Earthquake Resistance

OLD STATE HOUSE
BOSTON, MASSACHUSETTS

January 1980

consulting engineers

Simpson Gumpertz & Heger Inc.

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16. Abstract (Limit: 200 words)
The purpose of this study was to obtain bounds for the ground acceleration experienced in the vicinity of the Old State House in Boston during the Cape Ann Earthquake of November 1755. To achieve this objective, the earthquake resistance of the Old State House was estimated, and from this resistance the base accelerations that would cause no cracks or damage were calculated. Described are the structure and material properties of the building and its seismic response characteristics. The building's overall seismic behavior; motion of masonry walls, roof and floor diaphragms; and peak ground acceleration were analyzed. It was concluded that the most probable effective base acceleration in the vicinity of the Old State House did not exceed the range of 5-10 percent of gravity. The peak ground acceleration probably did not exceed the range of 2.5-5.5 percent of gravity. A lower bound on the Cape Ann Earthquake could not be established because of reports that the building was not damaged.

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A B S T R A C T

The purpose of this study is to obtain bounds for the ground acceleration that was experienced in the vicinity of the Old State House during the Cape Ann Earthquake of November 1755. The approach taken to achieve the purpose is to estimate the earthquake resistance of the Old State House, and from this resistance to estimate the base accelerations that would cause no cracks and those which would cause no damage.

The Old State House is a three-story plus partial basement masonry building with wooden roof trusses, floors, and interior columns. The masonry walls are made of common brick, laid in English bond, set in a lime mortar. These walls have a regular distribution of windows in the first and second floors, with full height piers between windows. The long side walls and the end gable walls are modeled as plane frames, with the masonry supported on the wood lintels over the windows taken as beams, and the piers as columns. The floors are taken as diaphragms, while the roof sheathing is considerd as either an effective or an ineffective diaphragm. Static equivalent lateral forces are applied, using both constant and linear distributions of acceleration with height. Several frame models, with different assumptions of beam effectiveness, are analyzed.

The results show that the building is weakest in the transverse direction. Earthquake inertia forces in this direction are carried by the two gable end walls with in-plane forces, and by the long side walls in out-of-plane bending; in addition, the flexible roof affects the transverse resistance more than it does the resistance in the long direction. The Old State House has a capacity to resist effective base accelerations in the transverse direction in the range of 3% to 12% of gravity. The lower bound corresponds to first cracking of beams and the upper bound corresponds to minor cracking of beams and piers, assuming the material properties and the connection details at their highest level.

The conclusion of this study is that the most probable effective base acceleration in the vicinity of the Old State House did not exceed the range of 5% to 10% of gravity. The peak ground acceleration probably did not exceed the range of 2.5% to 5.5% of gravity. A lower bound on the Cape Ann Earthquake cannot be established from this study because of the reports that the building did not suffer any damage.

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I. INTRODUCTION

1.1 Background

The earthquake requirements contained in the current Massachusetts State Building Code are based, to a large extent, on seismic risk studies performed for the New England region, in particular for Massachusetts.

Seismic risk studies in New England can be performed using approximately 300 years of historical data. Most of the information available is in the form of historical records of damage caused by earthquakes. There are no ground acceleration records available for any major earthquake. Historical information obtained from accounts of the earthquakes are used to obtain estimates of the magnitude and of the epicentral intensity of past earthquakes. The damage data is also used to draw maps showing the regions that suffered different levels of intensity. These isoseismal maps, generated using historical data, are the basis for the best current estimates of epicentral intensity and magnitude of historical earthquakes.

The largest earthquake to hit Boston occurred in November of 1755, west of Cape Ann. This earthquake has been studied by many researchers. A recent major study by Weston Geophysical Research, Inc., collected and interpreted a large number of documents available both in the U.S. and in England on the Cape Ann earthquake. Weston Geophysical Research, using these historical accounts of damage, then prepared isoseismals for the Cape Ann earthquake for intensities 4, 5, 6, and 7; in addition, they estimated Modified Mercalli Intensities for all cities for which damage accounts are available. Even though the Cape Ann earthquake has been studied exhaustively using historical data, there is still much controversy about the magnitude and epicentral intensity of this earthquake as well as about the level of ground accelerations that it caused in Boston.

The research project "Deducing Ground Motion Parameters by Analysis of Contemporary Construction with known Damage: The 1755 Cape Ann Earthquake" that is being carried out at MIT, is the first attempt to estimate the epicentral intensity of an historical earthquake using analytical techniques. The idea is conceptually simple: given the knowledge that a building survived the Cape Ann earthquake of 1755, and given historical records of damage or no damage for the building during the earthquake, it must be possible to estimate the ground acceleration that shook the building. This report attempts to do just that using the Old State House as a reference building.

To obtain the sought estimate of site ground acceleration, much information on the building has to be obtained from historical documents. The primary information that must be collected and analyzed, is as follows:

- Records of damage to the building during the earthquake
- Records of the structural configuration of the building at the time it was subjected to the earthquake
- Records of methods and materials of construction used at the time
- Records of local soil conditions.

The information thus assembled, with its inherent uncertainty, is then used to develop a structural model of the building, to perform a structural analysis, and to predict the levels of base acceleration which are compatible with the reported damage.

1.2 Purpose

The immediate purpose of this study is to obtain bounds for the ground acceleration in Boston in the vicinity of the Old State House that was experienced during the Cape Ann earthquake of November 1755. The ultimate objective of the research project, of which this study is a part, is to obtain better estimates of the epicentral intensity of the Cape Ann earthquake and hence to provide more reliable data to update the earthquake design requirements in Massachusetts.

1.3 Scope

The scope of work of this study is:

- Review historical records to determine damage levels of the Old State House experienced during the Cape Ann earthquake of 1755.
- Review available information on the history of the Old State House.
- Obtain a best estimate structural configuration of the Old State House, using information available for it as well as information available for buildings of a similar construction built in the mid-eighteenth century.

- Review historical records on methods of construction.
- Determine the probable range of material properties used in the construction of the Old State House.
- Perform approximate structural analyses of the best estimate structural configuration.
- Obtain best estimates for the base accelerations that would cause no damage to the building, and those which would result in some damage to the building.
- Perform sensitivity studies, primarily by varying the estimates of material properties, to obtain a reasonable range for the estimates of base acceleration.
- Present the findings obtained in a report.

1.4 Authorization and Acknowledgement

The work performed for this study was authorized by MIT Purchase Order No. SR103102. This study is a part of the larger project entitled "Deducing Ground Motion Parameters by Analysis of Contemporary Construction with known Damage: The 1755 Cape Ann Earthquake" that is being carried out at MIT under the sponsorship of the National Science Foundation.

Professor Robert V. Whitman is the principal investigator of the project, assisted by Professor James M. Becker. We are grateful for the assistance and direction provided by Professors Whitman and Becker. Ms. Catherine A. Bush and Ms. Hinghman Chan were research assistants for the project. These students performed in-depth studies of the methods of construction used for early American wooden and masonry houses. Their contributions were instrumental in defining the condition of the Old State House just prior to the year 1755.

Ms. Sarah Chase of the Society for the Preservation of New England Antiquities has assisted us with information on the Old State House and on other buildings of the same era. This information proved very useful to determine the most probable structural condition of the building the way it was in the year 1755.

Stahl Associates, Architects, performed renovations to the Old State House in 1974 for the Public Facilities Department of the City of Boston. Access to the drawings by Stahl Associates is acknowledged.

2. DESCRIPTION OF THE STRUCTURE

2.1 Sources of Information

Our primary sources of information for this project are listed below.

a. Damage information.

The best available source for damage information is "Historical Seismicity of New England" prepared by Weston Geophysical Research, Inc.

