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A Comparison Between Two-Dimensional and Three-DimensionalAnalysis, A Review of Horizontal Wood Diaphragms and a Case Study of the Structure Located at 89 Shrewsbury Street, Worcester, MA

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A Comparison Between Two-Dimensional Analysis and Three Dimensional Analysis, A Review of Horizontal Wood Diaphragms and a Case Study of the Structure Located at 89 Shrewsbury Street, Worcester, MA

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ABSTRACT

A two-dimensional structural analysis design approach has been the universally accepted method for a small structural engineering design firm. The tools to perform the analysis have been paper and pencil, calculators and more recently personal computers with two-dimensional software. With the introduction of three-dimensional software, a major shift is occurring on how small structural engineering firms approach analysis and design. This thesis research reviews the analysis of an existing building utilizing the standard two-dimensional approach, including horizontal diaphragm-action within wood floors. This study also reviews the research performed on horizontal diaphragms and investigates the use of three-dimensional, finite element modeling (RISA-3D) for the analysis of horizontal diaphragms. It is shown that the three-dimensional model can provide results similar to the two-dimensional hand calculations. However, the thickness of the diaphragm elements has to be significantly modified for flexible diaphragm action. The experience described herein is useful for structural engineer interfacing within three-dimensional CAD systems. The thesis concludes with a discussion on the challenges facing small structural engineering firms, including computer based technologies, engineering expertise to develop contract documents and review shop drawings, and outsourcing of design services.

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Over the last four years, I was fortunate to have attended Worcester Polytechnic Institute, a school that I will be proud of attending for the remainder of my life.

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1.0 INTRODUCTION

In a two-dimensional structural analysis of an existing building or a two-dimensional design of a completely new structure, the structural design profession has had an evolving, universally accepted approach that has been proven to avoid building collapses. In the current small firm structural engineering world, a two-dimensional engineering approach is the standard method of practice. Essentially, each structural component is analyzed and/or designed for its specific load, and the element is chosen and proportioned based on its strength characteristics. Factors of safety are utilized based on standard engineering practice and building code requirements. Three-dimensional analysis and design is rarely used and only for special projects. However, with the increased use of computers and advanced software, a three-dimensional approach appears to be emerging as the standard method of design of the future.

The goal of this thesis is to compare a three-dimensional analysis against the standard two-dimensional manual calculation approach. The intent is to develop a deeper understanding of the structural behavior and to try to make a clear determination as to whether or not the two-dimensional approach is conservative. A two-dimensional and three-dimensional, computer based analysis will be completed on an existing building. Once the three-dimensional model has been input into the structural analysis software, various scenarios can be investigated to cause a partial building failure or complete structural collapse. Elements can be reviewed in order to verify their true contribution to the performance of the structure. An additional area of study will be the contribution of horizontal diaphragm action with respect to the behavior and performance of the building structure. The performance of flexible and rigid diaphragms will be reviewed in order to compare the two systems as they affect the lateral design of the building. Research will indicate the current state of investigation of horizontal diaphragms. A review of existing literature will consist of what types of horizontal diaphragms have been studied, the types of study performed (simple calculations versus finite element analysis), and the current state of design of horizontal diaphragms.

The case study to be utilized will be the existing building located at 89 Shrewsbury Street in Worcester, Massachusetts. This structure was chosen due to the local location and it was recently a project at Johnson & Seaman Engineering. An analysis will be presented indicating a two-dimensional and three-dimensional analysis of the gravity and lateral systems. An analysis of a wood diaphragm will be performed in a three-dimensional model that will indicate the flexibility of the diaphragm, its relative stiffness as compared to materials of other diaphragms, and how it transfers lateral load to the lateral resisting system.

PHILOSOPHY OF DESIGN

In the standard two-dimensional structural analysis, the structural engineer is typically given a set of plans from the architect who is in charge of the project. The initial set of plans created by the architect will typically consist of floor layouts, preliminary elevations and possibly a building cross section. The contract documents are a work in progress and will change many times prior to the issue of the final set.

The structural engineer reviews the initial set of drawings and meets with the architect to determine the building composition. The exterior walls, roof, floor, building use, and multiple architectural and structural issues are discussed. The lateral load resisting system and foundation type will also be discussed at this time. If borings or test pits of the existing soils have not been completed, a request to the architect occurs at this time. With the initial information known, the structural analysis and design concept can be reviewed, and preliminary drawings can be created. Utilizing past experience, framing plans with approximate member sizes and spacing are given to the architect for their review in order for their work to continue.

The formal analysis can be started once a building code review has been completed. Utilizing tables and figures in the 6th Edition of the "Massachusetts State Building Code" (MA Code), the minimum live loads, snow loads, wind and seismic loads are calculated and used for design.

Upon completing the structural analysis and design, which is usually completed in multiple phases, the structural framing plans and details can be finalized. The completed structural plans are packaged with the architectural and other engineering plans for bidding purposes.

2.0 LOADS

Dead and live loads are gravity loads that act vertically on a structure, and in a two-dimensional analysis, individual structural members are designed to support the dead loads and live loads in their tributary areas. Live loads are moving loads and vary based on a building's use. Dead loads are non-movable loads that are permanently attached to the structure such as the structural members themselves, and the weight of decking, flooring materials, ceilings, lighting, sprinklers and miscellaneous other items. Snow loads are also gravity loads and act similar to live loads but act on the exterior roof surface. Lateral loads are wind and seismic (earthquake) loads that act horizontally.

2.1 Live and Dead Loads

The minimum uniformly distributed live loads are indicated in Table 1606.1 For example, assembly areas with moveable seats require a minimum live load of 100 pounds per square foot. Office minimum live loads are 50 psf in the office themselves, 100 psf live loads are used in the lobbies, and 80 psf in corridors above the first floor. Depending on the type of structural system, the dead loads will vary greatly based on the type of materials. They are typically added in square foot increments.

2.2 Snow Loads

Snow loads vary throughout Massachusetts, and the Massachusetts State Building Code (MA Code) divides the state into snow load zones. The towns are listed in each zone, and the City of Worcester, where the case study is reviewed, is located in Zone 3 as indicated in Figure 1610.1c. According to Section 1610.2, the basic snow load Pf is 35 pounds per square foot (psf). The engineer of record must also consider snow loading conditions under Section 1610 including conditions for sloped roofs and odd shaped roofs. Additionally, multiple changes in roof elevation can affect snow drifting loads, and sliding snow can be a factor if a lower roof is adjacent to a higher pitched roof. Section 1610 of the MA Code has equations to calculate the effects due to drifted snow, unbalanced snow, sliding snow and other scenarios.

2.3 Lateral Loads

Lateral loads such as wind or seismic must also be reviewed in the analysis of the existing building or the design of a new structure. Similar to snow loads, wind loads in the MA Code are also defined according to zones. The MA Code divides the state into three zones, and towns are listed in each zone depending on location. The City of Worcester is located in Zone 3 as per Figure 1611.1b. Table 1611.4 provides the reference wind pressure. The reference wind pressure is modified according to the height of the building and its exposure. For example, a structure located on the outskirts of downtown Worcester that is less than 50 feet in height has a reference wind pressure of 17 psf. The wind pressure increases to 24 psf for building heights above 50 feet to a height of 100 feet. Exposure is defined as a measure of terrain roughness. For example, Exposure C has the highest reference wind pressure and is used for areas of open, level terrain with only scattered buildings, structures, trees or miscellaneous obstructions, open water, or shorelines. Most practicing structural engineers choose Exposure C since it produces the more conservative wind pressure and the future of the surrounding terrain is unknown.

Uplift forces on flat roofs are also calculated using the reference wind pressure multiplied by factors according to Table 1611.8. For essentially flat roofs or roofs with minimum pitch, the reference wind pressure is multiplied by 0.6 giving a net suction force to be applied on the windward slope (for a minimum pitched roof) or 0.5 for a net suction force applied to the leeward slope. For a flat roof, the engineer of record would apply a factor of 0.6 multiplied by the reference wind pressure. There are also tables for reference wind pressures multiplied by factors to be utilized for signs, parts of structures and local supporting elements, etc.

Seismic loads are defined in Section 1612.0. The criteria for earthquake design and the construction of buildings subject to earthquake ground motions are broken down into multiple equations throughout this section. As the MA Code states in section 1612.1 "it must be emphasized that absolute safety and prevention of damage, even in an earthquake event with a reasonable probability of occurrence, cannot be achieved economically for most buildings."[1] In general, the MA Code criteria with respect to earthquake design is an attempt at preventing the loss of life while limiting catastrophic damage to a structure.

These minimum seismic criteria are considered to be prudent and economically justified for the protection of life safety in buildings subject to earthquakes. The "Design Earthquake" ground motion levels specified may result in both structural and non-structural damage. For most structures designed and constructed according to MA Code Section 1612.0, it is expected that structural damage from a major earthquake may be repairable but the repair may not be economical. For ground motions larger than the design levels, the intent of the MA Code is to promote a low likelihood of building collapse. Chapter 16 of the MA Code involves seismic loads related to new construction, and Chapter 34 involves additions and renovations related to existing buildings.

[1] William F. Galvin, Secretary of the Commonwealth, The Massachusetts State Building Code - 6th Edition, February 1997, Page 278

3.0 SEISMIC DESIGN/NEW CONSTRUCTION-CHAPTER 16, MA CODE

Seismic analysis for new construction utilizes multiple equations that result in horizontal forces applied to the building structure. The lateral resisting system of the building structure is designed to resist those forces. The seismic forces are compared to the wind forces which are also calculated according to the MA Code. The lateral resisting system is designed for the larger loads calculated for either wind or seismic.

3.1 Seismic Hazard Exposure Group

In earthquake design, a building is classified in MA Code Table 1612.5 according to its seismic hazard exposure group. There are three seismic exposure groups with Group III as the most restrictive (i.e. fire, rescue, emergency, etc.). As per MA Code Section 1612.7, the seismic performance category is defined based on the seismic hazard exposure group. Groups I and II are classified as seismic performance category C, and Group III is classified as seismic performance category D.

3.2 Seismic Performance Category

The MA Code divides buildings into either performance category C or D. Seismic performance category D relates to fire, rescue, police, and emergency related structures. Also included are primary communication facilities and toxic material storage structures. Seismic performance category C is all other structures.

The criteria for seismic performance category C and D are indicated in the following figure:

1612.4.4.3 Seismic Performance Category C:
The structural framing system for buildings assigned to Seismic Performance Category C shall comply with the building height and structural system limitations in Table 1612.4.4.

