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# ASSESSMENT OF THE SEISMIC VULNERABILITY OF EXISTING BUILDINGS

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## ABSTRACT

A simplified method of analysis to conduct the seismic evaluation of existing structures is presented. The method is based on the comparison of maximum lateral displacement and ductility demands with their corresponding capacities in order to assess the seismic vulnerability of the structure during severe earthquake ground motions. This comparison of demands and capacities is made both at the global level and at the local level. Global seismic deformation demands on the structure are obtained by computing the response on an equivalent nonlinear single-degree-of-freedom system through step-by-step numerical integration. Local deformation demands are computed using a pre-computed relationship between global and local deformation demands which is non-constant and varies primarily with the level of lateral deformation imposed in the building. The use of this simplified method is exemplified in the assessment of the seismic vulnerability of an instrumented existing ten-story reinforced concrete building located in southern California. Results are compared with results of nonlinear time history analyses.

## KEYWORDS

Seismic vulnerability; existing buildings; equivalent SDOF system; displacement-based evaluation criteria.

## INTRODUCTION

Typically, destruction caused by earthquakes results from a combination of severe earthquake ground motions and vulnerable man-made structures. Existing buildings vary from new buildings designed according to modern seismic design provisions to very old buildings erected before the advent of earthquake regulations. Moreover, modifications to seismic requirements have been so rapid that many buildings that are only 20 years old or so, despite having being designed according to codes which included seismic provisions, are now considered as potentially hazardous. One of the most effective ways of minimizing potential earthquake-related losses is to conduct reliable assessments of the vulnerability of existing structures and to develop and implement effective ways to upgrade structures identified as hazardous (Miranda, 1991). Although, it is recognized that earthquake-related structural damage is primarily produced by deformations demands larger than the deformation capacities of the members and connections of the structure, most methods currently used by practicing structural engineers to conduct the seismic evaluation of existing structures are based on comparing member strength demands (usually computed through linear elastic analyses using reduced lateral forces) with member strength capacities. On the other hand, detailed and rational evaluation methods of the seismic vulnerability have been proposed based on inelastic time-history analysis of the whole structure, however, these methods are often lengthy, complex and demanding. There is a need of simplified methods to assess the seismic vulnerability of existing structures that can provide valuable information with less effort but with a similar degree of confidence than that of detailed and rational evaluation methods based on inelastic time-history analysis of the whole structure.

Most vulnerability studies express the level of damage in a structure in terms of Modified Mercalli's intensity (MMI) or MSK insensity (Whitman et al., 1973; Sauter et al., 1980; Applied Technology Council, 1985). One disadvantage of expressing the level of damage expected as a function of MMI, is that this parameter is in itself a measure of damage. Moreover, MMI is a subjective parameter that depends on the local building code requirements and the asignment of specific intensity values to a damaged region varies from investigator to investigator. In order to overcome some of these disadvatanges it is common to correlate MMI with recorded peak ground accelerations (Sauter et al., 1980; Dong et al., 1988), however this ground motion parameter has very poor correlation with observed damage (Bertero et al., 1991).

Structural damage produced by earthquakes is primarily the result of lateral deformation, so the best way to measure the seismic hazard in a structure is through a displacement parameter and to express the vulnerability in terms of its deformation capacity. Recently there has been a growing interest in displacement-based seismic design procedures (Moehle, 1992; Bertero et al., 1991; Wallace, 1995; Calvi and Kingley, 1995; Kowalsky, et al., 1995), however these studies have been oriented to the design of new structures and not to the evaluation of existing ones.The objective of this paper is to present a simplified displacement-based method of analysis to conduct the seismic evaluation of existing structures.