Professor Whitman, in private communications, has also stated that the Old State House suffered no damage during the earthquake.

b. Information about the structural components of the Old State House.

Several students at the Massachusetts Institute of Technology, working through the MIT Undergraduate Research Opportunities Program, researched the condition of the Old State House in the year 1755 as part of a study of construction in Boston in that period. The report, "Residential Construction in Boston at the Time of the Cape Ann Earthquake of 1755," by Catherine A. Bush, gives much useful information on methods of construction used at the time. Ms. Hingman Chan performed a study of the Old State House to obtain and collect information on the structural configuration of the building, in particular, member sizes and their connections.

Drawings of the Old State House, prepared for renovation work by Stahl/Bennett Inc. in 1974, show a recent configuration of the building. Many details about the Old State House, and about construction methods and details used in the 1755 period were obtained from Ms. Sarah Chase of the Society for the Preservation of New England Antiquities. The drawings, together with the historical information supplied by Ms. Chan and Ms. Chase, was used to backfigure the condition of the Old State House in 1755.

In addition to the information provided by others, we made a field trip to the Old State House. During this trip a few measurements were taken and other questions were resolved. We did not, however, have access to most of the structure.

2.2 History of the Old State House

The first Boston townhouse was apparently built in 1658. This townhouse was a wooden structure of the half-timbered type so commonly associated with English buildings of the medieval period. In 1711 it was damaged by fire to an extent such that it had to be entirely replaced. This fire was probably part of the "Cornhill" fire of 1711 which destroyed some 100 buildings in the City of Boston. The replacement building was a brick masonry, load-bearing-wall, building with interior wooden floors, columns, and trusses. The type of masonry structure selected for the replacement building was part of the architectural trend towards a more fireproof form of construction in the increasingly densely populated center of the City of Boston. This masonry building was often referred to as the State House.

When this building was gutted by fire in the year 1747, it was rebuilt using the still-standing brick walls. Although no plans or construction drawings exist describing the configuration prior to the 1747 fire, there are available written descriptions which allow a determination of the floor plans.

When the earthquake occurred in 1755, the building configuration was that of the renovation after the fire in 1747. This is the configuration of most interest for this study, and is therefore the one to which we have devoted most attention.

In the 200-year history since the earthquake, many modifications of the Old State House were made; however, subsequent modifications, performed for historical preservation reasons, restored the building close to its original configuration.

The history of the Old State House, over a period of more than 300 years, is quite fascinating. Not only does it give a glimpse of the mood of the time, but it also serves as an eye-opener to any engineer who wants to design renovations of an old existing building. Literally every building goes through multiple renovations throughout its history, and it is important to associate the strength of building components with the period during which they were built.

2.3 Description of the Structure

The Old State House is a three-story plus partial basement masonry building. Approximate plan dimensions are 36 ft-4 in. x 112 ft-7 in. Overall height, excluding the central tower, varies from about 51 ft on the west side to 57 ft high on the east side. A central tower that tapers back in three steps extends about 70 ft above the attic floor; the plan dimensions at the base of the tower are about 15 ft x 15 ft, and the first taper has a plan of about 11 ft-6 in. x 11 ft-6 in. In the current configuration, there is a basement, a first floor, a second floor, and an attic; in 1755, however, most of the basement was probably an unfinished crawl space, and the attic had not been finished.

The masonry exterior walls that form the long side of the building have a regular distribution of windows in the first and second floors, with piers in between, and with an entrance door in the middle of the wall. The masonry walls are made of common brick, laid in English bond, set in a lime mortar. The piers between windows are 4 ft-6 in. wide with a thickness of 24 in. in the first floor and 20 in. in the second floor. The windows between the piers are also 4 ft-6 in. wide; an 8-in. thick masonry fill wall that lies between first and second floor windows is supported by wooden lintels. Each window sits approximately 4 ft-6 in. above the floor level; this is therefore also the height of the masonry fill wall.

The end walls, or transverse walls, are also masonry walls of pier construction. Each end wall is a gable-type wall that extends above the second floor to the attic floor level. The gable portion above the second floor is 16 in. thick. Each end wall has three windows per floor, except that the west elevation has a door entrance at the center of the first floor instead of a window, and two 30-in.-diameter eye windows. In the attic region there is a central window in each of the two end walls.

The foundations for the exterior masonry walls were 28 in. wide and consisted primarily of field stone packed with clay below grade, and set in lime mortar above grade. Most of the original foundations have been replaced during the construction of the subway, and no accurate information is available on the original foundations.

Floor and roof construction is all made of wood. The first and second floors have a similar type of construction. Floors consisted of two orthogonal layers of one-in. thick boards, each board being about 6 in. wide. The floor spanned a 21-in. space between the 4-in. x 5-in. wooden floor joists. Joists are supported on floor girders, or on one floor girder and the end gable wall. The end joists were set in pockets in the end walls; all other joist ends sit in mortised slots cut in the 12-in. x 13-in. floor girders. The girders span from the long side wall piers to central wood columns. These columns extend from the basement up to the second floor level. Since the roof trusses span between longitudinal walls, the second floor is open, with no interior columns. The columns, as well as the first floor girders, rest on foundation piers. There are eight such piers along the center of the building.

The roof of the Old State House consisted of 3/16 in. slate over 1 in. sheathing boards. The sheathing spanned between 4 in. x 5 in. deep purlins, spaced about 24 in. on centers. These purlins, in turn, spanned between adjacent roof trusses; these trusses were probably spaced about 9 ft-10 in. on center. The end spans, adjacent to the gable end walls, were slightly larger.

The roof trusses, which give the roof its double-pitch configuration, have a double-rafter design. The top chord or upper rafter has a 10 in. x 10 in. cross section. The top chord is notched to receive the purlin ends. The lower rafter runs almost parallel to the upper rafter and is connected to it by several struts. This lower rafter rests on the lower truss chord member, which has an 11-in. x 11-in. cross section, and on the central king post, an 11-1/2 in. x 12-1/2 in. member. A cross beam ties the upper rafters and the king post at mid-height.

Table 2-1 contains a summary of our best estimate of the type and size of structural components, and of some of the structural connections. The table also contains the source for our information and, where appropriate, a comment on the likelihood of a dimension or detail being as assumed. Uncertainties exist about both the structural components and their connection details. There is, however, more uncertainty in the connection details than there is in the sizes of major components which can be measured. Our best estimate of connections that existed in the building in the year 1755 is as follows. The floorboards and roof sheathing, which was probably 6 in. to 9 in. wide, were nailed with three hand-wrought nails over each purlin and floor joist. Joists and purlins were probably pegged to their respective girders and rafters. Wood members supported on the masonry walls, were set into pockets in the walls, and then grouted in. Metal strap anchors between masonry walls and wooden floor or wooden roof members, commonly seen in existing masonry buildings, were not in use by the year 1755. The detail of the support of the roof truss on the masonry wall piers is quite uncertain; it is likely that the wall was built up on either side of the truss members resting on the wall, and that grout then filled the spaces between the wood and the masonry. Whether this detail would rely entirely on friction, or whether it would somehow engage the masonry is not clear. We have not been able to determine the detail of the connection that would allow for uplift forces to be carried, other than the dead weight of the roof.

