SEISMIC RESPONSE ASSESSMENT FOR A MASONRY HIGH-RISE BUILDING

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SUMMARY

In seismic regions, high-rise buildings utilizing load-bearing walls composed of reinforced concrete-masonry are becoming commonplace. Mathematical procedures are available for evaluating seismic performance. Verification and refinement of these procedures is provided by comparison of the mathematically predicted and experimentally observed behavior of an existing building. The mathematical procedure is based on common structural analysis software. Biaxial stress criteria are used to evaluate the likelihood of structural damage due to several probable classes of ground motion. The analysis procedure proves to be both accurate and uncomplicated in application.

INTRODUCTION

Masonry has experienced widespread use as a structural material for low-rise building construction and, in regions of low seismic activity, for high-rise structures (> 6 stories). Only recently have buildings utilizing load-bearing reinforced masonry gained additional popularity in regions prone to significant earthquake occurrences. As a result of this expanded utilization of structural masonry, it becomes necessary to develop accurate analysis techniques for seismic loading conditions.

While other building materials, principally steel and reinforced concrete, have been the subject of extensive earthquake research resulting in improved analysis and design methods, masonry has not benefitted from similar attention, especially in the area of experimental investigations. This deficiency is particularly apparent for experiments involving full- or large-scale specimens and complete building systems.

The objective of this study is to provide a comparison between the observed seismic response of an existing high-rise masonry structure, utilizing load-bearing reinforced concrete-masonry as the principal structural element, and the response predicted by analytical techniques. In this way, a reliable and practical procedure can be developed for analysis of structures with similar configurations.

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ANALYSIS PROCEDURE

When masonry structures are carefully designed and are constructed with impeccable workmanship, some degree of ductility can be provided. If deficiencies exist, however, the structure may provide little reserve capacity beyond the point when initial structural damage occurs. In consequence, initial structural damage can be conservatively chosen to be synonymous with initial cracking of structural wall elements, provided that other structural components, such as connections and floor diaphragms, are deemed adequate in strength. Up to the point when initial cracking occurs in wall elements, it is reasonable to assume that the structure will exhibit linear force - deformation response. Thus, the chosen method of structural analysis need only model elastic behavior.

Computer Modeling

The linear-elastic models are developed in two phases using the computer programs ETABS and SUBWALL (Refs. 1 and 2). The analysis procedure begins with coarse modeling of the entire structure using ETABS, a linear analysis program for building systems (Figure 1). The modal behavior is compared with the experimentally observed properties and basic system identification concepts are used to refine the model prior to further analysis. Once the final model has been developed, the global force and displacement response of the building are generated and critical wall elements are identified. The wall elements are then modeled in detail using SUBWALL, a two-dimensional finite element program. The global displacements are applied to the wall model, permitting identification of the local stress field.

Damage Criteria

A comprehensive study at the University of California, San Diego, determined that biaxial failure criteria similar to those used for concrete are appropriate for the analysis of masonry cracking (Ref. 3). In this study, the failure surface of masonry was determined in the principal-stress space (Figure 2). The primary difference between the concrete and masonry failure surfaces is that the ratio of tensile to compressive strength is 0.05 for masonry, as compared with 0.10 for concrete.

Although masonry is an anisotropic material, the envelope was found to be relatively independent of the angle of the principal stresses with respect to the bed-joint. It is also not significantly affected by the amount of reinforcing steel for the small steel/masonry ratios typically found in masonry construction. Thus, the biaxial failure criteria for masonry are relatively straight-forward in application.

CASE STUDY

Since one of the objectives of this study is to provide a comparison between predicted and observed dynamic response, it was necessary to search for a study candidate with available records of response to seismic loading. Ideally, data for the selected building would be from seismic events of sufficient intensity to generate moderate amplitude response in the structure. This is often a difficult obstacle for experimental studies of full-scale and

