

Response of an unreinforced masonry building during the Loma Prieta earthquake

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ABSTRACT: A two-story unreinforced masonry structure located within 15 km of the epicenter of Loma Prieta responded with little damage despite roof accelerations as large as 0.79g. The historical building was constructed of clay brick masonry bearing walls, and timber floor and roof systems. Acceleration records are available on ground motions in three directions as well as on lateral motion of the roof. This paper addresses dynamic response and damage of the structure, and the applicability of various analytical models for estimating response.

1 INTRODUCTION

On October 17, 1989, a magnitude 7.1 earthquake occurred at a distance of 100 km south of San Francisco. The epicenter was at Loma Prieta, California. Significant damage of masonry buildings occurred in nearby Santa Cruz, Watsonville and Los Gatos, and as far north as Oakland. Entire city blocks were demolished in Santa Cruz because masonry buildings crumbled.

Gilroy California is approximately 15 km south east of Loma Prieta. A town hall in Gilroy constructed of masonry was severely damaged by the shaking. Two blocks north of the town hall stands another unreinforced masonry building that withstood the earthquake with little damage. The structure is a two-story historic building (Fig. 1) that



Figure 1. Southeast view of firehouse.

has served as a firehouse since the 1890's up until a few years ago. At the time of the earthquake, it was vacant

and undergoing rehabilitation for a change of building function. Before the earthquake, it had been instrumented with six accelerometers.

Since there were no seismic codes at the time of construction, the firehouse was not necessarily designed to resist sizeable earthquake forces. However, the building withstood the 1906 earthquake. Today, there are no indications that the building required repair in 1906.

Methods used to construct the historic firehouse were similar to those used throughout America at the turn of the century. Multiwythe, unreinforced brick walls formed the building envelope, and were joined by timber floor and roof systems. Door and window openings were placed in the masonry walls where ever they were needed without concern for symmetry or structural function.

In such a structural system, lateral inertial forces are transferred to the rather stiff, in-plane walls via floor or roof diaphragms that are relatively flexible. Walls bending normal to their plane are distorted from differential movements of upper and lower diaphragms, as well as from lateral inertial forces across a story height. Dynamic response of such a structural system cannot be characterized with the same numerical models that are commonly used for modern multistory construction. Because story masses are not concentrated at floor levels, diaphragms are not stiff, and large eccentricities of lateral force may exist about the center of mass, modes of vibration are much more complex than for conventional lumped-mass stick models.

Because moderate earthquakes are expected east of the Rocky Mountains, response of the firehouse can help foretell the earthquake hazard in the eastern and midwestern United States. Though the Loma Prieta Earthquake was considered to be moderate in intensity, an equivalent motion in the eastern U.S. would be considered strong. For example, if an earthquake at New Madrid Missouri were to occur some time within the next 250 years, there would only be a 10% probability that ground accelerations measured at Gilroy (maximum of 0.29g) would be ex-

ceeded (NEHRP, 1988). Thus, the Gilroy ground motion represents an upper bound for assessing possible hazards associated with similarly constructed buildings in the eastern United States.

Since the firehouse did not damage appreciably, even with these high accelerations, there is hope that similar historic buildings across the nation may survive a future earthquake. However, such an extrapolation is not warranted unless a detailed investigation is done to examine reasons for the superior performance. This paper provides a summary of such a study. Descriptions of the structure and the base motions are presented which are followed by a discussion of the measured building response, and the computational models used to study it.

2 DESCRIPTION OF STRUCTURE

2.1 Overall layout

The building was originally intended to be a firehouse. The first story was designed so that trucks could enter the building and park within it, and thus was open on the front (Fig. 2). The second story was intended for resi-

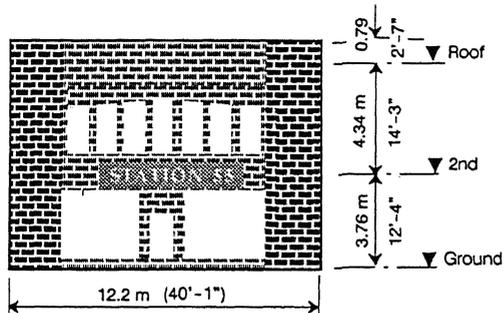


Figure 2. South elevation.

dence space, and has a number of windows on the street side and the east side (Fig. 3).

