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Behavior of steel frame buildings with infill brick

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ABSTRACT

The City of Oakland, California, has a substantial inventory of mid- to high-rise steel frame buildings with unreinforced brick infill forming the exterior walls. Many of these buildings suffered significant damage during the October 17, 1989, Loma Prieta Earthquake. These buildings were built about the 1920s and their designs are similar to buildings built in cities east of the Rocky Mountains.

The ground motion felt in downtown Oakland was of fairly short duration with maximum peak horizontal ground accelerations estimated to range from 10% to 20% of gravity. The observations made on the performance of the infill steel frame buildings during the earthquake can be of great value in addressing earthquake hazard mitigation issues in the eastern half of North America.

Data obtained from observations and evaluations of brick infill steel frame buildings are summarized. A generalized case study is given to illustrate damage assessment techniques and methods of repair. The City of Oakland criteria for assessing and repairing earthquake damaged buildings and criteria for evaluating and upgrading existing undamaged infilled steel frame buildings is reviewed.

INTRODUCTION

North America has a substantial inventory of mid- to high-rise steel frame buildings with unreinforced brick infill forming the exterior walls. These buildings were generally not designed with earthquake forces in mind; wind being the governing factor for lateral forces. Not much is known on how these buildings will perform in a major earthquake. They are difficult to evaluate by conventional analytical procedures because of the uncertainty of how the brick walls interact with the structural steel frames.

This type of construction was very common for residential and commercial buildings in the downtown areas of San Francisco and Oakland, California,

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during the early part of this century. Many were designed by Chicago and New York architectural/engineering firms. During the October 17, 1989 Loma Prieta Earthquake the San Francisco Bay Area experienced ground motion of fairly short duration with horizontal peak ground accelerations in the range of 10% to 20% of gravity. Reported damage to brick-infilled steel-framed buildings ranged from very minor to significant. The types of damage included cracked interior partitions and ceilings, spalled ornamental exterior terra-cotta and stone, and cracked and dislodged exterior brick. Although most of the buildings remained open, many were closed because of potentially hazardous conditions.

The post-earthquake damage assessments, structural evaluations, and rehabilitation proposals give us an opportunity to gain knowledge on how these types of buildings work.

BUILDING DESCRIPTION

The type of building discussed here is a steel frame structure with brick infill exterior walls.

The typical building has a complete three-dimensional steel frame that carries the gravity loads. The beams are generally rolled sections and the columns are generally riveted built-up sections. The exterior perimeter beams typically are offset from the columns' center lines so that the web of the beam lines up with the outer flange of the columns. An idealized representation of the brick-infilled steel frame is shown in Fig. 1. The riveted beam-column connections are primarily designed for gravity loads with some nominal moment resistant capacity for lateral forces. The moment resistant capacity is generally provided by two or four rivets attaching the top flange clip angle and the beam seat.

The infill masonry generally consists of three wythes of brick. Two interior wythes are supported by the top flange of the beam. The exterior wythe often includes decorative terra-cotta elements and its method of support varies for different buildings. In some cases a steel ledger plate cantilevers out from the steel beams. In other cases, it is supported by the keying action of header bricks interlocked with the interior wythes. Whereas the brick is generally tightly fitted above and below by the steel beams, the details of the brick interfacing with columns on each side are not usually well known.

The floor framing systems for these buildings may be wood or concrete. When wood is used, the floors generally have two layers of wood sheathing. When concrete is used, some of the structural steel framing may be encased in concrete fireproofing. Interior partitions may be plastered stud walls or hollow clay tile. Ceilings are lath and plaster hung from the floor framing above.

The buildings generally range in height from 6 to 7 stories for hotels to over 15 stories for office buildings. Floor areas range from 5,000 to 10,000 square feet. Most of the buildings have some shape irregularities such as flat-iron, L- or U-shaped in plan, light wells on the sides, light courts in the center, tall first stories, and open storefronts on one or two sides.