1612.4.4.4 Seismic Performance Category D:
The structural framing system for buildings assigned to Seismic Performance Category D shall comply with 780 CMR 1612.4.4.3 and the additional provisions of 780 CMR 1612.4.4.

1612.4.4.4.1 Limited building height:
Buildings having a structural system of steel or cast-in-place concrete-braced frames or shear walls are limited to a height of 240 feet where there are braced frames or shear walls so arranged that braced frames or shear walls in one plane resist not more than the following proportions of the seismic design force in each direction, including torsional effects:

1. 60% where the braced frame or shear walls are arranged only on the perimeter.
2. 40% where some of the braced frames or shear walls are arranged on the perimeter, or
3. 30% for other arrangements.

1612.4.4.4.2 Interaction effects:
Moment-resisting frames that are enclosed or adjoined by more rigid elements not considered to be part of the seismic-resisting system shall be designed so that the action or failure of the enclosing or adjoining elements will not impair the vertical load and seismic force-resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at building deformations corresponding to the design story drift (delta) as determined in 780 CMR 1612.5.5.

1612.4.4 Structural framing system:
The basic structural framing systems to be utilized are indicated in Table 1612.4.4. Each type is subdivided by the types of vertical structural elements that will resist the design lateral forces. The structural system utilized shall be in accordance with the seismic performance category and height limitations indicated in table 1612.4.4. The appropriate response modification factor (R) and the deflection amplification factor (Cd) indicated in Table 1612.4.4 shall be utilized in determining the base shear and the design story drift. Structural framing and seismic-resisting systems which are not contained in Table 1612.4.4 shall be permitted if analysis and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 1612.4.4 for equivalent response modification factor (R) values.

Figure 1: Seismic Performance Categories
(Adapted from The Massachusetts State Building Code - 6th Edition)

3.3 Structural Systems

Table 1612.4.4 – Structural systems details the basic structural systems (i.e. – seismic resisting systems including load bearing wall systems, building frame system, moment-resisting frame systems, duel systems and inverted pendulum structures) and corresponding response modification factor (R), deflections amplification factor (Cd) and structural system limitation and building height for seismic performance category C and D.

3.4 Structural Design Requirements/Structural Data

As per MA Code Section 1612.4 – the following figure indicates the structural design requirements:

> 1612.4.1 Design Basis "The seismic analysis and design procedures utilized in the design of the buildings and their structural components shall be in accordance with the requirements of 780 CMR 1612.4. The design seismic forces and their distribution over the height of the building shall be in accordance with the procedures of 780 CMR 1612.5 or 1612.6. The corresponding internal forces in the structural components of the building shall be determined using a linear elastic model. Further into this section states individual structural members shall be designed for the sheer forces, axial forces and moments determined in accordance with 780 CMR 1612.4. Connections shall be designed to develop the strength of the connected members or the analysis force, whichever is less. The design story drift of the building, calculated as specified herein, shall not exceed the allowable story drift of 780 CMR 1612.4.8, when the building is subjected to the design seismic forces. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to resist the forces developed and shall accommodate the movements imparted to the building by the design ground motions. The foundation design criteria shall account for the dynamic nature of the seismic forces, the design ground motions and the design basis for strength and ductility of the structure."

Figure 2: Structural Design Requirements/Structural Data
(Adapted from The Massachusetts State Building Code – 6th Edition)

Minimum structural data needed to perform the analysis of earthquake design are indicated in the following figure:

1.) Site co-efficient (s) which varies from 1.0 to 2.0 are indicated in Table 1612.4.1. A geotechnical engineer is typically involved to determine this value where S=1.0 is the better soil condition in terms of calculating a lower seismic force in the overall building design.
2.) Response modification factor (R) and deflection amplification factor (Cd) as per table 1612.4.4 – Values of R and Cd are based on the seismic resisting system.
3.) Building Weight – Total dead load of structure, attached loads, and a percentage of snow load.
4.) Review of Building Configuration (Table 1612.4.5.1 – plan structural irregularities) will contribute to seismic forces applied to building.
5.) Allowable story drift must be calculated, seismic coefficient (Cs), fundamental period (T) and other miscellaneous equations.

Figure 3: Structural Data – Earthquake Design
(Adapted from The Massachusetts State Building Code – 6th Edition)

3.5 Seismic Base Shear

Once the seismic base shear has been calculated, the force is distributed to each horizontal diaphragm based on the equation in Section 1612.5.3. The design philosophy of the horizontal shear distribution (seismic force distribution) is to apply a force at each level of the building structure and to review the force at each floor level and the structure in its entirety. The seismic design equations continue in detail in Chapter 16 for non-structural elements which include architectural components, mechanical and electrical components.

As an overview to Chapter 6 of the MA Code, the general intent with respect to seismic design is to provide a lateral resisting system that reinforces a structure in order to provide a safe building in a seismic event. Damage to the building structure is secondary.

4.0 SEISMIC DESIGN/EXISTING – CHAPTER 34, MA CODE

Repairs, alterations, additions and change of use of existing buildings are dealt with in a separate chapter of the building code. In the MA Code, Chapter 34 provides an in-depth approach to the seismic review of existing buildings. The provisions of Chapter 34 can make or break the economic feasibility of a potential project by increasing costs due to seismic upgrades.

In order to review a building and determine the seismic requirements with respect to Chapter 34 of the Massachusetts State Building Code, the hazard index must be defined as per Table 3403. The use group must be known, e.g., F stands for factory and industrial; and A-3 stands for restaurant. F use group has a hazard index of 3, and A-3 use group also has a hazard index of 3. As described earlier in Chapter 16 of the MA Code, the seismic hazard exposure group is also needed as per Table 1612.2.5. Once the hazard index and seismic hazard exposure group are known, the seismic hazard category is defined according to Table 3408.1 which is provided below as Table 1 in order to follow the provisions of Chapter 34.

4.1 Seismic Hazard Category

Table 3408.1 has three seismic hazard categories with Category 1 as the least restrictive and Category 3 as the most restrictive. Depending on whether or not occupancy increases, cost of the renovation and the changes in use, the seismic hazard category can be found.

Table 1: Seismic Hazard Categories – From Table 3408.1 of Chapter 34 (Adapted from The Massachusetts State Building Code – 6th Edition)

CHANGE IN USE (1)	CHANGE IN OCCUPANCY OR COST OF ALTERATIONS: Occupancy increased by more than 25% and to A total occupancy of 100 or more; or total cost of alterations exceeds 50% of the assessed valuation of the building. (2)	CHANGE IN OCCUPANCY OR COST OF ALTERATIONS: All other changes in occupancy and total cost of alterations less than or equal to 50% of assessed valuation of the building. (2)
Change from Use Group with Hazard Index less than 4 to Use Group with Hazard Index 4 or greater; or Seismic Hazard Exposure Group III per Table 1612.2.5	3	2
All other changes in Use Group, or no change in Use Group.	2(3)	1(3)

Note 1. Refer to Table 3403 and Appendix F, Table F-1 for the Hazard Index of any use group. Adjustments to the Hazard Index indicated in the footnotes to Table 3403 shall not be applied for determination of Seismic Hazard Category.

Note 2. Total cost of alterations shall include the cost of alterations proposed under the current building permit application, plus the cost of any alterations covered by building permits in the two-year period proceeding the date of the current permit application. The assessed evaluation shall be as of the date of the current building permit application.

Note 3. When there is no change in use, the following costs may be excluded from the total cost of alterations:
Costs incurred by requirements for compliance with the following:
Americans With Disabilities Act
Massachusetts Architectural Access Board Regulations, 521 CMR
M.G.L. c. 148 & 26A1/2 requiring sprinklers in existing high-rise structures.
Costs incurred for improvements in:
Sprinklering
Smoke and heat detection
Fire alarm systems
Exit enclosures

An example of the application of Table 3408.1 with respect to the renovation to 89 Shrewsbury Street is as follows: An existing factory building is to be renovated into an office building, and the construction work is to be considered major where it will be gutted and rebuilt and only the building shell remains prior to the renovation. As per Table 3403, the original Hazard Index is 3 (Factory and Industrial) and the proposed

Hazard Index is 2 (Business). The total cost of alterations will exceed 50% of the assessed valuation of the building. As per Table 3408.1, for changes in use group that do not increase from a Hazard Index of less than 4 to 4 or greater (or Seismic Hazard Exposure Group III), the seismic Hazard Category would be 2. A letter is included in Appendix A dated September 19, 2005 in regard to 89 Shrewsbury Street, written by Johnson & Seaman Engineering, Inc. and describes different renovation scenarios and how they affect the Hazard Index.

The Seismic Hazard Category will indicate the direction the project is headed with respect to the seismic requirements. The following list defines the design requirements for Seismic Hazard Categories 1 to 3.

3408.5.4.3 For Seismic Hazard Category 1:
Earthquake resistance need only comply with the requirements of 780 CMR 3408.3.5.

3408.5.4.4 For Seismic Hazard Category 2:
Earthquake resistance need only comply with the requirements of 780 CMR 3408.3.5, and the existing building shall be investigated for the presence of special earthquake hazards as described in 780 CMR 3408.6.3, and all such hazards that are present shall be corrected in accordance with the provisions of 780 CMR 3408.6.3.

3408.5.4.5 For Seismic Hazard Category 3:
Full compliance with 780 CMR 1612.0 is required, except as provided in 780 CMR 3408.5.4.6 and 3408.6.4, and except that existing structural systems not conforming to the requirements of 780 CMR 19 through 23 may be considered to participate in resisting lateral seismic loads, but only if the seismic design force is calculated in accordance with 780 CMR 3408.6.1.1.

Figure 4: Seismic Hazard Categories 1 – 3
(Adapted from The Massachusetts State Building Code – 6th Edition)

Many construction projects will fall into Seismic Hazard Category 2 since total renovations/costs often exceed 50% of the assessed valuation of the building and typically there will not be a change in use. Consequently, the requirements of sections 3408.3.5 and 3408.6.3 are thoroughly reviewed for compliance.