## SIMPLIFIED METHODOLOGY

The proposed methodology is based on the use of equivalent single-degree-of-freedom (SDOF) systems to evaluate the performance of multi-degree-of-freedom (MDOF) systems. Several methods for developing an equivalent SDOF system from a MDOF system have been proposed in the literature (Biggs, 1964; Saiidi and Sozen, 1979). The adequacy of SDOF systems to estimate the global response of MDOF systems has been studied by several investigators (Saiidi and Sozen, 1979; Qi and Moehle, 1991; Miranda, 1991). The simplified evaluation of existing buildings consits of the following steps:

1. Construct and calibrate linear and nonlinear mathematical models of the building.
2. Conduct nonlinear static-to-collapse (i.e., push-over) analysis of the building.
3. With the results of the static nonlinear analysis of the building, determine a relationship between global and local deformation demands for different levels of deformations. The parameters used are global and local (i.e., story) ductility demands, $\mu_G$ and $\mu_L$, respectively; global and local drift indexes, $\gamma_G$ and $\gamma_L$, respectively. These indexes are defined as follows:

$$\mu_G = \frac{\delta_{r,\,max.}}{\delta_{r,\,y}} \tag{1}$$

$$\mu_L = \frac{\gamma_{max.}}{\gamma_y} \tag{2}$$

$$\gamma_G = \frac{\delta_{r,max.}}{H} \tag{3}$$

$$\gamma_L = \left|\frac{\delta_{i+1} - \delta_i}{h_i}\right|_{max.} \tag{4}$$

where $\delta_{r,\,max.}$ is the maximum roof displacement; $\delta_y$ is the yield roof displacement; $\gamma_{max.}$ is the maximum interstory drift index; $\gamma_y$ is the yield interstory drift index; $\delta_i$ is the lateral displacement in the ith level; $H$ the roof height and $h$ the interstory height.

4. Develop an equivalent SDOF system of the MDOF model.
5. Conduct nonlinear time history analyses with the equivalent SDOF system to estimate global displacement and global ductility demands.
6. With the relationship between global and local deformation demands computed in step 3, estimate local deformation demands.
7. Determine the adequacy of the building using a vulnerability funcition which depends on the maximum interstory drift index.

For a linear elastic structure the relationship between global and local displacement indexes remains constant regardless of the level of lateral deformation. However, for a nonlinear structure, this relationship depends on the level of inelastic deformation. There will be a larger difference for structures that tend to concentrate inelastic deformation in only one or only a few stories (e.g., structures that tend to form soft stories because of vertical irregularities in strength and stiffness and/or structures that have been designed with a strong-girder/weak-column ). The relationship computed in step 3, which is a function of the level of inelastic deformation, is strictly valid for static loads and a particular loading pattern (i.e., distribution of lateral forces over the height of the building), however in this methodology is used to estimate local displacement demands during earthquake ground motions.

Alternatively to conducting time history nonlinear analyses with the equivalent SDOF system in step 5, one can use deterministic or probabilistic inelastic strength demand spectra and inelastic displacement demand spectra (Miranda, 1993a, 1993b) to estimate global ductility demands or maximum roof displacement demands.

## Vulnerability functions

In contrast to most vulnerability functions which relate the level of damage with MMI or with peak ground acceleration, it is proposed to relate the level of damage in the building with the maximum intersory drift index $\gamma_{max.}$. The proposed vulnerability function is given by

$$DI = 1 - exp\left[Ln(0.5)\left(\frac{\gamma_{max.}}{\bar{\gamma}}\right)^{\alpha}\right] \tag{5}$$

where DI is a damage index which varies from 0 (no damage) to 8 (maximum damage); $\bar{\gamma}$ and $\alpha$ are parameters which depend on the interstory drift index associated with the onset of damage in the building and the maximum interstory drift index that the critical story in the building can resist. Examples of vulnerability functions computed with eq. (5) are shown in Fig. 1 for pre-1971 reinforced concrete moment resisting frames (MRF) in California and post-1985 reinforced concrete special moment resisting frames (SMRF) in California. For pre-1971 RC frames an interstory drift index of 0.004 has been set as the onset ofdamage in the building (e.g., $DI=1$) and of 0.012 as the interstory drift capacity (i.e., maximum drift index that the critical story in the building can resist). The corresponding values of the parameters $\bar{\gamma}$ and $\alpha$ are 0.008 and 4.42, respectively. For post-1985 RC SMRF an interstory drift index of 0.005 has been set as the onset of damage and 0.030 as the interstory drift capacity. In this case, the corresponding values of $\bar{\gamma}$ and $\alpha$ are 0.0155 and 2.10, respectively.