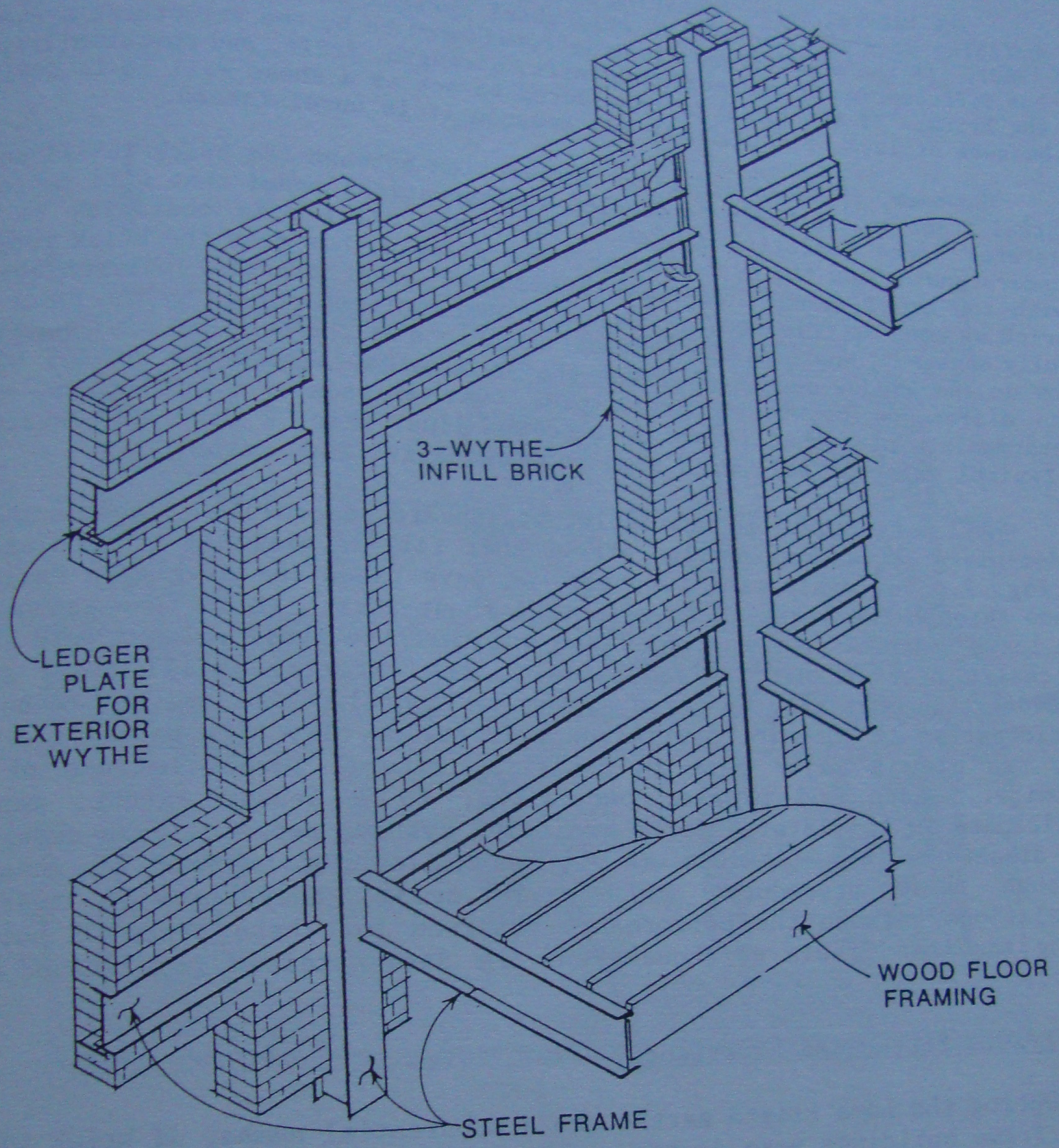

Figure 1. Schematic representation of brick infilled steel frame

Lateral Force Resisting System

The lateral force resisting system of an existing brick infill steel frame building does not conform to structural systems in current codes (e.g., UBC 1988). If the steel frame alone is considered to be the structural system, it has deficiencies in connection details, strength, drift, and compatibility with the brick. If the brick is considered to act as a shear wall it is deficient because of lack of continuity and because it is unreinforced.

However, if we consider the interaction between the brick infill and the steel frame we can begin to develop a mathematical model that will define the lateral force resisting system. There are still some obstacles to fully understanding how the system works. For example, how does the brick interface with the steel frame? Is there horizontal shear transfer between beam and brick by bond or friction, or is the column confined enough by the brick to be fully engaged? What are the vertical reactions at the beam-column connections? How do the window openings affect the load paths through the brick? What is the difference between a brick pier that encases a steel column and an intermediate pier that is not on a column line? Fig. 2 shows an elevation of a typical exterior wall with some representative load paths.