4.2 Reduction of Earthquake Hazards

The purpose of Section 3408.6.3 is to minimize hazards that may be a safety concern in a seismic event. The possibility of human harm is to be minimized by anchoring parapets, masonry walls and pre-cast concrete structural elements to the stiffer portions of the building structure (the roof or floor diaphragms). On existing buildings, the reduction of earthquake hazards (Section 3408.6.3) is accomplished by anchoring the structural roof and floor system to the masonry walls by the use of clip angles, lag bolts and/or adhesive anchors. By attaching the walls to the roof and floor diaphragms, the floors will help stabilize the walls in a seismic event. Section 3408.6.3 is written as:

3408.6.3 Reduction of Earthquake Hazards:
Where the provisions of 780 CMR 3408.0 require correction of special earthquake hazards, the following measures shall be taken to reduce hazards from parapets, masonry walls, and/or precise concrete structural elements which do not conform to the requirements of 780 CMR 1612.0:

1. **Parapets:** All parapets not meeting the requirements of 780 CMR 1612.0 **s**hall be removed or braced so as to meet the requirements of 780 CMR 1612.7 and, for un-reinforced masonry parapets, 780 CMR 3408.6.4.
2. **Masonry Walls:** All masonry walls shall be connected to floor or roof diaphragms or other elements providing their lateral support, so as to Conform to the requirements of 780 CMR 1612.7. The design force for the Connection shall not be less than 100 pounds per linear foot of wall. Connections shall not produce cross-grain bending in wood members.
3. **Pre-cast Concrete Structural Elements:** Interconnections of pre-cast concrete structural elements shall be investigated, and reinforced if necessary.

Connections shall conform to the requirements of 780 CMR 19.

Figure 5: Reduction of Earthquake Hazards
(Adapted from The Massachusetts State Building Code – 6th Edition)

4.3 Existing Lateral Load Capacity

The purpose of Section 3408.3.5 is to make the structural engineer of record and the owner of the building aware of the possible consequences of a building renovation that disrupts the existing lateral load resisting system. Section 3408.3.5 is written as:

3408.3.5 Existing Lateral Load Capacity:
Alterations shall not be made to elements or systems contributing to the lateral load resistance of a building which would reduce their capacity of resist lateral loads, unless a structural analysis conforming to 780 CMR 3408.3.4 shows:

1. That the lateral load resisting system of the building as altered conforms to 780 CMR 1611.0 and 1612.0 of the code for new construction, or
2. That the lateral load resisting system as altered conforms to all applicable minimum load requirements of 780 CMR 3408, and that there is no reduction in the lateral load capacity of the building as a whole.

Existing elements or systems may be reinforced or replaced with new elements or systems of equivalent strength and stiffness, in order to meet these requirements.

A building which complies with 780 CMR 1611.0 and 1612.0 except that the lateral load resisting system does not conform to the detailing requirements of 780 CMR 19 through 23 for the structural materials and seismic load resisting system employed, may be considered to be in compliance with 780 CMR 3408.3.5 if the lateral force calculated in accordance with the formula in 780 CMR 1612.4, but with lateral force factors ® and force modification factors as stipulated in Tables 3408.2 and 3408.3, respectively.

Figure 6: Existing Lateral Load Capacity
(Adapted from The Massachusetts State Building Code – 6th Edition)

4.4 National Historic Register Impact

A loophole within Chapter 34 is in regard to historic structures. The applicability of Chapter 34 is defined in Section 3400.3 and lists ten specific cases; however, at the end of the list is an exception which includes “Totally Preserved and Partially Preserved

Historic Buildings"[2] which are defined in Section 3409. In short, historic structures are exempt from the provisions of Chapter 34. An example of how the path of the structural design can change is indicated in a letter in Appendix B which illustrates the consequences of defining 89 Shrewsbury Street as a historic structure. The letter is dated November 8, 2005 and is written by Johnson & Seaman Engineering, Inc.

As an overview to Chapter 34 of the Massachusetts State Building Code (with respect to existing structures) the general intent is to ensure that the structural engineer designs upgrades to the existing lateral load resisting system should the existing system be altered. If the intent of the owner is to make major modifications, the structure's lateral system shall be upgraded and all potential seismic hazards shall be minimized with respect to human harm. Chapter 34 tries to reinforce an existing structure to provide a safe building in a seismic event.

[2] William F. Galvin, Secretary of the Commonwealth, The Massachusetts State Building Code-6th Edition, February 1997, Page 445

5.0 RESEARCH ON HORIZONTAL WOOD DIAPHRAGMS

The topic of horizontal diaphragms was researched for its contribution to the lateral load distribution system to the vertical lateral resisting system. Also, the type of horizontal diaphragm determines the load path to the lateral resisting system.

5.1 Literature Categories

There is a great deal of research that has been completed with respect to horizontal wood diaphragms. The following chart indicates a sample of the type of literature categories that are available:

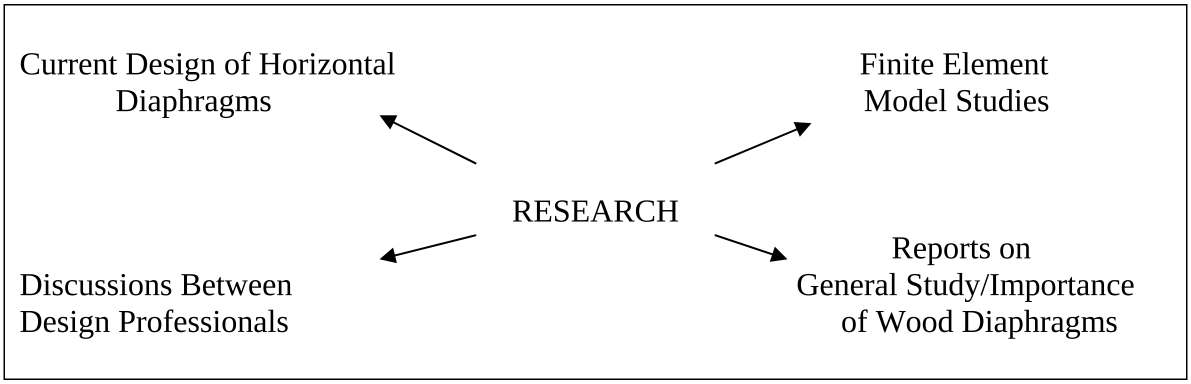

Figure 7: Research on Horizontal Wood Diaphragms

There were multiple research articles on nonlinear wood diaphragms utilizing finite element software. Many of the finite element models studied reviewed wood diaphragms and shear walls. The majority of the research investigates modern, light-framed wood construction consisting of 2x wood joists or studs with plywood attached. The diaphragms in the research are considered flexible, i.e. the shear walls resist their tributary portion of the lateral load. There were numerous articles on residential structures and their relationship to vertical shear walls and horizontal diaphragms.

One of the more useful sources of information was located in the APA Design and Construction Guide (APA, The Engineered Wood Association 1997). The tables within the design guide provide the allowable shear capacity for horizontal wood diaphragms and vertical wood shear walls. The horizontal or vertical shear capacity is based upon the nail size, nail spacing, and plywood thickness. There are additional criteria for the minimum thicknesses and spacing of the wood studs and floor joists.

5.2 Examples of Current Research

The research is the central focus point of Figure 7 and the material that was discovered is divided into four categories. Examples of current and pertinent research are indicated in Table 2.

Table 2: Current Research Examples

Current Design Articles	1. "Table 34" by the APA Design and Construction Guide (APA, The Engineered Wood Association 1997) which indicates design values for plywood shear walls. 2. "Acceptance Criteria for Wood Screws Used in Horizontal Diaphragms and Vertical Shear Walls" (ICC Evaluation Service, Inc. 2006).
Discussions between Design Professionals	1. An e-mail between engineers questioning the analysis procedure with respect to diaphragms and shear walls. (www.eng-tips.com 2005). 2. An e-mail between engineers discussing shear wall locations. (www.seaint.org 2003).
Finite Element Model Studies	1. "Role of Diaphragms in the Mitigation of Natural Hazards in Low-Rise Wood Frame Buildings" Written by Robert H. Falk, Chung K. Cheung and Rafik Y. Itani (Falk, Cheng and Itani 1984). 2. "Lateral Load Sharing by Diaphragms in Wood-Framed Buildings" written by Timothy L. Phillips, Rafik Y. Itani and David I McLean (Phillips, Itani and McLean 1993).
Reports on General Study/Importance of Wood Diaphragms	1. "Deflections of Nailed Shearwalls and Diaphragms" written by Chun Ni and Erol Karacabeyli (Ni and Karacabeyli). 2. "Wood Panel Diaphragms with Free Sheathing Joints" written by Martin H. Kessel and Michael Meyer (Kessel and Meyer).

It is obvious that there has been a great deal of study and communication between professional engineers on horizontal wood diaphragms. An area of study in the three-dimensional model to be created for this thesis will be to review the results of the model and try to compare them with the results of the two-dimensional hand calculations. Multiple values of the relative stiffness of the model diaphragm will be run in order to determine if flexible diaphragm behavior can be created similar to the two-dimensional analysis results. The comparison between the loads absorbed into the vertical braces of the model (utilizing multiple relative diaphragm stiffness) will determine how flexible the diaphragm behaves by using the two-dimensional analysis as the base line.

6.0 CASE STUDY - INTRODUCTION

The case study consists of a two-dimensional and three-dimensional structural analyses and partial design of 89 Shrewsbury Street, Worcester, Massachusetts. Also, a partial analysis of the existing wood diaphragm floor system located in both the newer steel-framed section of the building and the older wood-framed section of the building are compared to the results of a research article written by Robert H. Falk and Rafii Y. Itani titled "Finite Element Modeling of Wood Diaphragms."

The existing three story structure located at 89 Shrewsbury Street was constructed in two phases. The initial construction (Phase 1) consists of wood decking spanning to heavy timber interior support beams and wood columns. As the column loads increase from the roof, third floor and second floor, the wood column sizes increase. The exterior walls are solid brick, and they support the ends of the wood beams and small tributary areas of wood decking. It is obvious by studying the existing wall layouts, that the initial construction of Phase 1 had all exterior walls constructed of solid masonry. Phase 2 most likely was constructed shortly after the first phase with wood decking spanning to steel beams (not wood) and steel columns. The exterior walls are also solid brick and support the ends of the steel beams and small tributary areas of wood decking. One exterior wall of Phase 1 was modified with multiple openings and still utilized for bearing but as a common interior wall between the two phases.

6.1 Two-Dimensional Analysis of 89 Shrewsbury Street

6.1.1 Gravity System Analysis

The initial calculations consist of a gravity analysis of all levels of the structure. Example calculations of the Phase 1 and Phase 2 areas are located in Appendix D, pages

2B-1 and 2B-4. All of the existing roof beams were analyzed to determine their specific snow load carrying capacity, and the existing floor beams were analyzed to determine their live load capacity. In today's wood analysis and design, nominal allowable wood stresses are multiplied by adjustment factors to account for moisture content, load duration, repetitive member scenarios, and other usage conditions. In existing structures with older wood timbers, most of the information with respect to the nominal wood strength must be assumed based upon experience. Fine tuning an analysis with multiple allowable stress values multiplied by adjustment factors would be impractical since an assumption is made based upon the strength of the wood. The allowable stress values and elastic modulus values utilized for this analysis were as follows:

Fb = 1450 psi = Allowable stress in bending.