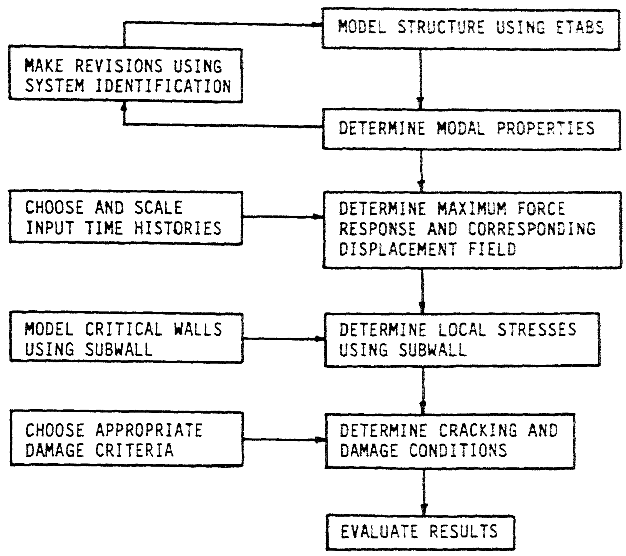

Figure 1: Analysis Procedure

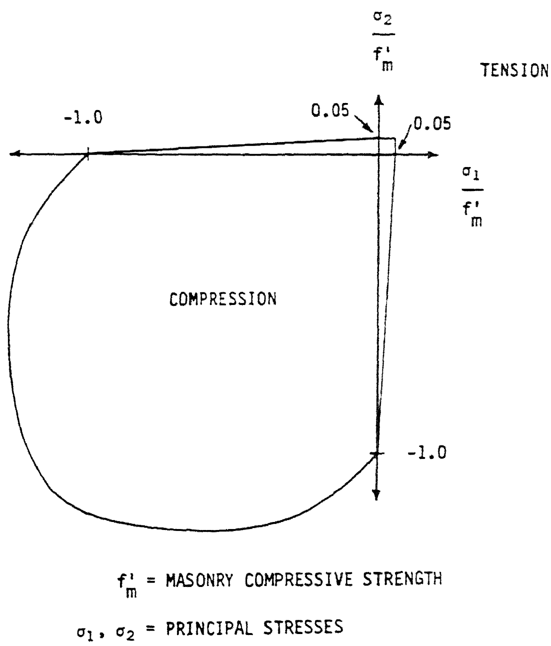

Figure 2: Biaxial Failure Criteria

complete structural systems.

In this case, an existing building located in Las Vegas, Nevada, was chosen for study. Buildings in Las Vegas have been subjected to dynamic ground motion produced by underground nuclear explosions (UNEs) at the Nevada Test Site, located approximately 160 km from the city. Seismic loading from UNE sources exhibits many similarities to earthquake-induced motion and is occasionally of sufficient magnitude to produce significant response of tall buildings in the area. Furthermore, since ground motion from UNEs occurs at predetermined times, building and ground stations can be successfully instrumented to capture the ground motion and building response.

Building Description

The building chosen for this investigation is a 10-story structure with rectangular plan dimensions of 72 m x 16 m (Figure 3). The building was designed according to the provisions of the 1970 edition of the Uniform Building Code (Ref. 4) for Seismic Zone 2, with base shears of 5 % and 4 % of the building weight in the longitudinal and transverse directions, respectively. Design calculations indicated a fundamental longitudinal period of 0.28 sec and a transverse period of 0.58 sec.

The lateral load supporting system consists of 200- and 300-mm-thick load-bearing walls formed from reinforced concrete-masonry units. Multiple walls provide resistance to transverse loading but in the longitudinal direction only a single 300-mm-wall, with a length of 7 m, and two walls of lesser length forming the elevator core act as primary lateral elements. Additional non-structural walls were not considered by the designer to provide any load resistance for longitudinal forces. Floor shears are distributed to the walls through a cast-in-place post-tensioned concrete slab with a thickness of 150 mm.

Since only three structural walls act in the longitudinal direction these walls can be expected to be more highly stressed than the transverse walls. Consequently, this text will only address the computer models that were developed for loading along the long building axis.

Loading Conditions

Response records have been obtained from the study building for a number of UNEs. Corresponding records also exist of free-field motion recorded near the building. Records from one of these events were chosen as a benchmark on the basis of the clarity of the recorded time-histories and the moderate level of response that was generated in the structure. The selected free-field ground motion is illustrated by the corresponding response spectrum in Figure 4.

