IMPORTANCE OF SEVERE PULSE-TYPE GROUND MOTIONS IN PERFORMANCE-BASED ENGINEERING: HISTORICAL AND CRITICAL REVIEW

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SUMMARY

A historical review of the recorded earthquake ground motions, EQGMs, that contain severe pulses and of the studies that have been conducted regarding the effects of these pulses on the seismic performance of buildings is carried out. The studies conducted in the 1970's and 1980's indicate clearly that there were recorded EQGMs with severe acceleration pulses which were critical for the design of structures in which significant inelastic behavior (ductility) was acceptable.

Normalized displacement and yield coefficient response spectra of idealized and recorded pulse-type ground motions are defined and it is shown that these normalized response spectra can be used to determine critical ground motions for given structures. Using the normalized response spectra the efficiency of added damping in controlling the lateral displacement of medium and long period structures is shown. This retrofitting strategy proven to be effective in controlling the response of multi-degrees of freedom systems. It is also shown that short and medium period structures could experience base shear coefficient at least as large as the peak ground acceleration.

INTRODUCTION

The importance of increasing the reliability with which earthquake ground motions (EQGMs) to be used for Earthquake-Resistant-Design (EQ-RD) are determined for the different probabilistic hazard levels cannot be overstressed enough in the development of any performance-based earthquake-resistance design (P-B EQ-RD) methodology [Bertero, 1997]. In spite of the fact that analyses of accelerograms recorded during earthquakes which have occurred since the 1950s show that the EQGMs can be of severe pulse-type, only recently the possibility of occurrence of EQGMs with one or more severe pulses and their importance for the EQ-RD of civil engineering facilities has been recognised and introduced into the seismic provisions for EQ-RD of such facilities through what has been called the near-source factor [UBC, 1991 & 1997].

NEAR FAULT RECORDS AND THEIR IDEALIZATION

The record obtained during the Port Hueneme (CA) EQ of March 18, 1957 was shown to be essentially a single pulse of energy [Housner and Hudson, 1958]. In spite of its small Richter Magnitude (M) of 4.7, it caused exceptional damage for a shock of so low M and Peak Ground Acceleration (PGA)=0.08g. Destructive single-shock EQGMs associated with moderate M (5.4 to 6.2) and shallow foci (<30 km) occurred in: Agadir in 1960 [Despeyroux, 1960]; Libya in 1963 [Minami, 1965]; Skopje in 1963 [Ambraseys, 1964]; and San Salvador in 1965 [Rosenblueth and Prince, 1965].

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Perhaps the first destructive EQ in the U.S. with recorded accelerograms that contained acceleration pulses with very severe damage potential, was the February 9, 1971 San Fernando EQ with an M=6.2. The derived records from the recorded accelerations at the Pacoima Dam (DPD) and at the lower Van Norman Dam (DLVND) have acceleration pulses that led to very large incremental (peak to peak) velocities of 158 cm/sec for DPD and 172 cm/sec for the DLVND.

Bertero et al. [1976] assessed the implications of the records obtained during this San Fernando EQ at sites located near the fault. They offered a series of conclusions and recommendations, including the following: "Near-fault records of the San Fernando earthquake, such as those for the Pacoima Dam and Van Norman dams contain severe long acceleration pulses which resulted in large velocity increments. This was found to be the characteristic of near-fault motions. Results of an analytical study of a building near the fault zone using the DPD record correlated well with the observed damage. This damage appears to have been the results of only a few large displacement excursions rather than of numerous oscillations". In 1979 the Imperial Valley (CA) EQ produced several records of EQGMs which contained severe acceleration pulses. Anderson and Bertero [1987] investigated the effects of several of these recorded EQGMs on flexible steel frame buildings. Based on the results obtained in their study they wrote: "Current design response spectra do not recognize two important factors that are the result of impulse-type ground-motions: (1) the increase in spectral response in the long-period region (1-3 sec) that results in significantly higher accelerations, velocities, and displacements and should be included in spectra used to design structures in close proximity to active faults; and (2) the increase in the ductility requirement for more rigid structures, which is due to the increase in the ratio of the pulse duration to the period of the structure".