Fv = 90 psi x 2/3 = Allowable stress in shear.

E= 1,400,000 psi = Modulus of elasticity.

Each wood roof beam was analyzed by applying the tributary area, the allowable bending stress, section modulus and length into the basic formulas of $M = WxL^2/8$ and Sx required = M/Fb. By finding the allowable moment and linear load per foot, the capacity of the member was established. Deflection and shear were also checked to verify that they remained acceptable. An analysis example of a roof beam is provided in Appendix D, PG. ER.1. The existing wood beam properties were calculated with an Area = 108.7 in^2, Section Modulus = 213 in^3 and a Moment of Inertia = 1250 in^4.

A trial calculation using 50 pounds per square foot was run in a simple program which is used at Johnson & Seaman Engineering. The spacing between the beams measured 9.75 feet and a linear load of 488 pounds/foot was applied in the program. The output indicates the reaction at each end, the maximum bending moment and the required area, section modulus and moment of inertia for design purposes. For this beam analysis, the output was R (Reaction) = 5,124 pounds, M (Bending Moment) = 26,901 foot pounds, A (Area) required = 85.4 in^2, Sx (Section Modulus) required = 222.62 in^3 and Ix (Moment of Inertia) required = 1453 in^4. The area and section modulus values required were greater than the existing beam properties; therefore, the analysis was re-run using 45 pounds per square foot or 439 pounds per linear foot to establish a benchmark capacity. The process was repeated for decreasing levels of load until satisfactory results were obtained for the existing roof beam. All of the existing wood roof beams were analyzed using this method, and the total uniform load capacity of the roof of the initial Phase 1 portion of the building ranged between 40 and 45 pounds per square foot.

The following figures indicate framing plans of 89 Shrewsbury Street. Figure 8 represents the roof (wood) framing plan with the additional reinforcing beams. Figure 9 represents the roof (steel) framing plan. Figure 10 represents the third floor (wood) framing plan, and Figure 11 represents the third floor (steel) framing plan.

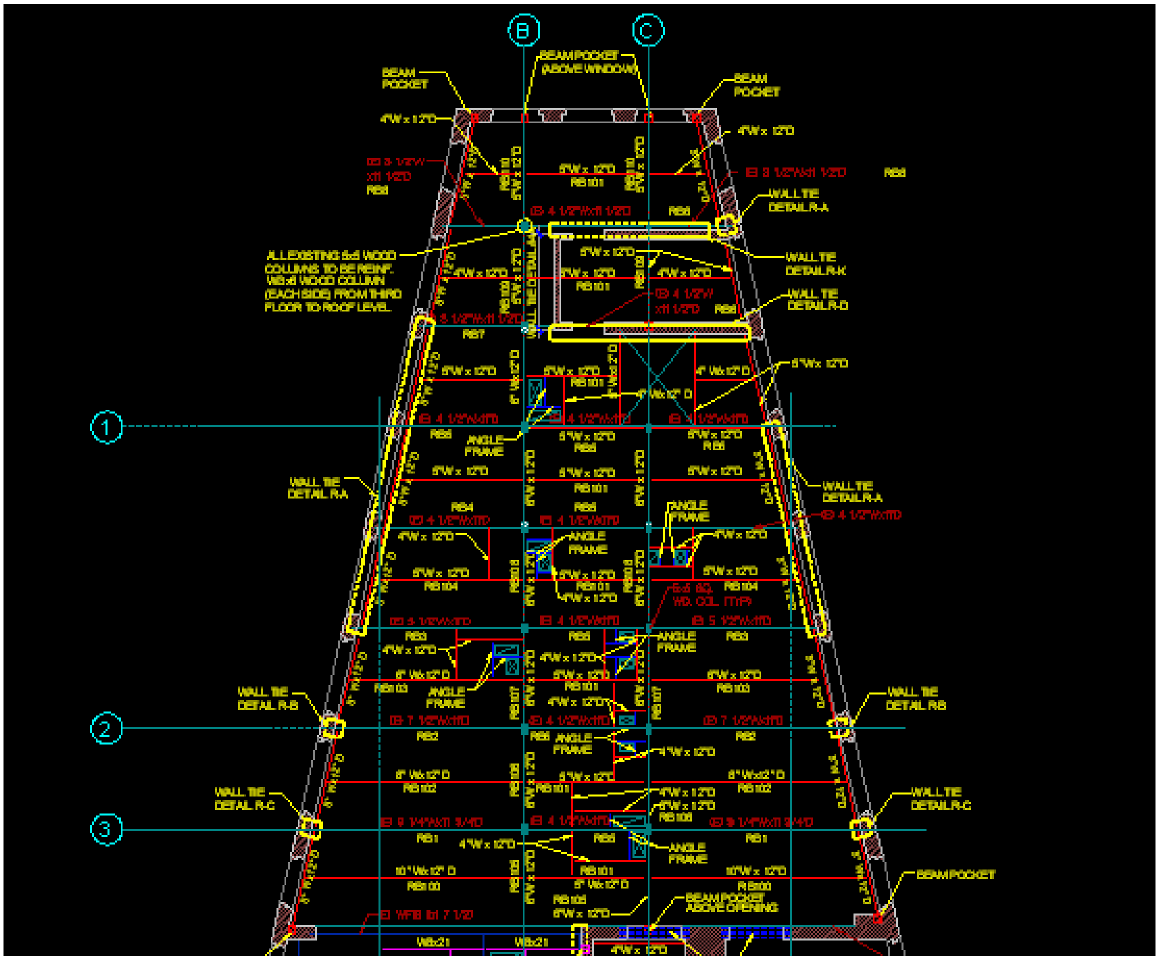

Figure 8: Wood Framing Plan – Roof Area
(Adapted from Roof Framing Plan
Created by Johnson & Seaman Engineering)

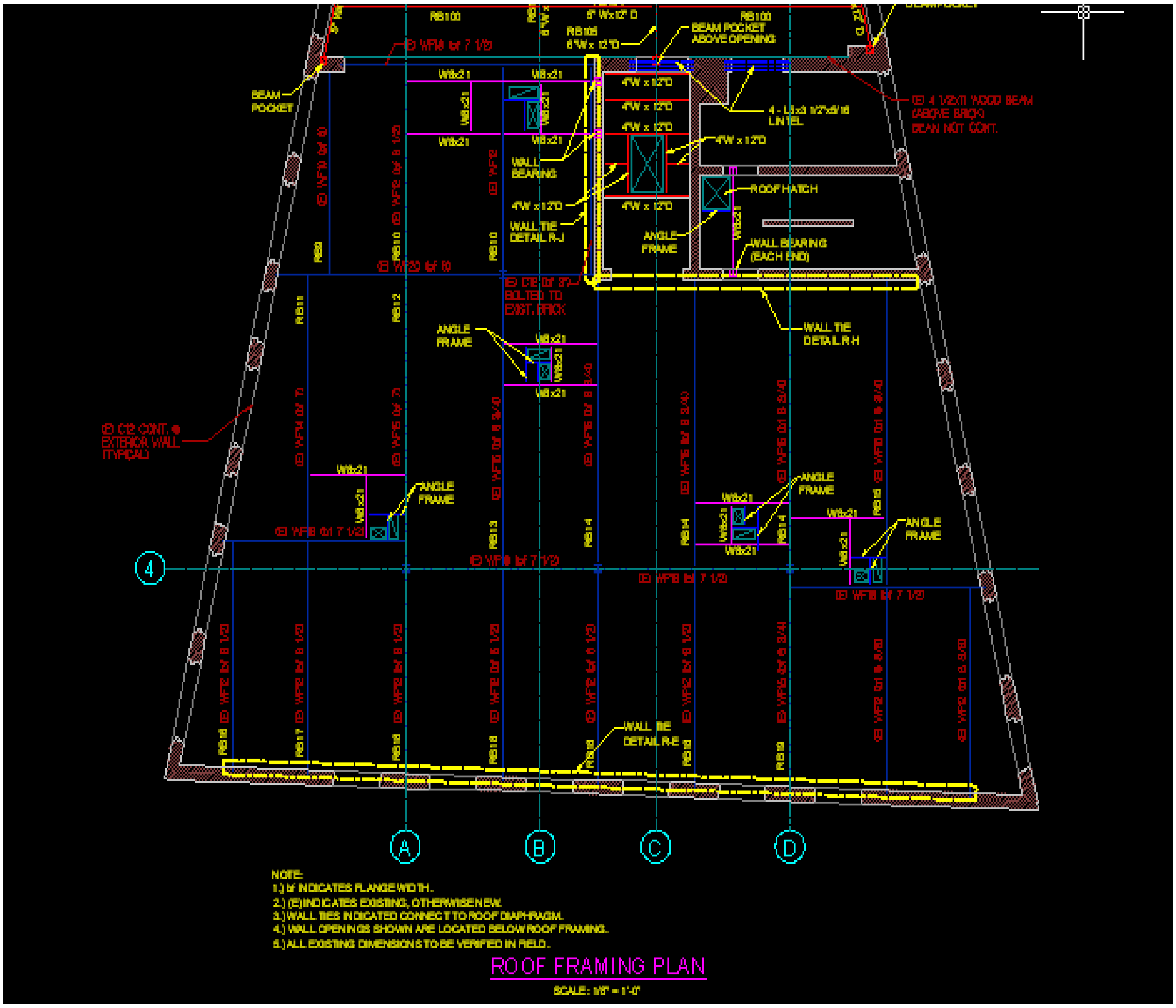

Figure 9: Steel Framing Plan – Roof Area (Adapted from Roof Framing Plan Created by Johnson & Seaman Engineering)

