DAMAGE POTENTIAL OF WHITTIER NARROWS EARTHQUAKE GROUND MOTIONS

H. Krawinkler
Professor of Civil Engineering, Stanford University

A. Nassar
Research Assistant, Stanford University

ABSTRACT

This paper summarizes parts of a project that is concerned with an assessment of the damage potential of the ground motions recorded during the October 1, 1987 Whittier Narrows earthquake. Damage potential is defined here as the seismic demand imposed on building structures with due consideration given to representative structural response characteristics. The demand parameters considered in this study include strength demand, ductility demand, and energy and cumulative damage demands. The seismic demands are predicted from ground motion recordings, utilizing simplified elastic and inelastic bilinear SDOF structural models.

INTRODUCTION

Although it was only of magnitude 5.9, the October 1, 1987 Whittier Narrows earthquake has caused considerable damage in the larger Los Angeles area. The question to be addressed is whether the ground motions generated in this earthquake justify the extent of damage and whether they are more severe than is anticipated for a magnitude 5.9 earthquake. The great number of ground motions recorded during this earthquake provide a great opportunity to address the issue of damage potential and evaluate predicted and observed performance. They also permit an assessment of attenuation of ground motion effects with distance from the epicenter.

The study summarized in part in this paper provides quantitative data on damage potential and much needed information for a correlation between predicted demands and observed performance of building structures. It addresses and provides partial answers to the following questions: How large was the seismic demand imposed by the Whittier Narrows earthquake and how did the demand attenuate with distance from the epicenter? Are presently used simplified models of demand prediction adequate for a global performance assessment? Does modern Code design provide the intended level of protection against damage and collapse? If not, what are the major lessons that can be learned for improvement of design practice?

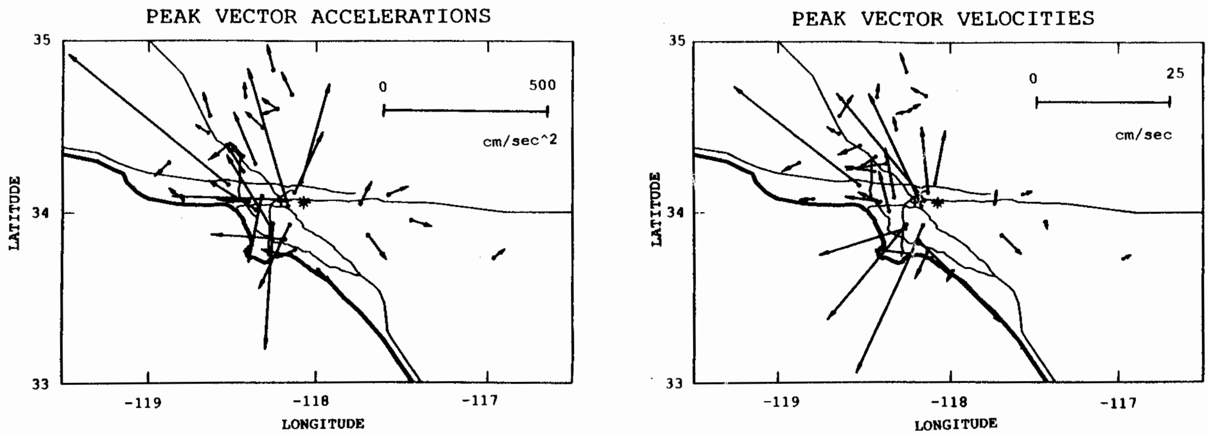
This paper focuses on an evaluation of ground motion and seismic demand parameters and their attenuation with distance from the epicenter. Seismic demand predictions are correlated with estimates of seismic capacity of generic structures in order to assess the damage potential of the ground motions. The results presented here are based on a comprehensive evaluation of the acceleration histories recorded at CSMIP stations located between 7 and 108 km from the epicenter.