The building was originally constructed as a box consisting of four exterior brick walls, and timber diaphragms at an intermediate floor level and at the roof (Fig. 4). An addition was constructed on the back of the building by constructing a third east-west wall and extending the two north-south walls.

The west wall, the interior wall and the north wall are nearly solid. The center of in-plane wall stiffness is therefore offset from the center of mass, and a potential for torsion of the structural system exists.

2.2 Masonry bearing wall construction

Bricks were typically placed in three wythes for a total wall thickness of 305mm (12"). The two walls adjoining the garage door at the front were nominally four-wythe walls with a thickness of 432mm (17"). Bricks were placed in running bond. Mortar bed joints were approximately

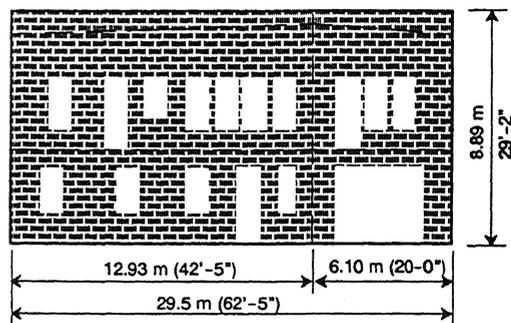


Figure 3. East elevation.

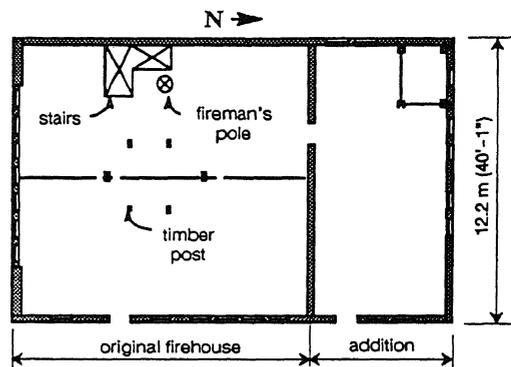


Figure 4. Plan view of second floor.

13mm (0.50 in.) thick. Brick headers were placed at every seven courses.

A few of the bricks were removed from the building and taken to a laboratory for testing. Standard ASTM tests were run to determine density, absorption, modulus of rupture and compressive strength. Measured properties are listed in Table 1.

Table 1. Measured brick properties.

density	17.5 kN/m ³ (112 lb/ft ³)
absorption	13.8%
initial rate of absorption	65.3 gms/min
saturation coefficient	0.74
modulus of rupture	1.79 MPa (260 psi)
compressive strength	37.0 MPa (5360 psi)

Petrographic, x-ray diffraction and chemical analyses were done on a few samples of the mortar. Results suggested that the mortar consisted of some cement, a hydrated lime and natural siliceous sand.

Two-unit high brick prisms were made using actual reclaimed half brick units and a surrogate mortar whose ingredients were based on the chemical analyses. The average compressive strength of the prisms was 9.1 MPa (1325 psi). The average elastic modulus was 389 times the prism compressive strength.

In-place shear tests were run throughout the building by a local materials testing firm. This nondestructive test consists of placing a small hydraulic jack in a cavity that is left by removing a single brick. The opposite head joint is also removed, so that a single test brick can be sheared relative to its upper and lower bed joint as well as any possible collar joint. Vertical compressive stress is estimated at the location of an in-place shear test, and is used to deduce a corrected value of shear strength for zero normal stress. The 20th percentile of reduced shear strength values for the 16 tests was equal to 0.59 MPa (85psi). The condition of the collar joints was uncertain based on visual observations made after extraction of these bricks, and was not assumed effective in resisting shear during the in-place shear test.

