system using reversed rectangular pulses

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ABSTRACT: Recent near-fault earthquake ground motions are characterized by pulse-like velocity waveforms, and they are destructive as the peak velocities may be several times larger than the ones considered in seismic design. Although time history analysis gives us detailed information about earthquake response, explicit formulas and simplified methods are efficient to highlight the relationship between structural response and characterized parameters of ground motions, this is important for a rapid damage evaluation of structures. This research proposed a method to relate the maximum response of SDOF elasto-plastic system with the peak values of ground motions. Reversed rectangular acceleration and velocity pulses are used to consider the effects of pulse-like ground motions, and some assumptions for the restoring force histories are made to obtain a series of simple explicit formulas for maximum response. Prediction accuracy of the proposed method is investigated by comparing with time history analysis using records of near-fault ground motions. The proposed formulas give an extremely simple correlation between structural responses with peak values of ground motion, and further researches are needed for different pulse types.

1 INTRODUCTION

The earthquake response of structural models can be accurately computed from time history analysis. However, there may be a large scatter accompanied with a small change of the characteristics of structural models. Together with the uncertainties of earthquake ground motions, it is important to develop generic and simple methods applicable for seismic design and damage evaluation of structures. For that purpose, integrated quantities of ground motions and assumption of structural response are needed. For example, in current seismic design, response spectra are used for defining the effect of ground motions, and structural response are predicted by the method of equivalent linearization (Caughey, 1963) with the assumption of stable vibration. Recently, instantaneous energy spectra are proposed for evaluating the inelastic response of structures (e.g., Nakamura & Kabeyasawa, 1998).

Comparing with response spectra, the simplest parameters of earthquake ground motions are their peak values; peak ground acceleration (PGA), peak ground velocity (PGV), and peak ground displacement (PGD). Over last twenty years, very large PGA and PGV values were recorded during intense near-fault earthquakes worldwide. The peak values may greatly exceed $5.0m/s^2$ in PGA and $0.5m/s$ in PGV. They are much larger than that considered in seismic design, and large inelastic deformations are expected. Besides the large peak values, near-fault earthquakes show strong pulse-like velocity waveforms, as described by Mavroeidis & Papageorgiou (2003).

Under pulse-like input, the transient vibration will be important, and response spectra method based on stable vibration may not be applicable.

There are many researches intend to represent pulse-like ground motion or their effects by simple functions or simple waveforms, (e.g., Hall & Aagaard, 1998; Sakai et al, 1999; Alavi & Krawinkler, 2000; Cuesta & Aschheim, 2001; Mavroeidis & Papageorgiou, 2003). Expression by simple function has great meaning for understanding the main characteristics of complicated earthquake waves. However, it is difficult to reflect the effects of high frequency ground acceleration on structural response by velocity pulses. Most importantly, in spite of the simple functional representation of waveforms, time history analysis does needed to obtain structural response. Simplification of earthquake input has great meaning if simple expression of structural response may be obtained.

For the purpose of damage evaluation, peak values of ground motions or their combinations have been used for correlating with the seismic damage to obtain fragility curves. Sasani (2004) proposed a new index called significant peak ground acceleration, and it is correlated with time history analysis to obtain an empirical formula.

We have proposed a method to theoretically relate the response of SDOF elasto-plastic system with the peak values of ground motions using rectangular acceleration and velocity pulses (Zhang et al, 2004). A series of simple explicit formulas were obtained by using some assumptions for the restoring force history. In this paper, calculation accuracy of the proposed method is investigated by comparing with time history analysis. And its applicability and limit are discussed for further study.

2 RESPONSE UNDER REVERSED PULSES

2.1 *Simple pulses*

Half century ago, Tanahashi (1956) and Housner (1956) considered the ground velocity and the potential energy of structures decide structural damages. The maximum response of elasto-plastic system subjected to constant velocity pulse or acceleration impulse may be obtained simply using the energy equilibrium (Shibata 1986), and the reduction factor still used in current seismic design may be also obtained (Veletsos & Newmark, 1960).

Another simplest pulse is a rectangular acceleration pulse. Newmark (1965) shown the response of a rigid-plastic system depends on the amplitude and the duration of an acceleration pulse.

One of the most important characteristics of near-fault ground motions is the existence of velocity pulse. Most often, the velocity pulses are not monotonic, but are cyclic. This is why the velocity response may be 2 or 3 times larger compared with the PGV value. Therefore, reversed pulses should be used in the simplification of velocity pulses.

The simplest velocity pulses are one cycle of sine or cosine waves; that correspond to ground motions with single predominant period. Makris & Black (2004) discussed the non-dimensional parameters controlling the inelastic structural response subjected those pulses, and applied the non-dimensional analysis for calibrating the numeric results.