EVALUATION OF GROUND MOTION PARAMETERS

Selection of Records

The overriding consideration in the selection of records was that each record could be viewed as a "free-field" record. Thus, only records from instrument shelters or single-story buildings were considered in order to avoid records that could be considerably contaminated by structural feedback. From the three extensive collections of records that were obtained from the earthquake (CDMG, USGS, USC), only the CSMIP stations maintained by CDMG were utilized so far in this study. Very few of the USGS records qualify as "free-field" records, and the USC maintained records from the Los Angeles Strong Motion Accelerograph Network were not available in digitized form early enough to be incorporated at this time.

The locations of the 36 CSMIP stations utilized in this study are shown in Fig. 1, together with vectors indicating the directions and magnitudes of the peak values of accelerations (Fig. 1(a)) and velocities (Fig. 1(b)). These vectors were obtained by vectorially combining the two horizontal components of each record and identifying direction and magnitude of the maximum time history value. Although the main purpose of this figure is to illustrate spatial attenuation, it is interesting to note the trend towards a radial pattern of the vectors, with the epicenter as the focal point.



(a) Peak Vector Accelerations

(b) Peak Vector Velocities

Fig. 1. Locations and Peak Values of CSMIP Records

There are three peculiar records in this set. One is the Mt. Wilson record (#24399, e = 19 km) which was eliminated from further consideration because it is the only rock site record. The other two are the Tarzana record (#24436, e = 44 km) and the Downey record (#14368, e = 17 km). Both records are unusual, the Tarzana record because of its very large PGA value (0.54g), and the Downey record because of its very small PGA/PGV ratio (6.7/sec). Because of their unusual characteristics both records are excluded from the later discussed regression analysis, although their effects on regression lines is relatively small as was tested by including and excluding them in regressions on PGA and PGV attenuations.

Figure 1 shows the maximum of the vector resultant of the two components of each record. However, in all analytical studies and in all results reported from here on, the larger of the two recorded components was used rather than the vector resultant.

Attenuation of Ground Motion Parameters

The following ground motion parameters were evaluated for the CSMIP records:

PGA: Peak ground acceleration of record

$RMSA_{max}$: Maximum value of cumulative root-mean-square function of acceleration record

PGV: Peak ground velocity of record

I_{sm}: Arias Intensity of strong motion portion of record, $I_{sm} = D_{sm}(RMSA_{sm})^2$

D_{sm}: Duration of strong motion portion of record, using the definition proposed by McCann and Shah, 1979.

It was attempted initially to look at spacial variations of these parameters, but it was concluded that the CSMIP records represent too small a sample set to draw definite conclusions on spacial variations. Trifunac, 1988, has reported on spacial variations of PGA, using the records from 68 stations of the Los Angeles Strong Motion Accelerograph Network. He came to the conclusion that the motions were largest to the south and north-west of the epicenter. It is planned to combine the CSMIP and USC records to check whether the combined set confirms the contours obtained by Trifunac from the USC set alone.

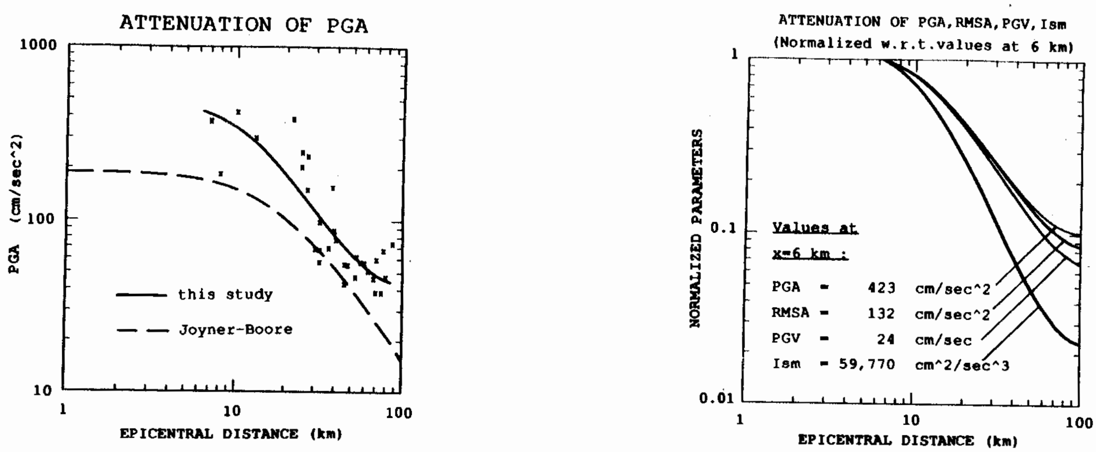
The CSMIP records are utilized here to evaluate attenuation with epicentral distance alone, without regard to geographic location. It is recognized that such an attenuation disregards variations in geological and site conditions. Although the surface geology of the Los Angeles basin is rather complex, most of the area is covered with a layer of recent or quaternary alluvium. Only records at alluvial sites are used in this study.