Table 2.1

Summary of Structural Component Sizes

Item	Information	Source
ROOF STRUCTURE		
Slate	3/16" thick	Ms. Chase
Sheathing	probably 1"	
Purlins	4" x 5"	Ms. Chan
Trusses	9'-10" o.c.	Ms. Chan
Upper Rafter	10" x 10"	
Lower Rafter	6" x 9"	
King Post	11-1/2" x 12-1/2"	
Lower Beam (chord)	11" x 11"	
Upper Cross Beam	11" x 11"	
Struts	?	
Anchorage to End Walls	– roof purlins in pockets, mortared in – floor joists in pockets, mortared in	Ms. Chase
ATTIC		
Plastered Walls	no	Ms. Chase
Ceiling	(was probably done in 1773)	
Floor Finished	no	
Subfloor	probably	

Table 2.1 (Continued)

MASONRY WALLS		
Gable End Walls	16" thick brick	Ms. Chan
2nd Story	20" thick brick	Ms. Chan & SGH
Wood Lintels over Windows Brick Facing	English Bond piers 4'-6" wide	inspection
Brick Fill Wall Below Windows	8" thick brick	
1st Story	24" thick brick	Ms. Chan
Wood Lintels over Windows Brick Facing	English Bond piers 4'-6" wide	Ms. Chan
Brick Fill Wall Below Windows	8" thick brick	Ms. Chan
Foundations	fieldstone 2'-4" packed with clay below grade and lime mortar above	Ms. Chan Ms. Chase
FLOORS		
2nd Floor Flooring	1" thick finished floor probably 6" wide	Ms. Chase
	1" thick sub floor	Ms. Chase
Plaster Ceiling	probably	Ms. Chase
Joists (in pockets in walls and girders)	4" x 5" at 21" o.c.	Ms. Chan
Girders	12" x 13"	Ms. Chan

3. MATERIAL PROPERTIES

3.1 Properties of Brick Masonry

There are no tests available from which the properties of the brick masonry in compression, tension, and shear can be obtained. In the absence of such data, we have reviewed tests performed using lime mortar, the type of mortar we assume was used for the Old State House, during the late nineteenth century. Most of the tests we could find were performed at the Watertown Arsenal between the years 1880 and 1890; other tests performed in Germany by Bohme and also by Bauschinger in the years 1870 to 1880 are available, as well as tests reportedly performed in England in the year 1837 (we do not know the name of the researcher who performed these latter experiments).

The first assumption that must be made in estimating material properties for an old existing structure, are the quality of the materials used and the quality of the workmanship. Considering that the Old State House was an important structure at the time it was built, we must assume that high quality brick (high quality for that period), good quality lime mortar, and careful workmanship was used during the construction. Deterioration is normally of concern when estimating the properties of an old existing structure; for this study, however, deterioration is of no concern because we are trying to estimate the properties of the building in the year 1755, which is only eight years after the original construction.

The axial compression of walls and columns, and the shear capacity of walls depends on the compressive strength of brick masonry (normally called f_m'). A series of tests performed at the Watertown Arsenal in 1883 using 12-in. solid Bay State brick piers with 1:3 lime mortar, gave results in the range of 750 psi to 1,300 psi. Experiments performed a year later gave an average ultimate strength of the 12-in. pier of 1,508 psi for a mortar composed of one part lime and three parts sand. In these latter experiments, the ratio of the strength of the pier to the strength of the brick went from 0.06 to 0.18, with a mean of 0.10. The ratio of the strength of the pier to the strength of the mortar was on the average equal to 12. These results are consistent with those obtained in experiments performed in Germany, using a mortar of one part lime and two parts sand; in these experiments a pier strength of 1,290 psi was obtained for ordinary brick with an average crushing strength of 2,930 psi, and a pier strength of 1,620 psi was obtained for selected brick of 3,670 psi crushing strength. It appears unlikely, based on our review of the quoted results and of some other results, that the compressive strength of the brick masonry in the Old State House was less than 750 psi. All of these results, of course, assume that a good quality brick was used. Light red brick, as manufactured by the Boston Face Brick Company, has a crushing strength in the range of 5,900 psi to 8,500 psi. For parametric studies of ultimate stresses in brick masonry, we therefore chose the range of 750 psi to 1,300 psi as a reasonable range.

The tensile strength of brick masonry, and to a lesser degree, the shear strength of brick masonry is normally assumed to depend on the adhesive strength of the brick to the lime mortar. Table 3.1 gives the results obtained in experiments performed in Germany for several lime mortars. The tensile strength of brick masonry is much larger than the adhesive strength. The adhesive strength of brick to mortar must be considered a lower bound to the tensile strength of brick masonry.

To obtain ultimate stresses for the brick masonry of the Old State House that appear reasonable, we computed the allowable stresses permitted by The Brick Institute of America and by the Uniform Building Code, for Type N mortar. Lime mortar is weaker than Type N mortar. In addition, we reviewed several test results used as basis for establishing allowable stresses, and found that the allowable stresses are somewhere between one-third and one-fifth of the ultimate stresses. We therefore took three times, and five times, the allowable stresses given by current codes for assumed compressive strength of 750 psi and 1,300 psi as guidelines. The results are given Table 3.2.

The Uniform Building Code makes a distinction between unreinforced brick masonry that is built using rule-of-thumb thicknesses, and engineered, unreinforced brick masonry, which is that masonry used in buildings designed using the rules of structural analysis, including all types of loads normally used in the design of a building. For unreinforced brick masonry that is not engineered, the allowable stresses given by UBC are much lower. Table 3.3 gives the values acceptable by the Uniform Building Code for non-engineered unreinforced brick masonry. From the results previously quoted for compressive strength of brick units, we believe that the brick in the Old State House lie either in the range of 2,500 to 4,500 psi, or possibly in the 4,500+ psi range.

A comparison of the test results we reviewed, and the results of Tables 3.1, 3.2 and 3.3, led us to select four cases to be used for our parametric studies, Table 3.4. Case I is an unrealistically low bound, since it considers zero flexural tension capacity. This case would be valid if the earthquake duration is long enough to cause cracks in the masonry that

Table 3.1

Test Results for Adhesive Strength of Brick and Hydraulic Lime, psi

Age in Days	Lime-to-Sand Ratio			
	1:1	1:2	1:3	1:4
7	21.0	18.7	15.3	13.2
28	30.4	25.5	20.9	17.5
90	41.9	38.9	28.1	22.6

Table 3.2

Allowable and Ultimate Stresses
in Engineered Non-Reinforced Brick Masonry, Type N Mortar[1]

Type of Stress	Allowable Stress psi		Allowable Stress x 3 psi		Allowable Stress x 5 psi	
Assumed compressive strength of brick masonry	750	1,300	750	1,300	750	1,300
Axial compression of columns	120 60	208 104	360 180	624 312	600 300	1,040 520
Flexural compression of columns[2]	195 98	338 169	585 293	1,014 507	975 488	1,690
Flexural tension normal to bed joint (columns)[2]	19 14		57 42		95 70	
Flexural tension parallel to bed joints (beams)[2]	37 28		111 84		185 140	
Shear[3]	$14 + 0.2 f_c \leq 28$ $8 + 0.2 f_c \leq 28$	$18 + 0.2 f_c \leq 28$ $11 + 0.2 f_c \leq 28$	$42 + 0.6 f_c \leq 84$ $24 + 0.6 f_c \leq 84$	$54 + 0.6 f_c \leq 84$ $33 + 0.6 f_c \leq 84$	$70 + f_c \leq 140$ $40 + f_c \leq 140$	$90 + f_c \leq 104$ $55 + f_c \leq 140$

Notes

1. Each entry in the table contains two values, the top is from BIA, 1969; the bottom is from UBC, 1976. The values are those for "Without Inspection."

2. Flexural tension is independent of compressive strength of brick masonry.

3. f_c is the average compressive stress due to dead load.