In view of the results obtained in the previous studies it has been very surprising that until 1991 the effects of severe pulse-type EQGMs had not been incorporated in the seismic code provisions for practical analysis and design of civil engineering structures, even for those located in the near-field of the source; an exception has been for the design of very special facilities like dams [Bolt, 1997]. In the 1991 UBC's regulations for seismic-isolated structures introduced for the determination of the seismic coefficient, near-source factors related to the proximity of the building or structure to known faults with established magnitudes and slip rate.

Although pulse-type ground motions are not only limited to sites located at the vicinity of faults and have been observed far from the seismic sources, such as the Bucharest accelerogram, Figure 1, which was recorded at about 150 km away from the source of the 1977 Vrancea, Romania EQ, to idealize these types of ground motions, only near source records are considered herein. The propagation of fault rupture toward a site at very high velocity (close to the shear wave velocity) causes most of the seismic energy from the rupture to arrive in a pulse-type ground motion. These velocity or displacement pulses sometimes are referred to as source fling, [Bolt, 1997]. It is well established that mainly based on the forward directivity effect of the propagated seismic waves and the expected types of the ground displacement near to the causative faults, the ground velocity may consist of one or two strong pulses for recorded motions parallel and normal to the fault, respectively [Somerville, 1997]. Figures 2 and 3 show the time histories of the fault normal ground motion recorded at Los Gatos station during 1989 Loma Prieta Earthquake and at the Takatori station during 1995 Kobe Earthquake, respectively. Figure 4 shows just the shape and not the values of idealized fault normal and fault parallel ground motions. As can be seen, for fault normal ground motion, the displacement at the end of motion is set equal to zero, while for fault parallel ground motion, there is a permanent displacement which is caused by the relative displacement of the ground at two sides of strike-slip faults. Note that t_d is the total duration of the strong velocity pulse(s) that could be as large as two to three seconds. It should be noted that the required conditions for forward directivity are also met in dip slip faulting, including both reverse and normal faults [Somerville, 1997].

Studying the wave propagation in shear type structures Iwan [1997] has shown that interstory drift, and as a result, lateral displacement of the structures could be calculated using the ground velocity and displacement (the ground velocity has a more important effect, particularly when damping coefficient is small). Therefore, in idealizing the near fault records, especial care needs to be given to reasonably modeling the recorded ground velocity. Comparing Figures 2 and 3 with Figure 4, it can be seen that the Los Gatos and Takatori fault normal ground motions have strong velocity pulses with durations equal to 2.7s and 1.7s, respectively. As shown in Figures 2, 3, and 4, the duration of severe fault-normal pulse-type ground motions, t_d, is equal to the duration of two consecutive severe velocity pulses.

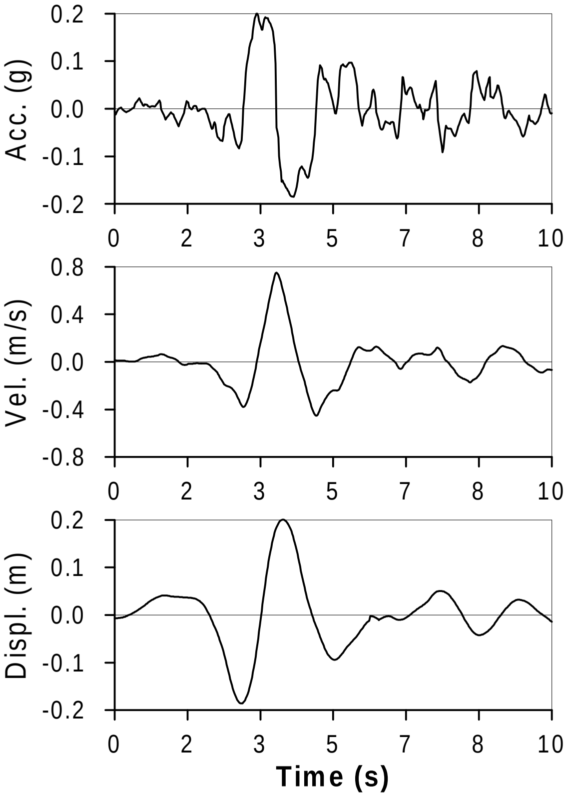

Figure 1. Ground motion at BRI station during 1977 Vrancea earthquake.