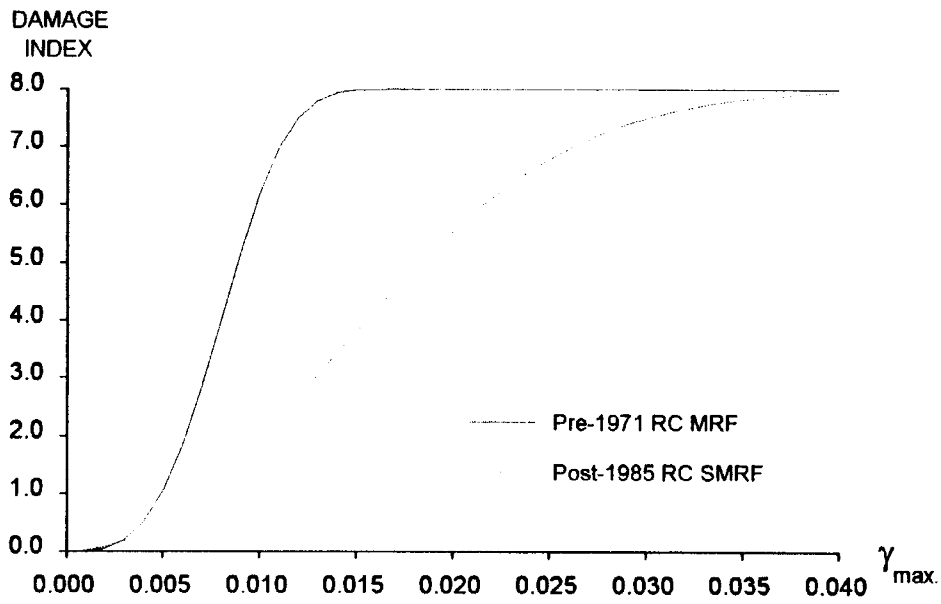


Fig. 1. Examples of displacement-based vulnerability functions.

## USE OF THE METHODOLOGY

### Description of the building and its instrumentation

The ten-story building studied herein is a reinforced concrete structure designed and constructed in 1972 according to the 1970 edition of the Uniform Building Code. It is located at latitude 33.98° N and longitude 118.04° W within the Los Angeles metropolitan area.

The soil conditions at the site consist of quaternary alluvium deposits of medium-grain sand sediments. The foundation of the building consists of spread footings. The height of the building is 27.4 m above the ground level. Interstory heights are 3.66 m in the first story and 2.64 m for the second through tenth stories.

In the longitudinal (north-south) direction the structural system of the building is a moment-resisting frame consisting of two external frames designed to carry most of the lateral loads and two interior frames designed primarily to carry vertical loads. Fig. 2 shows a typical floor plan of the building. The exterior frames consist of 50.8 cm by 50.8 cm reinforced concrete columns and 61 cm by 61 cm beams. The interior frames consist of 40.6 cm by 40.6 cm columns and a cast-in-place 16.5 cm thick concrete slab (flat plate). The structural system in the transverse (east-west) direction is a dual system composed of reinforced-concrete coupled shear walls in the north and south ends of the building, two smaller shear walls surrounding the elevators and flexible frames (columns and flat plate). Transverse reinforcement in the columns consists of square ties with 90° hooks at the corners. Similarly, ties in the beams are not closed, consisting of U-shaped ties with alternating caps. The amount of specified transverse reinforcement in critical regions of columns and beams exceeds minimum code requirements at the time of construction. However, by today's standards the amount of transverse reinforcement and the type of detailing would be considered as inadequate.

The building forms part of the National Strong-Motion Instrumentation Network (NSMIN) operated by the U.S Geological Survey (USGS). The building instrumentation consists of three SMA-1 analog accelerographs (each capable of recording three components of motion) located in the south end of the basement, 5th floor and 10th floor.

