Reducing Seismic Risk in Massachusetts

A program that evaluates the seismic risk of existing buildings is instrumental in developing structural modifications that will reduce seismic risk.

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No single factor clearly defines seismic risk in Massachusetts, nor for that matter for the rest of the Northeast United States. It is only when several factors are considered together that the risk becomes clear. This state of affairs presents a clear problem — the general public in the region is not aware of all these factors and, therefore, does not perceive the very real threat from earthquakes.

Viewed together, the following four factors portend a potential disaster. The first factor is that the mechanics of earthquakes in the Northeast are not well understood. There does not appear to be a correlation between epicenters and geologic features such as faults. This lack of correlation implies that an earthquake may happen anywhere in the region, and at any time. There is also no limit on the magnitude of the earthquake that may occur. Even though there have not been any significant earthquakes affecting the region in the recent past, history shows that significant earthquakes have occurred. In the colonial period the region was very active. Many earthquakes were reported in the written records of the period. For instance, the following was written in reference to an earthquake in early June 1638:[1]

> "Between three and four in the afternoon, being clear, warm weather, the wind westerly, there was a great earthquake. It came with a noise like a continued thunder, or the rattling of coaches in London, but was presently gone. It was at Connecticut, at Naragansett, at Pascataquack, and all the parts round about. It shook the ships, which rode in the harbour, and all the islands, &c. The noise and the shakings continued about four minutes. The earth was unquiet twenty days after, by times."

The second factor, also associated with geology, is the high transmissibility of earthquake energy in the region, which translates into the potential for widespread damage. This transmissibility is hinted at in the above quotation, and is further illustrated by the reports that both the New Madrid, Missouri (1811-1812), and the Charleston, South Carolina (1886), earthquakes were felt in Boston.

The third factor is the general lack of seismic resistance in the building stock. While recent versions of the Massachusetts building code

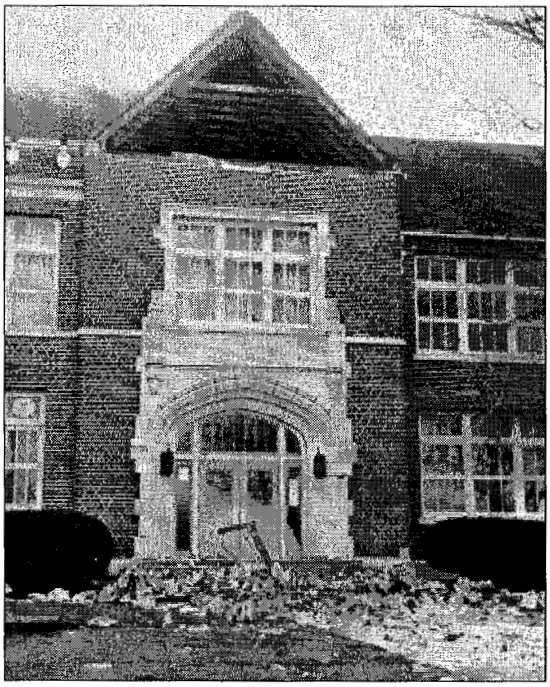


FIGURE 1. Destruction of unreinforced nonbearing structural elements from an earthquake.

have included seismic provisions, they pertain mainly to new construction. The majority of buildings in the state were not built with lateral loading in mind. Without regular seismic activity as a reminder (as occurs in California), the tendency is to ignore potential earthquake loading. The large number of unreinforced masonry structures heightens the threat to public safety. Under such conditions, even relatively mild earthquakes can result in significant risk for loss of life and property, as demonstrated by the March 1993 earthquake in Scotts Mills, Oregon.[2] Figure 1 shows damage that occurred to a high school from that magnitude 5.6 event. Imagine the consequences had a group of students been on the doorstep of the school.

The final factor is high population density. Maps of population in the U.S. clearly show population concentrations on the two coasts, with an especially dense band along the northeast coast. Were a large earthquake to occur (and there is no reason to believe it will not), the potential for a disaster is high due to the large number of structures that are vulnerable to earthquakes and the large number of people using those buildings.

Given the general lack of acceptance of seismic risk in the Northeast, it seems unlikely the public in the region will spend additional monies on seismic hazard reduction. Funds could be allocated from the current tax base, but given these economic times, that would probably mean cutting some other essential programs. Ultimately, the public will have to commit funds if large-scale remediation is to take place. However, such an allocation cannot occur until the public is sold on its necessity. Until that time, and as part of that selling effort, certain things can be done that can serve as the basis for large-scale remediation. These things can also help sell the public on the more cost-effective idea of taking advantage of opportunities for seismic retrofitting of essential buildings during other renovation activity.

The Beginnings of a Solution

In summary, a very focused effort at reducing seismic risk in Massachusetts involves a number of activities:

1. Conduct a rapid visual screening of essential facilities. This screening will help define the magnitude of the task, and will serve as a planning tool for implementing the rest of the program.
2. Perform a detailed seismic evaluation of typical structures from each group of essential facilities with the goal of identifying common features of the buildings in each group that make them seismically vulnerable. For example, the large openings found in the first story of every fire station make these buildings especially vulnerable.
3. Design retrofit techniques that are catered to the region and to specific types of buildings. It is hoped that techniques can be developed that will be usable, with minor variations, over and over again. At this point, a good estimate of retrofitting costs can be made.
4. Institute a widespread retrofit program for all essential facilities in the state. Much of this retrofitting can be done in conjunction with other programs. For instance, many of these essential facilities are also historically significant and are often restored/renovated as part of historic preser-

vation programs. Seismic retrofit should be incorporated into such programs. In many cases, it would only add moderately to the cost. Also, schools are often renovated and should at the same time be seismically retrofitted.

While implementing such programs, close attention should be paid to the actions of the federal government. The Earthquake Hazards Reduction Act of 1977 and Executive Order 12699 outline the actions the federal government is to take with regard to the buildings it owns and leases. Executive Order 12699 mandates that all new buildings must be designed and built in accordance with "appropriate seismic design and construction standards," and work is underway to implement this order. More significant in regard to scope and cost is the provision in Sec. 8. (a) in the Earthquake Hazards Reduction Act of 1977 which states that:

> "The President shall adopt, not later than December 1, 1994, standards for assessing and enhancing the seismic safety of existing buildings constructed for or leased by the Federal Government..."

Some valuable lessons will surely be learned by studying the implementation of this legislation. It should also be noted, as it was in the Earthquake Hazards Reduction Act of 1977, that much of the work done towards reducing seismic risk would also reduce the risk from other natural hazards such as hurricanes.

Work based on the first three steps noted above has begun on a smale scale in Massachusetts. The rapid screening of fire stations in Middlesex County described in the next section was conducted in the summer of 1992. This rapid screening was followed by a detailed seismic evaluation of one of the fire stations, and research on externally bonded reinforcement as a means to strengthen unreinforced masonry walls — also described below.

Rapid Screening of Fire Stations in Middlesex County

As previously noted, despite a significant earthquake hazard in the eastern United States, few structures in this region have been constructed to withstand the lateral loads that an earthquake may impose. The Earthquake Hazards Reduction Act of 1977 (amended November 1990) outlines several steps that can mitigate this threat, one of which is to "conduct seismic safety inspections of critical structures" that may fail in an earthquake.[3]

One tool designed for seismic safety inspections is the Federal Emergency Management Agency (FEMA) publication "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook" (FEMA-154), hereinafter noted as the RSP.[4] This document outlines a procedure to identify buildings that may sustain serious damage in an earthquake, and any that fail the screening are noted as requiring inspection by a professional engineer experienced in earthquake engineering.

The RSP document is intended for "local building officials, professional engineers, registered architects, building owners, emergency managers, and interested citizens."[4] Obviously, the range of expertise among this group could vary from none to complete. For this project, civil engineering undergraduate students were employed to exercise the RSP. Engineering students at the upper undergraduate level have been taught dynamics, basic structural analysis, and the characteristics of building materials — all of which make understanding and using the RSP easier.

