ANALYSIS OF THE SEISMIC COLLAPSE CAPACITY OF UNREINFORCED MASONRY WALL STRUCTURES

BY

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Summary

Buildings constructed of unreinforced masonry are normally regarded as brittle and susceptable to severe damage from earthquakes. The purpose of this paper is to present the reserve energy method applied to buildings of this type and show the seismic lollapse capacity calculated by this relatively simple equivalent nonlinear method is consistent with observation as well as more rigorous analytical techniques.

Introduction

Historically, unreinforced masonry construction has been considered as one of the types of construction most vulnerable to seismic damage and collapse. This is evident from an evaluation of the structural failures resulting from nearly any actual earthquake. In fact, failures of masonry walls with varying levels of workmanship constitute one of the principal effects described in the Modified Mercallii and other seismic intensity scales. Despite this, many buildings have been and continue to be constructed in this manner, even in zones of moderate to relatively high seismic risk throughout the world. Even with very severe ground motion, unreinforced masonry bearing wall structures rarely collapse as long as the floors remain connected to and supported by the walls. When collapse occurs, the failures are essentially in-place, vertical collapse with no evidence of even multistory strucutures moving laterally by significant amounts. Rather, the horizontal ground motion causes the bearing walls to crack and rotate at the base which, along with sliding of the roof structure after successive cycles, leads to loss of vertical support and collapse. Even with the wide use of digital computers, the rigorous analysis of this highly nonlinear response is time consuming and expensive. The use of the reserve energy method for systems involving rigid body rocking with little hysteretic effect provides a simple and reasonably accurate means of assessing the dynamic response characteristics of unreinforced wall-roof structures. In this paper, the reserve energy method is applied to a typical building constructed of unreinforced masonry walls. The results are compared with observations of similar structures from several past earthquakes and also with those from a time history, nonlinear response analysis.

Structural Behavior

At very low levels of seismic excitation, the response of most masonry structures is essentially elastic. As the response of the structure increases, however, effects such as sliding of the roof and cracking and sliding of walls begin to occur which introduce significant nonlinear effects into the system. Figure 1 shows a typical deformation pattern which often occurs for unreinforced masonry construction of one or one-and-one-half-story construction. Even for multistory structures the non-linear effects are normally confined to the lower elevations so that single-degree-of-freedom analysis methods will often provide sufficiently accurate results. As shown in Figure 1, cracking normally occurs at the base of the walls followed by rigid body rocking of the wall-roof system as an inverted pendulum. Often the details of the roof attachment method will provide some stabilizing effect as a result of the shift in bearing location due to rocking even though no means of positive retention are employed. Once the deflection reaches a point at which the dead weight restoring forces become zero, any additional deformation results in the wall and roof dead weights contributing to the overturning moment. Instability and essentially in-place vertical collapse are then imminent.

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At the initiation of cracking at the base or other location in a wall, the total cracking capacity is determined by adding the compressive dead weight and vertical seismic stresses to the flexural Modulus of Rupture (MOR). The direct and bending stresses are calculated on the net horizontal section through the mortar. Although there is normally a fairly large uncertainty associated with both the strength and effective area of the mortar, the energy associated with the initial crack formation is not usually singnificant compared to the overall energy associated with collapse capacity. This is illustrated in Figure 2 which shows the nonlinear force-deformation relation for a typical masonry wall. The amount of energy associated with mortar cracking during the first cycle is seen to be small and the force-deflection relation after cracking is very close to that of the initial cycle.

Reserve Energy Method

For rigid body rocking dynamic systems which exhibit little hysteretic energy dissipation such as described above, the Reserve Energy (RE) technique (Ref. 1) provides a tractable method of accounting for the nonlinear response. The RE method consists of analyzing an equivalent elastic system by the modal spectral method of analysis. The equivalency is maintained by equal energy capacity of the nonlinear system and its equivalent elastic system. This equivalence is obtained as shown in Figure 2 by comparing the nonlinear and linear diagrams. Failure of the linear system thus occurs at the force F_{RB} and deflection δ_{RB} which corresponds to the force and deflection capacity of the nonlinear wall-roof system. The energy equivalence is shown in Figure 3 together with the equivalent linear and nonlinear models.