In order to obtain a directly quantitative expression of inelastic response, this research use reversed rectangular pulses either in acceleration or in velocity to represent the effects of near-fault ground motion. The method of Newmark (1965) for single acceleration pulse and the method by energy equilibrium (Shibata 1986) for single velocity pulse are extended to obtain simple formulas for maximum response.

2.2 *Definition of reversed pulses*

Fig.1(a), (b) and (c) show the ground acceleration, velocity and displacement of Tabas ground motion (1978, Iran). Simple function like sine function may shows good correlation with the ground displacement, and some extent with the ground velocity of the main part of waves. However, it is impossible to reflect the large peak ground acceleration.

As we know, ground acceleration, velocity and displacement are sensitive to systems with different elastic period. It is important taking account PGA into the simplified input.

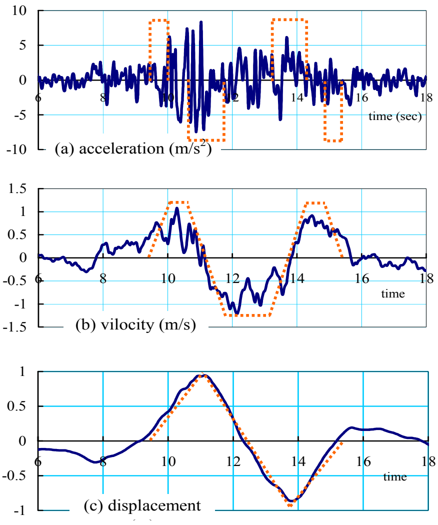


Figure 1. Ground motion of Tabas (TR component)

In Fig.2(b), continuous trapezoidal velocity pulses are assumed to generate a symmetric approximately triangular ground displacement as shown by Fig. 2(c). Here, V_p and D_p are the peak velocity and displacement of pulse inputs. Velocity pulses need intermittent rectangular acceleration pulses with peak values of A_p as shown in Fig.2(a). If the displacement, velocity and the acceleration have the same peak values in the positive and negative directions, both the second velocity pulse and the second acceleration pulses should have 2 times of duration compared with the first pulses.

Firstly, the duration T_{pv} of the first acceleration pulse that results peak velocity V_p is decided by the following equation.

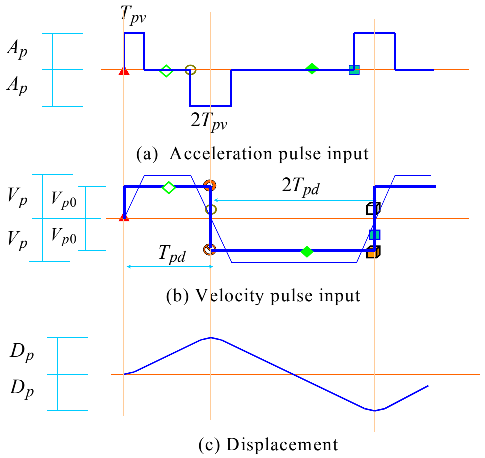
$$T_{pv} = V_p / A_p \quad (1)$$

As the peak displacement may be expressed by Equation(2), the duration of velocity pulse T_{pd} should be decided by Equation (3).

$$D_p = V_p (T_{pd} - T_{pv}) \quad (2)$$

$$T_{pd} = (D_p + V_p T_{pv}) / V_p = (D_p + V_p^2 / A_p) / V_p \quad (3)$$

As shown in Fig.1, acceleration pulses are not corresponding to the actual acceleration waveforms, but produce similar velocity pulse and displacement.

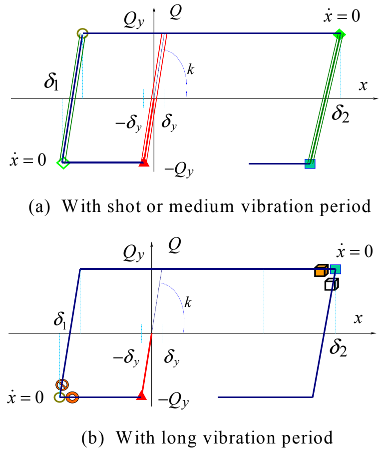


(a) Acceleration pulse input

(b) Velocity pulse input

(c) Displacement

Figure 2. Earthquake input simplified as reversed pulses



(a) With shot or medium vibration period

(b) With long vibration period

Figure 3. Response model of elasto-plastic systems

For later easy formulation of maximum response, trapezoidal velocity pulses are further simplified as rectangular shapes. Their amplitude is decided by Equation(4) to generate almost the same ground displacements.

$$V_{p0} = D_p / T_{pd} = V_p \frac{1}{1+V_p^2/(A_p D_p)} \tag{4}$$

Sometimes, ground displacement may be in one direction, or even finally residues. As the pulses shown in Fig.2 have large instantaneous energy and are more severe for structural response, they are utilized for discussion in this research. Later, peak values (PGA, PGV, PGD) of ground motions will be used for replacing (A_p, V_p, D_p) of the defined reversed pulses.