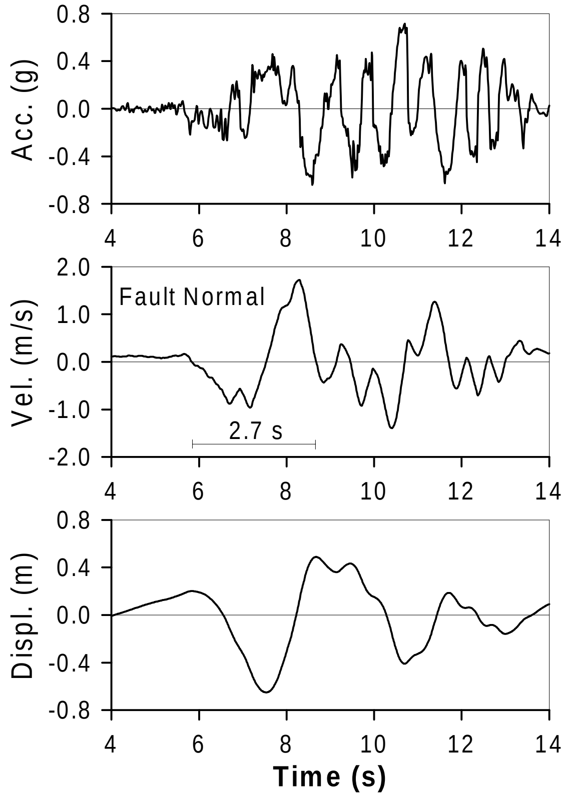

Figure 2. Ground motion at Los Gatos station during 1989 Loma Prieta earthquake.

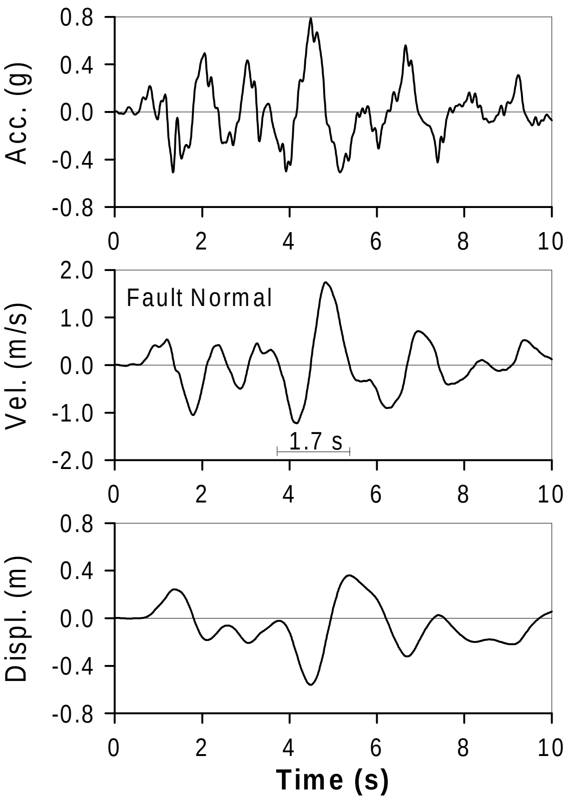

Figure 3. Ground motion at Takatori station during 1995 Kobe earthquake.

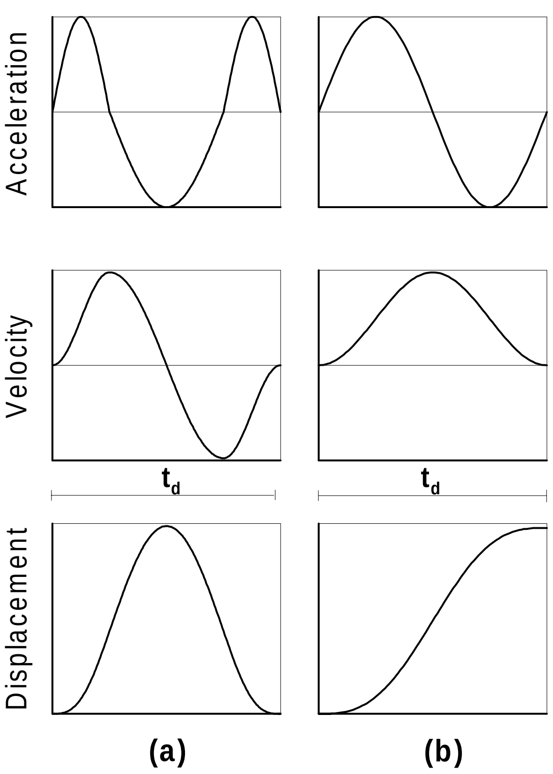

Figure 4. Idealized ground motions: (a) fault normal; and (b) fault parallel.