Regression analysis was performed on all five parameters listed above. For the first four parameters the following relationship between the parameter y and the distance r is assumed:

$$\log y = a + d \log r + kr \tag{1}$$

where r = distance from station to the hypocenter of the earthquake, with the focal depth estimated as 14 km (Hauksson, et al., 1988)
a,d,k = regression parameters.

This equation is of the form proposed by Joyner and Boore, 1988, without consideration of a site soil correction factor. Also, Joyner and Boore set the value of d equal to -1.0, whereas in this study d was a free regression parameter. Results of the regression analysis, plotted versus epicentral distance, are shown in Fig. 2.



(a) Regression of PGA

(b) Normalized Regression Lines for PGA, RMSA, PGV, and I_{sm}

Fig. 2. Attenuation of Ground Motion Parameters with Epicentral Distance

Figure 2(a) shows the data points for PGA values and the corresponding regression line as well as the regression line from Joyner and Boore for a magnitude 5.9 earthquake (they used a focal depth of 8 km in their regression). The figure shows that the Whittier Narrows earthquake generated ground motions that were considerably higher at all epicentral distances than predicted by Joyner and Boore.

Figure 2(b) illustrates the relative attenuation of the four basic ground motion parameters. The rate of attenuation for PGA, RMSA, and PGV is very similar, whereas the Arias Intensity I_{sm} attenuates at a much faster rate.

Regression was also performed on the strong motion duration D_{sm}, but using a second order polynomial. The data points and regression line shown in Fig. 3 indicate a clear trend towards an increase in duration with epicentral distance. The strong motion duration becomes an important parameter when cumulative damage is evaluated, an issue that is part of this study but is not addressed in this paper.

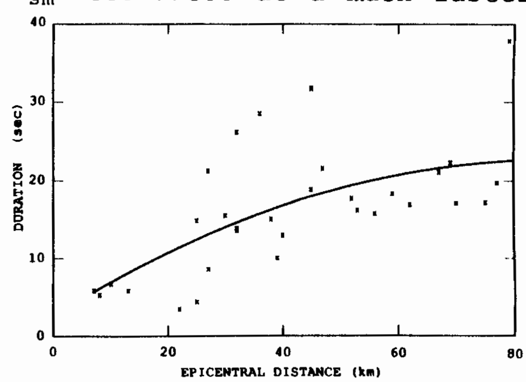


Fig. 3. Attenuation of D_{sm}

EVALUATION OF SEISMIC DEMAND AND CAPACITY OF BUILDING STRUCTURES

It is the main objective of this study to gain a clear understanding of the seismic demands imposed by the Whittier Narrows earthquake on buildings in the larger Los Angeles area The predictions will serve as a basis for assessing the global damage potential of the earthquake as well as the performance of a small number of actual buildings.

For this purpose, seismic demands are predicted from the CSMIP ground motions and capacities of generic code designed structures are estimated.

The global damage potential is assessed through combining the information on demand predictions and capacity estimations.

Seismic Demand Predictions

Seismic performance depends on a number of demand parameters, which may be conveniently classified as follows:

Elastic Strength Demand, $F_{y,e}$. The elastic response spectra provide the needed information on this parameter, which serves to assess the force level at which brittle failure modes have occurred in this earthquake.