Though Las Vegas is not situated in an area of high seismicity, the region is prone to motion from natural seismic events (Ref. 5). Considering the probable characteristics of ground motion in Las Vegas based on a 10 percent chance of exceedence in 100 years, a local source can be expected to produce a peak ground acceleration (PGA) of 0.10 g at a predominate frequency of 5 Hz, with a strong-motion duration of approximately 1 second. An earthquake

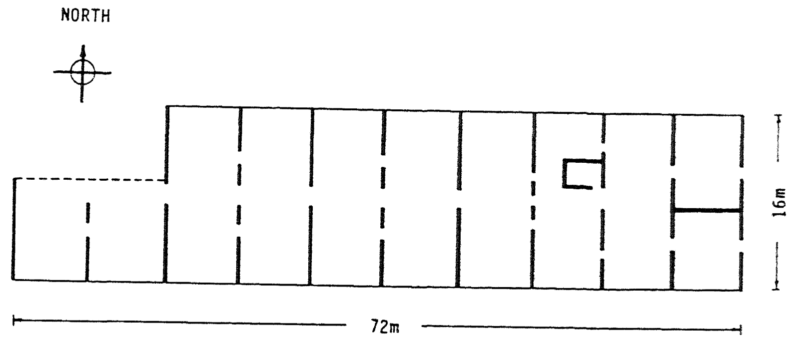

Figure 3: Typical Building Floor Plan

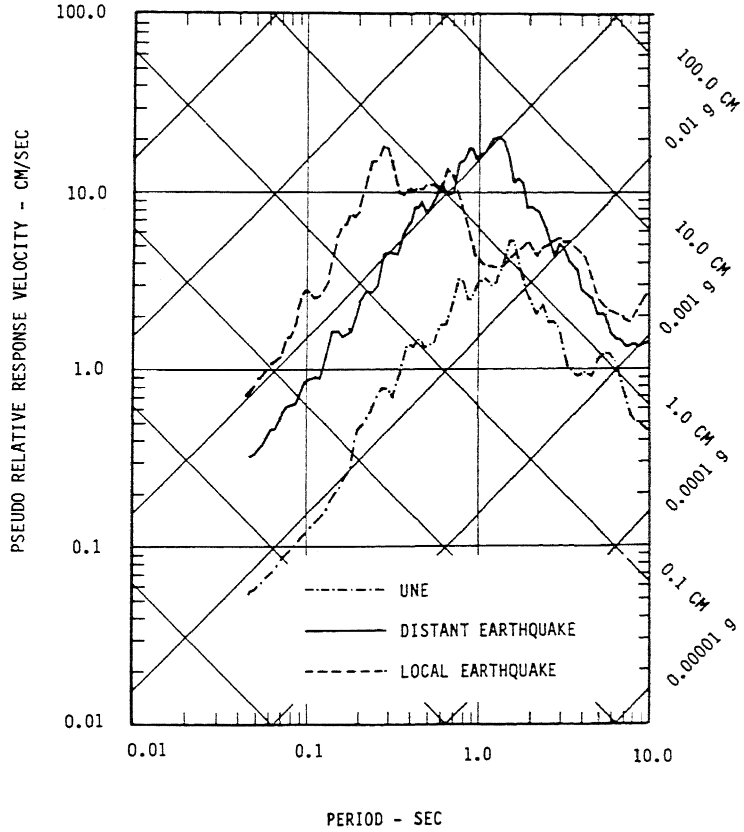

Figure 4: Response Spectra of Input Motion

originating at a distant source with the same level of probability would produce 0.04 g peak acceleration with a duration of 40 seconds and 2 Hz being predominate.

These levels of ground motion have the potential for producing building response of sufficient amplitude to reach the damage criteria. Consequently, once the UNE - induced motion has been used to refine the computer model, reliable estimates of stress demands from natural seismic sources can be made. Two records from historic earthquakes were selected as being representative of the expected ground motion; the San Francisco State Building record from the San Francisco, 1957, earthquake (M = 5.3, PGA = 0.09 g, 4.0 Hz predominate) and the Las Vegas SE-6 record from the Hiko, Nevada, 1971 earthquake (Mr = 4.8, PGA = 0.008 g, 2.0 Hz predominate). These records from the San Francisco and Hiko earthquakes were scaled in amplitude to match the expected amplitude of motion in Las Vegas from local and distant sources, respectively (Figure 4).