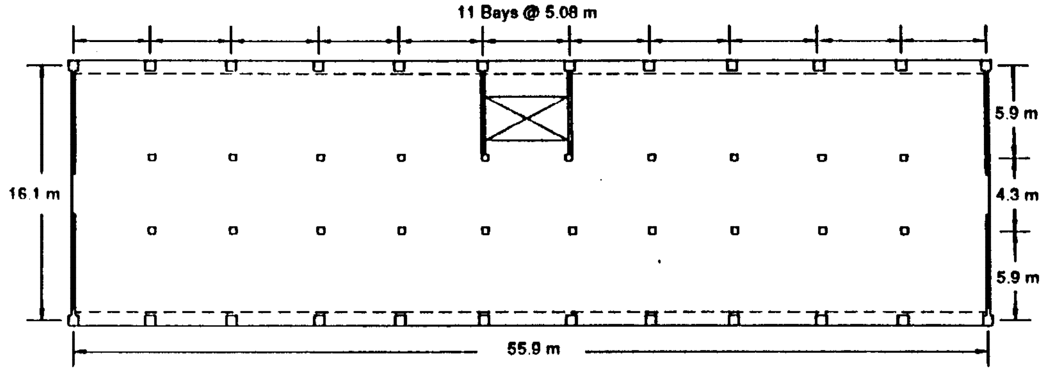


Fig. 2. Typical floor plan of the building.

The largest magnitude earthquake that has shaken the building is the October 1st, 1987 Whittier Narrows earthquake. The epicenter of this magnitude 5.9 $M_L$ earthquake was approximately 10 km (6.2 miles) north of the building. Among more than 250 strong-motions accelerograph stations (operated by the USGS, the California Strong Motion Instrumentation Program, and the University of Southern California), that were triggered in this earthquake, the largest peak ground acceleration was recorded in the basement of this building. In the transverse (east-west) direction peak accelerations of 0.63g, 0.62g, and 0.53g were recorded in the basement, 5th floor, and 10th floor, respectively. In the longitudinal (north-south) direction peak accelerations of 0.43g, 0.55g, and 0.40g were obtained in the basement, 5th floor, and 10th floor, respectively (Etheredge, and Porcella, 1987). Major damage occurred within 5 km (3 miles) of the building, including several partial collapses in the Whittier downtown shopping area (Whittier Village). A Modified Mercalli Intensity (MMI) of VII was assigned to the area where the building is located. No damage was reported in the building during.

## Results

A linear elastic model of the building was calibrated with the dynamic characteristics of the building inferred from the earthquake records obtained during the 1987 Whittier earthquakes using system identification techniques (Miranda and Bertero, 1991).

Only the longitudinal direction was used to study the efectiveness of the proposed methodology. The mathematical nonlinear model consisted of 289 nodes, 519 members and 780 degrees of freedom. The first three translational periods in this direction are 1.43 s, 0.49 s and 0.32 s. Load-deformation relations were determined by imposing assumed shapes for lateral load distributions over the height of the structure and increasing the total load monotonically from zero up to incipient collapse. For this purpose two loading patterns were used, triangular and rectangular (uniform). Fig. 3 shows the relationship between the base shear and the roof displacement corresponding to an inverted triangular load distribution. The maximum lateral strength of the building is 22% of its weight when subjected to a triangular load and the ratio between the maximum base shear and the base shear at first significant yielding is 1.38. Structural damage is initiated at a global drift index, $\gamma_G$, of 0.0038, which corresponds to a maximum interstory drift index of 0.005 occurring in the fourth story.

Fig. 3 shows a comparison between the load deformation relationship computed through nonlinear static analysis and the one assumed in the equivalent SDOF system. Fig. 4 shows the distribution of interstory drift index along the height of the building for different levels of global deformation. It can be seen that this distribution is constant while the building remains elastic (i.e., $\gamma_I < 0.005$) and changes significantly for larger displacement demands, experiencing a concentration of inelastic deformations between the third and sixth stories. The maximum deformation capacity of the building in the longitudinal direction is expected to be