2.3 *Response under reversed pulses*

It is well known that a response spectrum is acceleration sensitive for shorter periods, velocity sensitive for medium periods and displacement sensitive for long period structures. For inelastic structure, the yielding strength may be another factor deciding the sensitivity of response spectra. In this research, reversed rectangular acceleration pulses are applied if the elastic periods are shorter and the yielding strength are large. Otherwise, for systems with longer elastic period and lower yielding strength, reversed rectangular velocity pulses are applied.

Indeed, structural response includes elastic vibration and vibration after yielding. However, it is difficult to obtain a simple formula if elastic vibration is included, that is in the form of trigonometrical function. Moreover, rigorous analysis using the simplified pulse waveforms does not have much meaning. For above reasons, simplification and assumption will be made for the restoring force history associated with the pulse inputs.

2.3.1 *Response for shorter period systems using acceleration pulses*

The real ground acceleration waveforms are very complicated. Actually, SDOF model vibrated before the start of assumed pulses; initial displacement and velocity may exist or even experienced yielding. In order to obtain simple formulas, in Fig.3(a), the history of restoring force is assumed corresponding to the reversed acceleration pulses in Fig.2(a).

Considering SDOF model with mass m and restoring force Q, the equation of motion will be expressed by Equation (5) if subjected to ground displacement x_0, where x is the relative displacement.

$$\ddot{x} + Q/m = -\ddot{x}_0 \tag{5}$$

At yielding states, let the yielding force be F_y, and suppose the initial velocity is zero and initial displacement is δ_0. Then, when subjected to a rectangular acceleration pulse α_p with a duration of t_p, the displacement and velocity at the end of pulse will be expressed by Equations (6) and (7).

$$\dot{x}_a = (\alpha_p - F_y/m)t_p \tag{6}$$

$$x_a = \delta_0 + \frac{1}{2}(\alpha_p - F_y/m)t_p^2 \tag{7}$$

Afterwards, the system will vibrates with the above initial velocity and initial displacement. With a further time increment expressed by Equation (8), the response velocity will be zero and displacement will be the maximum expressed by Equation (9).

$$\Delta t = \frac{m\dot{x}_a}{F_y} = (\frac{m\alpha_p}{F_y} - 1)t_p \tag{8}$$

$$\delta_b = \delta_0 + \frac{1}{2}(\frac{m\alpha_p}{F_y} - 1)\alpha_p t_p^2 \tag{9}$$

When subjected to the first acceleration pulse, let the system start vibrating from the negative yielding point. For ($\delta_0 = -\delta_y, \alpha_p = -A_p, F_y = -Q_y$) and $t_p = T_{pv}$, the maximum displacement at the negative direction is expressed by the following Equation.

$$\delta_1 = -\delta_y - \frac{1}{2}(\frac{mA_p}{Q_y} - 1)A_p T_{pv}^2 \quad (10)$$

After free vibration, suppose the system started at the opposite yielding point rightly when subjected to the second acceleration pulse. For ($\delta_0 = \delta_1 + 2\delta_y$, $\alpha_p = A_p$, $F_y = Q_y$) and $t_p = 2T_{pv}$, the maximum displacement will be decided by the following Equation.

$$\delta_{max} = \delta_2 = \delta_y + \frac{3}{2}(\frac{mA_p}{Q_y} - 1)A_p T_{pv}^2 = \delta_y + \frac{3}{2}(\frac{m}{Q_y} - \frac{1}{A_p})V_p^2 \quad (11)$$

2.3.2 *Response for medium period systems using velocity pulses*

For simplified rectangular velocity pulses in Fig.2 (b), response history in Fig.3(a) is also assumed to obtain a formula for maximum response. Here, the elastic vibrations are also ignored and the initial vibration states are assumed as the same as in 2.3.1.