The actual implementation of the RSP is simple since it is intended to be a "sidewalk survey" procedure based almost entirely on information gathered visually. Figure 2 shows the form that is filled out during an inspection. A building is identified based on its type of construction (unreinforced masonry, timber, steel, reinforced concrete, etc.) and is accordingly assigned a base seismic score. A seismically good construction type, such as wood, will have a high score, and a potentially poor type, such as unreinforced masonry, will have a low score. The basic scores can then be modified based on the physical characteristics of the building. For example, a building with a long wing projection may have its basic score reduced one point for a plan irregularity, since significant damage may occur at the re-entrant corner. A building may have its score increased only if it was built after seismic provisions were

ATC-21/ (NEHRP Map Areas 3,4, Moderate)
Rapid Visual Screening of Seismically Hazardous Buildings

Address_____
Zip_____
Other Identifiers_____
No. Stories_____ Year Built_____
Inspector_____ Date_____
Total Floor Area (sq. ft.)_____ Use_____
Building Name_____

OCCUPANCY

Residential	No. of Persons
Commercial	
Office	
Industrial	0-10
Pub. Assem.	11-100
School	100+
Govt. Bldg.	
Emer. Serv.	
Hist. Bldg.	

Non Structural Falling Hazard ☐

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data
DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RCSW)	C1 (MRF)	C2 (SW)	C3/S5 (URMINF)	PC1 (TU)	PC2	RM	URM
BASIC SCORE	6.0	4.0	3.0	6.0	4.0	3.0	3.5	2.0	3.5	2.0	3.5	2.0
HIGH RISE	N/A	-1.0	-0.5	N/A	-1.0	-0.5	-1.0	-1.0	N/A	0.0	-0.5	-0.5
POOR COND	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
VERT. IRREG.	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-0.5	-1.0	-1.0	-1.0	-0.5	-1.0
SOFT STORY	-1.0	-2.0	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-1.0	-2.0	-1.0
TORSION	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
PLAN IRREG.	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
POUNDING	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
LG.HVY.CLAD.	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
SHORT COL	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
PST BNMK YR	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 8-20 FLRS	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	-0.8	N/A	-0.8	-0.8	-0.8
FINAL SCORE												

COMMENTS

Detailed Evaluation Required?

YES NO

FIGURE 2. A blank RSP form.

introduced in the governing building code — hence, determining the year of construction is vital. A final score is determined and compared to a cut-off score of 2.0. A score of 2.0 or less means that the building may be seismically hazardous and that it requires further evaluation.

In addition to the basic structural scores and modifiers, the RSP facilitates consideration of other characteristics of the building. The occupancy of the building and its general use can be noted in order to prioritize any further evaluations. A non-structural falling hazard can be

noted, and is typically a portion of the building (such as ornamentation like parapets and chimneys) that does not carry a load. Since these features are non-structural, they are often not anchored sufficiently to resist lateral loads and can be thrown from the building.

Teams of two or three have been most effective in implementing the RSP. The work of interviewing the building's occupants, photographing and measuring the building, and collecting global positioning data can be split among the team members.

Due to the large number of buildings in Massachusetts, some limits had to be placed on this project. The Earthquake Hazards Reduction Act of 1977 notes that priority for inspection should be given to "power generating plants, dams, hospitals, schools, public utilities, and other lifelines, public safety structures, high occupancy buildings, and other structures which are especially needed in time of disaster."[3] Similarly, the seismic provisions of the Uniform Building Code (UBC 1988) specifically mention the need for fire stations and police stations to withstand a higher level of lateral loading.[5]

By the above definitions, the number of essential buildings that should be screened is still in the thousands. To contain project duration to one summer, the decision was made to consider only fire stations. Fire stations are clearly essential facilities since fires often occur as a consequence of a seismic event, as witnessed by the aftermath of the 1906 San Francisco and 1989 Loma Prieta earthquakes.

Initially, the intention was to inspect all the fire stations in Massachusetts. Phone calls to the fire departments of each town in Middlesex County yielded an estimate of approximately 148 fire stations, though ultimately 151 were found. It was then decided to limit the screenings to fire stations in Middlesex County, which proved to contain a good cross section of urban, suburban and rural communities that have fire stations of various ages and types of construction.

As the inspections proceeded, the data that were collected in the field were stored on a personal computer. A word processor was used to reconstruct the RSP form, and the field data for each fire station were entered onto the computerized form. Highlighting (redlining) in the word processor mimicked marks made on the form in the field, and comments could be typed in the provided spaces.

The RSP calls for the inclusion of a photograph of the inspected building and a sketch of the building in plan. Room for two photographs was provided on the word-processed version of the form. The photographs were taken with a digital camera, each showing two of the four sides of the building whenever possible. Once the photographs were downloaded from the camera, they were touched up (sharpened, brightened, etc.) and converted into a format that could be imported into the word-processed form.

The plan view of the building was drawn with a CAD program using the measurements made in the field. These drawings were also converted into a format that could be imported into the form. The form is only one page in length, and has an average size of 65 kilobytes. Figure 3 is an example of a completed computerized form.

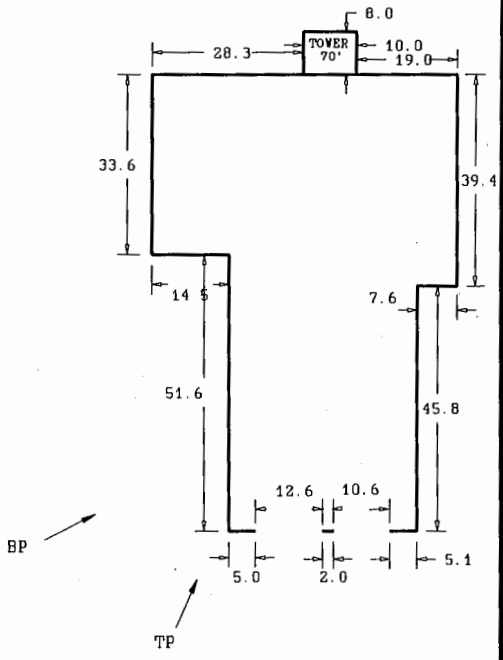
For the purposes of quick retrieval, the data from each form were stored in a database. Each fire station has its own record. Use of this database allowed queries to be made. It is a simple matter to use a logical query in order to get a listing of all fire stations built of unreinforced masonry, all the fire stations built before 1975 (the year that earthquake provisions were included in the Massachusetts building code), all fire stations meeting both of these criteria, or any other query desired for which data have been collected.

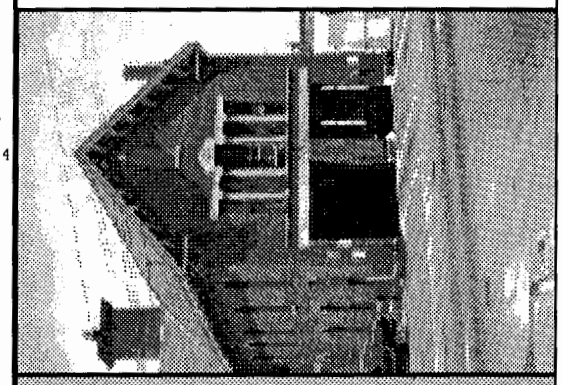
A geographic information system (GIS) was also used to present the results of the inspections. A GIS is a spatial database that links information about a feature on the earth's surface to the location of that feature. As noted previously, a global positioning unit was used to collect data at each fire station which yielded a longitude and latitude of each fire station that, with differential corrections, are within ten to 30 feet of the fire station's true location. The GIS converted these locations into a point coverage (a digital map composed of point locations) and linked the location of each fire station to its record in the database. The GIS can display the relative position of each fire station and, by selecting a fire station's icon, the record for that fire station can be displayed showing the RSP data.