Table 3.3

Allowable and Ultimate Stresses

in Unreinforced Brick Masonry, Type N Mortar

	Allowable Stress psi			Allowable Stress x 3 psi			Allowable Stress x 5 psi		
Compresive strength of brick	4,500 plus psi	2,500-4,500 psi	1,500-2,500 psi	4,500 plus psi	2,500-4,500 psi	1,500-2,500 psi	4,500 plus psi	2,500-4,500 psi	1,500-2,500 psi
Axial compression	200	140	100	600	420	300	1,000	700	500
Flexural tension normal to bed joint (columns) (1)	15 7.5	15 7.5	15 7.5	45 22.5	45 22.5	45 22.5	75 37.5	75 37.5	75 37.5
Shear (1)	15 7.5	15 7.5	15 7.5	45 22.5	45 22.5	45 22.5	75 37.5	75 37.5	75 37.5

Note 1.
For flexural tension and shear two values are given. The top is "With Inspection," the bottom is "Without Inspection."

Table 3.4

Ultimate Strength Estimates Used for Parametric Studies, psi

Case	Ultimate Stress 1	2	3	4
Axial compression of columns	300	400	700	1,000
Flexural tension normal to bed joint (column)	0	30	40	50
Flexural tension parallel to bed joint (beam)	0	45	60	75
Shear	$22.5 + 0.8\ f_c \leqslant 100$	$45 + 0.8\ f_c \leqslant 100$	$60 + 0.8\ f_c \leqslant 100$	$75 + 0.8\ f_c \leqslant 100$

decrease the flexural tension to essentially zero. Case 4 is an upperbound, but by no means an unrealistically high bound. Our best estimate corresponds to Cases 2 and 3.

3.2 Properties of Wood and Wood Diaphragms

The species of wood used for the construction of the Old State House is probably white oak. Oak timber is no longer used in American construction; its properties are not listed in currently used handbooks such as The Timber Construction Manual. A number of tests performed by the Forestry Division of the U.S. Agricultural Department in the period from 1880 to 1900, was published in the U.S. Forestry Circulars. Some results for white oak, obtained from the Forestry Circular #15, which summarizes results obtained in the years 1891 to 1896, are reproduced here in Table 3.5.

For reference, allowable stresses for certain species currently used are given in Table 3.6. The allowable stresses published by the American Institute of Timber Construction are based on a factor of safety of approximately 4 against ultimate. A comparison of the white oak strength given in Table 3.5, and the allowable stresses of Table 3.6, shows that oak is about twice as strong as the softwood timber used today.

The publication "Seismic Design for Buildings," published by the Departments of the Army, Navy, and Air Force, gives an allowable shear of 50 pounds per linear foot for 1-in. straight sheathing horizontal diaphragms. If the factor of safety against ultimate is in the range of 3 to 5, the ultimate strength of a horizontal diaphragm with straight sheathing boards is in the range of 150 pounds per linear foot to 250 pounds per linear foot. Tests performed by the U.S. Department of Health, Education, and Welfare on straight sheathed panels, show that the wood and not the nailing determines the ultimate shear of the panel. This conclusion, however, is based on the use of alternate nailing schemes, all of which are good. For example, 8d and 10d nails were tested; and also, two nails and three nails per sheathing board at each rafter were tested.

To evaluate the strength and stiffness of the straight sheathing used in the Old State House, one must consider that the oak used is much stronger than any species in current use, but also, that the hand-wrought nails used at the time may not provide the strength of 8d or 10d nails used today. These two effects lead us to believe that the strength of straight sheathed diaphragms used 200 years ago is similar to that of diaphragms built today. A value of ultimate shear in the range of 150 to 250 pounds per linear foot is the value we adopted for the upper range of capacities of straight sheathing.

Table 3.5

Ultimate Strength of White Oak at Standard Moisture of 12% of the Dry Weight, psi

	Average	Average of highest 10%	Average of lowest 10%	Highest test result	Lowest test result	Fraction within 10% of average	Fraction within 25% of average
Cross-bending test:							
Apparent elastic limit	9,600	14,100	6,100	15,700	4,400	0.37	0.73
Ultimate strength	13,100	18,500	7,600	20,300	5,700	0.39	0.75
Modulus of elasticity	2,090,000						
Crushing endwise	8,500	11,300	6,300	12,500	5,100	0.40	0.81
Crushing across grain at 3% deformation	2,200						
Shearing along the grain	1,000						

Table 3.6

Allowable Stresses for Certain No. I Species in Current Use
Used at 15% Moisture Content, psi

Species	Extreme Fiber Bending, single member	Tension parallel to grain	Horizontal Shear	Compression perpendicular to grain	Compression parallel to grain
Balsam fir	1,250	725	65	170	975
Eastern hemlock	1,650	975	90	365	1,250
Eastern spruce	1,400	800	70	255	825
Northern pine	1,500	875	75	280	1,150
Northern white cedar	1,000	600	65	205	675

In current building practice, straight sheathed horizontal diaphragms are considered as very flexible diaphragms. This type of diaphragm is not acceptable for laterally supporting masonry walls.

Diaphragms with two orthogonal layers of flooring, can develop the shear strength of one of the two layers. Test results given in Table 3.5 show 1,000 psi as ultimate shear strength of white oak. A review of the shear strength of all species of oak, shows that the ultimate shearing strength is in the range of 900 psi to 1,100 psi. We do not have available the distribution of test results for shearing strength, as shown in Table 3.5 for the apparent elastic limit or the crushing strength. We believe from Table 3.5 that a value of 800 psi may be used for floor diaphragms that have two layers. The value of 800 psi must, however, be applied to one layer only. This means that we are using an ultimate shear in the diaphragm of 800 pounds per linear foot. As a comparison, the Seismic Design Manual gives an allowable of 600 pounds per linear foot for two layers of diagonal sheathing at 90° to each other and on the same face of the supporting member; the ultimate shear for two layers of diagonal sheathing is, of course, much higher than the allowable.

In summary, for one layer of horizontal sheathing we are using an upper bound ultimate shear in the range of 150 to 250 pounds per linear foot, and for diaphragms that have two layers of floorboards, one layer orthogonal to the other layer, we are using an ultimate shear value of 800 pounds per linear foot.

4. SEISMIC RESPONSE

4.1 Overall Behavior and Assumptions of the Analysis

The building behaves much as a box structure, with the roof and the wooden floors acting as diaphragms which deliver the loads to the end walls, or to the longitudinal walls, which in turn bring the loads down to the foundation. There is little doubt that the floors acted as very effective diaphragms because of the two orthogonal layers of flooring. There is more uncertainty as to the effectiveness of the roof ceiling to act as a diaphragm. We have assumed the roof to be either effective or ineffective as a diaphragm. The results obtained, of course, are quite different; the real behavior of the roof's sheathing must lie somewhere in between the conditions of no diaphragm and a perfect diaphragm. The results obtained therefore must be considered as lower and upper bounds for the possible response.

The masonry is assumed to behave as a linear elastic material in compression, and as a linear elastic material with a cut-off in tension. Since the reports we have seen indicate that the Old State House suffered no or little damage during the 1755 earthquake, we are interested primarily in the elastic range. The inelastic range, and the ultimate capacity, which are substantially more difficult to estimate in a masonry wall building, are of concern only when predictions of base acceleration are sought that produce a known level of damage.

The base accelerations in any one direction are carried by two load paths that are in parallel. The first one is that normally considered in earthquake analysis of a box structure; loads go from a point in the structure, to a roof or floor diaphragm, to a shear wall, and from there to the foundations and the ground. The other, is the local out-of-plane bending of the walls which are orthogonal to the load carrying shear walls. The out-of-plane capacity of transverse walls is normally neglected relative to the in-plane capacity of shear walls. With thick masonry walls, of the order of 2 ft thick, the capacity of walls to carry some of the earthquake inertia by their own bending strength, while small, is by no means negligible. The relative stiffness of the two load-carrying mechanisms described is not easily determined; a judgment must be made to assign a capacity to the building in excess of that computed based on a box-type behavior, because of the strength contribution of the out-of-plane bending of masonry walls.