For the first velocity pulse, energy equilibrium is applied for the negative yielding state, and it is expressed as follows.

$$-Q_y(\delta_1 + \delta_y) = \frac{m}{2}V_{p0}^2 \quad (12)$$

Maximum displacement in negative direction will be solved as follows.

$$\delta_1 = -\delta_y - \frac{m}{2Q_y}V_{p0}^2 \quad (13)$$

Then the system is unloaded and vibrates in elastic state. Assuming rightly at the opposite yielding point, the second velocity pulse is applied, and we have energy equilibrium as follows.

$$Q_y(\delta_2 - 2\delta_y - \delta_1) = \frac{m}{2}(2V_{p0})^2 \quad (14)$$

It should be noted that effective velocity input is $2V_{p0}$ as the velocity pulse is reversed. Maximum displacement is obtained from the following equation.

$$\delta_{max} = \delta_2 = \delta_1 + 2\delta_y + \frac{2m}{Q_y}V_{p0}^2 = \delta_y + \frac{3m}{2Q_y}V_{p0}^2 \quad (15)$$

After elastic vibration, next yielding occurs when subjected to the following velocity pulse. Obviously, the second velocity pulse decides the maximum displacement.

2.3.3 *Response for long period systems using velocity pulses and consider forced unloading*

If the system has long elastic period or low yielding strength, its velocity will remain when the next velocity pulse started; and the system will be forced for unloading. Here the response history is assumed as shown by Fig.3(b) corresponding to velocity pulse in Fig.2(b).

At the end of the first velocity pulse, we have the following structural response.

$$\delta_1 = -\delta_y - V_{p0}T_{pd} + \frac{Q_y}{2m}T_{pd}^2 \quad (16)$$

$$V_1 = -V_{p0} + \frac{Q_y}{m}T_{pd} \quad (17)$$

Then the second velocity pulse applied, it results initial velocity as follows.

$$V_{20} = 2V_{p0} + V_1 = V_{p0} + \frac{Q_y}{m}T_{pd} \quad (18)$$

At the end of the second velocity pulse and just before the start of the third velocity pulse, the maximum displacement is reached and is solved as follows.

$$\delta_{max} = \delta_2 = 2\delta_y + \delta_1 + 2V_{20}T_{pd} - \frac{Q_y}{2m}4T_{pd}^2 = \delta_y + V_{p0}T_{pd} + \frac{Q_y}{2m}T_{pd}^2 = \delta_y + D_p + \frac{Q_y}{2m}(D_p / V_{p0})^2 \quad (19)$$

2.4 *Required strength spectra*

The formulas about maximum response are developed under several assumptions, and more cases may exist. E.g., full velocity input or forced unloading may be possible for the first and second velocity pulses. Nevertheless, we chose the smallest from Equations (11, 15 and 19) for response prediction.

For an elasto-plastic system, the yielding displacement δ_y is represented by elastic period T and yielding strength coefficient q_y as follows.

$$\delta_y = \frac{T^2 g}{4\pi^2}q_y \quad (20)$$

where g is the gravity acceleration.

Equations (11, 15 and 19) may be reformulated for required strength spectra.

(a) In the case of shorter period

Using Equation (11), the ductility factor μ is expressed as follows.

$$\mu = \frac{\delta_{max}}{\delta_y} = 1 + \frac{3}{2}(\frac{A_p}{g}\frac{1}{q_y} - 1)\frac{A_p}{g}T_{pv}^2\frac{4\pi^2}{T^2}\frac{1}{q_y} \quad (21)$$

The required strength is obtained from the following formula.

$$q_y = \frac{A_p}{g} \cdot \frac{2}{1 + \sqrt{1 + \frac{2(\mu - 1)}{3\pi^2}\left(\frac{A_p}{V_p}T\right)^2}} \quad (22)$$

(b) In the case of medium period

Using Equations (15 and 20), we have

$$q_y = \frac{2\pi}{g}\frac{V_{p0}}{T}\sqrt{\frac{3}{2(\mu - 1)}} \quad (23)$$

(c) In the case of long period

Using Equations (19 and 20), we have

$$q_y = \frac{4\pi^2}{\mu - 1 - 2[\pi D_p / (TV_{p0})]^2}\frac{D_p}{g}\frac{1}{T^2} \quad (24)$$

Fig.4 shows an example of required strength spectra (relation between q_y and T) for pulse input with ($A_p = 1.0g$, $V_p = 1.0m/s$, $D_p = 0.5m$). In this case, q_y is decided by acceleration pulse for $T < 1.2\sec$, by fully inputted velocity pulse when $1.2\sec \le T \le 2.7\sec$, and by partially velocity pulse while $T > 2.7\sec$. However, the border between shorter, medium or long period depend on the peak values of pulse and ductility factor μ, they can not be decided only from the pulse inputs (earthquake characteristics).