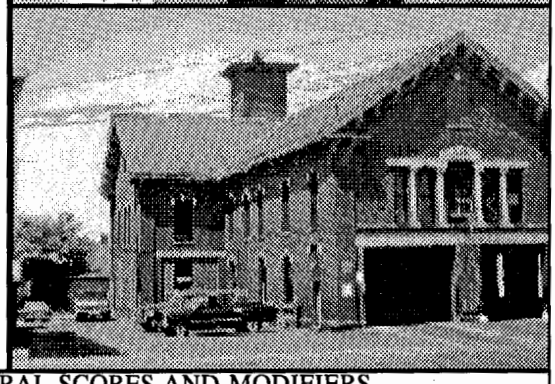
In order to further clarify the location of a fire

ATC-21/ (NEHRP Map Areas 3,4, Moderate)
Rapid Visual Screening of Seismically Hazardous Buildings

Address Branch Street LOWELL
Zip 01851
Other Identifiers Lowell Fire Dept., Station #2
No. Stories 2 + tower Year Built 1877
Inspector S. McElligott Date 4-10-92
Total Floor Area (sq. ft.) 7600 * Use FS
Building Name Station #2







OCCUPANCY

Residential	No. of
Commercial	Persons
Office	
Industrial	0-10
Pub. Assem.	11-100
School	100+
Govt. Bldg.	
Emer. Serv.	
Hist. Bldg.	

Non Structural Falling Hazard ☑

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data
DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RCSW)	C1 (MRF)	C2 (SW)	C3/S5 (URMINF)	PC1 (TU)	PC2	RM	URM
BASIC SCORE	6.0	4.0	3.0	6.0	4.0	3.0	3.5	2.0	3.5	2.0	3.5	2.0
HIGH RISE	N/A	-1.0	-0.5	N/A	-1.0	-0.5	-1.0	-1.0	N/A	0.0	-0.5	-0.5
POOR COND	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
VERT. IRREG.	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-0.5	-1.0	-1.0	-1.0	-0.5	-1.0
SOFT STORY	-1.0	-2.0	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-1.0	-2.0	-1.0
TORSION	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
PLAN IRREG.	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
POUNDING	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
LG.HVY.CLAD.	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
SHORT COL	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
PST BNMK YR	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 8-20 FLRS	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	-0.8	N/A	-0.8	-0.8	-0.8
FINAL SCORE												-0.5

COMMENTS
-Unreinforced Chimney on right side of building
-2 different constructions (front and back) both are URM
-Floor to wall anchorage is provided, however not sufficient to protect from seismic loading (too small + sparse)

Detailed Evaluation Required? YES NO

FIGURE 3. A completed RSP form (for the Branch Street Fire Station).

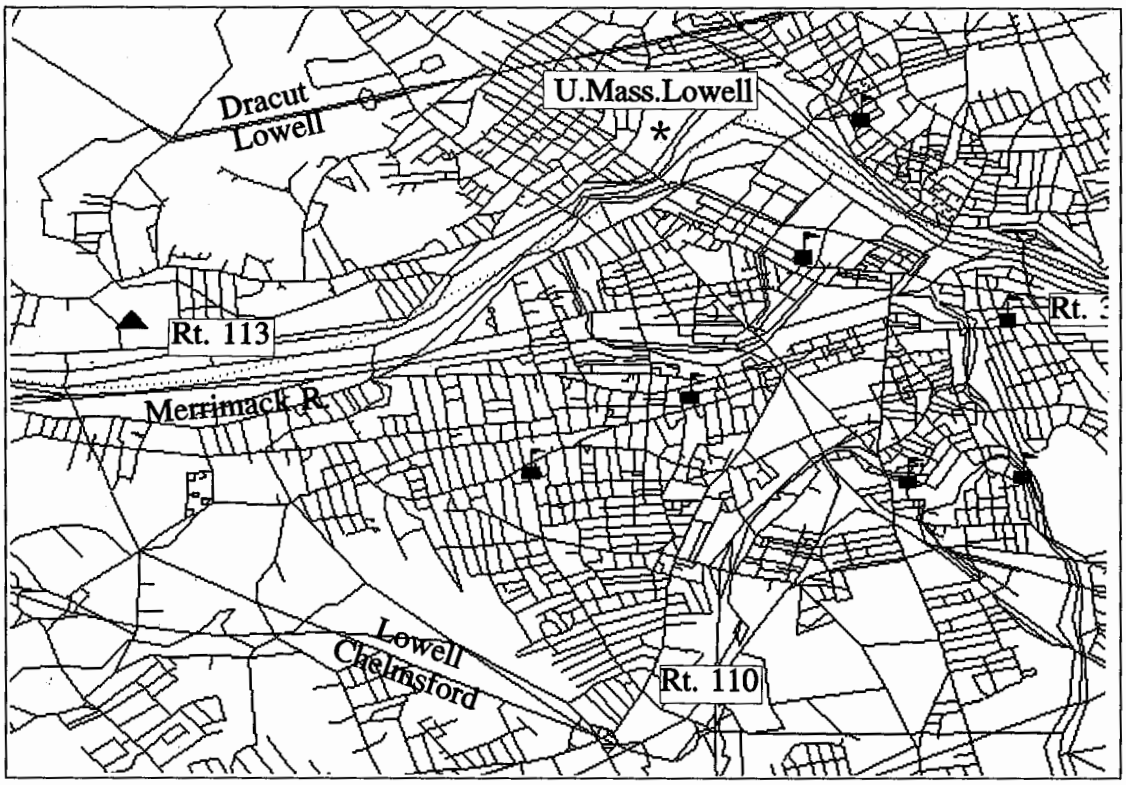


FIGURE 4. TIGER/Line digital map file for Lowell.

station to a user unfamiliar with the GIS database, a digitized street map of Middlesex county is also used. Data for the map were obtained from TIGER/Line data files from the U.S. Census Bureau that are stored on CD-ROM. The GIS converted the data file into a line coverage (a digital map composed of lines). The coverage for the fire stations could then be laid on top of the coverage for the street map, and specific data for either the fire stations or the streets can be displayed. Since the GIS program is large, it does not execute quickly on a personal computer. Therefore, for the purposes of just displaying the coverages and the databases, a quicker program with read-only capabilities is used.

The coverage of streets in the county is displayed and then the coverage of the fire stations is displayed on top of the streets. Figures 4, 5 and 6 demonstrate the use of the spatial database. The user can zoom into a particular city, such as Lowell, as shown in Figure 4. The text labels were not originally displayed and were added to clarify the street map. There are eight fire stations in Lowell, seven of which failed the RSP inspection and are represented by the flagged boxes. The one fire station that passed is represented by a solid triangle. A user can select a fire station with a mouse and a portion of the fire station's database record is displayed as shown in Figure 5. The database display can be blown up to full screen size as shown in Figure 6, and all the data from the RSP form, except for the photographs and plan drawing, can be read.

The data from the inspections yielded some interesting results. Of the total of 151 fire stations inspected, 130 received scores of 2.0 or less, indicating that the structure may present a threat to life-safety in the event of an earthquake.