The natural frequency of the equivalent system is calculated based on the stiffness and mass of the equivalent SDOF linear model. The stiffness K_e is as shown in Figures 2 and 3. The equivalent mass of the single-degree-of-freedom system is based on the sum of the contributing roof weight and a fraction of the distributed wall weight. The spectral acceleration capacity S_{ac} of the wall-roof system is simply the ratio of the maximum inertial foree capacity (F_{RB}) to the effective weight which contributes to the applied inertia forces (W_c). The capacity of the system in terms of peak ground acceleration, a_{gc}, is obtained utilizing the response spectrum corresponding to the base motion. With the natural frequency of the equivalent linear system known, the spectral amplification factor, AF, can then be obtained. Using the spectral acceleration capacity and the spectral amplification factor, the peak ground acceleration capacity may be determined.

Confirmation of the Reserve Energy Approach

The confirmation of the RE method of analysis of the collapse capacity of wall/roof transverse systems was obtained in two ways. First, a review of the data from past earthquakes to determine collapse capacities of similar structures was conducted (Refs. 2,3 and 4). Percentages of unreinforced buildings collapsed or subsequently torn down were established. Particular emphasis was devoted to Long Beach, Santa Barbara, Kern County, and San Fernando Earthquakes since more accurate ground acceleration levels were available than for many of the older earthquakes. A qualitative estimate of median collapse levels for unreinforced masonry buildings from these data is in the range of 0.15g to 0.4g. This is consistent with the Modified Mercalli Intensity of VIII which indicates damage and partial collapse for "Masonry C" buildings in the range of 0.15g and 0.3g.

Analyses of a series of unreinforced masonry structures expected to fail in the rigid body rocking mode were then conducted (Ref.2) using the RE method. Calculated collapse capacities in the range of 0.21g to 0.38g were typical, and correlated well with observed behavior of similar structures to actual earthquakes.

A second confirmation of the approximate analysis procedure was provided by a series of nonlinear time history verification analyses. In order to demonstrate that the RE method of analyses provides a good estimate of wall/roof system capacities considering rigid body response modes, verification analyses were made for a simple but representative wall/roof system. The RE analysis method was verified by showing that the capacity of a representative system obtained using the RE method of analysis compares well with the capacity of the system as determined independently using nonlinear time history analysis. The effect of concurrent vertical motion was also investigated using uncorrelated artificial earthquake time history input motions.

The representative simplified system selected was a single wall supporting a uniform roof load. The 12 inch hollow block wall with corresponding wall and roof weights is shown in Figure 4. Although the actual seismic capacity in transverse shaking of the structures was determined utilizing more complex models involving an entire structural system comprised of several walls and the roof structure, the single wall was considered to provide a representative check of the method. Actual systems consisting of more than one wall have higher seismic capacities.

The system was first analyzed by means of the reserve energy method. Results of the RE method indicate collapse at .09g peak ground acceleration. The equivalent rigid body system has a maximum force F_{RB} capacity of 47.8 lbs/in and a peak top displacement, δ_{RB} (at the point of instability) of 7.9 inches. The same system was then analyzed using a nonlinear time history method. The solutions for horizontal ground motion with no vertical input are shown in Figure 4. Stable response was obtained for input scaled to 0.09g or less peak acceleration. Unstable response was obtained for 0.095–0.110g motion with a stable response occuring for 0.115g scaling. Unstable response was obtained for 0.120g or greater peak acceleration. Thus, the 0.09g capacity estimate provided by the RE method is verified by the time history analysis. For ground motions exceeding 0.09g, the peak displacement was found to increase very rapidly and the response was found to be highly sensitive to exact phasing of the pulses of the applied time history.

The effects of concurrent vertical ground motion on the stable 0.09g peak horizontal response case were also determined for ± 0.06g and ± 0.14 g peak vertical ground motion. While the general character of the response was somewhat changed by the concurrent vertical motion, the stability of the rigid system was not affected.