Two methods are used to estimate the strength of the masonry elements. In the first method, the masonry is assumed to have a specific tensile capacity. Based on this tensile capacity, the overall strength of the member is then computed. This method can be applied both to beams and to columns. The results of this method correspond to first cracking of the masonry. In the second method, the masonry is assumed to be pre-cracked, and therefore unable to carry any tensile loads. For columns, an equilibrium condition is sought in which both the axial force and the bending moment are carried entirely by compression in part of the cross section. Shear is carried only in the compressed region. The results of this second method of analysis, must be considered an upper bound on the elastic capacity of the masonry members of the building, and are an indication of the capacities which, if exceeded, would cause much damage in the building. The precracking assumed in this method is due either to poor workmanship or poor material quality, or it is a cracking induced by the earthquake shaking. This method cannot be applied to beams, because a pre-compression is required to establish the equilibrium stress state.

The analysis performed is strictly a static equivalent lateral load approach. The acceleration distribution with height is taken first as constant with height, and then as linearly increasing with height. The actual force distribution with height is probably somewhere in between. For overall deformations of the building, the two assumptions of acceleration distribution with height are reasonable to establish acceleration bounds at the base; for local behavior, however, these acceleration distributions are not necessarily valid. In a code type design, the out-of-plane bending of a wall is considered as a "part or portion" of the building with an acceleration that is reasonably high, but independent of the location of the wall in the building. For the analysis of interest in this study, this code-type approach is not valid; we have, therefore, adjusted the out-of-plane accelerations of wall elements in accordance with their location in height in the building.

4.2 Masonry Walls with Out-of-Plane Motion

The longitudinal masonry walls are, from a structural point of view, a series of parallel piers connected by windows, window lintels, and light infill masonry. When the building is vibrating during an earthquake, one mode of vibration is an out-of-plane bending of the walls. A pier, in out-of-plane vibrations, behaves much as a two-span continuous beam. The supports at the second floor level are the 12 in. x 13 in. deep girders; for movement of the wall towards the girder, the girder acts as a positive restraint. For movement away from the girder, the grout in the pocket surrounding the girder restrains the pier by shear. We checked the range of reactions that may be expected at a second floor girder, and concluded that the available shear in the grout is sufficient to consider the girder as a full restraint both for inward and for outward movement.

At the third floor level, just below the trusses, the connection detail is more uncertain. The action of the roof sheathing as a diaphragm is also less determinate. The roof trusses sitting on the wall piers are, because of their dead weight, sufficient to restrain the top of the second floor pier. Whatever strength the grout surrounding the bottom chord of the truss has is added to this restraint. When the roof sheathing acts effectively as a diaphragm, the roof trusses provide support to the piers vibrating out of their plane. To bound the behavior of the piers, we have assumed that either there is a perfect roof diaphragm, and hence, a rigid support for the pier at its top; and, we have also assumed that the roof diaphragm is ineffective, and hence, the second floor pier acts as a cantilever with no restraint at its top.

The 8 in. fill walls between piers behave as one-way slabs for out-of-plane inertia forces. The depth-to-span ratio of these slabs is high, so that these infill walls are not expected to fail in out-of-plane bending before the piers do.

The end walls, in out-of-plane bending, are expected to have a support at each floor level, and also at the roof line. The plane that contains the bottom chord of the roof trusses provides restraint for local behavior because of the bending stiffness of the bottom chord truss members. At the roof line, there is lateral restraint when the roof acts as a diaphragm. In the analysis of the local out-of-plane bending behavior of an end-wall member, three assumptions were used: full, partial, and no restraint provided by the roof.

Selected results for out-of-plane motion are given in Table 4.1. An entry in the table gives the level of base acceleration, expressed as a fraction of **g** (acceleration of gravity), which is required to reach the ultimate capacity of the member for a given set of assumptions of tensile strength and shear strength of the masonry. From Table 4.1 we can see that in the absence of an effective roof diaphragm, the capacities to resist base shear accelerations of the piers of the long side walls lie between 2.8 and 7.8% of g; these same piers, when effectively restrained by a roof diaphragm, can withstand a base acceleration in the range of 16.9 to 70.5% of g. The range of these results is disconcertingly large. From the results obtained for the roof diaphragm, given below, it follows that the roof sheathing, together with the roof purlins and the upper chords of the roof trusses, act as a flexible diaphragm. This flexible type of diaphragm provides to the top of the piers an elastic restraint; even though it is difficult to estimate the stiffness that a roof truss provides at the top of a wall pier, as an elastic restraint, only a small amount of restraint is required to significantly increase the capacities of the wall piers. The results obtained with the assumption of no roof diaphragms, are therefore, an unrealistically low bound; we believe that a more refined

Table 4.1

Capacity Ratios for Exterior Walls
with Out-of-Plane Acceleration, Percent g[1]

Member	Roof as Diaphragm	First Cracking[2]				Minor Cracking
		Case 1	Case 2	Case 3	Case 4	Case 1
Longitudinal walls[3]	No	2.8	6.3	7.4	7.4	7.8
Longitudinal walls[3]	Partial	6.	13.	15.	15.	17.
Longitudinal walls[3]	Yes	16.9	37.3	44.1	50.9	70.5
Gable End Walls[4]	No	4.2	7.5	8.6	9.7	10.4
Gable End Walls[4]	Partial	8.	15.	17.	19.	21.
Gable End Walls[4]	Yes	51.3	83.8	94.6	105.4	117.7

Notes:

1. Based on a uniform acceleration with height.

2. See Table 3.4 for definition of cases 1 to 4.

3. Typical interior pier at the bottom of second floor.

4. Interior pier at the bottom of the second floor.

analysis, including flexible supports, would indicate capacities which are about two to three times those obtained for the piers unrestrained by a diaphragm. The results for long side walls with partial restraint from the roof represent our best estimate for the behavior under consideration.

The range of results obtained for the different material strength assumptions for a certain postulated roof diaphragm behavior, could only be resolved with more accurate descriptions of the damage that was observed during the earthquake. The sensitivity of the results is such that it would be important to know whether no damage means no hairline cracks, or some small amount of visible cracking. The results for pier capacity, obtained for some elastic restraint to the side wall from the roof, give the range of the base acceleration the Old State House might have survived without damage as 6 to 15% g. The end wall piers have a slightly larger out-of-plane bending capacity. The estimated range of base accelerations, based on out-of-plane capacity of end walls is 8 to 19% g.

4.3 Masonry Walls with In-Plane Motion

The masonry walls, for in-plane inertia forces, can be modeled as shear walls with openings, as equivalent frames, or as a set of parallel interconnected piers. The elements that determine the behavior are the beams, specifically, the moment capacity of the beams.

An equivalent frame model of the longitudinal side walls, has two levels of beams. The top row of beams, consists primarily of the wood lintels. While these wood lintels have bending capacity in their length, the bending restraint provided by the masonry in the pockets where these lintels sit in the piers, is quite low. The beams at the second floor level, consist of the wood lintel and the 8 in. fill wall that extends above the lintel and below the second floor window. The bending capacity of these elements depends primarily on the tensile capacity of the lime mortar. An assumption of zero tensile capacity in the mortar makes these beams ineffective; however, because these beams are very deep (length to depth ratio is about 1:1) a small tensile capacity gives the beams appreciable bending resistance. This means that an assumption of even a small allowable tension in the masonry, makes it possible to justify a model of the longitudinal side walls as moment resisting frames.