Figure 7 presents a histogram showing the number of fire stations built within certain decades. The advanced age of the stock of fire stations can be clearly seen since a significant number were built prior to the turn of the century. It is interesting to note that only 17 of the fire stations were built after the inclusion of seismic provisions in the Massachusetts building code. Most of the fire stations were built in

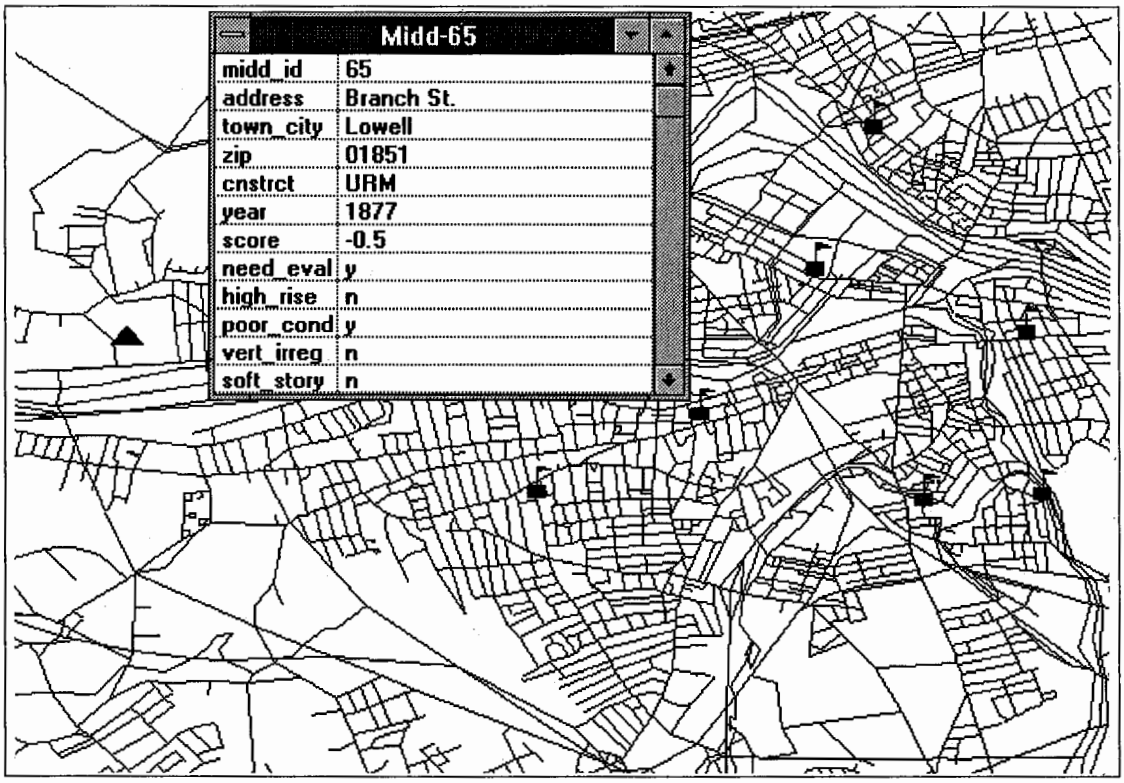


FIGURE 5. Response to selecting a fire station screen icon.

the 1950s and 1960s, before the inclusion of seismic provisions in 1975. A breakdown of the construction types found is shown in Figure 8. Unreinforced masonry construction, which has repeatedly demonstrated poor performance in past earthquakes throughout the country, makes up a clear majority of the fire stations.

Detailed Evaluation of the Branch Street Station

FEMA-154 recommends that structures receiving low scores in the visual screening should be subjected to a more detailed seismic evaluation. As noted earlier, many of the fire stations received low scores. However, conducting in-depth evaluations of each would not only be time consuming but also currently impractical. Therefore, one building possessing typical characteristics of the majority of those that "failed" screening was evaluated. The building chosen for this evaluation was the city of Lowell Fire Department's Branch Street Station No. 2 (shown in Figure 3) which was built in 1877. This building was singled out only for convenience — fire stations of similar construction and condition exist throughout Middlesex County, and probably the state.

Characteristics of this building that are typical of the others that received low screening scores were: unreinforced masonry bearing wall construction, plan irregularity, flexible wooden floor diaphragms, relatively tall hose tower, large openings along one wall and many non-structural falling hazards.

The evaluation of this building was conducted according to guidelines set forth in "A Handbook for the Seismic Evaluation of Existing Buildings" (FEMA-178 [ATC-22]).[6] This seismic evaluation is intended to determine the specific deficiencies in a structure's lateral force-resisting system. Once these deficiencies are identified, the engineer and owner must examine cost-efficient means of reducing the life-safety hazard of the structure. This evaluation typically involves examining the construction drawings and conducting an in-depth physical investigation of the building. Thus, this type of evaluation requires a large effort.

Midd-65	
midd_id	65
address	Branch St.
town_city	Lowell
zip	01851
cnstrct	URM
year	1877
score	-0.5
need_eval	y
high_rise	n
poor_cond	y
vert_irreg	n
soft_story	n
torsion	y
plan_irreg	y
pounding	n
lg_hvy_cld	n
short_clmn	n
pst_bnc_yr	n
floor_area	7600
use	Emer.Serv.
occupancy	0-10
nstrct_haz	y
comment_1	URM on right side of building
comment_2	2 different constructions (front and back) both are URM
comment_3	Floor to wall anchorage is provided, however not sufficient to protect
comment_4	from seismic loading (too small + sparse)
comment_5	
comment_6	

FIGURE 6. Data displayed for the selected fire station.

In accordance with FEMA's guidelines, evaluation forms pertaining to the building construction type (unreinforced masonry) were completed. These forms consist of statements that attempt to address life-safety issues. The statements are broken into various categories such as: building systems, masonry walls, diaphragms, connections, etc. Each statement is intended to lead the evaluator through a series of investigations and calculations in order to determine the safety of the building. Statements that evoke a true response are deemed acceptable according to the evaluation procedure. A false response indicates the need for further investigation. A response of "NA" signifies that this criterion is not applicable to this specific structure, and a response of "NC" denotes that this criterion was not considered as part of the evaluation.

Below are listed explanations of why certain evaluation statements were noted as false. Statements such as these must be remedied in order to reduce the life-safety threat. In order to clarify the description of the deficiencies of the evaluated building, the transverse direction is taken as the short direction of the building.

Weak Story. A deficiency was noted in the strength of the first-floor walls. Since both the first-floor walls and the second-floor walls were of the same thickness and construction, they possess the same unit shear strength. The ratio of strengths between the two stories is essentially the ratio between areas resisting shear in each story. It was shown that in the transverse direction, the strength of the lower story was 74 percent that of the second, less than the allowable difference of 80 percent. This deficiency is due mostly to the presence of the large garage door openings at one end of the structure.

Torsion. A severe torsion problem exists within the structure. There is an eccentricity between the center of gravity of the building and the center of rigidity of the lateral force resisting system. This eccentricity was shown to be 23.86 feet, more than the maximum allowable 20 percent of the width of

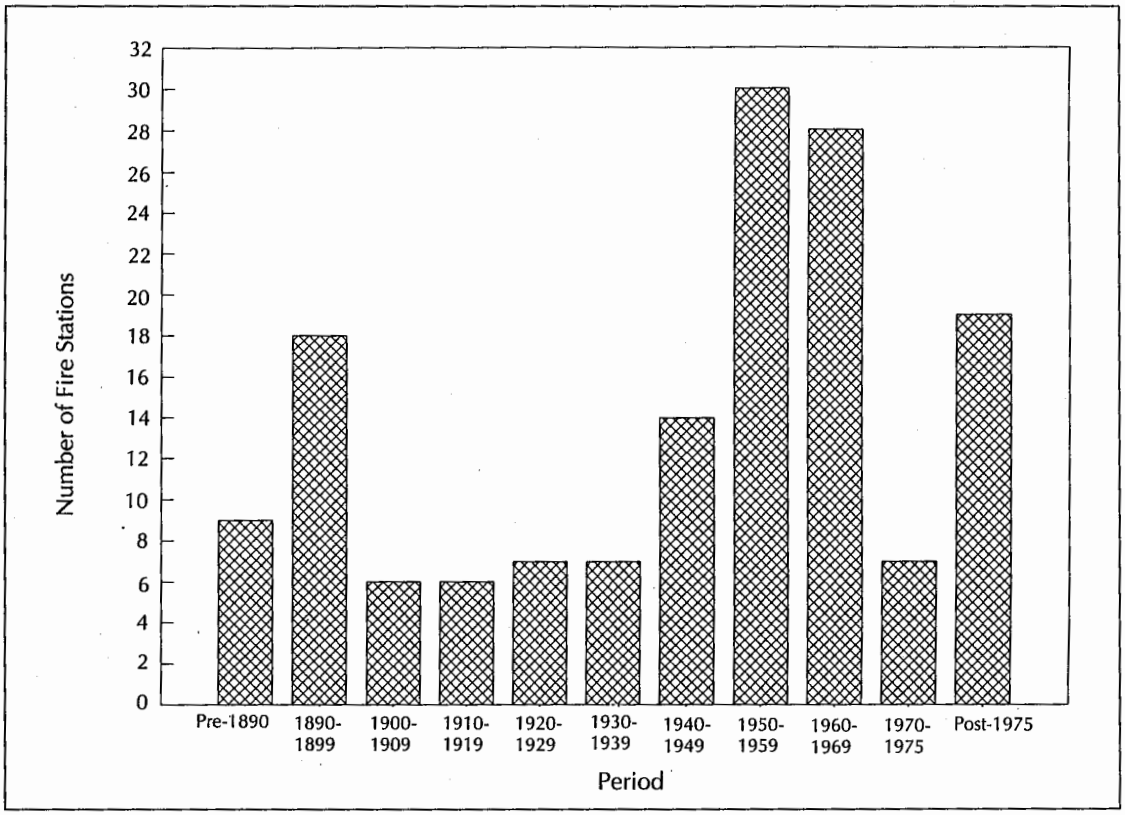


FIGURE 7. Distribution of construction dates among fire stations in Middlesex County.