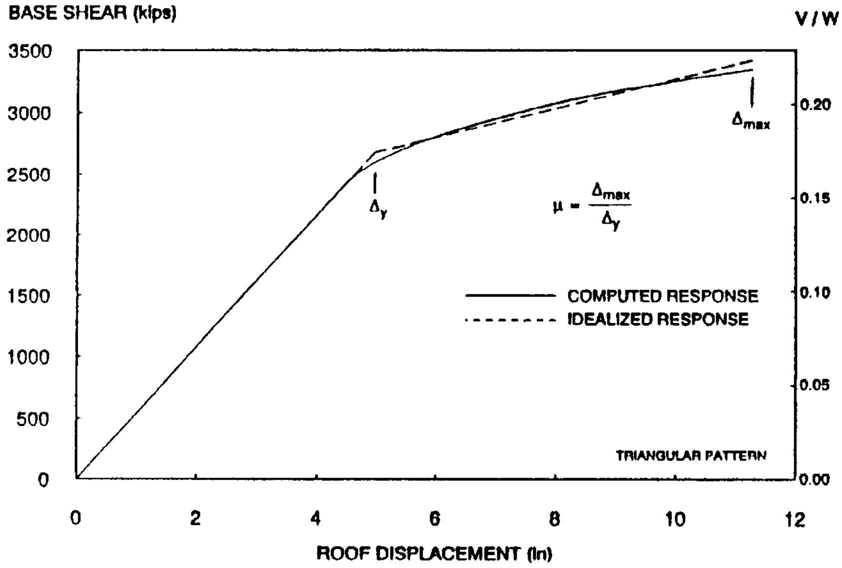


Fig. 3. Load deformation relationship of the building.

controlled by the available rotation capacity of the columns and by the shear capacity of their corresponding beam-column joints in the 3rd through 6th floors where important concentrations of inelastic deformations occur. For the maximum deformation shown in Figs. 3 and 4, rotations in excess of 0.016 rad are computed in the fourth story columns. While the amount of transverse reinforcement at the end of these columns is, in general, superior to that observed in buildings designed according to "pre-San Fernando" detailing requirements, the effectiveness of the ties to provide confinement to the concrete core for these levels of deformation is not expected to be good because of the 90° hooks at the corners of the section. It has been observed that concrete columns with this type of detailing do not exhibit good behavior, since when the cover concrete cover is lost the end of the tie leg at the 90° hook moves away from the longitudinal bar it engages, resulting in complete loss of anchorage of the tie (Park, 1990) and therefore there is a loss of confinement and restraining capacity against local buckling of the longitudinal reinforcement.

With the information shown in Fig.4 it is possible to establish a relationship between global deformation demands, $\mu_G$ and $\gamma_G$ , and local deformation demands $\mu_L$ and $\gamma_L$ . The relationship between global and local displacement ductility demands for the building is shown in Fig. 5. It can be seem that for global ductilities larger than 1.2, local inelastic deformations concentrate between the 3rd and 6th stories. When the maximum deformation capacity is reached in the fourth story, the global ductility demand is 2.0 and the local is 3.51.

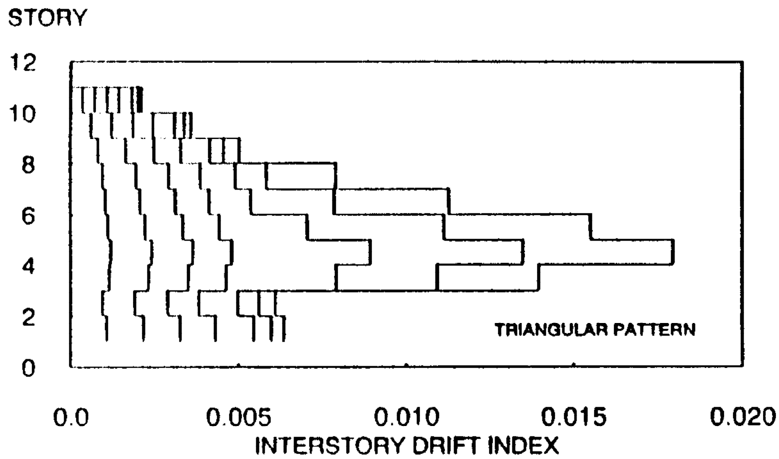


Fig. 4. Intersoty drift index profiles in the building for different levels of global deformation.