2.3 Floor and roof diaphragms

Floor and roof systems consist of 12 mm (1/2") thick plywood sheets that are nailed to and through 1" x 4" diagonal sheathing to timber floor beams that are nominally 2" x 14" @ 16" on center. The first floor south diaphragm has 16mm (5/8") thick plywood. The ceiling of the second story is 1.02 m (40") below the roof level.

Timber floor beams are well anchored to masonry bearing walls with steel tie rods. In general, the building was constructed well with adequate attention given to connection details for tying walls with floor and roof diaphragms.

3 BASE MOTIONS

3.1 Measured acceleration histories

Accelerometers were placed in each orthogonal direction on the ground immediately adjacent to the north-east corner of the building. Histories of the recorded ground accelerations for the first 20 seconds are shown in Fig. 5. Peak base accelerations were 0.24g, 0.14g and 0.29g for the north-south, vertical and east-west directions, respectively.

It is interesting to note that ground accelerations in the east-west direction included a single pulse with a fairly long period of approximately 1.5 seconds near the 4.0 second point. Peak response of the structure in the east-west direction was associated with this pulse.

3.2 Linear response spectra

Spectral response curves representing ground motions measured in the east-west direction are shown in Fig. 6 for various damping percentages. The set of spectral response curves indicate that a single-degree-of-freedom oscillator with a period of approximately 0.42 seconds would amplify the base motions more than any other. As-

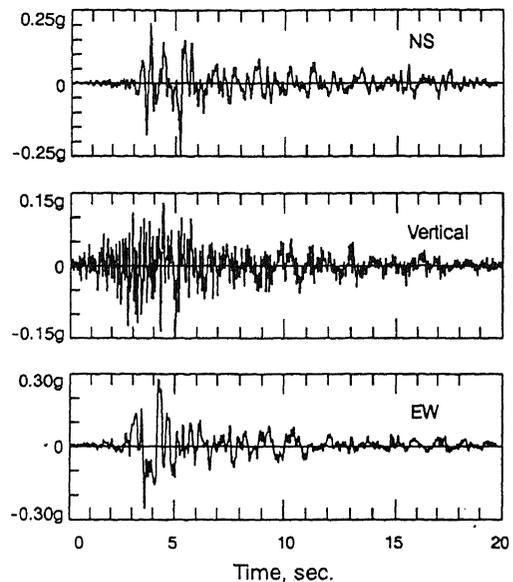


Figure 5. Recorded ground accelerations.

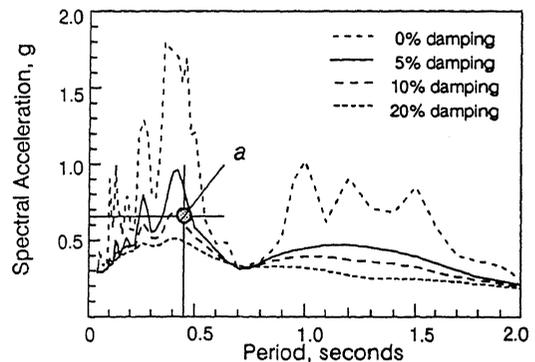


Figure 6. Response spectra for east-west motion.

suming 5% damping, peak ground acceleration would be amplified by a factor of 3.3 for this oscillator.

4 MEASURED BUILDING RESPONSE

4.1 Measured acceleration histories

Horizontal accelerations were measured at three locations on the roof. One accelerometer measured east-west motions at the top of the interior wall (sensor 4). Two others were placed at the center of the long diaphragm span and measured motions in each horizontal direction (east-west, sensor 5 and north-south, sensor 6). Measured re-