the structure. Due to the large open front of the building, the center of rigidity was drastically shifted towards the rear of the building. When a lateral load is applied to the structure in the transverse direction, a torsional moment occurs. This moment causes a torque, or "twist," of the entire building.

Masonry Units. It was seen through visual inspection that much of the masonry was cracked and deteriorated. The deficiency is in the strength of the unit. No attempt was made to determine the reduced capacities of these masonry units since that would involve destructive testing.

Masonry Joints. Mortar could easily be scraped away from the joints with a key. Soft mortar has been shown to have reduced shear and bond strength. In some cases, the presence of mortar in the joints in the outer wythe of bricks was non-existent. Destructive tests would be required to determine the in-plane shear strength of the walls.

Shearing Stress Check. The average shearing stresses obtained through the "quick check" procedure[6] were in excess of the allowable value of ten pounds per square inch (psi), a conservative value based on the observed strengths of actual buildings with similar construction. The values obtained in the transverse direction were two to four times this allowable value, while those in the longitudinal direction were twice this value. A more detailed analysis was not conducted, realizing that shearing stresses resulting from torsion (which was not included in the "quick check") will increase the level of shear in the walls more than a detailed analysis will decrease it.

Plan Irregularities. The irregular plan of the structure poses a problem. When exposed to a lateral load in the longitudinal direction, the floor diaphragm will deflect as shown in Figure 9. At the re-entrant corners of the building (the circled regions), the deflection of diaphragm 1 does not coincide with that of diaphragm 2. Unless the dia-

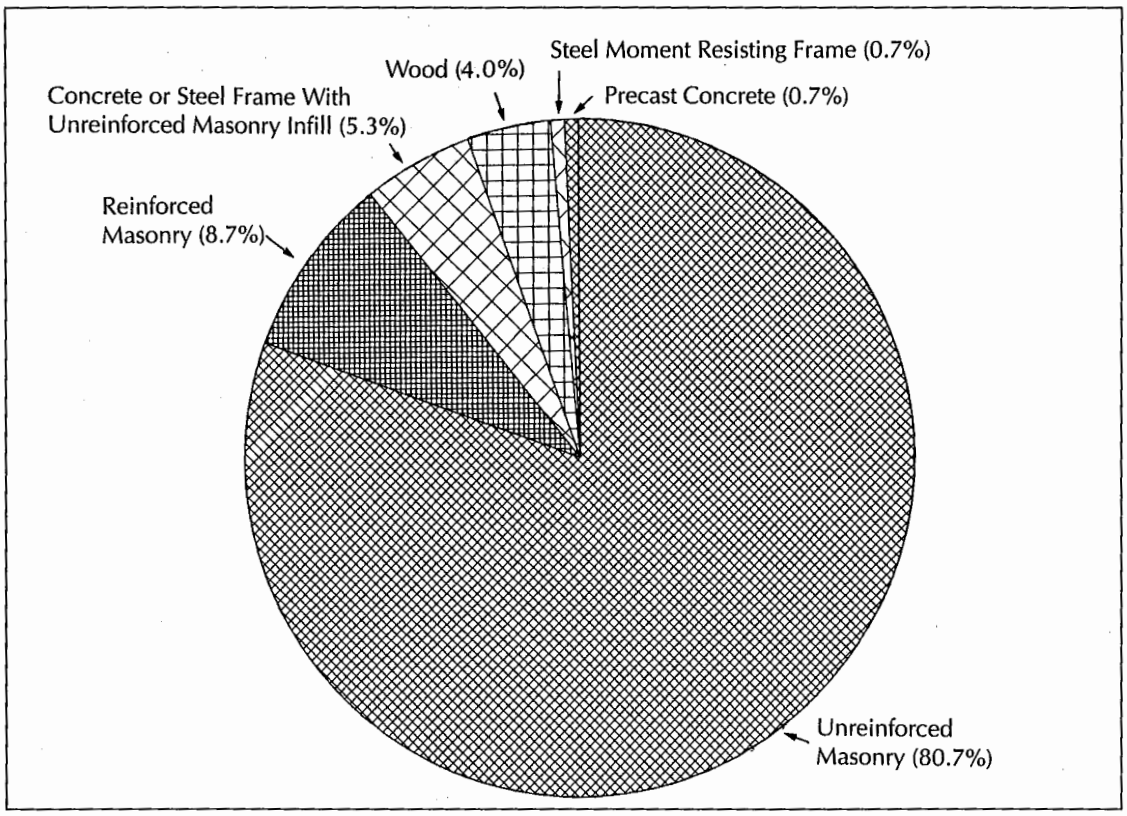


FIGURE 8. Distribution of construction types for fire stations in Middlesex County.

phragm sections have significant tensile capacity to resist their pulling apart in these areas, they will tear. The deficiency is in the strength of the diaphragm in the area of the corners.

Masonry Wall Anchors. Since finish carpentry covered the diaphragm to wall connections, the strength of those connections (if there is any) is unknown. Three built-up steel beams supporting the second level floor were attached to the wall with anchors (noticed from the exterior). The cover plates on these anchors are clearly too small to maintain stability of the normal walls during a transverse loading.

Anchor Spacing. The wall anchors present are spaced at approximately 13 feet. This spacing is far in excess of the four feet recommended in the FEMA guidelines. This lack of sufficient anchors can lead to partial collapse of normal walls when exposed to a transverse loading.

Lateral Support. The gable walls at the building ends could be laterally unstable. Height to thickness ratios of these gable walls is approximately 14:1, assuming the walls are one foot thick. Since no access was gained to the attic, the nature of how (or even if) these walls are laterally supported is unknown.

Anchorage. The gable walls and elaborate cornices present along the building's roof line are considered to be insufficiently anchored to the structural system. Again, since access to the connections (or absence thereof noted) was not gained, confident evaluations could not be made.

Unreinforced Chimneys. Two unreinforced chimneys, one of which is in very poor condition, extend above the roof level. These chimneys are non-structural falling hazards.

It is evident through this evaluation, that this building has numerous deficiencies in resisting lateral loads. The more critical of these deficiencies are a weak first story, torsion, shear strength and flexural strength of the walls.

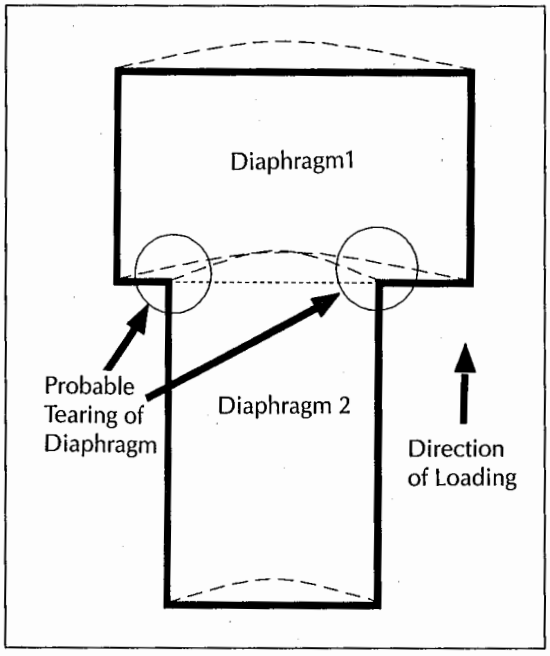


FIGURE 9. Diaphragm deflections at re-entrant corners.