Conclusions

The displacements from nonlinear analyses for the scaled ground motion as predicted from the RE method (0.09g) are below collapse values and the wall is stable. However, with a very small increase in ground acceleration (less than 5 percent increase) beyond the capacity predicted by the RE method, the displacements increase radically and response, in general, is a function of ground motion phasing as the system nears instability. Thus for this simple system the RE method is shown to predict a collapse capacity very close to the capacity predicted by time history analysis. Though the RE method is based on systems being dynamically equivalent when the energies available are equal in the inelastic and equivalent elastic systems, there is no assurance that this equivalence holds for all types of inelastic systems. However, the verification analysis results as discussed above provides a strong indication that the equivalence does hold for the linear degrading models which represent the rigid body systems considered herein. The reserve energy method is therefore believed to provide a simple but valid method of determining collapse capacities of unreinforced masonry structures which fail by rigid body rockig.

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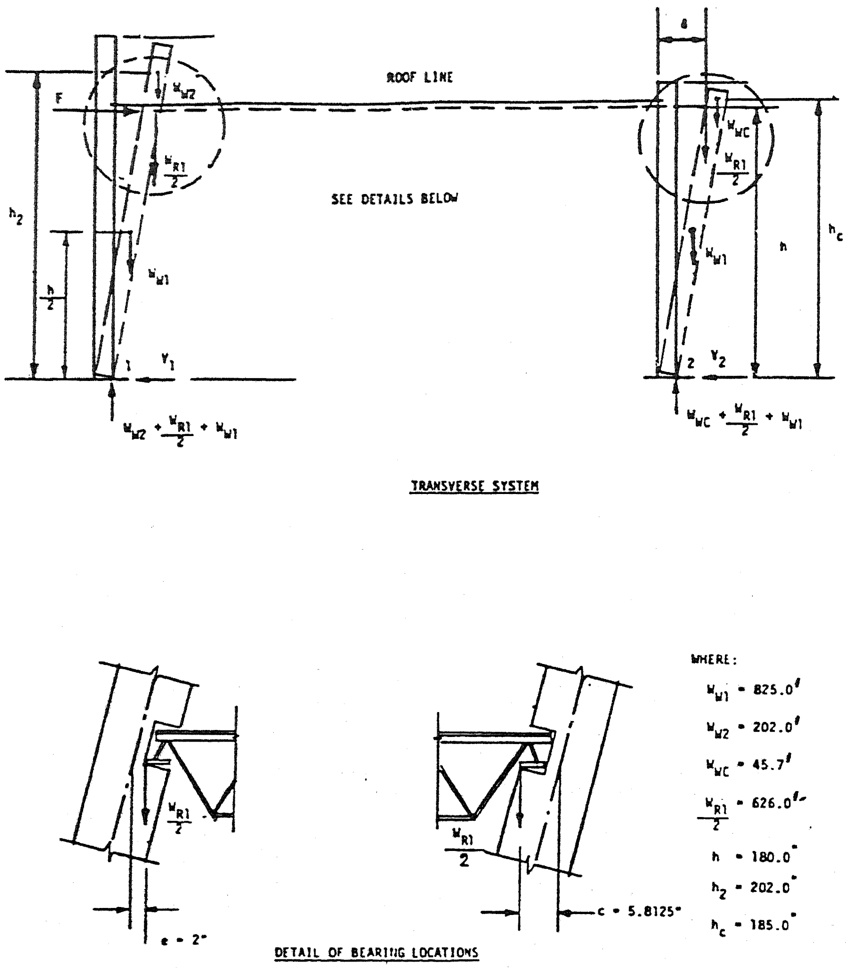
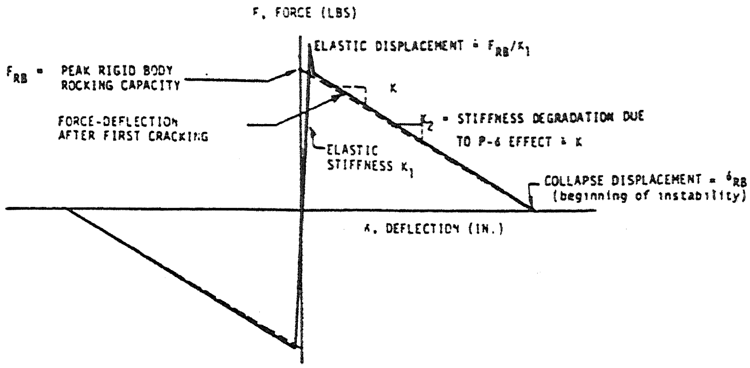
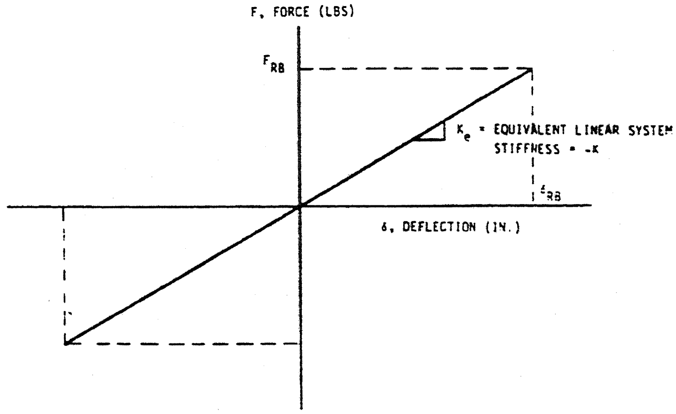