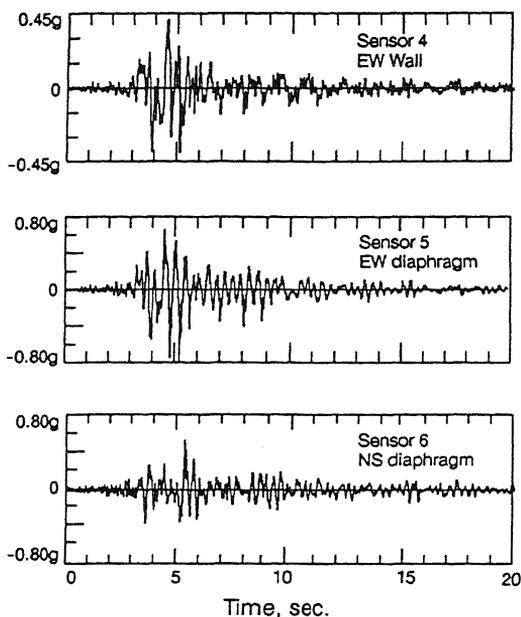


Figure 7. Recorded roof accelerations.

records are shown in Fig. 7.

Peak acceleration response was 0.41g at the top of the interior east-west wall, and 0.79g at the center of the diaphragm resulting in amplification factors of ground motions equal to 1.4 and 2.7 respectively. Accelerations as large as 0.55g were measured in the north-south direction at the center of the diaphragm indicating an amplification factor of 1.9.

The sequence and frequency content of the east-west wall motion (sensor 4 in Fig. 7) was remarkably similar to that of the ground (third waveform in Fig. 5) suggesting that the wall acting parallel to its plane was quite stiff. The sequence and frequency content of the east-west diaphragm accelerations was also similar to that of the ground and the shear wall accelerations, with the exception that the long-period pulse at the 4.0 second point was not obvious. Apparently the diaphragm, once excited at its own natural frequency was not affected by this pulse.

Fourier spectra have been computed for each of the three measured records of roof acceleration. They are shown in Fig. 9 for the east-west diaphragm acceleration (sensor 5) along with spectra for the east-west ground motion. The dominant period for the diaphragm response was 0.45 seconds. The ground motion was also rich in this period range, however, its dominant period occurred near 1.5 seconds which corresponded to the long period pulse at the 4.0 second point. The Fourier spectra for the east-west wall accelerations (not shown) matched that of the ground in the long-period range, but agreed with the diaphragm spectra in the short-period range as well. This suggests that the stiff, in-plane wall was faithfully following the ground motion, but also was accelerating with the diaphragm.

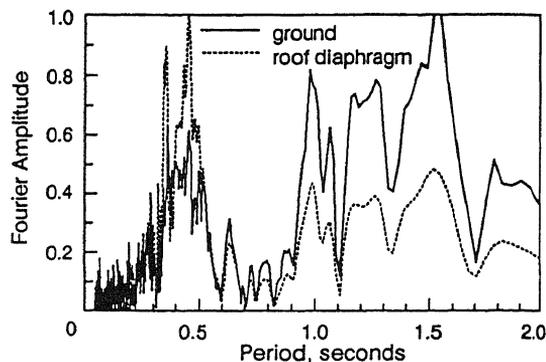


Figure 8. Fourier spectra for east-west motions.

4.2 Correlation with linear response spectra

The dominant apparent period of the structure (0.45 seconds from Fourier spectra) was very close to the characteristic period of the ground motion (0.42 seconds from response spectra, Fig. 6). This either infers that (a) by coincidence, the natural period of vibration matched that of the input motion, or (b) the structure was sufficiently stiff to respond at the ground-motion frequencies. It is likely that the natural period of the vibrating roof diaphragm could have been in the range of 0.4 seconds so both hypotheses are likely.

The suitability of using linear response spectra can be checked by plotting the apparent spectral acceleration and period, as measured, on spectra computed from measured base motions (Fig. 6). For simplicity, the apparent dominant period of the roof record (0.45 seconds) is assumed equal to the natural period of the system. If a modal participation factor of 1.2 is assumed (based on a triangular mode shape for a two-story building), then the apparent spectral acceleration is the peak measured acceleration of the diaphragm, 0.79g, divided by this factor, or 0.66g. Point *a* in Fig. 6 is plotted based on these two values. The point lies very near to the spectral response curves in the range of 5% to 10% damping. In light of the approximations used, the correlation is excellent suggesting that linear response spectra are indeed a viable means for estimating peak response, if the building period can be estimated.