These deficiencies could be remedied by the following:

- Tying a moment resisting steel frame to the inside of the front wall (and floor diaphragms) to resist lateral loads in the transverse direction. By doing this, the stiffness and shear strength of the lower story will increase, eliminating the weak story and shear stress deficiency, and the center of rigidity of the structure will shift forward, thus eliminating the torsion problem.
- Strengthening the floor diaphragms in the re-entrant corners, thereby eliminating the plan irregularity problem.
- Strengthening diaphragm-to-wall connections by adding more closely spaced and larger masonry wall anchors, thereby stabilizing walls with large height to thickness ratios and providing a continuous load path to the foundation.
- Strengthening the walls. One possible system is described in the section below. It involves bonding reinforcing to the wall surfaces. This system improves both the in-plane and out-of-plane strength of the walls.
- Bracing any gable walls and parapets. The bonding method mentioned just above would be applicable. This alternative would not only be effective, but also be cost efficient to building owners in the Northeast.

An Alternative Wall Strengthening System

Although many wall strengthening techniques are currently practiced in the Western states, their cost would seem unjustifiable in the minds of many building owners in the East. Methods such as bonding steel reinforcement to walls with shotcrete tend to physically detract from the appearance of the masonry, add considerable mass to a building and are stronger than what is probably justifiable in the East.

A lighter reinforcing system that would be cost effective, less obtrusive and still maintain the structural integrity of a building in the event of an earthquake needs to be developed. One such system could be the bonding of thin polymer films or filaments with epoxy to the face of the masonry wall. This concept grew out of the laboratory testing of plastics that demonstrated potential for high strength and large ductility, and a subsequent literature review on the use of polymers and fiber-reinforced plastics.

In order to determine whether the process of externally bonding reinforcement to a building's walls will eliminate their stability deficiency, the out-of-plane bending moments and tensile force resultants within the walls were evaluated. Different types of reinforcement materials were then investigated to determine whether they were capable of resisting these tensile resultants.

The out-of-plane lateral forces on each individual pier in the first story of the fire station were calculated according to the following UBC equation:[5]

$$F_p = ZIC_pW_p$$

Where:

F_p = The total lateral seismic force on an element in lbs

Z = Seismic zone coefficient, 0.15 for Massachusetts (Zone 2A)

I = Importance factor, 1.25 for an essential structure

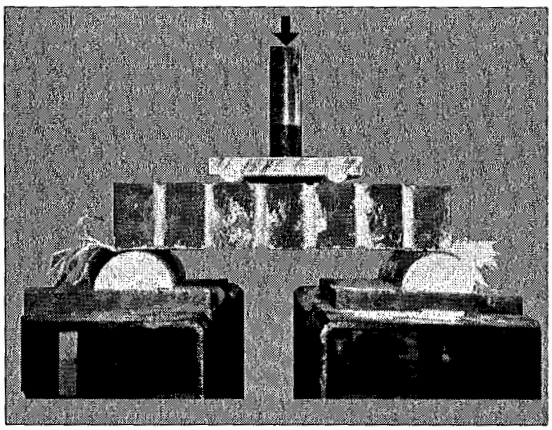


FIGURE 10. Flexural beam test setup.

TABLE 1
Flexural Beam Test Results

Reinforcement	Load (lbs)	F_g (psi)
Polyvinylbutyral	467.29	74.52
Glass Fiber—2-in. Mesh	1,925.86	293.31
Glass Fiber—2.67-in. Mesh	1,887.85	287.61
Carbon Fiber	705.92	110.30
Monofilament	614.13	96.55
Plastic Straps	335.20	54.71
Unreinforced	292.21	48.26

C_p = Horizontal force factor equal to 0.75
W_p = Weight of element or component (lbs) is equal to 120 lbs/ft^2 times the effective load width times height (effective load width is the width of wall whose weight is tributary to the pier in question)

Conservatively assuming each pier to act as a uniformly loaded simple span beam, the bending moments caused by these forces were determined to range from 10,283 to 132,891 in-lbs, with an average of 39,868 in-lbs (excluding the pier between the large door openings). Next, assuming that the masonry itself possessed no tensile strength, the resulting tensile force that was assumed to act at the face of the wall (where the reinforcement would be applied) was determined for each pier. The calculated forces ranged from 27.5 to 71.4 lb/in, with an average of 59.7 lb/in. Any acceptable reinforcing scheme must be capable of developing these forces.

An experimental program was developed to determine whether the external bonding of reinforcing materials would increase the flexural and diagonal tensile strength of unreinforced masonry walls. The materials studied were polyvinylbutyral film, glass fiber, carbon fiber, monofilament fishing line and plastic packaging straps. These materials were chosen mainly because they were readily available and/or inexpensive, added only 0.03 inch to the thickness of the walls (and correspondingly little weight), and could be applied to a building while it was still in use. These alternatives were bonded to the face of the masonry with a two-part epoxy.

Flexural beam tests were performed to determine the effect of the reinforcing on the flexural strength of the masonry. The flexural beam test was used because of its versatility and ability to produce data with little effort and cost. This test is, potentially, a simple and economical representation of the behavior of walls subjected to out-of-plane lateral loadings. The flexural beam test was conducted in accordance with ASTM E518-80.[7] The specimen was placed horizontally across a 15.75-inch span so that the reinforced face would be placed in tension (see Figure 10). A third point load was applied to the specimen manually with a hydraulic actuator until failure.

The gross area flexural tensile bond strength of the specimens was calculated using:[7]

$$F_g = ((P + 0.75P_s)L)/bd^2$$

Where:
F_g = Gross area flexural tensile bond strength
P = Maximum total applied load
P_s = Weight of the flexural beam specimen
L = Span length
b = Average width of the specimen
d = Average depth of the specimen

Average values of applied load and gross area flexural tensile strengths obtained from testing the flexural beams are presented in Table 1.

An average flexural bond strength of 74.52 psi was observed for the polyvinylbutyral film reinforced specimens. Although more signifi-

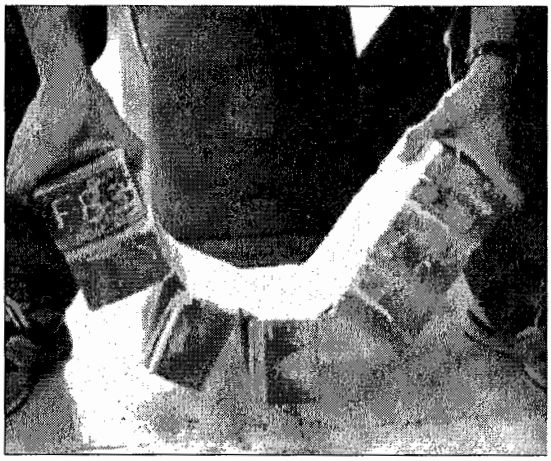


FIGURE 11. Failed polyvinylbutyral reinforced specimen.

cant increases in strength were observed with other reinforcements, the great post-cracking ductility imparted by the film is worth special note. The specimen failed successively in three joints. It was obvious that all the tensile force in the specimen was being developed in the film. Even though complete failure of the mortar to brick bond was observed, the polyvinylbutyral film held the specimen intact, as illustrated in Figure 11.