Ductility Demand, μ. This parameter is defined as the ratio of maximum deformation over yield deformation for a system with a yield strength smaller than the elastic strength demand $F_{y,e}$.

Inelastic Strength Demand, $F_y(\mu)$. This parameter defines the required yield strength of an inelastic system whose ductility demand is equal to μ.

Strength Reduction Factor, $R_y(\mu)$. This parameter defines the reduction in elastic strength that will result in a ductility demand of μ.
Thus, $R_y(\mu) = F_{y,e}/F_y(\mu)$.

Energy and Cumulative Damage Demands. Repeated cyclic loading is known to have a detrimental effect on inelastic response characteristics. Many cumulative damage models have been proposed in the literature, the simplest one being of the form (Krawinkler, 1987)

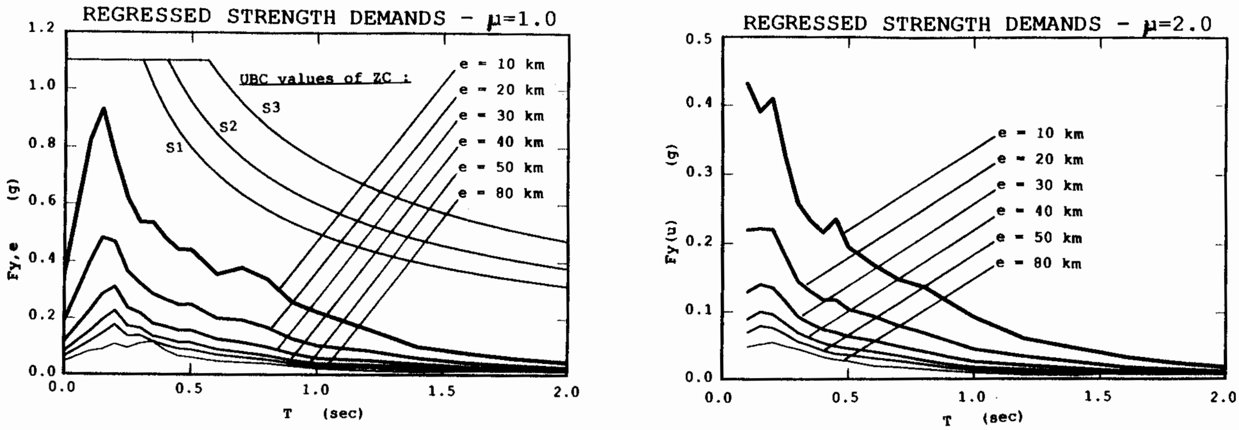
$$D = C\ \Sigma\,(\Delta\delta_{pi})^{c} \tag{2}$$

where D = cumulative damage
C, c = structural performance parameters
N = the number of inelastic excursions experienced in the earthquake
$\Delta\delta_{pi}$ = the plastic deformation range of excursion i.

For bilinear systems this expression reduces to the total hysteretic energy if the coefficient C is taken as the yield strength F_y and the exponent c is taken as 1.0. In this study the cumulative damage demands using exponents of c = 1.5 and 2.0 were evaluated, as well as the hysteretic energy, damping energy, and input energy.

Time history analysis of bilinear Single Degree of Freedom (SDOF) systems was performed to predict these seismic demands, using all of the CSMIP records. The natural period and yield level of the systems were varied to cover the full range of interest. The strainhardening stiffness and damping were kept constant at 10% of elastic stiffness and 5% of critical, respectively.

Because of space limitations, only elastic and inelastic strength demands ($F_{y,e}$, $F_y(\mu)$) are discussed here. Two examples of spectra of these demands are illustrated in Fig. 4. The solid lines represent site specific distance for the elastic demand and the inelastic demand for $\mu = 2.0$. The shapes of the spectra are relatively smooth and change very little with distance. The elastic spectra (Fig. 6(a)) exhibit one consistent large protuberance in the short period range, with a peak that moves with distance from 0.15 sec. at 10 km to 0.35 sec. at 80 km. The high values of elastic strength demand in the short period range of near-source spectra help to explain the large damage experienced by stiff masonry buildings in Whittier and other nearby communities. Superimposed in Fig. 6(a) are graphs of the UBC Code values for the product ZC (for soil types S1, S2, and S3), which are a measure of the elastic strength demand for severe earthquakes implied by code design.