Until better data is available, it appears reasonable to assume that the wider piers act as compression struts that fail in diagonal tension as shown in Fig. 2. For evaluation purposes, we have taken the area of a horizontal plane through the piers as an effective shear area. It is then assumed that the loads transfer to the steel frame and that the steel columns work axially to resist overturning. In addition to the exterior infill frame system, consideration must also be given to the horizontal diaphragms and to possible participation of interior partitions. If hollow clay tile partitions exist, they can play a major part in resisting lateral forces because of their rigidity. Lath and plaster partitions, to a lesser extent, can also participate in the lateral force resisting system; especially in cases where the diaphragms are flexible (e.g., wood flooring instead of concrete). Although these procedures may appear crude, they provide reasonable correlations between the performance observed for these types of buildings during the Loma Prieta earthquake and the data obtained from ground motion records.

Loma Prieta Earthquake Experience

During the Loma Prieta earthquake a substantial number of brick infilled steel frame buildings were damaged in San Francisco and Oakland. The results of the studies covered by this paper are primarily based on data from buildings in the downtown area of the City of Oakland.

The strong motion instrument located closest to the downtown area recorded peak ground accelerations of roughly 10% of gravity in a north-south direction and 20% of gravity in an east-west direction. These values were consistent with other recordings in the Bay Area. Response spectra developed from these

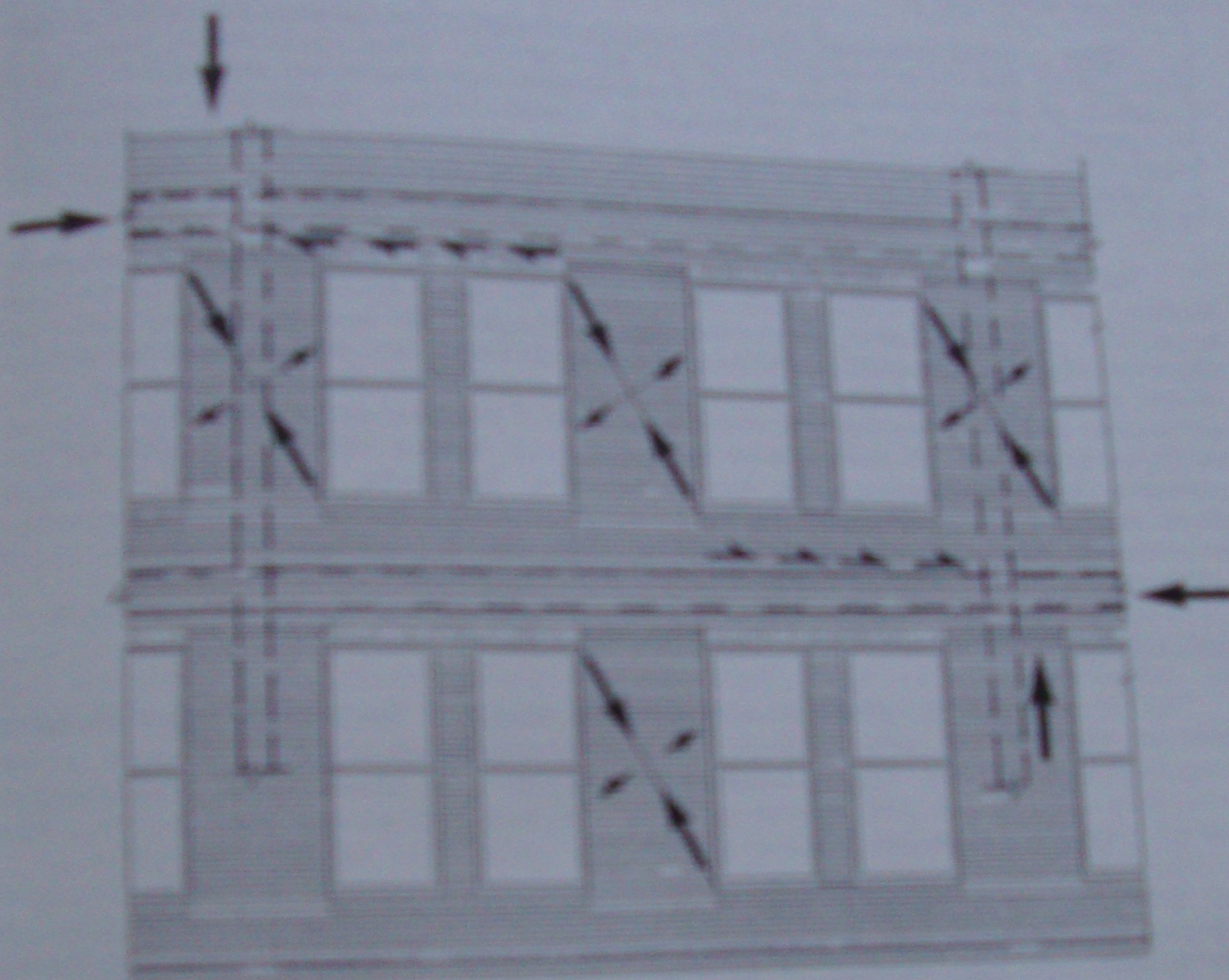

Figure 3. Partial wall elevation with representative load paths

recordings were fairly broad in shape indicating a significant ground motion amplification in the 0.5 to 1.0 period range (Fig. 3).