RESPONSE TO RECORDED AND IDEALIZED PULSE-TYPE GROUND MOTIONS

Since strong velocity pulses could have different durations, t_d, to facilitate the choice of the critical EQGM, the effects of t_d on the response of single degree of freedom, SDOF, systems as well as how to normalize the response spectra to generalize their applications are discussed bellow.

Nonlinear Dynamic Analysis of SDOF Systems

Assuming linear variation of acceleration over each time step of solving the differential equation of motion, the incremental equation of motion for a SDOF system can be written as below [Clough and Penzien, 1993]:

$$\left[\frac{k(t)}{m}+\frac{6}{\Delta t^2}+\frac{3}{\Delta t}\frac{c(t)}{m}\right]\Delta u=-\Delta\ddot{u}_g(t)+\left[\frac{6}{\Delta t}\dot{u}(t)+3\ddot{u}(t)\right]+\frac{c(t)}{m}\left[3\dot{u}(t)+\frac{\Delta t}{2}\ddot{u}(t)\right] \quad [1]$$

where k(t), c(t), and m are the stiffness, damping, and mass of the SDOF system, respectively. u_g and u are the ground displacement and relative structural displacement, respectively. Assume that the duration of motion is scaled by γ. Dividing both sides of the above equation by γ^2 results in the following equation,

$$\left[\frac{k(t)}{\gamma^2 m}+\frac{6}{\gamma^2\Delta t^2}+\frac{3}{\gamma\Delta t}\frac{c(t)}{\gamma m}\right]\Delta u=-\frac{\Delta\ddot{u}_g(t)}{\gamma^2}+\left[\frac{6}{\gamma\Delta t}\frac{\dot{u}(t)}{\gamma}+\frac{3\ddot{u}(t)}{\gamma^2}\right]+\frac{c(t)}{\gamma m}\left[\frac{3\dot{u}(t)}{\gamma}+\frac{\gamma\Delta t}{2}\frac{\ddot{u}(t)}{\gamma^2}\right] \quad [2]$$

Having $c(t)=2m\sqrt{\frac{k(t)}{m}}\xi$, Equation 2 could be considered as the incremental equation of motion for a new structure with $\left(\sqrt{\frac{k(t)}{m}}\right)_{new}=\frac{1}{\gamma}\sqrt{\frac{k(t)}{m}}$ (this could be interpreted as $T_{new}=\gamma\,T$) solved at new time steps equal to $(\Delta t)_{new}=\gamma\,\Delta t$, under $\left(\ddot{u}_g(\gamma t)\right)_{new}=\ddot{u}_g(t)/\gamma^2$. This new ground acceleration is found by modifying the original ground acceleration as following: (1) the time axis is scale by γ, $(t_d)_{new}=\gamma\,t_d$, and (2) the ordinate of the ground acceleration is scaled by $1/\gamma^2$. Therefore, in terms of the ground velocity, if the time is scaled by γ and the ground velocity is scaled by $1/\gamma$ (PGV t_d remains unchanged) the response will be such that $\left(u(\gamma t)\right)_{new}=u(t)$, $\left(\dot{u}(\gamma t)\right)_{new}=\dot{u}(t)/\gamma$, and $\left(\ddot{u}(\gamma t)\right)_{new}=\ddot{u}(t)/\gamma^2$. In other words, the displacement spectra versus T/t_d remains unchanged. Obviously, acceleration spectra versus T/t_d is scaled by $1/\gamma^2$.