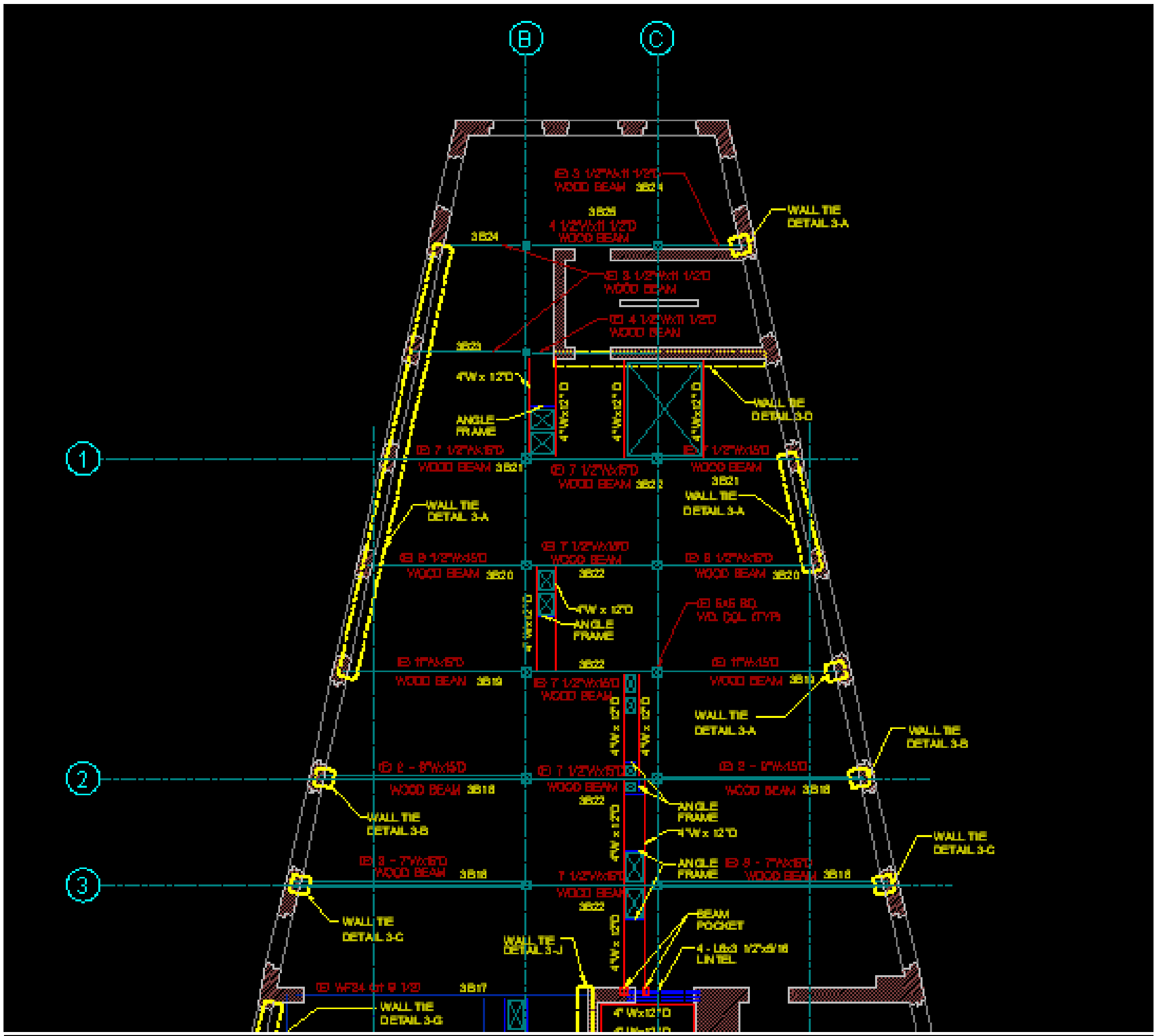

Figure 10: Wood Framing Plan – Third-Floor (Adapted from Third Floor Framing Plan Created by Johnson & Seaman Engineering)

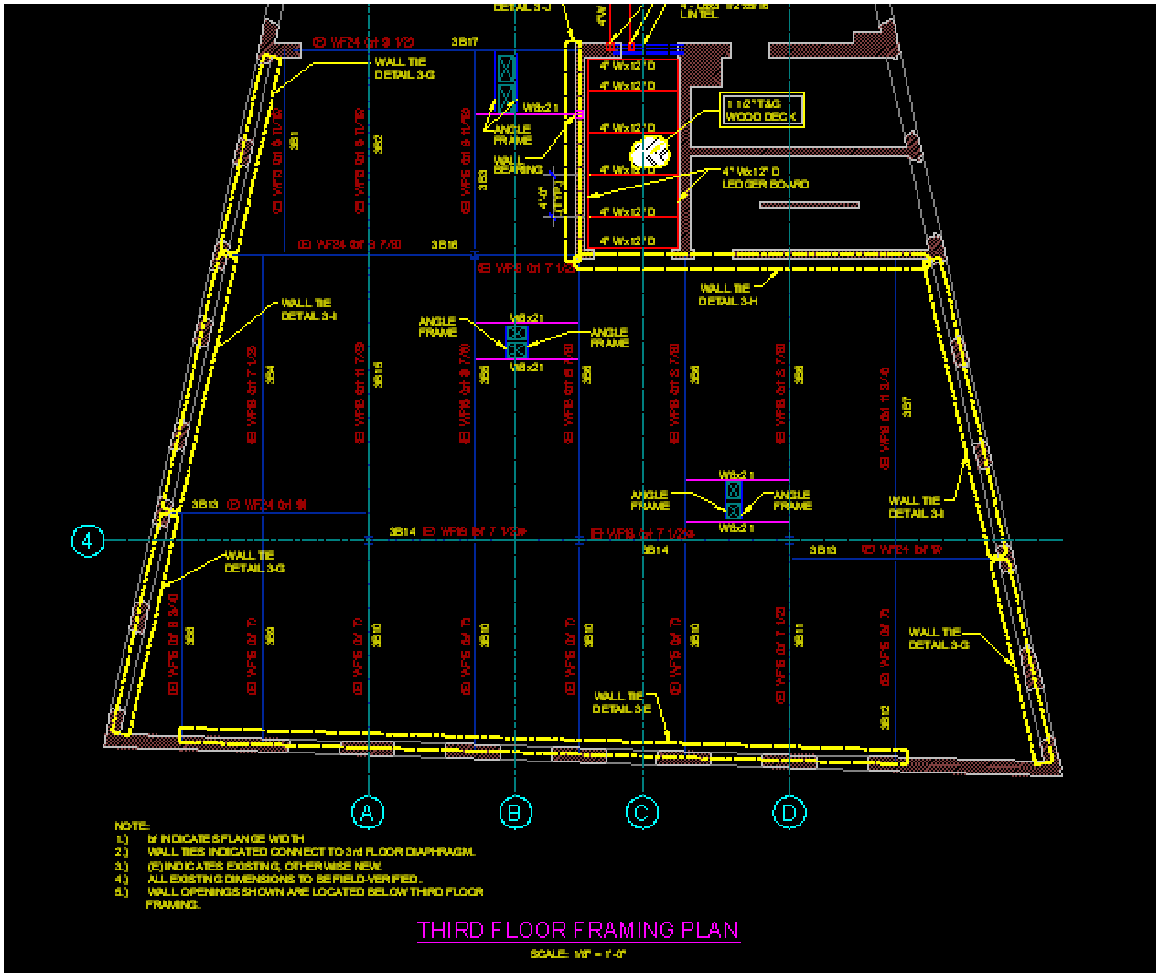

Figure 11: Steel Framing Plan – Third Floor (Adapted from Third Floor Framing Plan Created by Johnson & Seaman Engineering)

The Phase 2 portion of the building was also analyzed to establish the capacity of the existing members. The existing structural steel design was assumed to have a yield stress of 30 $kips/in^2$ with the following allowable stresses utilized for this analysis:

Fb = 18 ksi = Allowable stress in bending.

Fv = 12 ksi = Allowable stress in shear.

E = 29,000 ksi = Modulus of elasticity.

All of the steel roof beams were analyzed with the same analysis concept as the wood beams in Phase 1, and the total load capacity was found to be greater than 65 pounds per square foot.

The dead load of the roof consisted of a tar and gravel roof system, minimal insulation consisting of fiber board, wood decking, the structural members themselves and miscellaneous mechanical, electrical and architectural loads. These latter loads included sprinklers, lighting, ductwork, and electrical conduit. The total roof dead load summed to approximately 20 pounds per square foot.

In Phase 1 of the existing building analysis, the total load capacity was approximately 45 pounds per square foot with 25 pounds per square foot remaining for the snow load capacity which is inadequate for the current MA Code. The MA Code requires 35 pounds per square foot of snow load capacity. Therefore, additional structural wood members were designed to be placed between the existing wood members. The new members were analyzed and designed for their tributary areas which reduced the live load loading on the existing members by a factor of two. The roof plan in Figure 8 indicates the existing structural wood members and the new ones that were installed.

Capacities of the existing second floor and third floor members were investigated utilizing the same analysis method as for the roof beams. The third floor total load capacity was found to range between 70 pounds per square foot and 210 pounds per square foot in the Phase 1 area. In the Phase 2 area, the total load capacity ranged between 175 pounds per square foot and 600 pounds per square foot. The second floor framing members were identical to the third floor framing members. The intended use of the building is for office space on the second and third floors for which the required capacity according to Table 1606.1 of the MA Code is 50 pounds per square for offices, 100 pounds per square foot for lobbies and 80 pounds per square foot for corridors above the first floor. With the correct layout and use of space, the existing structure should be adequate for the proposed loads.

6.1.2 Two-Dimensional Analysis of Proposed Lateral System and Related Building Modifications for Code Compliance

The actual recent history of the structure (according to Johnson & Seaman Engineering, Inc.) is that the building was classified as a historical structure during the design process, and a lateral analysis was not performed. The cost savings to the owner could not be ignored despite the additional costs incurred for classification of a historic structure; i.e. – window modifications, color changes, etc. If the building was not listed on the historical register, a complete seismic upgrade would have been required due to the change in use of the first floor from Factory/Industrial to a Restaurant.

According to Table 3403 (Chapter 34 of the MA Code), the Hazard Index would change from 3 to 5. For the second and third levels, the change in use is Factory/Industrial to Business. The Hazard Index changes from 3 to 2. According to

Table 340.8.1 (Chapter 34 of the MA Code) the change in use and cost of alterations would classify the structure as Seismic Hazard Category 3. Section 3408.5.4.5 states "For seismic Category 3: Full compliance with 780 CMR 1612.0 is required, except as provided in 780 CMR 3408.5.4.6 and 3408.6.4, and except that existing structural systems not conforming to the requirements of 780 CMR 19 through 23 may be considered to participate in resisting lateral seismic loads, but only if the seismic design force is calculated in accordance with 780 CMR 3408.6.1.1."[3]

Due to the overall lack of lateral capacity of the existing structure (un-reinforced masonry walls) to meet the design seismic loads, a completely new lateral system is required. The new type of lateral load resisting system must be identified and locations must be chosen. The type of lateral system chosen that has been proven economical on other projects is a system consisting of steel tube bracing and the locations of which are indicated on the attached framing plans. The vertical bays span floor to floor, from the first floor slab to the roof framing level. The wood columns and beams in the Phase 1 area will need to be changed to steel members where the vertical braced bays will be located due to the increase in vertical loads.