(a) Elastic Strength Demands

(b) Inelastic Strength Demands for $\mu = 2.0$

Fig. 6. Attenuation of Regressed Strength Demand Spectra

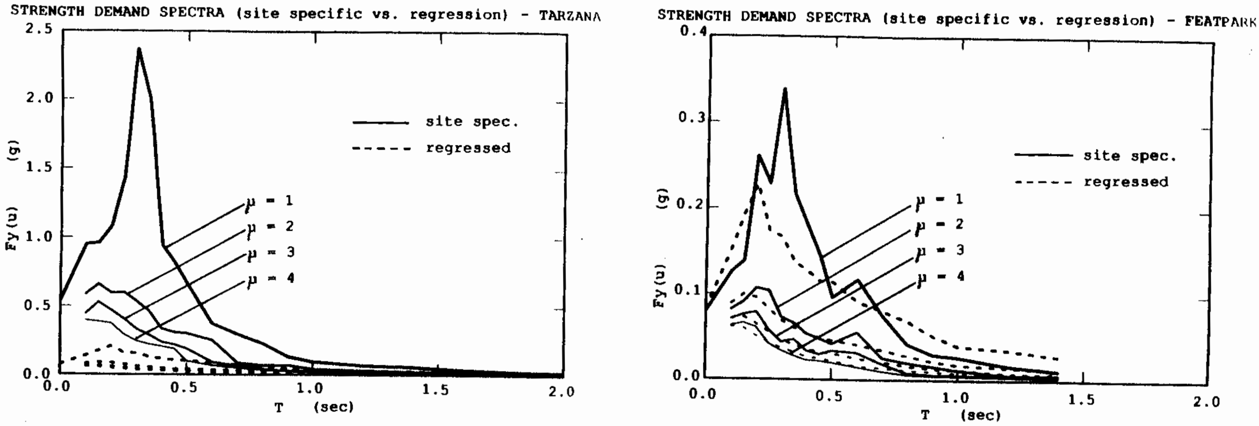
The inelastic strength demand spectra attenuate at a rate that is very similar to that of the elastic spectra. Regardless of distance, these spectra show a relatively high demand for structures with a natural period of 0.25 sec. or smaller. This can be seen from Fig. 6(b) and other spectra for $\mu = 3.0$ and 4.0 which are not shown here.

Figure 6 shows clearly that the strength demands of the Whittier Narrows ground motions are large, particularly near the source. If one would consider modern code seismic design forces as a measure of strength capacity (e.g., V/W = ZC/Rw in the 1988 UBC), then the damage potential of the ground motions appears to be very large. However, damage observations do not confirm high ductility demands for modern structures for the reasons discussed in the next section.

Strength Capacity of Building Structures

It is easy to show, and to some degree intended by code design, that actual structures have a significantly larger lateral strength than is indicated by the code seismic design forces. Structures have overstrength due to a variety of sources, including effects of gravity loads on member strength, stiffness (drift) requirements, increase in structure strength due to redistribution of internal forces in the inelastic range, as well as contributions of structural and nonstructural elements that are not considered as part of the lateral load resisting system.

spectra for μ equal to 1, 2, 3, and 4, whereas the dashed lines represent spectra obtained from a regression analysis discussed next.



(a) Tarzana Record (b) Featherly Park Record

Fig. 4. Site Specific and Regressed Strength Demand Spectra

A global assessment of damage potential cannot be achieved from site specific spectra whose details may be affected considerably by local site conditions. In order to smoothen local site effects and obtain continuous expressions for distance dependent strength demands, regressionanalysis was performed on each spectral ordinate as a function of hypocentral distance. Equation (1) was again utilized for this regression analysis.