Normalized Response Spectra

Using the results of the above discussion, the normalized displacement response spectra of the idealized fault normal ground motion and the two recorded fault normal ground motions for ξ=0.05 and for displacement ductilities, μ, from 1.0 to 3.0 are shown in Figure 5. Note that the horizontal axes are the normalized structural periods with respect to t_d, i.e. T divided by t_d. It should be also noted that in calculating the response spectra, no strain hardening is assumed and the free vibration after the ground motion ends is considered. As can be seen in Figure 5, the response spectra of the idealized ground motion could be considered as representative of those of the recorded motions. Comparing Figures 5a and 5b, around T/t_d=0.3 ($T \approx 0.8$s) the Los Gatos ground motion has higher displacement spectra which could be explained by the higher frequency content, as can be seen from the acceleration time history given in Figure 2. For T/t_d>0.6, the response spectra of the idealized ground motion is higher than those of Los Gatos ground motion which is attributed to the fact that the recorded motion in the first strong velocity pulse has a maximum velocity that is smaller than the PGV. The reasons for the sudden drops in the values of the response spectra observed in Figure 5 for the case of μ>1 are discussed later.

As shown in Figure 6a, in the case of normalized yield coefficient response spectra, the spectral ordinate is in terms of C_y / (PGV / t_d). Note that C_y is the seismic yield coefficient multiplied by g, i.e. is the coefficient when is multiplied by the mass, gives the yield lateral load. For a given period of a structure the idealized response spectra could be used to find the most critical ground motion for the structure under study. To facilitate the application of normalized response spectra, in Figures 6b and 6c, the vertical axes are scaled by T instead of t_d.

As shown in Figures 6b and 6c, the peak values of spectra occurs at T/t_d = 0.75~1.0 (i.e. t_d=1.0~1.33T) and the spectra reduces either by increasing or decreasing the duration of the motion out of the above range.

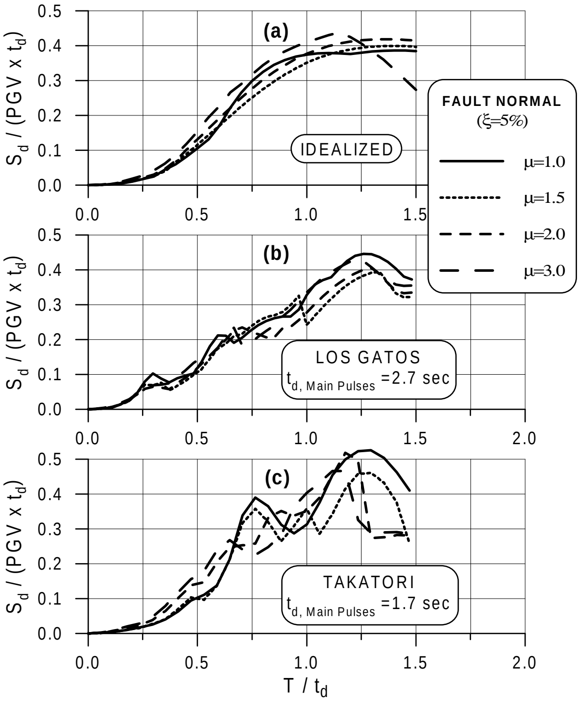

Figure 5. Normalized displacement response spectra for dealized and recorded ground motions.

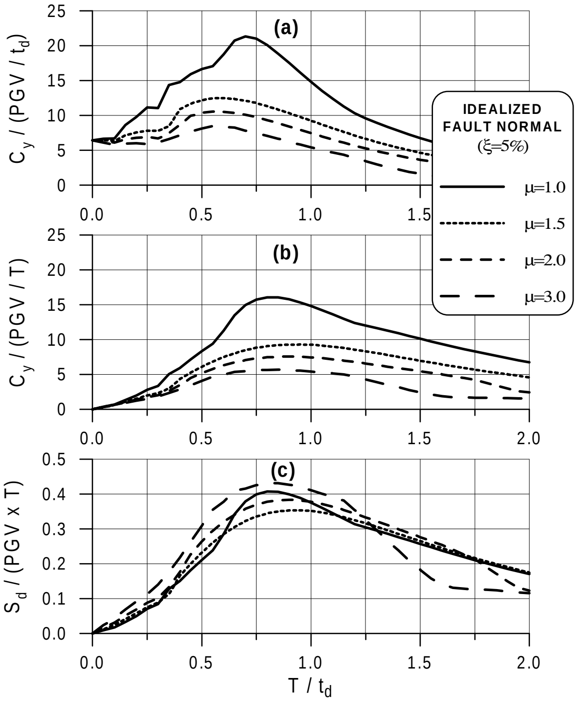

Figure 6. Normalized (a) and (b) yield coefficient response spectra; and (c) displacement response spectra.