In order to design the braced bays, the dead weight and half of the required design snow load of the structure must be calculated. The lateral calculations (pg. LAT 3 in Appendix E) indicate a total dead load weight of 2,220 kips which includes half of the snow load that is required by design on the roof. The following seismic criteria were used to determine the lateral forces at each level (reference Appendix E – calculation pages LAT 2, 3 & 4):

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[3] William F. Galvin, Secretary of the Commonwealth, The Massachusetts State Building Code - 6th Edition, February 1997, Page 455

Section 1612.2.3: Av, Aa = .12 g

Table 1612.2.5: Seismic Hazard Exposure Group II (use group A – first floor)

Table 3403: Hazard Index – 5

Section 1612.2.7: Seismic Performance Category – C

Table 1612.4.1: Site Coefficient S2 (assumes S2; S = 1.2)

Table 1612.4.4: Load Bearing Wall System (concentrically braced frames)

- Response Modification Factor R = 3.5
- Deflection Amplification Factor C_d = 3.5

Figure 12: Seismic Criteria
(Adapted from The Massachusetts State Building Code – 6th Edition)

The design concept utilized is that the lateral resisting system is completely new and therefore Table 1612.4.4 factors are used. The factors from the table are actually more conservative than the factors found in Table 3408.2 for a steel system with steel braced frames without gravity loads in the braces. (R = 5.0 and C_d = 4.5). The seismic base shear equation is:

V = Cs x W - Equation 1.0

Where W is the weight previously calculated and Cs is calculated on Page LAT.3 in the Appendix as 0.0831.

The seismic base shear is calculated as:

V = 0.0831 x 2220 kips = 184.48 kips.

A wind load calculation is performed and the total force is calculated as 74.6 kips which is less than the seismic base shear. An additional 5 percent is applied to the seismic base shear to approximate torsion (see MA Code Section 1612.5.3.1). The revised seismic base shear is calculated as:

V = 184.48 kips x 1.05 = 193.7 kips.

The vertical distribution of forces is calculated on Page LAT 4 in Appendix E in accordance with Section 1612.5.2.

The lateral load at the roof is calculated as 85.4 kips, 69.84 kips at the third floor and 40.74 kips at the second floor. The analysis of the vertical braced bays is then performed. This building is constructed of wood floors in lieu of concrete. Wood floors or wood diaphragms are not nearly as stiff as concrete slabs. Consequently, wood diaphragms are considered flexible diaphragms while concrete slabs are known as rigid diaphragms. Due to the flexibility of the floor and roof diaphragms, the approximate tributary loads are applied to each brace by assuming that the weight of the building can be distributed evenly per linear foot of length. In the long direction of the building (braced bay frame lines A, B, C and D) the lateral loads were applied equally to each frame. Figures 8,9,10 and 11 indicate the column lines of the vertical braced bays. The total load at each level was divided by four and applied accordingly. The free body diagram (page LAT 5 of Appendix E) of the frame is the same for the four braced bays since the dimensions of each frame are the same.

In the short direction of the building, the lateral loads were divided based upon the spacing between the adjacent braced bays. The tributary forces are calculated on page LAT 6 of Appendix E and the applied force is calculated. The closer the braced bay is to another braced bay, the lower the lateral force. The free body diagrams of the four braced bays are indicated on pages LAT 7 through LAT 10 Appendix C. On each free body diagram, the member forces in the braces are calculated, and the member force is labeled tension or compression.

With the member forces defined, the member sizes can now be designed. Section 1612.2.1 indicates the load combinations that should be applied. To verify overturning in the footing, load combination 6 should be applied which consists of 0.67 x dead load plus 0.8 x seismic load. The steel braces can be designed for the tension or compression force multiplied by 0.75 (an allowable reduction factor that categorizes the seismic force as a temporary load). The total dead load must be found that the columns supporting the braced frames must support. The allowable dead load to resist overturning, which would also include the weight of the footing and any soil or concrete that may bear on the footing, must be multiplied by a factor of 0.67. The 0.67 factor will provide a factor of safety of 1.5 for overturning and help eliminate error in the calculation of the applied dead loads. Due to the nature of this type of construction, the footing sizes will require modification and the structural members that bear on the braced bay column footings will require shoring (temporary support) during construction.

6.1.3 Diaphragm Analysis

Two areas of analysis were performed on the existing building by hand calculations and are included in Appendix F. The Phase 1 area (wood portion of the building) was analyzed in a small area footprint of 9’- 6” by 12’- 0”. The Phase 2 area (steel portion of the building) was analyzed in a small area footprint of 20’- 0” x 20’- 0”. The hand calculations were performed on the third floor. The lateral load at that level was calculated as 751.3 kips. The total floor area is summed to equal 8584 square feet. The load per square foot of diaphragm was calculated as 87.5 pounds per square foot. The calculation of the diaphragm uniform load assumes that the lateral force is applied evenly to the floor system. A simple beam analysis is applied to a 1 foot wide plank for an analysis using simply supported ends. The load per nail is found and then compared to the allowable shear as per the NDS (National Design Specification) Supplement for Wood Construction (ANSI/AF&PA 2005).

6.2 Three-Dimensional Analysis of 89 Shrewsbury Street

6.2.1 Modeling of Building

In order to create the three-dimensional model, all geometries had to be established for the building. The exterior masonry bearing walls were changed to steel or wood columns depending on where they were located (Phase 1 or Phase 2). The program cannot be modeled easily for continuous masonry bearing walls. Joints, joint coordinates, members, member sizes and restraints were generally easy to input. The RISA-3D Manual (RISA Technologies 2006) made the ease of input straight forward and having previous knowledge of STAAD (Research Engineers 2007), which is a more complicated, non-user friendly program, made the spread sheet based RISA Program relatively easy to comprehend. Error messages are created if a member or joint number has been used previously. The load input is also similar to STAAD in that the loads can either be entered globally or locally. The global system reflects the load as it pertains to the models X, Y and Z axes versus the local system which corresponds to the X, Y and Z axis of the individual member.

6.2.2 Lateral Loads – Proposed Building Use

The initial run of the model, once the lateral loads in one direction were input, produced instability errors at certain joints. All of the intersections between the columns and beams at each floor level were modeled the same as BenPIN but only certain joint intersections came up with the instability error. BenPIN releases My and Mz, AllPIN releases Mx, My and Mz. Risa-3D has a note with the AllPIN and BenPIN definition which states "RISA-3D will not allow you to release the member torsion at both ends. This is because it will be unstable as it would be free to spin about its centerlines. For this reason, pinned end conditions should be modeled using the BenPIN entry instead of AllPIN."[4] Upon reading the RISA-3D Manual, the error was caused by member and release issues. The manual states "Overuse of member end releases and/or boundary conditions is by far the most common cause of instability. The solution is to either remove a member end release or change a boundary condition so that the joint is restrained. At least one member or boundary needs to be fixed at each joint to prevent instability. If you think of a joint as the end of a member and specify no release for that member end, this member still will not experience moment at the end if all other elements are left unfixed."[5]

Although the RISA-3D Manual states that one member and a joint intersection must be fixed, the program ran the RISA-3D model with only a handful of members with fixed ends. In the member description, the end condition of certain members was changed to fix although the other ends were kept as BenPIN. Once the member ends were modified, the program ran the analysis without errors. For this study, the analysis RISA-3D program was only utilized for analysis purpose; its design functionality was not explored. As with the two-dimensional analysis, the columns that supported the braced frames were changed to steel if they were existing wood columns.

[4] RISA Technologies, RISA 3D Rapid Interactive Structural Analysis – 3 Dimensional, Version 6 General Reference, RISA Technologies, 2006, Page 262
[5] RISA Technologies, RISA 3D Rapid Interactive Structural Analysis – 3 Dimensional, Version 6 General Reference, RISA Technologies, 2006, Page 341

6.2.3 Diaphragm Analysis

The options for the diaphragm composition came from a list of essentially rigid diaphragms or plates. An option to choose a flexible diaphragm was not available. The RISA-3D Manual states that "the diaphragms in RISA-3D are extremely rigid and will not allow for relative displacement of the joints in that plane. While this is a common modeling assumption, it may not be appropriate for all circumstances. When you want to model diaphragm flexibility or when you are most interested in the force and deflection results of the concrete slab itself, then you would have to model the slab as a mesh of plate elements."[6] The RISA-3D Manual also states the following under the heading Flexible Diaphragms: "Using plate elements to create a concrete slab is an accurate way to model your floor slab. The relative rigidity of the floor slab to the shear walls and lateral frames will determine if you get rigid, semi-rigid or flexible diaphragm behavior."[7] Under the heading Diaphragm Stiffness, the RISA Manual states "you may alter the stiffness of the diaphragm, though this value should almost never be changed. Arbitrarily changing the diaphragm stiffness without understanding the ramifications on the stiffness solution can produce solution results that are inaccurate."[8] The load distributed to each brace will depend upon the flexibility of the plates.

Two trial runs were completed with a concrete floor thickness of 1 inch and .001 inches in order to review the different results of the loads absorbed into the vertical braces and to determine if flexible diaphragm behavior could be achieved.

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[6] RISA Technologies, RISA 3D Rapid Interactive Structural Analysis – 3 Dimensional, Version 6 General Reference, RISA Technologies, 2006, Page 70
[7] RISA Technologies, RISA 3D Rapid Interactive Structural Analysis – 3 Dimensional, Version 6 General Reference, RISA Technologies, 2006, Page 70
[8] RISA Technologies, RISA 3D Rapid Interactive Structural Analysis – 3 Dimensional, Version 6 General` Reference, RISA Technologies, 2006, Page 70

7.0 COMPARISON BETWEEN TWO-DIMENSIONAL ANALYSIS AND THREE-DIMENSIONAL ANALYSIS

The results from the lateral analyses of the two-dimensional hand calculations and the three-dimensional model are compared in the following tables.