In a frame model, either the piers or the beams will reach their capacity first when the building is subjected to base accelerations, and will be the governing elements. The analysis shows that the beams reach their ultimate capacity, governed by tension in the mortar, before the piers do. When this occurs, a redistribution of forces must take place in the wall. The level of base shaking that causes internal forces in the beams in excess of their capacity, may be such that it will not cause stresses in the frame, after redistribution of forces, that exceed the column capacities. In this case, exceeding tension strength in a beam will lead to only minor cracking, since the remaining frame members have the strength and stiffness to absorb the loads imposed on them. The capacity of the frame with cracked beams is then of interest. A different situation arises when the level of base shear which causes internal forces in a beam that exceed its capacity, causes internal forces in the remaining members of the frame, after redistribution, that exceed the capacity of the columns. In this latter case, major cracking of the beams must be expected. The level of base acceleration that causes overstress in the beam, is then a measure of the capacity of the wall.

Tables 4.2 and 4.3 give capacity ratios for the longitudinal walls. Table 4.2 is for a uniform acceleration at all levels of the building; Table 4.3 is for a linear distribution of acceleration with height, with the acceleration increasing from the base to the top. Each table contains the results obtained from a model in which the second floor beams are considered effective, and from a model in which these same beams are considered to be cracked. A review of the results of both of these tables shows that the lower bound on base acceleration that would cause column distress is 9 to 10% of **g**. With a uniform acceleration assumption, beams may resist cracking for base accelerations of up to 14% g, and with linear accelerations the beams may resist up to 20% **g** prior to cracking. Should beam cracking occur, however, then the capacity of the piers governs. From a design point of view, no more than 9% **g** ultimate would be allowed for the long side walls; but, in this study, we must accept the possibility that the walls acted with no loss in continuity, and that the base accelerations could have reached 14% **g** without any visible distress to the wall.

The gable end walls can also be considered as shear walls with openings or, as equivalent frames. The gable ends extend beyond the roof level; therefore, in an equivalent frame model, all beams have substantial depth. For the end walls, the assumed tensile capacity of the masonry will also govern the capacity of the idealized beams.

In the event that the beams are unable to carry the moment delivered to them, the end walls behave as a series of four piers in parallel. When applied moments on a beam cause tensile stresses that exceed the tensile capacity, a brittle crack may form in the beam. This brittle behavior of the beams makes a determination of the ultimate load on the end gable walls

Table 4.2

Capacity Ratios for Longitudinal Walls
Uniform Acceleration with Height, Percent g

Member	Minor Cracking	
	Second Floor Beams not Cracked (1)	Second Floor Beams Cracked (1, 2)
Exterior column at 2nd floor	43	—
Interior column at 2nd floor	22	—
Exterior column at 1st floor	27	19
Interior column at 1st floor	23	10

Notes:

1. Lintels at top of wall are assumed to carry no moment.

2. Beam cracking occurs for base accelerations smaller than 14 percent g.

Table 4.3

Capacity Ratios for Longitudinal Walls
Linear Acceleration with Height, Percent g

Member	Minor Cracking: Second Floor Beams not Cracked (1)	Minor Cracking: Second Floor Beams Cracked (1,2)
Exterior column at 2nd floor	32	—
Interior column at 2nd floor	15	—
Exterior column at 1st floor	31	17
Interior column at 1st floor	23	9

Notes:

1. Lintels at top of wall are assumed to carry no moment.

2. Beam cracking occurs for base accelerations smaller than 20 percent g.

uncertain. The bounds are established by the columns, under the assumptions of effective and ineffective beam behavior. The capacity of the gable end lies within these bounds.

Tables 4.4 and 4.5 give the capacity ratios for members in the gable end walls obtained for a uniform acceleration with height distribution, and with a linear acceleration with height distribution. Both Tables 4.4 and 4.5 indicate that the weakest elements are the second floor beams, which are expected to crack at base acceleration levels of about 3% g. If, contrary to the results obtained for the beams, we assume that the frame works as a unit, then the results of Tables 4.4 and 4.5 imply that the exterior columns extending from the second to the third floor, would crack at the top at base acceleration levels of about 4 to 6% of g. Such cracking is not considered serious, because it would lead to a redistribution of moments to the bottom of the column. The results for the exterior column extending from the second to the third floor, after redistribution has occurred, and further assuming that the beams allow this behavior to take place, are shown as the bottom line on Tables 4.4 and 4.5. The results, for the stated assumptions, and after redistribution has occurred, show that the piers of the gable end walls can resist base acceleration levels in the range of 11 to 16% of g.

Table 4.6 shows the capacity ratios for the gable end walls after the beams have cracked. The results are a lower bound for the wall because the model assumes that all beams are cracked. If cracking of beams occurred at a level of base acceleration of about 3% g, then cracks may be expected at exterior columns of a gable end, at the first floor, for base accelerations of about 6% of g.

The governing beams are on the second floor. A first crack in a second floor beam does not necessarily imply progressive cracking of all beams. The third floor beams experience bending moments which are about one-half to one-third those that occur in the second floor beams. The second floor beams might all develop cracks while the third floor beams are intact. A frame with intact upper story beams and with cracked lower story beams will develop bending moments in the columns or piers that are within the upper bounds for piers given in Tables 4.4 and 4.5 and the lower bounds for piers given in Table 4.6. The results of Tables 4.4, 4.5 and 4.6, are all based on a model that delivers the entire inertia force to the end gable walls. In this model there is no contribution from the long side walls, acting in out-of-plane bending. Very approximate calculations for this effect show that the long side walls may carry somewhere between 2 and 5% of gravity in an out-of-plane bending mode. Whichever number is accepted within this range, would be additive to the results given in Table 4.6.

Table 4.4

Capacity Ratios for Gable End Walls
Uniform Acceleration with Height, Percent g

Member	First Cracking				Minor Cracking	
	Case 1	Case 2	Case 3	Case 4	Case 1	Case 4
Exterior column at 2nd floor						
Top	3.	32.	42.	52.	6.	6.
Bottom	16.	46.	55.	65.	29.	29.
Interior column at 1st floor						
Top	2.	4.	4.	5.	12.	12.
Bottom	2.	4.	5.	6.	16.	16.
Exterior beam at 2nd floor	0.	1.8	2.4	3.0	—	—
Exterior column at 2nd floor						
Top: cracked	—	—	—	—	—	—
Bottom	—	—	—	—	16.	17.

Table 4.5

Capacity Ratios for Gable End Walls,
Linear Acceleration with Height, Percent g

Member	First Cracking				Minor Cracking	
	Case 1	Case 2	Case 3	Case 4	Case 1	Case 4
Exterior column						
at 2nd floor						
Top	2	23	30	37	4	4
Bottom	12	32	39	46	21	21
Interior column						
at 1st floor						
Top	2	4	4	5	11	11
Bottom	2	4	5	5	15	15
Exterior beam						
at 2nd floor	0.	1.6	2.1	2.7	—	—
Exterior column						
at 2nd floor						
Top: cracked	—	—	—	—	—	—
Bottom	—	—	—	—	11	12

Table 4.6

Capacity Ratios for Gable End Walls, Cracked Beams, Linear Acceleration with Height, Percent g

Member	Minor Cracking	
	Case 1	Case 4
Exterior column at 1st floor, bottom	5.4	5.9
Interior column at 1st floor, bottom	7.8	8.5

A reasonable estimate for a base acceleration parallel to the end walls that would cause no visible damage (cracking of beams may not be visible after the earthquake stops) must consider both the effect of out-of-plane bending of side walls and the possibility of cracking of second floor, but not third floor, beams. A base acceleration in the range of 6% to 11% of gravity is therefore applied, based on the capacity of the end gable frames acting in their own planes.