It should be noted that for T/t_d =0, the normalized C_y, i.e. C_y / (PGV / T) is zero. This could be explained by the fact that as T/t_d approaches zero, either T goes to zero or t_d goes to infinity. In the former case obviously, the normalized C_y approaches zero. In the latter case, as the duration of the ground motion approaches infinity, for a given value of PGV, the corresponding PGA approaches zero as well, and as a result the yield coefficient drops to zero. To illustrate the application of these normalized response spectra of idealized ground motions, assume T=1.5 sec. As shown in Figures 2 and 3, both the Los Gatos and Takatori ground motions have the same PGV of about 1.73m/sec (68in/sec). Since the duration of the main velocity pulses of the Los Gatos and Takatori ground motions are about 2.7 sec (1.80 T) and 1.7 sec (1.13 T), respectively, it is expected that the Takatori ground motion be more critical for the structure with a period of 1.5 sec. As a matter of fact, from Figure 6b or 6c, one can conclude that for μ=2, the spectral value for the Takatori ground motion being about 1.28 times that of Los Gatos ground motion. Using the response spectra of these two-recorded ground motions, Figures 5a and 5b, the displacement spectra of the Takatori ground motion is about 1.45 times that of Los Gatos ground motion.

Another important observation that one can make from Figure 6a is that structures with a displacement ductility up to 3 and a period of up to about 3/4 of the duration of pulse-type ground motion, will experience at least the PGA of the ground motion as the base shear coefficient. This value that could be quite significant suggest that, unlike what could be interpreted from the UBC-97 near source amplification factor, as far as pulse-type ground motions are concerned, short period structures may as critical structures as long period ones.

The displacement and yield coefficient response spectra of the idealized fault parallel ground motion are shown in Figures 7a and 7b. The PGV of fault parallel ground motions is usually smaller than that of fault normal ground motions. Comparing Figures 5a and 7a, even if the fault normal and fault parallel ground motions had the same PGV, if the duration of each single velocity pulse of the two types of ground motions are assumed to be equal, i.e. $(t_d)_{FN} = 2(t_d)_{FP}$, the fault normal ground motion is more critical than the fault parallel ground motion.

As mentioned before, the response spectra of the pulse-type ground motions experience some sudden drops. For instance, assuming that t_d =2.0sec, for μ = 1.5 and at T=1.9 sec (T/t_d = 0.95) there is a drop in the response spectra shown in Figure 7b. However, as shown in Figure 7c, if the cyclic ductility rather than the cinematic (or monotonic) ductility (which is the one used in Figure 7b and that is the one usually used) is used, in which the maximum cyclic displacement is measured from the previous zero crossing (Figure 8) those drops disappear. This could be explained by studying the hysteresis of SDOF systems with T=1.9 sec and 2.0 sec, shown in Figure 8. For the structure with T=1.9 sec, first it moves to the left, but just before yielding, the direction of motion changes and the ductility 1.5 is reached in the second half cycle, i.e. to the right. By increasing the period of structure to 2.0 sec, the structure experiences yielding in the first and second cycles. As a result of plastic deformation in the negative direction, even though there is a larger plastic deformation to the right, the absolute maximum displacement occurs in the first half cycle and the corresponding yield coefficient drops.

A STRATEGY TO CONTROL LATERAL DISPLACEMENT AND DAMAGE TO STRUCTURES

The normalized displacement response spectra under the idealized fault normal ground motion for 5% and 30% damping coefficients are shown in Figures 5a and 9a, respectively. The ratio of the spectra are shown in Figure 9b. As can be seen, in spite of the severe pulse-type of the ground motion, for T/t_d between 0.4 and 1.2, increasing damping coefficient from 5% to 30%, the lateral displacement is reduced by at least 40%. This means that adding dampers to the system could be used as a strategy to control the lateral displacement of structures and the imposed damage, at different performance levels. The effectiveness of the added damping becomes less, for smaller and larger values of T/t_d, which for a given t_d means shorter and longer period structures.