FIGURE 1. TYPICAL UNREINFORCED WALL AND ROOF SYSTEM SEISMIC RESPONSE MODE

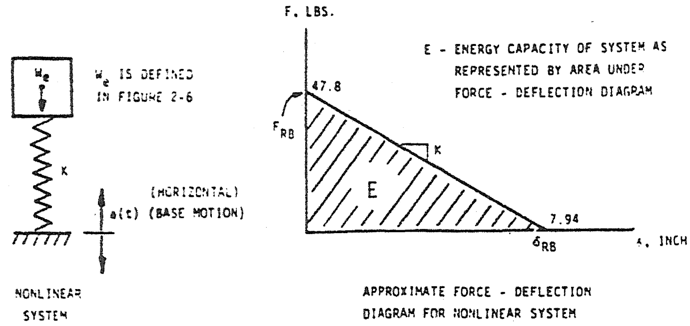


NONLINEAR FORCE-DEFLECTION RELATION



FORCE-DEFLECTION RELATION OF EQUIVALENT LINEAR SYSTEM

FIGURE 2. NONLINEAR AND EQUIVALENT LINEAR FORCE-DISPLACEMENT RELATIONSHIPS



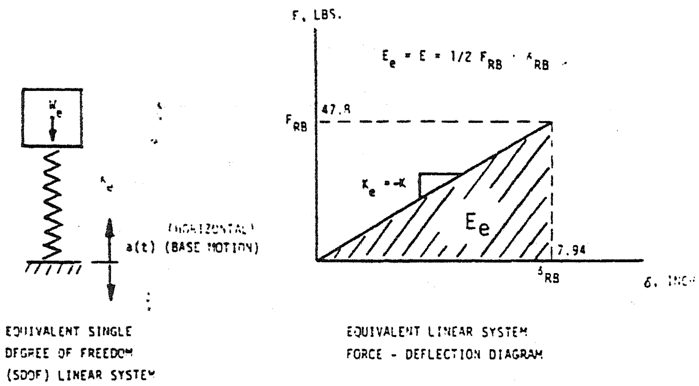
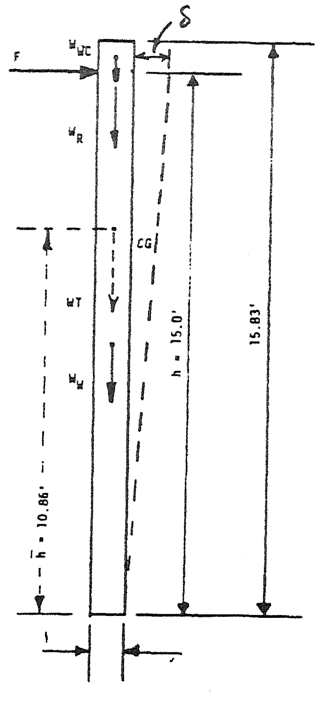