4.4 -Roof and Floor Diaphragms

The Old State House has very effective floor diaphragms on the first and second floors, and a diaphragm of uncertain behavior in the sloping roof planes. The floor diaphragms are effective because of the two layers of sheathing which are placed orthogonal to each other. In the two planes of the roof, there is one layer of sheathing boards which probably are 1 in. thick.

The simplified theoretical model of the floor diaphragms, which considers the boards running in one direction as sheathing, and which takes the boards in the orthogonal direction as ties, shows that the sheathing boards would be effective even though they may be placed with a small gap between boards. The gaps between boards may also develop after construction and be caused by the shrinkage of the wood. Even if we accept the existence of small gaps between the sheathing boards, the floor would act as diaphragm; in this case, however, the diaphragm would allow for a certain amount of movement prior to providing a load path. The force-deformation behavior of the diaphragm could be modeled by a bi-linear curve, the first line starting from the origin being very flat (elastic modulus close to zero) and the second line having the stiffness that corresponds to a 1 in. thick wooden diaphragm.

The base accelerations that stress the floor diaphragm to the ultimate capacity are close to 1 g, if both layers of sheathing are considered to be effective. When only one layer is assumed to be effective, then the diaphragm would not reach its ultimate strength until the base acceleration reaches 30% to 40% g. Whichever assumption is used for the floor diaphragm, the conclusion is reached that the floor diaphragms are not a weak link in the system. Because of this conclusion, no further refinements in the ultimate strength of floor diaphragms were attempted.

The roof sheathing is probably nailed with hand wrought nails to the roof purlins. A single layer of sheathing that runs normal to the supporting members has been shown in the load tests to be four to seven times weaker than a similar diagonal sheathing. Gaps between sheathing boards, that must be expected due to a combination of shrinkage and casual workmanship, have a degrading effect on the stiffness of the diaphragm that cannot easily be evaluated.

A commonly accepted value of allowable shear in horizontal sheathing is 50 lbs/ft. This allowable is based on a factor of safety of 5, which means that the ultimate shear of a parallel sheathed floor diaphragm is about 250 lbs/ft. A base acceleration in the range of 5% to 6.5% **g** will cause forces in the roof diaphragm which approach its ultimate value. The value of 250 lbs/ft is applicable to a diaphragm that is built with controlled workmanship; the conditions of workmanship and nailing used to build the roof diaphragm on the Old State House is uncertain, therefore, the assumed ultimate shear for the diaphragm is probably an upper bound for the diaphragm.

The strength and stiffness of the roof diaphragm determines in part the behavior of the longitudinal external masonry walls when vibrating out of the plane of the wall, and it also determines the load distribution that reaches the gable end walls, and hence, the behavior of these walls for in-plane loads. Both the side walls out-of-plane and the end walls in-plane are weak links of the building; the behavior of these weak links is in turn governed by another weak link, which is the roof diaphragm.

4.5 Peak Ground Acceleration

All results presented up to this point are for a static equivalent lateral force system. In this static equivalent approach the base shear capacity of the building was computed; the base shear capacity was then divided by the weight of the building to obtain a base acceleration. The problem to be addressed in this section is that of obtaining an estimate of the peak ground acceleration on firm ground given the base shear capacity.

An approach that may be used is to compute a base shear from a modal analysis of the building using a Newmark-type elastic response spectrum normalized to a 1 **g** peak ground acceleration. The base shear thus obtained is scaled to the base shear capacity; the scaling factor is an estimate of the peak ground acceleration. The approach described is applicable to buildings that have a first mode response governed by peak ground acceleration; such buildings will have a first mode natural period shorter than about 0.5 seconds.

The accuracy of the peak ground acceleration estimate obtained as described above depends, in order of decreasing importance, on the assumed damping of the building, on the accuracy of the first mode shape, and on the period of the second mode. The reasons for this dependence are: the damping sets the acceleration amplification; the first mode shape is required to compute the effective modal mass of that mode; and the period of the second mode is used to establish whether full or partial amplification of that mode is expected. The accuracy of the first mode period is not critical so long as the period falls in the constant amplification region of the Newmark-type elastic spectrum; this spectrum is bilinear in the region that depends on peak ground acceleration a_g (increasing from a_g at zero period to the maximum amplification at about 0.1 seconds, and remaining constant thereafter for periods up to about 0.5 seconds).

The computations performed to estimate the peak ground acceleration from the static equivalent base shear were based on a two degree of freedom model with the following range of parameters:

- Maximum amplification: $2.0\ a_g \leqslant Sa \leqslant 2.6\ a_g$.
- Mode shape of first mode: linear or parabolic.
- Period of second mode: $\frac{T_1}{4} \leqslant T_2 \leqslant \frac{T_1}{3}$

in which:

Sa: maximum response acceleration, percent **g**

a_g: peak ground acceleration, percent **g**

T_1: first mode period, seconds

T_2: second mode period, seconds

The effective modal mass of the first mode of the Old State House is 90-percent of the total mass for a linear first mode and 72-percent of the total mass for a parabolic first mode. The amplification coefficients C for base shear in the formula:

$$V = C\ a_g\ W$$

are given in Table 4.7. The modal base shear was taken as the root-sum-square of the modal base shears. The ratio of second- to first-mode period did not affect the results. Results of Table 4.7 show that the amplification result is linear with assumed maximum amplification.

Table 4.7

Base Shear Amplification Coefficients, C

Maximum Amplification	Linear First Mode Shape	Parabolic First Mode Shape
2.0	1.81	1.48
2.6	2.35	1.92

Our knowledge of the structural damping and of the structural behavior of the Old State House is not sufficiently precise to select an amplification value from Table 4.7. We have therefore assigned a Bayesian probability of 0.4 to the amplification coefficient 1.5 and a probability of 0.6 to the amplification coefficient 2.4. These subjective probabilities are used in the discussion where our best estimate of static equivalent base acceleration is given in probabilistic terms.

5. DISCUSSION OF SEISMIC RESISTANCE

5.1 Sources of Uncertainty

The preceeding Chapter has given some of the results obtained for the resistance and seismic response of several elements of the Old State House building. This Chapter will discuss the effects on seismic resistance of the uncertainties considered, and will interpret some of the results obtained.

The primary sources of uncertainty that must be considered to establish probable bounds for peak ground acceleration that occurred in the proximity of the Old State House during the Cape Ann earthquake of 1755 are as follows:

- damage suffered by the building;
- material properties of the building elements;
- size of the structural elements;
- connection details and quality of workmanship;
- dynamic response of the building; and
- estimate of peak ground acceleration obtained from static equivalent base shear.

The assessment of damage that the building suffered during the earthquake is important to establish where, within the possible range of results for no damage to results for minor damage, the actual results should lie. We believe that the results for little damage are most meaningful; by "little damage," we mean that the masonry walls could have suffered some cracking, but that after the earthquake the cracks closed or at least were not obvious to the casual observer. The acceptance of minor damage implies that the building could have reached, but could not have exceeded, the ultimate capacity of its structural elements.

The properties of masonry are more important to establish first cracking, and the consequent behavior of beams, than they are to establish the ultimate capacities of columns. The reason for this is that the ultimate capacity of columns is more dependent on the member size and on the ratio of bending moment to axial load in the member, than it is to ultimate shear or ultimate compressive strength. Take for example a column that is 24 in. wide, 60 in. deep, and has an axial load of 100 kips; if this column were governed by compression, the ultimate capacity of the column would increase by about 20% when the compressive strength of the masonry is increased from 300 psi to 600 psi. This same column capacity, however, is inversely proportional to the eccentricity (moment divided by axial load). For the same column, and assuming further an eccentricity of 100 in., the ultimate capacity of the column when governed by shear, increases by 17% when the ultimate shear increases from 50 psi to 100 psi. The dimensions chosen for this example are similar to those of a pier in the longitudinal wall. The results of this simple example indicate that for the type of analysis of interest in this study, the uncertainty that arises because of unknown masonry properties is small when compared to the uncertainty that arises from other sources.