Soon after the earthquake the City of Oakland adopted an Emergency Order for Abatement of Hazardous Structures Associated with Earthquake Damage. This emergency order requires that a Damage Assessment Report (DAR) be prepared for earthquake damaged buildings. The DAR is to include an estimate of the loss of capacity to resist lateral forces that the building sustained during the earthquake. If the loss of capacity is more than 10%, the order requires a total seismic upgrade.

This process requires the engineer to estimate the lateral force resisting capacity of the structure prior to the earthquake. This capacity is stated in terms of an equivalent base shear coefficient calculated as the ratio of the horizontal seismic force to the weight of the building. If the post-earthquake capacity is at least 90% of the pre-earthquake capacity, the building need only be repaired to its former condition. However, if the capacity is less than 90% (i.e., the loss is greater than 10%) a full seismic upgrade is required.

The capacity of the building is determined in a rational, consistent manner, taking into account relative strengths and rigidities of the various materials. This includes materials not conforming to current codes if it is apparent that their participation was significant in resisting the forces caused by the earthquake. The results of the evaluations of both the pre-earthquake and post-earthquake structure should be consistent with the observed earthquake damage. In order to be rational, the values of the strengths of the elements have to reasonably represent the in-place materials. Sample guidelines were established to aid engineers. For example, a tentative recommendation of 75 psi was suggested for the shear capacity of infilled brick.

Sample Evaluation

As a result of studies of several damaged buildings some general observations can be made. Each of the studied buildings had some shape irregularities of the types described earlier. In each case, the initial observable damage did not appear severe. It was not always obvious that the loss of capacity exceeded 10%.

However, upon further evaluation of the load paths and the overall response characteristics, a pattern of the sequence of damage could usually be established. In this sample all buildings were about seven stories in height. Periods of vibration were estimated to be about 0.5 to 0.6 seconds in their pre-damaged state.

When buildings were evaluated on the basis of a code type allowable shear stress of 12 psi for brick infill, the resulting pre-earthquake equivalent base shear capacities ranged from about 2% to 4% of the weight of the building. For the steel frames, with their nominal moment resisting capacities, the base shears coefficients generally ranged from 1% to 3%. However, the brick infill was much stiffer than the frames and major loss of brick would have to occur before the very flexible steel frames would participate. If ultimate strengths, such as 75 psi for the brick infill, were used in the evaluation, the base shear equivalent would be about 15% to 25% of the building weight. In its cracked (or damaged) state the periods of vibration were estimated to be about one (1) second. Damping values were assumed to be 5% of critical damping for the uncracked state and 20% for the cracked state. Using this data with the smoothed response spectra in Fig. 3, approximations of the building capacities in terms of peak ground accelerations can be made as shown below.

	C_B	T	B	S_A	A_G
Uncracked	.02	0.5	5%	.025	.01g
Uncracked	.04	0.6	5%	.05	.02g
Cracked	0.15	1.0	20%	.20	0.13g
Cracked	0.25	1.0	20%	.30	0.20g

C_B = base shear coefficient
T = fundamental period of vibration
B = percent of critical damping
S_A = spectral acceleration = $C_B/0.8$ (approximate)
A_G = peak ground acceleration (acceleration at zero period of response spectra) obtained from ratio of S_A value above to S_A value in Fig. 3

This indicates that a peak horizontal ground acceleration of 1% or 2% of gravity could result in brick stresses of 12 psi; however, it would take a ground acceleration of 13% to 20% to reach an ultimate cracking strength of 75 psi. On the assumption of 10% peak ground acceleration in the north-south direction and 20% in the east-west direction during the Loma Prieta earthquake, it appears that one would expect minor or no damage in the north-south direction and significant cracking in the east-west direction of the building. This agrees with the observations. If the results did not seem reasonable, one would take a closer look at the assumptions and try again.

REPAIR AND UPGRADE

If a building loses more than 10% of its pre-earthquake capacity, Oakland requires that the entire structure be made to substantially comply with the structural requirements of the current code (i.e., 1988 UBC). However, some allowances are made because of the difficulties of bringing an existing building up to current standards. This includes the application of the equivalent of the 1991 UCBC for unreinforced masonry bearing wall buildings and the provision that the total design base shear coefficient need not exceed 0.133. The repair ordinance also states that the building official may approve an alternative procedure if it can be demonstrated by rational analysis that the modified structure provides the intended level of safety.