7.1 Lateral System

Table 3: Seismic Analysis Results Table (Concrete Floor “t” = 1”)

	Hand 3rd Floor	**3D** 3rd Floor	**Hand** 2nd Floor	**3D** 2nd Floor	**Hand** 1st Floor	**3D** 1st Floor
Col. Lines A,D	T=C=12.31	T=11.86 11.61 C=13.29 13.13	T=C=22.39	T=21.31 20.55 C=22.00 21.34	T=C=28.41	T=27.84 28.46 C=28.02 28.78
Col. Line B **Computer Model – 2 Bays**	T=C=12.31	T= 6.64 6.33 C= 8.61 8.42	T=C=22.39	T=15.17 12.23 C=14.31 16.54	T=C=28.41	T=23.34 14.32 C=15.99 23.64
Col. Line C **Computer Model – 2 Bays**	T=C=12.31	T= 7.20 6.49 C= 9.68 8.71	T=C=22.39	T=16.73 12.66 C=16.05 17.23	T=C=28.41	T=20.87 12.54 C=14.39 22.01
Col. Line 1 **Computer Model – 2 Bays**	T=C=15.04	T= 6.29 10.48 C= 7.44 4.90	T=C=27.30	T=13.20 15.15 C=12.78 12.16	T=C=34.65	T=19.09 15.96 C=14.41 18.95
Col. Line 2 **Computer Model – 2 Bays**	T=C=6.18	T= 5.32 10.68 C= 7.19 6.05	T=C=11.23	T=11.88 14.24 C=11.94 12.03	T=C=14.23	T=16.86 14.60 C=14.29 16.02
Col. Line 3 **Computer Model – 2 Bays**	T=C=10.12	T= 2.95 9.43 C= 5.09 6.62	T=C=18.39	T=10.76 13.11 C=11.05 12.61	T=C=23.28	T=17.44 14.10 C=14.35 15.91
Co**l. Line 4** **1st Bay** **2nd Bay**	T=C=7.99	T= 5.94 6.50 C= 6.70 6.88	T=C=14.52	T=12.01 11.53 C=11.82 12.32	T=C=18.44	T=16.52 14.17 C=14.78 15.97

The table above (Table 3 Seismic Analysis Results) summarizes the member force results for the seismic analysis from both the two-dimensional hand calculations and three-dimensional model. Comparison of results between the hand calculations and the three-dimensional model indicate that the two methods have similar results when the braced bays are evenly spaced, and the results varied when the bays were not evenly spaced. Upon reviewing the results of the braced bay on Column Line A, (Column Lines A, B, C & D are approximately evenly spaced throughout the structure) the analysis results between the hand calculations and the computer model were within ten percent from one another. The analysis results of the braced bay on Column Line 4, when comparing the hand calculations and the computer model, were approximately twenty-five percent from one another. For comparison purposes, the following tables were created which include a list of the total tension in each of the braces in Column Lines 1, 2, 3 and 4 at the 3rd floor level.

Table 4: Results from RISA-3D

Column Line 1	6.29 kips + 10.48 kips = 16.77 kips
Column Line 2	5.32 kips + 10.68 kips = 16.00 kips
Column Line 3	2.95 kips + 9.43 kips = 12.38 kips
Column Line 4	5.94 kips x 2 Bays = 11.88 kips

The hand calculation results utilizing the flexible diaphragm method are as follows:

Table 5: Results from Hand Calculations

Column Line 1	= 15.04 kips
Column Line 2	= 6.18 kips
Column Line 3	= 10.12 kips
Column Line 4	7.99 x 2 Bays = 15.98 kips

There are discrepancies between the results due to the horizontal diaphragm constraints. The hand calculations use the tributary load concept with a flexible diaphragm which would apply a larger load on Column Line 4 and a lower load on Column Line 2. The computer model utilized a rigid diaphragm to calculate the load distribution and therefore, the load was more evenly distributed between the four braced bays based on the relative stiffness of the braces.

Figure 13 indicates the three-dimensional model created in RISA. The loads are not indicated in this figure for clarity purposes.

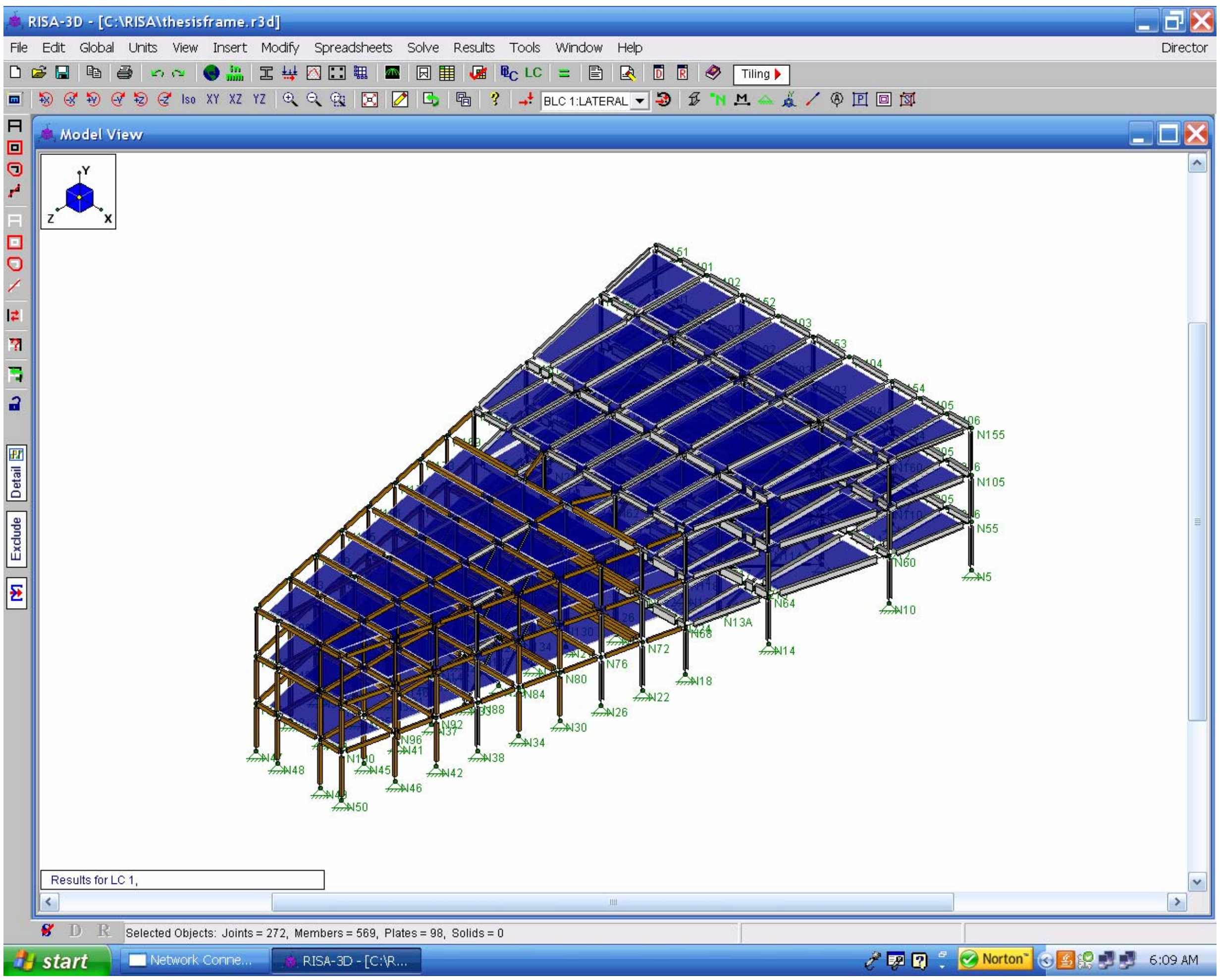

Figure 13: Three-Dimensional RISA Model
(Adapted From RISA Technologies)

7.1 Horizontal Diaphragms

Table 6: Seismic Analysis Results Table (Concrete Floor "t" = .001")

	Hand 3rd Floor	**3D** 3rd Floor	**Hand** 2nd Floor	**3D** 2nd Floor	**Hand** 1st Floor	**3D** 1st Floor
Col. Lines A,D	T=C=12.31	T=14.25 13.73 C=15.05 15.27	T=C=22.39	T=26.63 25.79 C=26.66 26.89	T=C=28.41	T=31.29 35.92 C=31.31 36.52
Col. Line B **Computer Model –** **2 Bays**	T=C=12.31	T= 5.47 5.25 C= 6.80 6.92	T=C=22.39	T=11.69 9.43 C=10.54 12.77	T=C=28.41	T=17.94 10.81 C=11.72 17.97
Col. Line C **Computer Model –** **2 Bays**	T=C=12.31	T= 5.84 5.15 C= 7.71 6.89	T=C=22.39	T=12.41 9.51 C=11.87 13.05	T=C=28.41	T=16.85 10.23 C=12.01 17.64
Col. Line 1 **Computer Model –** **2 Bays**	T=C=15.04	T= 5.78 10.13 C=11.57 8.11	T=C=27.30	T=14.61 16.52 C=16.99 16.10	T=C=34.65	T=22.74 18.06 C=18.18 22.87
Col. Line 2 **Computer Model –** **2 Bays**	T=C=6.18	T= 1.30 6.04 C= 3.49 6.95	T=C=11.23	T= 3.85 9.66 C= 5.19 9.56	T=C=14.23	T= 9.40 9.14 C= 7.16 11.02
Col. Line 3 **Computer Model –** **2 Bays**	T=C=10.12	T= 2.00 7.27 C= 7.98 8.39	T=C=18.39	T= 9.68 12.76 C=12.15 13.55	T=C=23.28	T=17.42 13.86 C=15.19 16.15
Co**l. Line 4** **1st Bay** **2nd Bay**	T=C=7.99	T= 6.60 7.46 C= 9.08 8.17	T=C=14.52	T=14.03 13.53 C=14.53 14.60	T=C=18.44	T=19.11 16.19 C=17.31 18.62

When comparing the results referenced in above Table 6 Seismic Analysis Results, the stiffness of the diaphragm (concrete thickness = .001”) did change the load to the braces and more closely resembled the hand calculations than the previous analysis utilizing a concrete thickness of 1”. Most notable were the forces in the braces on Column Line 4 which resulted in values closer to the hand calculations. Although the RISA-3D Manual clearly states using caution when varying the stiffness of diaphragms, it appears that forming flexible diaphragm stiffness similar to an actual wood diaphragm is possible.

8.0 CONCLUSIONS

The following conclusions are the writer's opinion of the future of three-dimensional modeling. A comparison is made between the previous software revolution with respect to CAD drafting and the new tree-dimensional software of analysis, design, and drafting. Also, true life experiences are of the writer are presented with first hand examples of engineering outsourcing and opinions of the future of structural engineering are provided.

8.1 Three-Dimensional Modeling and Computer Aided Drafting:

The hand calculations that were performed were completed in a manner that has been used for 13 years at Johnson & Seaman Engineering. The experience that was utilized at Johnson & Seaman Engineering was adopted from many years of experience at a small firm in West Springfield, MA and additional years at a large firm in Cambridge, MA. In other words, the design and analysis procedures performed for the two-dimensional analysis have many years of experience backing the results.