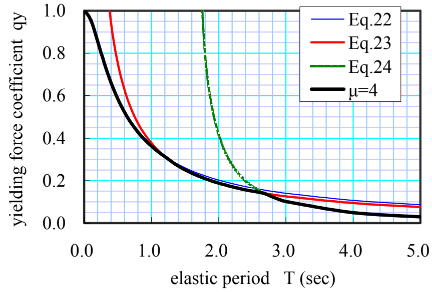


Figure.4 Required strength spectra

2.5 *Effects of peak values of reversed pulses*

Equations (22-24) evidence the effects of each peak values of pulse input as well as the effects of structural characteristics. From the example shown by Fig.4, we can see Equation (22), i.e., the acceleration pulses decide the response of structures with not very long elastic period, such as when $T < 2.0\sec$ for the assumed peak values. Here, let us using Equation (22) to discuss how each parameter influence the inelastic response. For convenience, Equation (22) is reformulated and normalized as follows.

$$\frac{q_y g}{A_p} = \frac{2}{1+\sqrt{1+\dfrac{2(\mu-1)}{3\pi^2}\left(\dfrac{A_p}{V_p}T\right)^2}} \qquad (25)$$

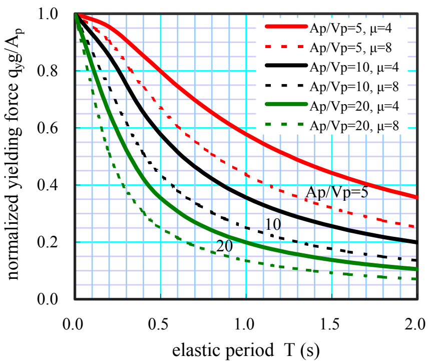


Figure.5 Normalized required strength spectra obtained from acceleration pulse

It means acceleration A_p has a major effect, but it does not have a linear effect on q_y, as A_p also appear in the denominator of the equation. Moreover, the ratio A_p/V_p, elastic period T and the term $\sqrt{\mu-1}$ have the same effect on q_y. Examples are shown by Fig.5, the effect of velocity increases with the increase of elastic period.

3 COMPARISION WITH RESULTS OF TIME HISTORY ANALYSIS

The formulas for predicting the maximum inelastic response of SDOF model are checked with results from time history analysis. An open program of time history analysis (Hachem, 2000) is employed to obtain required strength spectra for constant ductility factors. Viscous damping ratio is assumed as 0.05.

3.1 *Pulse-type ground motions*

Fifteen ground motion records are used as shown by Table 1 (No.11 is a random, non-pulse ground motion). They are provided within the program, and peak values are also quoted from there. These pulse-type ground motions may be categorized as three groups.

For velocity-pulse type ground motion with high-frequency component, type C has symmetric displacement as well as one or two cycles of velocity pulses; while type C.M has only half cycle displacement or one cycle of velocity pulse. Type L is the long period pulse-type ground motion, where simple continuous function such as sine function could represent very well not only the velocity but also the acceleration; it has been investigated by many researches. The effect of Type L ground motion could not be accounted by pulse input assumed in this research, as the intermittent rectangular acceleration pulses could not consider the resonant effect of a sine wave.

3.2 *Discussion on the calculation accuracy*

The calculation accuracy is evaluated using the following Equation.

$$err = \frac{q_{y(cal)} - q_{y(resp)}}{q_{y(cal)}} \qquad (26)$$

where $q_{y(cal)}$ and $q_{y(resp)}$ are results calculated using the proposed formulas and from time history analysis respectively.

err are calculated for ductility factor $\mu = 2,4,6$ and elastic period $0.1\sec \le T \le 3.0\sec$. Their mean and standard deviation for a specific ground motion are listed in Table 1.

Examples of required strength spectra q_y and calculation accuracy *err* are shown by Figs.6-8. Here, peak values (PGA, PGV, PGD) of ground motions are used for replacing (A_p, V_p, D_p) of the defined reversed pulse inputs for calculation.

As shown in Table 1, for most C type ground motion, the mean values of *err* are large than *zero,* the standard deviation of calculation errors are under *0.314*. For L type, CM and M type ground motions, calculation errors exhibit different tendency and have larger standard deviation of calculation errors. In a special case, the calculation errors of No.11 random ground motion are especially large.

Fig.6 shows the results of No.1 (fn component of Tabas) ground motion (Type C). In spite the prediction formulas somewhat underestimate q_y, i.e., the inelastic displacement for smaller ductility factor ($\mu = 2$) and short elastic period ($T \le 0.4\mathrm{sec}$). In overall, they give a consistent evaluation of inelastic response subjected to this type of ground motion. The underestimation partially occurs as we assumed same peak values for the input pulses in the positive and negative directions. The prediction accuracy may be improved as described later.