A typical example of a regression line and the corresponding data points are shown in Fig. 5 (for T = 0.2 sec and μ = 4.0). The regression line is similar to that for PGA but, again, the data show considerable scatter. This example is shown because the period of 0.2 sec. is in the most sensitive part of the spectra where the scatter is largest.

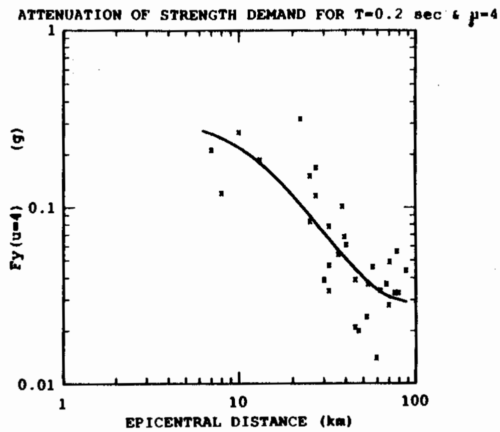


Fig. 5. Attenuation of Strength Demand (T = 0.2 sec., μ = 4.0)

Regressed strength demand spectra at any selected epicentral distance are then obtained from the values of the regression lines for different periods at the selected distance.

Figure 4(a) clearly shows the unusual strength demands of the Tarzana record. The site specific demands exceed the demands obtained from regression by about a factor of 10 in the short period range. The same is not observed in the Featherly Park record (Fig. 4(b)) which is at a similar epicentral distance (40 km vs. 44 km) but to the S-E of the epicenter rather than the N-W. For this record, site specific and regressed demands are similar, particularly for the inelastic strength demands.

An illustration of the attenuation of strength demands is presented in Fig. 6, which shows the variation of regressed strength demand spectra with

In the simplest case, structural behavior can be modeled as shown in the diagram of Fig. 7 (Osteraas and Krawinkler, 1989). In this figure, E represents the seismic design loading, E_1 is the capacity at the member strength level, and E_2 is the capacity at the structure strength level including overstrength. When ductility demands of structures are evaluated, due consideration must be given to this overstrength.

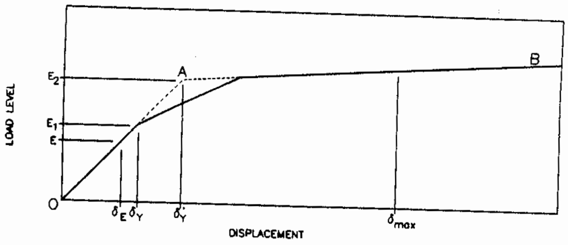


Fig. 7. Simplified Structural Model

The overstrength available in real structures varies widely, dependent on the material and type of structural system, structural configuration, number of stories, and detailing. Although general rules for assessing overstrength cannot be developed for all structures, the need exists to estimate overstrength in order to assess the damage potential of ground motions. Such an attempt has been reported by Krawinkler and Osteraas, 1988, for certain generic types of steel building structures. Efforts are being made as part of this study to estimate overstrength for a limited set of generic reinforced concrete buildings. It would be desirable to do the same for masonry buildings, but not much success can be expected from this exercise because of the great variation in lateral resistance of these structures. Thus, they have to be addressed on a case by case basis.

Information is presently available on the real strength, including overstrength, for three types of generic steel structures designed according to the 1988 UBC (Krawinkler and Osteraas, 1988). These structures are 3-bay by 5-bay buildings of two or more stories, designed as all moment resisting frame structures with Rw = 12 (MF), perimeter frame structures with Rw = 12 (PF), and braced frame structures with two braced bays and Rw = 8 (BF). The bay width was assumed to be 24 ft, altough it was found that the results were rather insensitive to the assumed bay width. For the designed structures the overstrength factor E_2/E is highest for the MFs and lowest for the BFs, decreases with the number of stories, and is as high as 6 for 2-story MFs. Braced frames (BFs) were only used to a period of 0.9 sec. since this period corresponds to the height limitation of 160 ft.

These structures can be viewed as well designed structures that follow basic code requirements, have no excessive waste in their member sizes, but have also no undesirable features such as plastic hinges in columns or weak connections. The information on strength capacity for these generic structures is combined in the next section with the previously discussed strength demands to assess damage potential.