The conventional strategies for upgrading structures are based on increasing the stiffness, strength, and ductility. Among different techniques that have been used to achieve such strategies are adding steel braces or reinforced concrete shear walls and/or steel jacketing of the structural elements. However, even if significantly larger μ is used (which means greater local damage) all these methods could result in an excessive base shear demands and thus larger demands on the foundation. As indicated above, in spite that the critical EQGM for the safety performance level is that of severe pulse-type and thus it could appear that increasing damping could not be an efficient strategy, the results of the studies conducted on SDOF systems have shown that it is indeed an efficient strategy. This has been confirmed by implementing it in the retrofitting of a thirteen-story reinforced concrete frame structure with a period of 3.2 sec [Sasani, M. et al., 1999]. The results of nonlinear dynamic analyses show that under the Los Gatos ground motion, as expected from the study of the response spectra, increasing damping from 5% to 30% reduces the lateral displacement response of the structure by at least 40%.

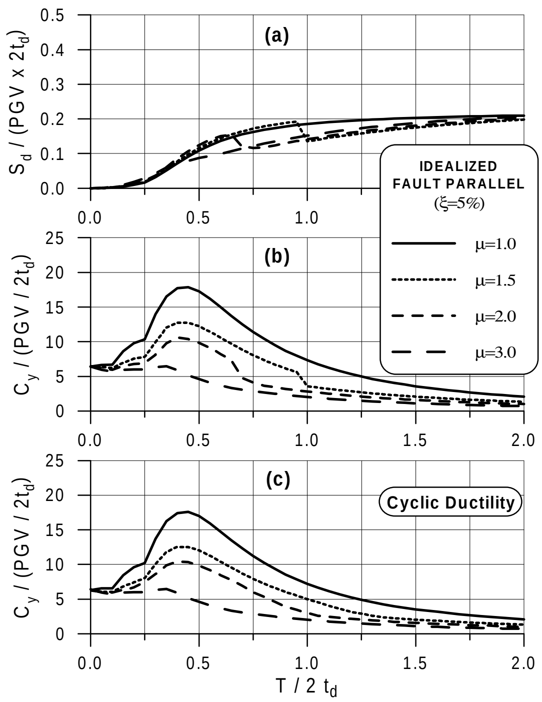

Figure 7. Normalized (a) displacement response spectra; and (b) and (c) acceleration response spectra.

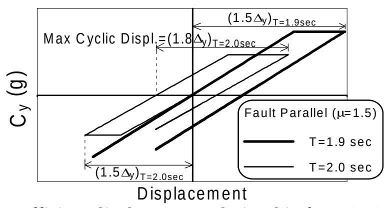

Figure 8. Yield coefficient-displacement relationship for a SDOF system.

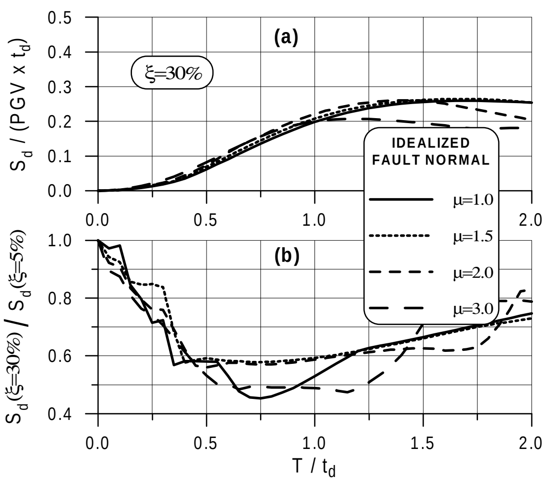

Figure 9. (a) Normalized displacement response spectrafor ξ=30%; and (b) ratio of response spectra for 30% and 5% damping.

CONCLUSIONS

The normalized response spectra for idealized severe pulse-type EQGMs have shown to be not only a good representative of the response spectra of the recorded ground motions, but also could be used as a useful tool in design and retrofitting of structures. They could be used as guides to select the most damaging pulse-type ground motions, and perhaps more general random types of ground motions, in order to have the critical lateral displacement and base shear demands.

Based on the yield coefficient response spectra, it is concluded that short and even medium period structures could experience very large lateral forces, even if they may deform well beyond their yield displacements (i.e. for large ductilities). In other words, as far as pulse-type ground motions are concerned, unlike what seems to be indicated by the near source amplification factor introduced by the UBC-97, short and medium period structures may be as critical as long period structures.

It is also concluded that in spite of the severe pulse nature of the near field ground motions, adding damping to medium and moderate long period structures could be an efficient strategy to control the lateral displacement of structures.

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