4.3 Estimated nominal wall stresses

In an effort to estimate stress levels, nominal values of shear, flexural and axial stresses resisted by each wall were estimated from measured roof accelerations. Tributary masses were estimated based on assumed values for weights of the structure and any attachments. As an approximation, horizontal acceleration at the top of all walls

was assumed equal to that measured at the top of the interior wall in the east-west direction.

Table 2. Summary of wall stresses, MPa (psi)

Wall	Shear Stress	Flexural Stress	Gravity Stress
South	0.26 (38)	0.43 (62)	0.30 (44)
Central	0.24 (35)	0.71 (103)	0.28 (40)
North	0.09 (13)	0.27 (39)	0.17 (25)
East	0.24 (35)	0.54 (79)	0.28 (41)
West	0.15 (22)	0.31 (45)	0.20 (29)

Shear stresses given in Table 2 are average values across an entire wall section. Flexural stresses have been estimated by dividing overturning moments by section modulus based on gross, uncracked sections. Gravity stress is the minimum vertical compressive stress based on dead loads alone.

On the basis of these approximate stresses, it is credible that the masonry should not have cracked. Shear stresses were less than 45% of measured in-plane shear stresses. The difference between flexural stress and gravity stress (net flexural tensile stress) was nearly always less than code allowable values. The net tensile stress for the central wall was the highest at 0.43 MPa (63 psi) which was approximately 1.3 times allowable values given by current UBC and ACI masonry codes of practice.

5 COMPUTATIONAL STUDIES

Seldom is so much information available on system description, material properties and dynamic response of an actual building system. Though it may be easily rationalized in terms of nominal stresses why the masonry did not crack appreciably, two intensive computational models were run to study their applicability. Unfortunately, with the limited scope of this paper, details of the analytical modeling cannot be explained, nor can correlations between computed and measured response be made. Such descriptions can be found in Tena-Colunga and Abrams (1992). The purpose of providing an overview of these two models is to suggest how advanced modeling techniques may be applied to a historic structure.

5.1 Discrete MDOF dynamic model

Dynamic response was computed for each horizontal direction using a discrete multi-degree-of-freedom model. For east-west response, the 10 d.o.f. system shown in Fig. 9 was used. Dynamic degrees of freedom were assigned to lumped masses at the centroid of each wall as well as at the center of both diaphragms at each story. Soil flexibility was modeled using generalized translational and rotational springs at the base.

In-plane lateral stiffness of each of the three east-west walls was condensed to that for equivalent beam elements with translational degrees of freedom in the direction of loading using two-dimensional finite element models. In this way, effects of window and door openings could be

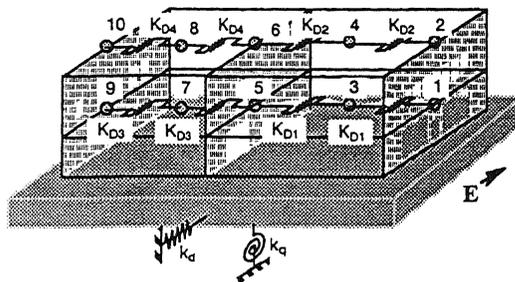


Figure 9. Discrete model for east-west response.

modeled. With the condensation approach, wall rotations were incorporated with the translational dynamic degrees of freedom. The same finite element models were used after the dynamic analysis to estimate peak stress states in the in-plane walls.

Results of several analyses using the discrete MDOF model illustrate sensitivities of dynamic response to variations of diaphragm stiffnesses, wall stiffnesses, mass discretization and damping assumptions. Measured acceleration histories at the roof could be replicated reasonably well with the discrete MDOF model, provided that diaphragm stiffnesses and soil flexibilities could be estimated accurately.