Damage Potential of Ground Motions

Figure 8 shows the strength capacities E_2 of the three generic types of steel structures superimposed on the spectra of strength demands for different ductilities at an epicentral distance of 10 km. It can be seen that, even with the large overstrength available in the generic structures, there are period ranges in which each one of the structure types is expected to experience inelastic deformation. It is fortunate that the overstrengths are highest in the short period range where the strength demands are highest. As can be seen from the figure, for the epicentral distance for which this figure applies, i.e., 10 km, the global ductility demands are not expected to exceed a value of 2.

The attenuation of ductility demands can be evaluated by comparing Fig. 8 with Fig. 9 which shows similar information at an epicentral distance of 20 km. Capacity curves are shown in this figure only for the perimeter frame (PF). In addition to the E_2 curve, the E_1 curve (see Fig. 7) for this structure type is shown as well. Figure 9 indicates that at this distance localized inelastic deformations have to be anticipated for PF structures with periods of less than 0.5 sec. The figure also contains a curve identifying the code seismic design force level for this structure type denoted by E. The differences between E and the capacity curves E_1 and E_2 illustrate the overstrength present in PF structures.

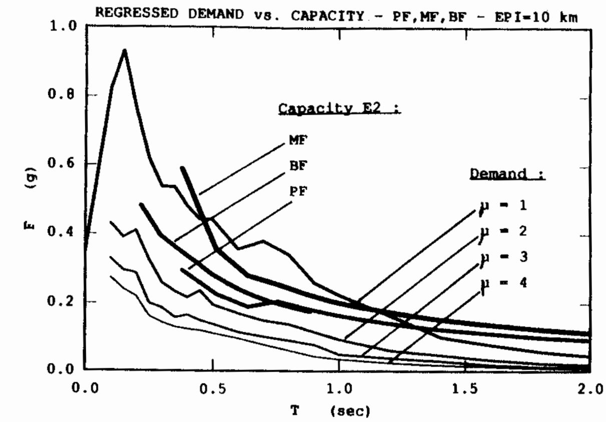


Fig. 8. Strength Demands and Capacities for Generic Types of Steel Structures (e = 10 km)

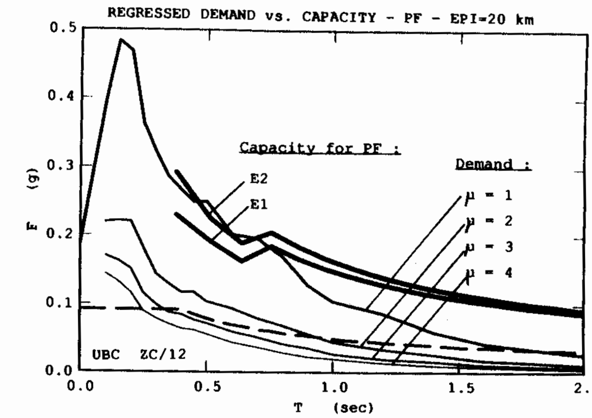
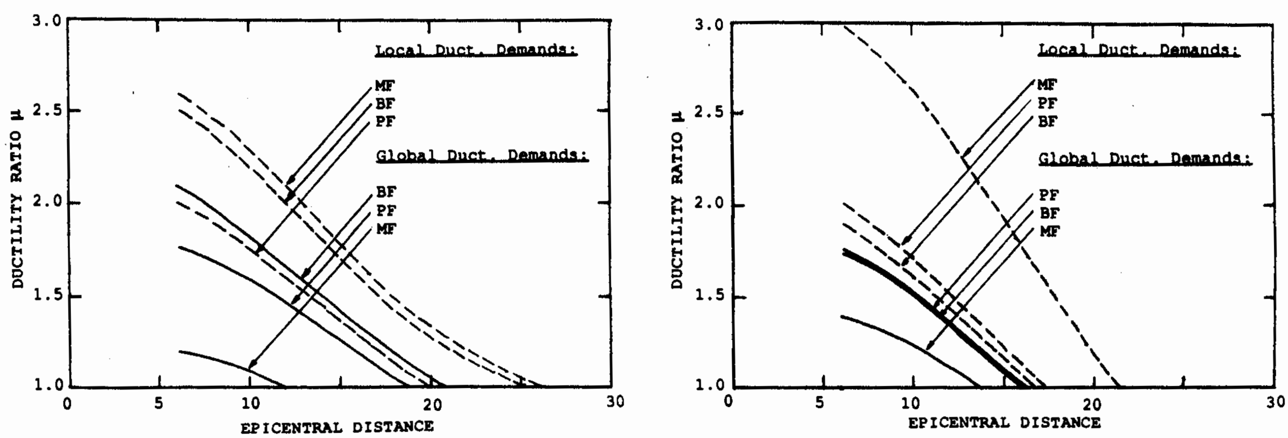