FIGURE 3. WALL/ROOF SYSTEM MODELS



F — LATERAL FORCE APPLIED AT ROOF LINE

W_{WC} — WEIGHT OF THE WALL ABOVE ROOF LINE
= 0.83' x 55 #/1' = 46#

W_W — WEIGHT OF THE WALL BELOW THE ROOF LINE
= 15' x 55 #/1' = 825#

W_R — TRIBUTARY ROOF WEIGHT
= 43.2 #/1' x 29'/2 = 626#

WT — $W_{WC} + W_W + W_R$ = 1497#

W_e — WEIGHT CONTRIBUTING TO INERTIA FORCES
= 1497 x 10.86/15 = 1084#

FIGURE 4. REPRESENTATIVE RIGID BODY SYSTEM FOR VERIFICATION ANALYSES

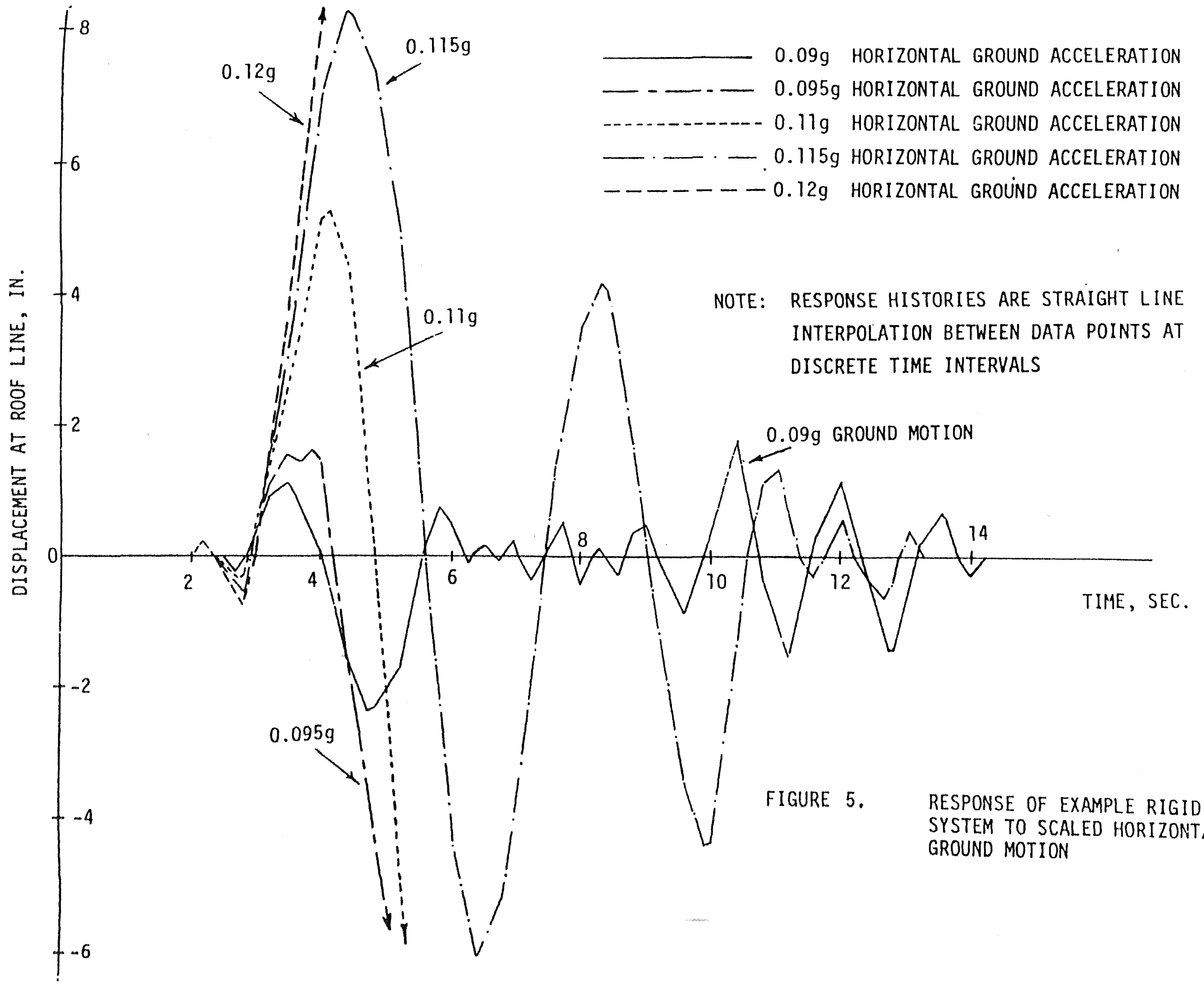


FIGURE 5. RESPONSE OF EXAMPLE RIGID BODY SYSTEM TO SCALED HORIZONTAL GROUND MOTION