5.2 3D Finite element model

Because the discrete MDOF model was limited to two dimensions, and significant torsional motions were likely, the complete system was modeled with the finite element mesh shown in Fig. 10. The ABAQUS program was used with thick shell, isoparametric 8-node elements. Floor and roof diaphragms were modeled with an orthotropic formulation while masonry walls were modeled with an isotropic formulation. Linear behavior was assumed since the structure was observed to be essentially uncracked following the earthquake.

The first-mode shape (Fig. 10) depicts the deformation of the roof diaphragm in the east-west direction as it spans between the relatively stiff in-plane walls. Distortion of the east wall resulting from the diaphragm movement is also evident. It is clear from the system deflected shape that response cannot be represented with a simple lumped-mass, stick model as is commonly used for dynamic analysis of multistory steel or concrete buildings.

The finite element model was used to determine frequencies and mode shapes. In addition, wall and diaphragm stresses were determined using the FEM model resulting from (a) equivalent static forces (obtained from the MDOF dynamic discrete model), (b) response spectra analysis (determined from measured ground motions) and (c) a time-step integration of 20 modal coordinates (determined from solving Eigenvalues for the FEM mesh) using the measured ground motions.

Shear stress contours are shown in Fig. 11 for the time of peak response from the time-step integration. Maximum wall shear stress was in the range of 0.38 to 0.56 MPa (55 to 81 psi) which is indicative of the range for allowable

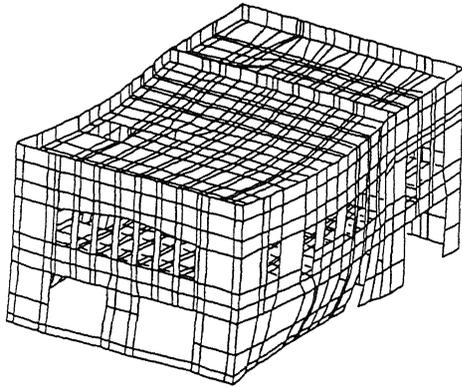


Figure 10. Finite element model.

stresses in masonry given by American codes. It is reasonable to assume with these stress levels that little or no cracking should have occurred.

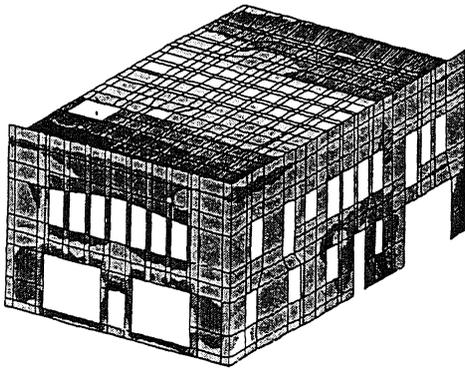
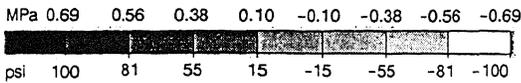


Figure 11. Shear stress contours.

Stress contours are not shown for the cases where the FEM model was run with equivalent static forces, or with a response spectra analysis, however, good agreement was seen between them and those for the time-step integration analysis. This suggests that the discrete MDOF model may be suitable for estimating peak lateral forces despite its simplistic assumptions. Moreover, the linear response spectra approach proved admissible once modes of vibration could be identified.

6 CONCLUSIONS

Various analytical techniques used to study response and damage of the historic unreinforced masonry structure helped to explain why the building withstood the Loma Prieta Earthquake with little damage. Wall shear stress levels were shown by both crude and sophisticated methods to be within reconcilable ranges.

The recorded response and the overall behavior of the structure during the earthquake were estimated well with the discrete MDOF linear dynamic analyses. Though it was far simpler than the finite element model, salient characteristics of response such as diaphragm flexibility, soil-structure interaction and lateral stiffness of walls with openings could be represented.

The study has inferred that the vulnerability of unreinforced masonry construction east of the Rockies may not be as severe as believed with the common consensus. There is promise that future earthquake hazard studies for specific buildings may show that demolition or expensive rehabilitation may not be necessary.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

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