This last provision appears to allow for innovative solutions and the use of performance criteria or limit state procedures for seismic upgrading of existing structures.

CONCLUDING REMARKS

There is still a lot to learn on how steel frames with brick infill perform when subjected to major earthquakes.

If we take the opportunity to learn from data obtained from past earthquakes, we can improve our design procedures and criteria for upgrading existing buildings.

REFERENCES

UBC 1988. Uniform Building Code, International Conference of Building Officials, Whittier, California.

UCBC 1991. Uniform Code on Building Conservation, International Conference of Building Officials, Whittier, California.

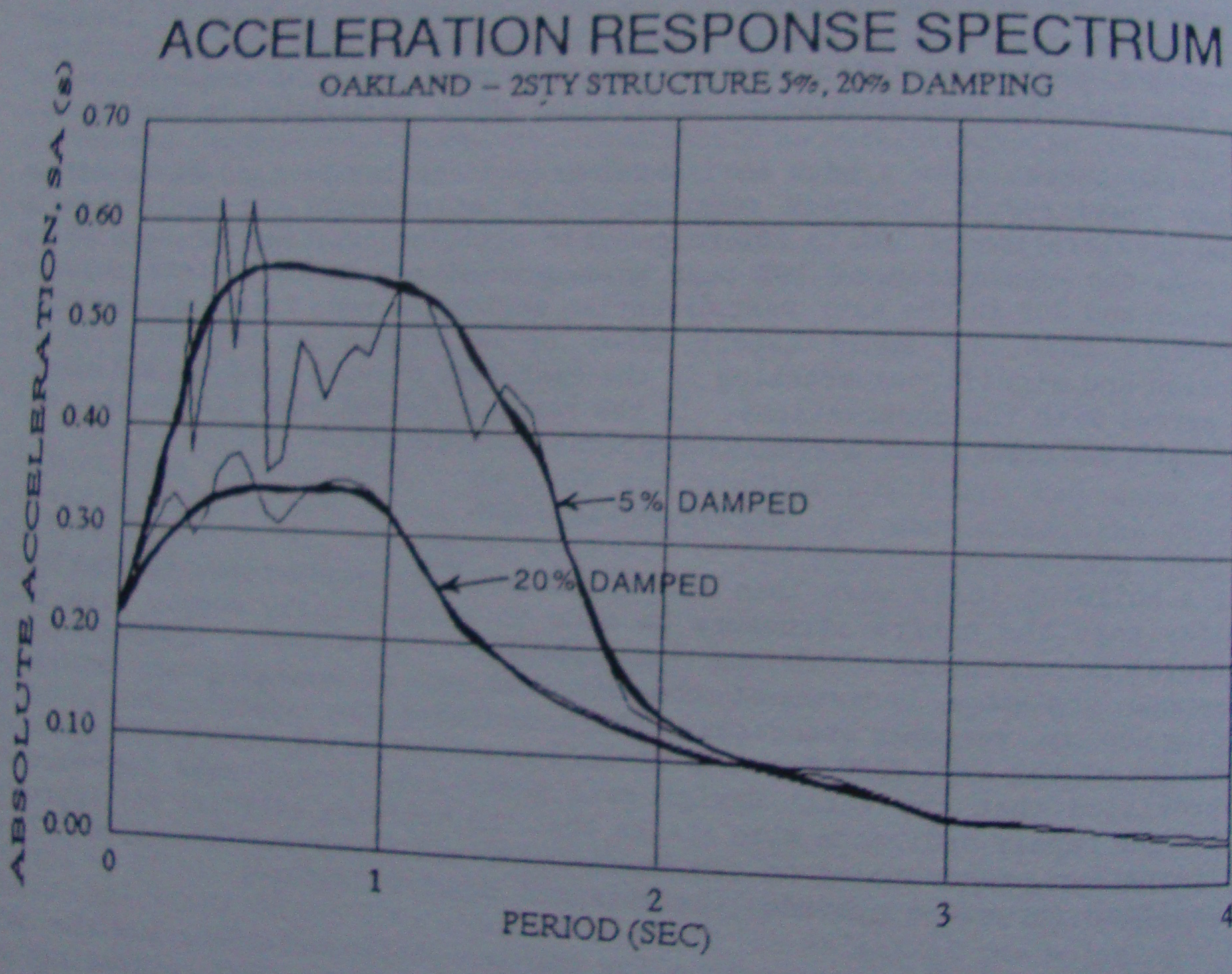

Figure 3. Smoothed response spectra, Oakland, October 17, 1989