Model Development

The mathematical models of the study building were developed using standard ETABS procedure. The modulus of elasticity for the structural walls was determined from the commonly accepted equation, $E = 1000 \times f_m$, where f_m is the masonry s ultimate compressive strength. An E value of 15.2 GPa was thus defined. A Poisson s ratio of 0.25 was assumed in order to define the shear modulus.

The model obtained by considering only structural walls exhibited a fundamental period of 0.93 s versus a measured value of 0.66 s. Non-structural elements were judged responsible for this discrepancy, however, the stiffness contribution of these elements is normally difficult to define. Since actual records of response were available for the building, the stiffness of the non-load-bearing walls, consisting primarily of gypsum, could be obtained by considering the fundamental relationship, Frequency = $[\text{Stiffness} / \text{Mass}]^{1/2}$. In this way, an E value of 1.27 GPa was determined as appropriate for the non-structural walls. Although this value is less than 10 percent of the modulus for the structural walls, these elements contribute significantly to lateral stiffness as a result of their large number.

The damping of the structural system was assumed to be modal. Once again, system identification techniques were used to define a modal damping ratio of 5 percent.

Results of Computer Analysis

The results of the ETABS analysis indicate that the primary 7-m-long wall is the most critically stressed for all three sources of seismic excitation. The displacement field which produces this condition is generated primarily by response at the fundamental mode. The stress, which is produced by large values of overturning moment and shear force, is most severe at the lowest building level, as might be expected for first-mode response.

Three sources of loading contribute to the wall stress; shear force, overturning moment, and vertical loads. The shear force results in a nearly uniform shear stress over the wall cross-section. The moment causes vertical tension on one side of the wall and vertical compression on the opposite side. Vertical loads produce a uniform vertical compressive stress which, when combined with the moment stress, results in a reduced value of tensile stress and an increased value of compressive stress. Since the biaxial stress criteria indicate a masonry compressive strength twenty times the tensile strength, the vertical loads prove to have a beneficial effect on seismic performance.

The critical values of stress produced by the three seismic input motions are summarized in Table 1. For the UNE, the wall compression produced by vertical loads tributary to the wall was sufficient to offset the tensile stress developed by the overturning effect. This was not the case for the two earthquake loadings, which generate tensile stresses exceeding the values corresponding to the damage criteria.

Table 1: Analysis Results - Ground Floor

Input Motion	Wall Location	Principal Stress Ratios		Factor Above Damage Criteria
UNE	E End	.00	-.03	-
	W End	.01	-.12	-
Local Earthquake	E End	.31	-.02	6.2
	W End	.01	-.37	-
Distant Earthquake	E End	.23	-.03	4.6
	W End	.01	-.31	-

CONCLUSIONS

The two-stage analysis procedure, based on coarse structural modeling to generate global response parameters followed by detailed modeling of critical subassemblages, was found to be an effective and efficient approach. For general analysis purposes, the procedure would be slightly modified from that discussed here. The primary change for practical application would result from deletion of the system identification step. In order to compare the analytical results to experimental data it was necessary to apply a time-history approach to analysis. In practice, a response spectrum approach would prove more efficient and nearly as accurate.

For the study building, both earthquake sources produce wall stresses exceeding the damage criteria. This does not necessarily imply that the structure is likely to suffer catastrophic collapse. Several factors will contribute to the ability of the building to support seismic loads over its useful life. One factor is that the adopted damage criteria are conservative and the structure can be expected to exhibit some reserve capacity. The second important point is that the earthquakes used in the analysis correspond to a low level of probability, 10 percent in a 100-year-period. A more probable earthquake occurrence would be much smaller in amplitude and would result in a corresponding decrease in response amplitude.

The primary factor contributing to the high stress in the study building is the use of one long wall as the primary lateral force resisting element in the longitudinal direction. The length of the wall contributes to a large component of tensile stress due to bending without an offsetting compressive stress due to tributary vertical loads. Increasing the length of this wall would not necessarily improve performance, since the vertical loads would not experience a proportional increase. Excessively long walls may also lead to crushing of the masonry at the toe of the wall due to high compressive stress produced by a combination of bending and vertical loads. As a result, multiple walls of moderate length exhibit superior seismic characteristics as compared to a few walls of great length.

ACKNOWLEDGEMENTS

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