Members sizes must be known for an accurate estimate of element capacities. The uncertainties in this study arise because of the historical nature of the investigation, as well as from our lack of access to measuring many of the still-existing members. We believe, however, that the best estimate sizes used in the analysis are a reasonable reflection of the actual element sizes; moreover, we know that there are no gross inaccuracies in the sizes of masonry elements.

The connection details, and the quality of the workmanship are some of the most important factors that determine the structural modeling, and hence, the structural response computed. For certain details it is difficult, if not impossible, to reconstruct the original configuration. We have considered the effect of uncertainty in details by using alternate structural models where appropriate. Some of the bounds obtained depend on the modeling assumption used, for example, the assumptions were made that the roof diaphragm was ineffective, and also that it was fully effective for the size and type of configuration of the sheathing boards.

The structural modeling of the building, and the computed response for pseudo-static forces, may lead to results which are different from those that would follow from a more complex analysis. The inaccuracy of the static analysis can be justified on the basis that the structural model is not well defined. It does not make sense, therefore, to perform a very detailed and complex analysis of a structural model that is an uncertain representation of the real structure. The results obtained, with their corresponding bounds, must be considered as good estimates for the question at hand. Other estimates could only be obtained in a project of a much expanded scope, and would require a large budget.

The results obtained are for a static equivalent lateral force system, except that an estimate of peak ground acceleration based on a modal analysis with a Newmark-type spectrum was obtained in Section 4.5. A valid extrapolation from static equivalent analysis to peak ground acceleration should be based on comparisons obtained from the results of static equivalent analyses and time-history integration of response analyses of similar structures. Such analyses are not available; the few studies performed of this type, are for buildings on the west coast, which were subjected to earthquake-type base accelerations that caused damage. In this study the pre-ultimate state of the elements in the building is of interest that is, a state of no damage or minor cracking.

5.2 Discussion of Results Obtained

The results presented in the proceeding Chapter, showed that a possible range of static equivalent base accelerations that caused no damage to minor damage of the long side walls are 6% **g** to 15% **g**, and that corresponding results for the gable end walls are 3% **g** to 12% **g**. The range in results for the long side walls is due primarily to the uncertainty of the connections of the roof trusses to the masonry walls and to the strength and stiffness of the roof diaphragms. The range in results for the gable end walls is due to the uncertainty of the roof diaphragm ability to distribute lateral loads to the end walls, on the uncertain tensile strength of the masonry (which determines whether the end walls behave as frames) and on the out-of-plane contribution from the side walls. Given the results obtained, and the reports of no damage suffered by the Old State House during the Cape Ann earthquake, the earthquake must have produced an equivalent static base shear that did not exceed the range of 5% to 10% g. The lower value would be true if the building in fact developed no cracks during the earthquake, and the upper result would hold true if minor cracks occurred which were not recorded by the observers of the earthquake. Historical accounts always tend to emphasize the major damages that occur in a city, with little or no mention of non-critical damage. It is entirely possible that some damage occurred to the Old State House, but because of its minor nature, it was not reported.

A review of the information gathered and developed during this project can be used to establish subjective probabilities for the upper bound effective base acceleration which would have caused no damage. These are given below.

2	100
3	95
4	90
5	80
6	70
7	50
8	30
9	10
10	6
11	4
12	2
13	1
14	0.5
15	0

The results of Section 4.5, that relate peak ground acceleration and effective base acceleration, can be combined with the subjective probabilties listed above to obtain subjective probabilities for peak ground acceleration on firm ground. The results obtained are listed below.

Peak Ground Acceleration Percent g	Probability that Acceleration Caused no Damage
0	1.00
2	0.86
4	0.31
6	0.04
8	0.008
10	0

The expected peak ground acceleration from this distribution is 2.5 percent g and the standard deviation is 1.5 percent g. The peak ground acceleration did not exceed, in probability, the mean plus four standard deviations, that is, 8.5 percent g.

Several correlation expressions have been developed by researchers to relate peak ground acceleration, Modified Mercalli intensity, and Richter magnitude; and also to relate epicentral intensity and intensity at a distant site. There is much scatter in correlation expressions and in attenuation laws, therefore, results obtained from them are only guidelines.

The following expressions will be used as guides to obtain intensities and magnitudes from the computed accelerations:

$$\log a = -0.18 + 0.3 \ I$$

$$I = I_o + 3.1 - 1.3 \ln D$$

$$M = 1.0 + \frac{2}{3} I_o$$

in which:

a: peak ground acceleration, cm/sec/sec

I: site intensity, Modified Mercalli scale

I_o: epicentral intensity, Modified Mercalli scale

D: site to epicenter distance, miles

M: magnitude, Richter scale

The distance from Boston to the Cape Ann epicenter will be taken as 30 miles. The epicentral distance is not known precisely, so a range of distances could be used; but, since the intensities and magnitudes computed are for reference only, no refinement will be attempted. Results of the probability distribution are combined below with results from the correlation expressions. The peak ground accelerations listed correspond, approximately, to the mean, and to the mean plus one, two, and four standard deviations.

Probability that Acceleration Caused no Damage	Peak Ground Acceleration, Percent g	Site Intensity, M.M. Scale	Epicentral Intensity, M.M. Scale	Magnitude, Richter Scale
0.80	2.5	5.2	6.6	5.4
0.30	4.0	5.9	7.2	5.8
0.05	5.4	6.3	7.7	6.1
0.	8.5	7.0	8.3	6.5

As a reference, the Cape Ann earthquake is listed in Earthquake History of the United States with an epicentral M.M. intensity of about 8; Fr. Linehan of Weston Observatory has estimated the epicentral intensity as 7 to 8; and Weston Geophysical Research has estimated a site M.M. intensity of 7 for certain locations in Boston.

6. CONCLUSIONS

The conclusions that are drawn from this study of the earthquake resistance of the Old State House are as follows.

1. It is feasible to estimate the ground acceleration that occurred during a past earthquake in the vicinity of a specific building given historical records of damage to the building during the earthquake, of the structural configuration of the building at the time it was subjected to the earthquake, of the methods and materials of construction used when the structure was built, and a knowledge of the local soil conditions.

2. Reasonably wide bounds are obtained for the estimates of base acceleration due to uncertainty in the damage suffered by the building during the earthquake, in the material properties of the building elements, in the size of structural elements and of their connection details, in the structural dynamic response of the building, and in the relationship between peak ground acceleration and static equivalent base acceleration. The results obtained are relevant even though there are wide bounds on the estimate for base acceleration.

3. The probable range of the maximum effective base acceleration that occurred in the vicinity of the Old State House during the 1755 Cape Ann Earthquake is 5% to 10% of gravity. There is a probability of 0.8 that an effective base acceleration of 5% of gravity caused no damage to the building, and a probability of 0.05 that an effective base acceleration of 10% of gravity caused no damage.

4. The peak ground acceleration that occurred in the vicinity of the Old State House during the Cape Ann Earthquake is smaller than the estimated effective base acceleration. There is a probability of 0.8 that a peak ground acceleration of 2.5% of gravity caused no damage to the building, and a probability of 0.05 that the peak ground acceleration of 5.4% of gravity caused no damage.

5. There is a probability of 0.8 that a site intensity of 5.2 caused no damage, and a probability of 0.05 that a site intensity of 6.3 caused no damage. Site intensities are obtained from peak ground acceleration using correlation expressions, thereby neglecting scatter.