Fig.7 shows the results of No.4 (fp component of Array 5, Imperial Valley 1979) ground motion (Type C.M). The prediction formulas greatly overestimate q_y for medium elastic period. This is due to the prediction formulas are derived based on pulse inputs shown by Fig.2, where the assumed reversed velocity pulse overestimate the input energy of Type C.M ground motion. We think this may be improved by using more types of pulse shape.

Fig.8 shows the results of No.14 (n component of Takatori, Kobe) ground motion (Type L). The prediction formulas underestimate q_y for smaller ductility factor ($\mu = 2$), while give an overestimation for large ductility factor ($\mu = 6$). The prediction accuracy is not so satisfactory for this type of ground motion with single predominant period。

Table 1. Peak values of pulse-type ground motions

No.	Ref.*	Earthquake	Year	Station	Comp.	PGA(m/s^2)	PGV(m/s)	PGD(m)	char.[*2]	computation error[*3] Mean	Std. dev.
1	NF01	Tabas, Iran	1978	Tabas	fn	8.83	1.10	0.56	C	0.179	0.260
2	NF02	Tabas, Iran	1978	Tabas	fp	9.59	1.06	0.75	C	0.267	0.314
3	LA03	Imperial Val., USA	1979	Array 5	fn	3.86	0.83	0.33	C	-0.015	0.174
4	LA04	Imperial Val., USA	1979	Array 5	fp	4.79	0.77	0.48	C.M	0.255	0.355
5	LA05	Imperial Val., USA	1979	Array 6	fn	2.96	0.89	0.48	C	0.089	0.209
6	LA06	Imperial Val., USA	1979	Array 6	fp	2.30	0.48	0.30	C	0.032	0.248
7	NF09	Erzincan,Turkey	1992	Erzincan,M	fn	4.24	1.19	0.42	L	0.050	0.277
8	NF13	Northridge, USA	1994	Rinaldi	n	8.73	1.74	0.38	L	-0.010	0.332
9	NF14	Northridge, USA	1994	Rinaldi	p	3.81	0.61	0.17	C	-0.081	0.297
10	NF15	Northridge, USA	1994	Sylmar(O.V.)	n	7.18	1.22	0.31	C	0.020	0.270
11	NF16	Northridge, USA	1994	Sylmar(O.V.)	p	5.84	0.54	0.09	Random	-0.313	0.592
12	LA13	Northridge, USA	1994	Newhall	fn	6.65	0.96	0.20	L	0.047	0.324
13	LA14	Northridge, USA	1994	Newhall	fp	6.45	0.81	0.36	C.M	0.103	0.293
14	NF19	Kobe, Japan	1995	Takatori	n	7.71	1.74	0.56	L	-0.073	0.320
15	NF20	Kobe, Japan	1995	Takatori	p	4.16	0.64	0.23	L	-0.070	0.319

[*1] Ground motion name in Bispec software, from where the peak values are cited. [*3] error=(cal.-resp.)/cal.

[*2] [C: symmetric one or two cycle pulse; C.M: half cycle or monotonic pulse] with high-frequency component; L: long period pulse.

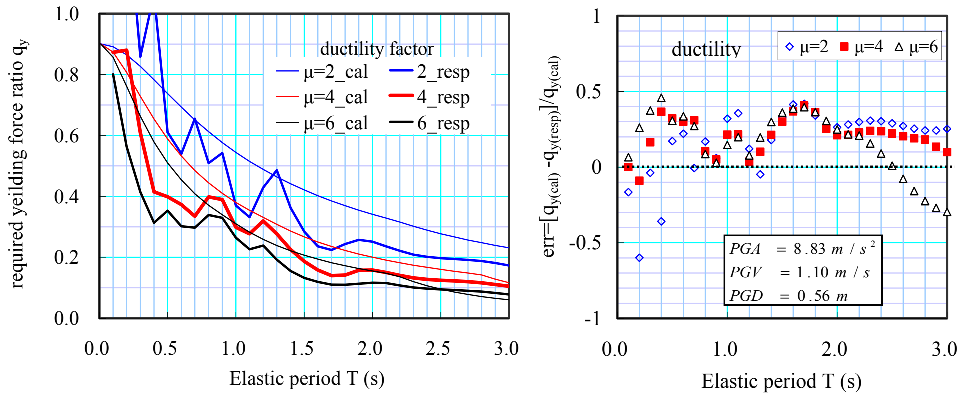


Figure 6. Requied yielding strength factor by calculation and response analysis [left], relative error [right]
(No.1: fn component, Tabas, Tabas 1978, mean=0.179, standard deviation=0.260)