Flexural cracks were observed in the mortar joints of the two-inch-spaced glass fiber reinforced specimens at a load level of approximately 300 lbs, corresponding to 49 psi. These specimens ultimately failed in shear at their first joint before failing in bending at midspan. Even though less reinforcement was present in the 2.67-inch-spaced glass fiber reinforced specimens, their flexural bond strength was approximately the same as that of the two-inch-spaced specimens. Since all glass fiber reinforced specimens failed in shear outside of the middle third of the span, it was concluded that the amount of reinforcing had little effect on the strength of the beam. It was felt that the masonry failed in shear before the true flexural capacity of the specimens was obtained.

As the shear and moment diagrams in Figure 12 indicate, failure within the middle third of the beam would be caused by bending stresses. Outside this region, the beam is exposed to both shear and bending. The applied load could cause a shear failure in the region before the ultimate moment capacity of the beam is reached in the middle section.

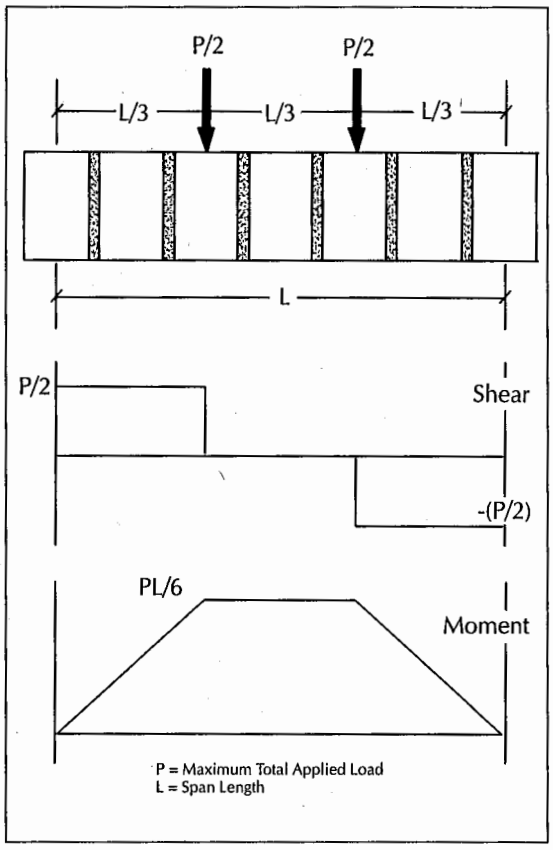


FIGURE 12. Shear and bending moment diagrams of third point loading.

To determine if this was the case for the glass fiber reinforced specimens, the shear strength of the masonry was estimated and compared to the actual shear stress in the specimens at failure. In place of actual testing, Brick Institute of America formulas were utilized in estimating the shear strength of the masonry. Based on the compressive strength of the brick of 10,940 psi (obtained from the manufacturer), the compressive strength of the masonry, f'_m, was determined to be 1,727 psi.

The shear strength of the masonry was then taken as:[8]

$$f_v = \frac{0.5\sqrt{f'_m}}{0.75}$$

Where:

f_v = The shear strength of the masonry (psi)

f'_m = The compressive strength of the masonry (psi)

0.75 = Factor to increase value from the ultimate stress level to the working stress level

The shear strength and corresponding load to cause shear failure were determined to be 27.7 psi and 1,558 lbs, respectively. The experimental results of the glass fiber reinforced specimens indicated applied loads ranging from 1,875 to 1,976 lbs. It is obvious that the shear strength of the specimens was exceeded, thus inducing failure.

The average flexural bond strength of the carbon fiber reinforced specimens was determined to be 110.3 psi. Brittle failure of the specimen was observed within the interior third of the span.

Neither the monofilament line nor the plastic strap reinforced specimens were able to develop their ultimate strengths. Poor bonding between the reinforcement and the masonry allowed the material to "peel" from the brick. If better bonding agents are discovered, these two materials could prove effective.

The increase in strength due to the various reinforcing methods ranged from 37 to 537 percent. It is obvious that the glass fiber showed the greatest increase in flexural bond strength. Although the polyvinylbutyral reinforcing increased the strength only 66 percent, it was able to contain the masonry after failure (as was the glass fiber reinforcing).

An analysis was conducted to convert the tensile forces in unreinforced masonry walls due to out-of-plane bending (calculated for the fire station earlier) to a flexural beam load that would produce the same magnitude of force. Once calculated, these flexural beam loads were compared to those obtained through testing of the reinforced specimens, to determine which, if any, reinforcement scheme(s) will produce the flexural capacity needed for seismic retrofit.

A free body diagram illustrating what occurs at failure in the beam specimen due to third point loading is presented in Figure 13. In order to simplify the analysis, the effects of the self-weight of the specimen as well as the weight of the loading attachment were neglected. Also, the mortar was assumed to have no tensile strength, a commonly made assumption in the analysis of masonry structures.

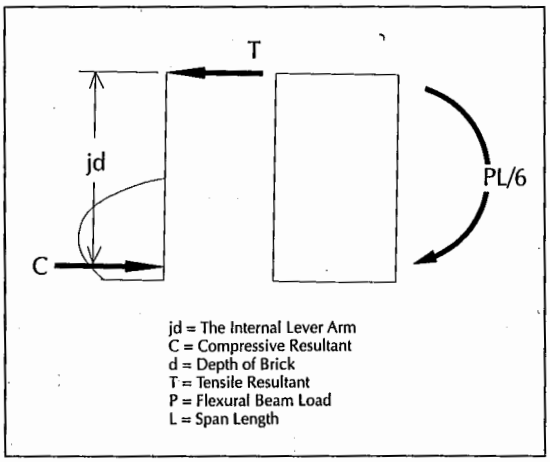


FIGURE 13. Free body diagram of a flexural beam specimen.

An equilibrium condition was obtained by summing moments about the compressive resultant, *C*. Assuming that $j=1$, the flexural beam load, *P*, required to produce the resultant tensile force in the beam, *T*, was calculated using:

$$P = (6Td)/L$$

Where:

P = Equivalent flexural beam load, lbs
T = Width of flexural beam (7.5) times the out-of-plane bending force, lbs per inch calculated earlier
L = Span length of the flexural beam, inch
d = Depth of the brick specimen, inch

These calculated flexural beam loads ranged from 275.4 to 765 lbs. A value of 1,960.7 lbs (corresponding to the load level in the pier between the large door openings) was not considered to be critical. If a complete retrofit of this structure were undertaken, a different strengthening procedure, such as the implementation of a steel frame, could and probably would be used to retrofit the wall containing this pier.

As seen in Table 1, the reinforcing of specimens with glass fiber spaced at both two inches and 2.67 inches, with flexural beam loads ranging from 1,875 to 1,976 lbs, were the only alternatives that were adequate to develop the required strengths with a factor of safety on the order of two.