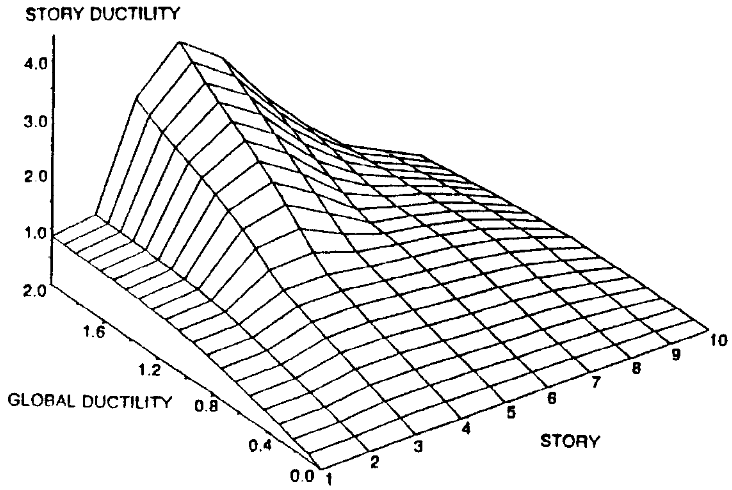


Fig. 5. **Relationship between global and local displacement ductility demands for the building.**

In order to evaluate the proposed method three ground motions were selected. The first ground motions is the base motion recorded in the 1987 Whittier, Califronia earthquake; the second ground motion is the north-south component of the Hollister record, obtained during the October 17, 1989 Loma Prieta, California earthquake ($M_S$ = 7.1); and the third ground motion is the S50W component of the James Road record, obtained during the October 15, 1979 Imperial Valley earthquake ($M_S$ = 6.5). The results obtained with the simplified methodology are compared with those of detailed nonlinear time-history analysis of the model of the building in Table 1. It can be seen that in the case of the Whittier Narrows earthquake, despite being subjected to significant peak ground accelerations, the interstory drift index demands are relavitely small, thus explaining the absence of damage. The differences between results of the time history analysis and the proposed methology are 5.8%, 5.1% and 15.8%, for the first, second and third records, respectively.

| STORY | WHITTIER RECORD | | HOLLISTER RECORD | | JAMES ROAD RECORD | |
|---|---|---|---|---|---|---|
| | SIMPLIFIED METHOD | DETAILED ANALYSIS | SIMPLIFIED METHOD | DETAILED ANALYSIS | SIMPLIFIED METHOD | DETAILED ANALYSIS |
| 10 | 0.0011 | 0.0013 | 0.0022 | 0.0030 | 0.0023 | 0.0026 |
| 9 | 0.0018 | 0.0020 | 0.0037 | 0.0046 | 0.0037 | 0.0041 |
| 8 | 0.0025 | 0.0026 | 0.0051 | 0.0055 | 0.0051 | 0.0053 |
| 7 | 0.0029 | 0.0028 | 0.0078 | 0.0089 | 0.0078 | 0.0066 |
| 6 | 0.0031 | 0.0030 | 0.0111 | 0.0118 | 0.0113 | 0.0076 |
| 5 | 0.0033 | 0.0033 | 0.0130 | 0.0131 | 0.0155 | 0.0112 |
| 4 | 0.0036 | 0.0034 | 0.0166 | 0.0158 | 0.0176 | 0.0152 |
| 3 | 0.0035 | 0.0033 | 0.0133 | 0.0132 | 0.0138 | 0.0131 |
| 2 | 0.0029 | 0.0030 | 0.0048 | 0.0048 | 0.0050 | 0.0050 |
| 1 | 0.0032 | 0.0031 | 0.0059 | 0.0062 | 0.0058 | 0.0058 |

Table 1. Comparison of results of the simplified methodology with detailed analyses.

## CONCLUSIONS

The simplified method to assess the seismic vulnerability of existing structures presented in this paper has the following advantages:

(i) The inherent relationship between seismic demands and supplies is taken into account by computing the seismic demands as a function of the lateral strength of the structure.

(ii) Inelastic behavior is explicitly considered in the estimation of both deformation demands and deformation capacities of the structure.
(iii) It produces good estimates of both global and local inelastic deformation demands. Thus, identifying the location and severity of the structural deficiencies in the existing structure.
(iv) Since the computational effort involved in the simplified method is only a small fraction of that involved in nonlinear time-history analyses of the whole structure, it allows the consideration of a larger number of input motions to the structure. Thus, allowing to spend more time in studying the vulnerability of the structure considering the large uncertainties on the characteristics of future earthquake ground motions.
(v) Using this simplified method the seismic vulnerability of the structure can be easily checked using dual-level criteria in which the performance of the building is computed for a serviceability-level earthquake and for a safety-level earthquake.

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