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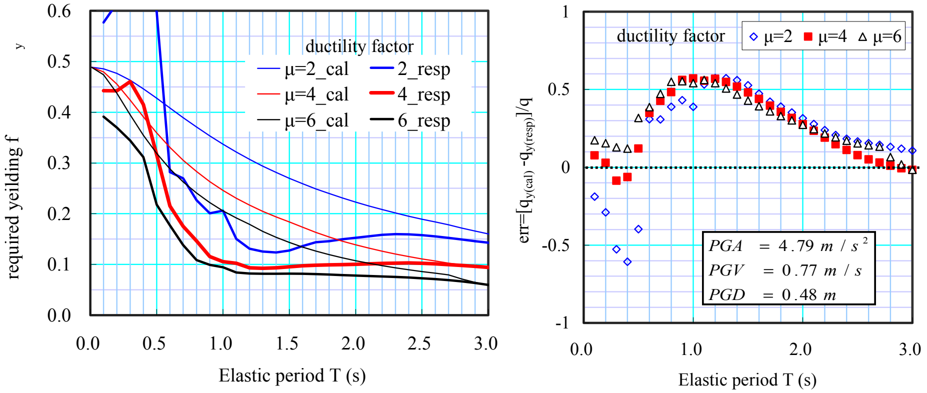

Figure 7. Requied yielding strength factor by calculation and response analysis [left], relative error [right]
(No.4: fp component, Array 5, Imperial Valley 1979)

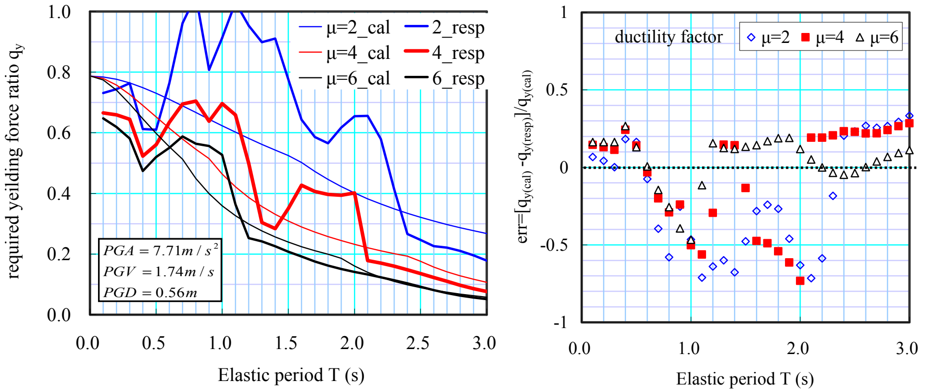

Figure 8. Requied yielding strength factor by calculation and response analysis [left], relative error [right]
(No.14: n component, Takatori, Kobe 1995)

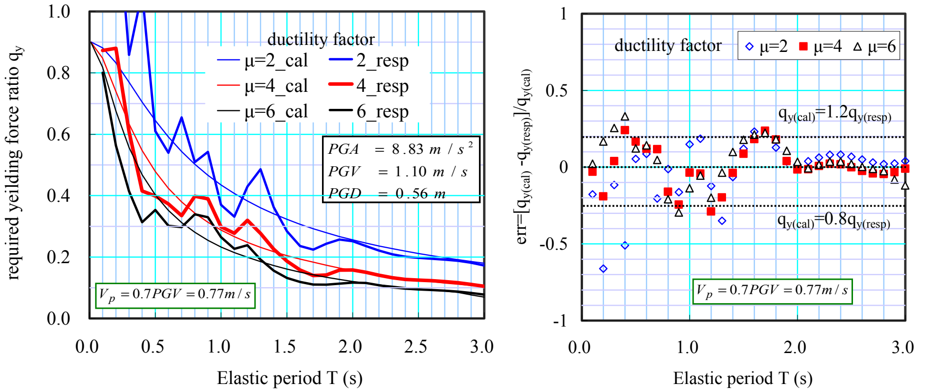

Figure 9. Requied yielding strength factor by calculation and response analysis [left], relative error [right]
(Using revied pulse velocity, No.1: fn component, Tabas, Tabas 1978)

3.3 *Future research for improving the calculation accuracy*

This research assumed one cycle symmetric ground displacement that decides the duration of continuous trapezoidal velocity pulses, and intermittent rectangular acceleration pulses are further assumed to generate the velocity pulses. This simplification was intended to reflect the effects of Type C ground motion with pulse-type velocity and high-frequency acceleration component. Future works are needed to categorize ground motions with several different pulse shapes,

In order to predicting the large inelastic response with explicit formulas, the elastic response is ignored, that may result significant errors if the inelastic response is small. Amplification during elastic vibration should be included in the future.