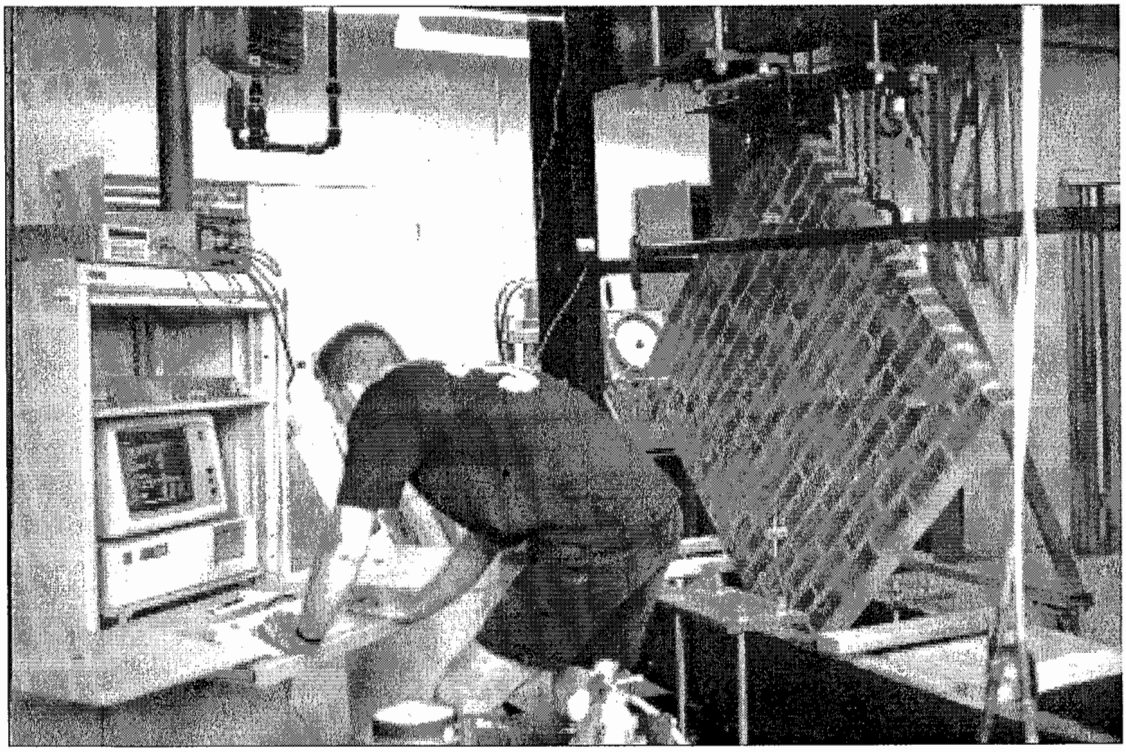


FIGURE 14. Diagonal tension test setup.

The flexural beam tests addressed walls subjected to out-of-plane loading. Diagonal tension tests were also conducted to investigate the resistance of walls to in-plane loading. It is a deficiency in a combination of this diagonal tensile strength and shear along horizontal bed joints that leads to the failure of a wall exposed to an in-plane lateral loading.

These tests, shown in Figure 14, were conducted in accordance with specifications set forth in ASTM E519, where a four-foot square panel is loaded to failure along one of its diagonals.[9] Tests were performed on four test panels — two unreinforced, one reinforced with a two-inch glass fiber mesh (laid up by hand with glass fiber roving) and one reinforced with polyvinylbutyral film. As a result of previous flexural testing, the glass fiber reinforcement was chosen for its strength, while the polyvinylbutyral film was chosen for its ductility.

A common interpretation of the diagonal tension test results was made. The average shear stress at diagonal tension failure, considered to be the component of the applied load parallel to the bed joint divided by the cross-sectional area of the bed joint, was calculated. Table 2 contains the values of the load and the average shear stress corresponding to failure.

Upon failure of the unreinforced specimens, each "exploded" into two large pieces with a loud noise. A large difference in ultimate strength was observed between these two specimens. Since each was constructed by a different person, one an experienced mason and the other a novice, this difference in strength was attributed to a difference in workmanship (the stronger of the specimens was constructed by the experienced mason). This assumption lends support to the significance of the masonry joint deficiency noted earlier.

Failure of the glass fiber reinforced specimen, also accompanied by a loud crunching noise, was initiated at the center of the panel and propagated along the vertical diagonal. Despite the fact that some of the glass fiber broke across the plane of the crack, enough fiber remained intact to hold the failed pieces of the panel together. It is also interesting to note that as load was applied to the panel, cracks became clearly visible on its unreinforced side.

No cracks became visible on the reinforced side of the panel until failure was observed. This differential movement between the two sides of the panel caused an outward bow or bend in the wall. Ultimately, if this system is used as a retrofit technique, the reinforcing may be put on both faces of the wall to support the tension developed on the two faces due to the cyclic nature of seismically induced loads.

Failure of the polyvinylbutyral reinforced panel, similar to that of the unreinforced specimens, initiated at the lower loading shoe and propagated upward along a bed joint. Since failure did not initiate in the center of the panel, it is hypothesized that failure was not due to diagonal tension, but rather to bed joint shear failure, originating at a joint where the load shoe bears on the panel. At failure, the polyvinylbutyral film was able to hold the broken portions of the wall together as a single unit. Although the film was in a plastic state at the time, it still managed to prevent the total collapse of the panel.

Both reinforcement methods were seen to increase the shear capacity of the masonry. When compared to the required increase in shear necessary for retrofit of the evaluated building (60 psi — calculated to be 1.5 times the average shear stress in the walls) both reinforcing schemes were found to be adequate.

TABLE 2
Diagonal Tension Test Results

Reinforcement	Load (lbs)	Shear Stress (psi)
Unreinforced	4,835	18.99
Unreinforced	21,478	84.36
Glass Fiber	24,325	95.50
Polyvinylbutyral	24,525	96.33

The Future

Taking into consideration the age of most of the fire stations that were screened, the limited maintenance that many of them have received, and the fact that 121 of them are constructed of unreinforced masonry, it can be concluded that 80 percent of the fire stations in Middlesex County are likely to be damaged in the event of a significant earthquake.

As shown through the experimental program, externally bonded reinforcement in conjunction with conventional bracing and anchoring could be utilized to strengthen the unreinforced masonry fire stations, as well as other unreinforced masonry buildings. An added observed feature of some of the bonded reinforcements was their ability to contain the bricks after failure. Since falling bricks pose a large threat to life-safety and property, this is a beneficial attribute. Containment of the bricks also results in a higher level of overall building integrity, therefore allowing safer evacuation of the occupants of damaged buildings.

Due to limitations of both the flexural beam and diagonal tension test procedures, further testing should be done prior to adopting the bonded reinforcing system. In the flexural beam tests, shear failures were sometimes occurring before flexural failures, which resulted in underestimating possible increases in the flexural strength of retrofitted walls. In the case of the diagonal tension tests, it is doubtful that the test is a good representation of the actual behavior of a wall subjected to seismically induced in-plane loads. In both cases, larger specimens with more realistic boundary conditions and loading are needed.

The practice of rapid visual screening of essential facilities in Massachusetts continues. Funding has been obtained from the Massachusetts Emergency Management Agency to continue this work during the summer of 1993. A broader range of essential buildings over a larger geographic region are being examined. Schools, hospitals, fire stations, emergency operating centers and emergency shelters in a distribution of communities in Middlesex, Norfolk, Essex and Suffolk counties are being screened, with an estimated total number of structures being examined as high as 300. Even though that is a relatively small number compared to the total number of essential buildings in the state (which is estimated to be in excess of 3,500), valuable information on the general character of these buildings is being gathered.

Once collected, these data will be reviewed with the goal of identifying common problems among the different types of essential buildings. With or without common problems, cost

studies for retrofitting "typical" essential buildings could begin using the data from the screenings as a planning tool for that study.

This work represents an initial effort. Much more work needs to be done. It is hoped that this study will help promote further work towards reducing seismic risk in Massachusetts.

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REFERENCES

1. *The History of New England from 1630 to 1649 by John Winthrop, Esq.*, by James Savage, Reprinted Edition 1972, Arno Press, New York, NY, 1972.

2. *EERI Newsletter*, Volume 27, Number 5, Earthquake Engineering Research Institute, Oakland, CA, May 1993.

3. "National Earthquake Hazards Reduction Program; Five-Year Plan for 1992-1996," Federal Emergency Management Agency, United States Geological Survey, National Science Foundation, National Institute of Standards and Technology, Washington, DC, September 1991.

4. "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook," FEMA-154, Federal Emergency Management Agency, Washington, DC, July 1988.

5. "Uniform Building Code, 1988 Edition," International Conference of Building Officials, Whittier, CA, 1988.

6. "A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)," FEMA-178, Federal Emergency Management Agency, Washington, DC, June, 1989.

7. "Standard Test Methods for Flexural Bond Strength of Masonry," ASTM E518-80, American Society for Testing and Materials, Philadelphia, PA, 1991.

8. *An Educational Syllabus on Structural Masonry*, International Masonry Institute, Washington, DC, 1984.

9. "Standard Test for Diagonal Tension (Shear) in Masonry Assemblages," ASTM E519-81, American Society for Testing and Materials, Philadelphia, PA, 1991.