Fig. 9. Strength Demands and Capacities for Perimeter Frame Structures (e = 20 km)

The information of the type presented in Figs. 8 and 9 can be utilized to derive estimates of ductility demands for different types of steel structures as a function of epicentral distance. Figure 10 shows two typical examples, for PF, MF, and BF structures with periods of 0.5 and 0.9 sec. The figure shows global ductility demands (δ_{max}/δ_y' in Fig. 7) as well as local ones (δ_{max}/δ_y in Fig. 7).



(a) For a Natural Period of 0.5 Seconds

(b) For a Natural Period of 0.9 Seconds

Fig. 10. Variation of Ductility Demands with Distance for Three Types of Steel Structures

Information of the type presented in Fig. 10 permit an assessment of the damage potential of the Whittier Narrows earthquake. It is conluded that this earthquake was indeed a severe test for modern structures located near the source. At specific sites, at which localized site conditions led to greater amplification of motions than is indicated by the regressed spectra, the ductility demands may even have been considerably higher.

CONCLUDING REMARK

Regression analysis on ground motion and seismic demand parameters was employed to obtain information that permits a global assessment of the damage potential of ground motions. A realistic assessment of damage potential can be obtained provided that the real strength of structures, including overstrength, is considered. Even when the overstrength is considered, this study has shown that the October 1, 1987 Whittier Narrows earthquake produced seismic demands that were more severe than expected from a magnitude 5.9 earthquake.

ACKNOWLEDGEMENTS

This study was supported by the California Department of Conservation through a research grant provided by the California Strong-Motion Instrumentation Program.

REFERENCES

1. McCann, M.W., and Shah, H.C., 1979, "Determining Strong-Motion Duration of Earthquakes," Bulletin of the Seismological Society of America, Vol. 69, No. 4, August 1979.

2. Trifunac, M.D., 1988, "The Whittier Narrows, California Earthquake of October 1, 1987 - Note on Peak Accelerations During the 1 and 4 October Earthquakes," Earthquake Spectra, EERI, Vol. 4, No. 1, February 1988.

3. Hauksson, E., et al., 1988, "The 1987 Whittier Narrows Earthquake in the Los Angeles Metropolitan Area, California," Science, American Association for the Advancement of Science, Vol.239, March 1988.

4. Joyner, W.B., and Boore, D.M., 1988, "Measurement, Characterization, and Prediction of Strong Ground Motion," Proceedings of Earthquake Engineering & Soil Dynamics II, GT Div/ASCE, Park City, Utah, June 27-30, 1988.

5. Krawinkler, H., 1987, "Performance Assessment of Steel Components," Earthquake Spectra, EERI, Vol. 3, No. 1, February 1987.

6. Krawinkler, H., and Osteraas, J., 1988, "Steel Building Design," Seminar on Evolving Earthquake Hazard Mitigation Practices, 1988 Annual Meeting, EERI, Mesa, Arizona, Feb. 4, 1988.

7. Osteraas, J., and Krawinkler, H., 1989, "The Mexico Earthquake of Septemner 19, 1985 - Behavior of Steel Buildings," Earthquake Spectra, EERI, Vol. 5, No. 1, February 1989.