Even for Type C ground motion, the peak values of ground motion may be different. In Fig.9, a smaller peak value of velocity pulses is assumed, then the calculation accuracy is greatly improved compared with Fig.6 (Mean: from *0.179 to 0.005*; standard deviation: from *0.260 to 0.162*).

4 CONCLUSIONS

Reversed rectangular acceleration and velocity pulses are assumed to represent the effects of pulse-like ground motions. With some simplifications of restoring force history, three simple formulas are obtained for predicting the maximum response of SDOF elasto-plastic systems; they are corresponding to the acceleration sensitivity, velocity sensitivity and displacement sensitivity respectively.

The prediction formulas reflect the composite effects of peak values of pulse acceleration A_p, velocity V_p and displacement D_p against all characteristics of inelastic structures, i.e., elastic period T, ductility factor μ or yielding strength q_y.

The proposed formulas are applied for predicting inelastic response subjected to recorded pulse-like ground motions by using their peak values.

For ground motion with pulse-type velocity and high-frequency acceleration component, the prediction formulas give a larger required strength, or overestimation of inelastic displacement. The maximum of computation errors are *0.267* for mean, and *0.355* for standard deviation. The prediction accuracy may be significantly improved if we have information about the waveforms of ground motions. This includes using several different types of pulse shapes, adjusting the reversed velocity values or their duration.

Comparing with statistical index using only one peak values of ground motion for damage evaluation, the method demonstrated may be useful for judging the destructive power of earthquake. The proposed method may also be used to correlate with time history analysis for developing empirical formulas. These need further studies.

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REFERENCES

Alavi,B., Krawinkler,H. 2000. Consideration of near-fault ground motion effects in seismic design, *Proceedings of 12th World Conference on Earthquake Engineering, Auckland, New Zealand,* (Paper No.2665).

Caughey,T.K. 1963. Equivalent linearization techniques, *Journal of the Acoustical Society of America,* Vol.35, No.11: 1706-1711.

Cuesta,I., Aschheim,M. 2001: Isoductile strengths and strength reduction factors of elasto-plastic SDOF systems subjected to simple waveforms, *Earthquake Engng Struct. Dyn.*: 1043-1059.

Hachem,M.M. 2000. Bispec help manual, Version 1.0, Pacific Earthquake Engineering Research Center.

Hall,J., Aagaard,B. 1998. Fundamentals of the near-source problem, *Proceedings of the 5th Caltrans Seismic Research Workshop.*

Housner,G.W. 1956. Limit design of structures to resist earthquakes, *Proceedings of 1st World Conference on Earthquake Engineering*, San Francisco, 5-1-12.

Makris,N., Black,C. 2003. Dimensional analysis of inelastic structures subjected to near fault ground motions, *Earthquake Engineering Research Center report,* No.2003-05.

Mavroeidis,G.P., Papageorgiou,A.S. 2003. A mathematical representation of near-fault ground motions, *Bulletin of the Seism. Soc. Am.*, 1099-1131.

Nakamura,Y., Kabeyasawa, T. 1998. Estimation of inelastic displacement response from equivalent linearization using instantaneous energy, *proceedings of the 10th Japan Earthquake Engng. Symposium,* 2573-2578.

Newmark,G.W. 1965. Effects of earthquakes on dams and embankments, Fifth Rankine lecture, *Geotechnique 15*: 139-160.

Sakai,Y., Minami,T., Kabeyasawa, T. 1999. A method to simplify strong ground motion to a sine wave by taking into account inelastic response of structures, *Journal of Struct. Engng*, Vol.45B, Architectural Institute of Japan, 81-86 (in Japanese).

Sasani, M. 2004. Seismic demand on short period RC structures utilizing a new measure of pulse-type ground motion severity, *the 13th World Conference on Earthquake Engineering,* Vancouver, Canada , (Paper No.2838).

Shibata A. 1981. Advanced Seismic Analysis, Morikita Press: 118-120. (in Japanese)

Tanahashi,R. 1956. Studies on the nonlinear vibrations of structures subjected to destructive earthquake, Proceedings of 1st *World Conference on Earthquake Engineering*, San Francisco: 6-1-12.

Veletsos,A.S., Newmark,G.W. 1960. Effect of inelastic behaviour on the response of simple systems to earthquake motions, *Proceedings of 2nd World Conference on Earthquake Engineering*, Tokyo: 895-912.

Zhang,F., Sakai,H. et al. 2004. Prediction of earthquake response of SDOF elasto-plastic system using reversed pulses as ground motion, *Journal of Structural Engineering*, Vol.50B, Architectural Institute of Japan, 435-440 (in Japanese).