The "Basic Programming" tools used to facilitate the hand calculations are approximately 20 years old and have changed little. The gravity calculations that are part of this thesis paper are drawn from a real-life situation, and the case study with respect to the gravity analysis is the result of a successful project. The analysis and design of the necessary roof reinforcing is at least two years old, and there have been no issues to date. A comfort level at Johnson & Seaman Engineering has been established due to experience. The test of time has proven that the design and analysis process actually works.

The concept of three-dimensional modeling for analysis and design is very exciting as compared to the older analysis methods. The timing of the three-dimensional software is reminiscent of a similar change that occurred when computer drafting became the industry standard and hand drafting became obsolete. Johnson & Seaman Engineering had a chance to have a clientele due to the purchase of equipment that provided computer drafting. As a witness and part of a firm that went through that major shift, the story of three-dimensional modeling and analysis may unfold similar to what occurred with computer drafting.

In the early 1990's, multiple software programs were made available that could provide drafting capabilities. Plotters consisted of multiple ink pens that often ran out of ink prior to the completion of a plan. A typical plot of one sheet would take up to one hour to produce and the computers that ran the drafting software were incredibly slow as compared to today's computers.

As an employee at Harvey & Tracy Associates in West Springfield, MA, the company was set up very simply with respect to technology. The structural designs consisted of the design of new or renovated buildings with typically one, two or three stories. The drafting was accomplished by hand on traditional drafting tables, and a majority of the work was on mylar plans (plastic film in lieu of paper). The only way to reproduce a plan was to utilize the ammonia-based blueprint machine with one copy at a time.

Although computers were available, this firm initially utilized powerful hand calculators that would run simple basic programs. As the programs were developed with increasing complexity, they took up more memory and unfortunately some calculators were reserved for only one program such as a column analysis. These basic programs created excitement for all levels of engineers, and made for a better product than what had previously been produced. Less time was spent trying to group beams and columns to reduce analysis time, and more time was spent on more accurate analyses. Often, the basic programs were checked against hand calculations to find a comfort level with the new system which had not changed since the invention of the calculator. The simple programs did not eliminate jobs or rid the company of experienced professionals; the product simply became more accurate. Hand drafting remained the norm during this phase (early 1990's) although the architectural clients were trying to upgrade to the computer drafting system. Designer drafts-people in architecture and engineering had spent a majority of their careers crafting their drafting techniques and their expertise with the design details that were essential to build buildings. There was an architectural client in West Springfield that was heavily involved in the design of schools. They had some of the most articulate hand drafters in the profession in that area.

By the early 1990's, the United States was in the middle of a severe recession. A majority of design work had disappeared due to a lack of work, and firms were downsizing and trying to become more technology savvy. The small structural engineering firm in West Springfield purchased one computer to load all of the basic programs and to adopt some small rigid frame analysis programs. Due to the climate of the economy and lack of work, the writer took a job in Boston for a short career on the Central Artery Tunnel Project and then to a large engineering firm in Cambridge, MA. Both firms had the resources to modify their drafting from hand drafting to computer aided drafting. They had multiple training seminars for their hand-picked employees whom they chose to teach. For the most part, the ages of the computer illiterate group of workers were generally in the 50's, and their jobs utilizing hand drafting were phased out. The middle aged and younger workers took over for all computer drafting.

At the Cambridge design firm, many of the draft-persons were relocated into a group that was responsible for the review of shop drawings. At least the firm had the foresight to keep most of the experienced staff in house and for quality control; the shop drawing reviewers were the last line of defense prior to the project actually being constructed. The Cambridge design firm could afford ink jet plotters that replaced pen plotters. A typical plot could be created in 7 minutes or less, the ink lasted much longer and the plotter paper was a continuous roll feed. Their system was now upgraded as compared to the previous workforce, which consisted of many hand drafters that were now replaced by a handful of CAD operators.

The larger companies transferred well experienced designer draft- persons to the shop drawing department. The engineers at the Cambridge design firm remained the reviewers of the contract documents but had little to do with the shop drawing review process. The new CAD operators also had little to do with the shop drawing review process. As the writer made visits to the employees of the shop drawing department, conversations would be overheard with respect to the contract documents created by the CAD operators and reviewed by the engineers. The saying was "The contract documents were a good set of plans to create a final set of contract documents." The shop drawing reviewers would take a full set of finished contract documents and mark then up with red pens in order to produce a set of drawings that they felt were qualified to process shop drawings.

The department head in the structural group would overhear discussions of the previous designer drafts-people reviewing what details were more applicable for a certain project and how those details would apply. Now the discussions among the CAD operators consisted of what layer is to be used and whether or not the company will purchase the latest version of AutoCAD.

Technology created new opportunities for younger computer-oriented personnel but also forced more experienced staff out of the industry. An experienced engineer had to be aware of the experience level of their support staff to avoid potential problems. Technology also has had a global effect on our economy where ideas and information can be transferred almost instantaneously due to the Internet.

8.2 Engineering Outsourcing:

As the Cambridge engineering design firm went through hard times towards the middle 1990's, another round of transformation would occur. They were now forcing the older, more experienced structural engineers to retire. They also were trying very hard to create new engineering offices in the Middle East and India due to the significant savings in labor costs. In the winter and spring of 1994 and 1995, the Cambridge structural department eliminated their most experienced engineer and then had a meeting to discuss why overseas engineering offices were important to the survival of the firm. The structural group sat in a conference room to listen to how and why the firm had to survive (even if some members of the group would not be there in the future). Costs had to be controlled and thanks to the Internet, overseas offices were very feasible. It was now the company's goal to reduce costs with foreign labor and if employees did not like it, too bad. After one of the more critical meetings in which my direct supervisor said, "Isn't it better to have a job in which we work with our staff overseas (even if we just send work and then check it) then not have a job at all?" The writer went into the boss's office and asked point blank "Can I kept my current job and not be part of the team that trains and sends work to India?" The response was no loss of job would occur but it would be a negative impact for the writer's future.

Fortunately, the writer had kept in touch with the engineers in the small structural firm in West Springfield who stated that they were winding down their careers and that the creation of a new firm should be considered. The employees were of retirement age and were in their sixties and did not want to go through with the expense of purchasing multiple computers, drafting software and a plotter. A site visit was made by the writer in early 1995, and they were drawing all of their projects by hand although a majority of their architectural clients had converted to computer drafting. Out of curiosity, the writer also visited the architectural firm that had the more experienced designer drafters and was surprised at what had occurred. The older employees had been let go, and the firm had transformed into a CAD-only production firm. The drafts-people creating the plans were the least experience personnel.

8.3 Creation of Johnson & Seaman Engineering:

The decision to create a structural engineering design firm was easy for the writer. A lot of that decision stemmed from the Cambridge firm sending work to India and leaving the American employees with a feeling that they could not control their own destiny. In time, most of the work would be sent to India and other overseas offices and a majority of the jobs in the United States would be eliminated. The architects working with the small structural engineering firm in West Springfield were aware that the firm would not be in business much longer.

Johnson & Seaman Engineering in reality was given a chance to have clients because the firm purchased computers, software, and a plotter. However, in order to keep the clients, the engineering and contract documents would have to be correct with the level of service the architects' expected. At initial client contact meetings, the architects would ask repeatedly "Are you using CAD drafting and do you have your own plotter?" They rarely asked if Johnson & Seaman Engineering was technically prepared to do their work.

8.4 Final Statement

In a very confined time period (1990 – 1995) the experienced designer drafts-people were either eliminated or transferred to other positions and lesser experienced (and fewer of them) were trying to create contract documents. Work was now also being shipped to India by way of the Internet, and many years of experience had been eliminated with reductions in workforce through layoffs and attrition.

Now we come to next major shift. Johnson and Seaman Engineering has settled into utilizing basic programs, two-dimensional analysis for rigid frames and CAD drafting for contract document creation. The experience of the architects and engineers has developed since the loss of the older group of designer draftspersons. The larger firms transfer as much work as possible to overseas firms to maximize profit. Quality control is most likely accomplished by reviewing the work in the United States by the engineers that stamp the plans. The smaller firms have not moved to outsourcing work to overseas firms, although many are contacted by American middle-men who would like to assist in moving work overseas for their own profit and to maximize the owner's profit here.

The dilemma that faces a firm such as Johnson & Seaman Engineering is whether or not to move to three-dimensional design, analysis and drafting. The contractors can afford to purchase three-dimensional modeling software and verify its efficiency for coordination purposes between architectural and engineering disciplines. The software companies are in the process of providing three-dimensional only programs, and drafting can be accomplished by training existing personnel by adding to their knowledge of the software. The real issue is how far the three-dimensional software should be used and trusted. RISA-Floor (RISA Technologies 2006) is widely utilized for gravity analysis and design. Once the framing plan is created for analysis and design purposes, the file can be transferred to a CAD package and framing plans are generated. The three-dimensional software does not create specific details which are unique to each building. This is where experience is critical in how to construct a building.

The basic programs utilized by Johnson & Seaman most likely take longer than RISA-Floor but a comfort level has been formed. The three-dimensional model as part of this thesis was fun to create but as a practicing structural engineer, the writer has little faith in the results; unless, the model can be checked by hand. How can the results be trusted when the input must be adjusted to produce an analysis? Also, the quantity of time to produce the model may far outweigh the fee to produce the product.

The realities are that our industry will move towards a three-dimensional system. The older and more experienced engineers will need to formulate and follow a check-and-balance process to feel comfortable with the model creation. The true "gut feeling of engineering" can get lost by transferring drafting and design to others. The use of computers and computer-aided drafting allows for a good looking product that may be partially incorrect or even totally incorrect. The forced elimination of experienced engineers in favor of less experienced engineers who have more facility with complex computer analyses would be a dangerous path for the future of our industry. As the baby boom generation ages and retires, their level of experience will become a commodity. The results of the computer runs may be impressive; however, without experienced eyes reviewing the results the solutions may be totally incorrect.

Now that a three-dimensional model has been created and a trial analysis has been performed with respect to lateral loads, the following method for design should be implemented in addition to the concept of three-dimensional drafting which in time will be incorporated. Three-dimensional drafting will occur in the very near future due to a push by the software firms to sell more products. A small engineering firm such as Johnson & Seaman Engineering should pursue the use of higher level gravity analysis and design programs to improve efficiency and limit errors. The use of a three-dimensional analysis and design software package should only be utilized as a check of two-dimensional work. The three-dimensional software has not been utilized long enough to have a comfort level for the small firm structural engineer.

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APPENDIX A

Appendix A

JSE JOHNSON & SEAMAN ENGINEERING, INC.

30 Faith Avenue, Auburn, MA 01501 (508) 832-3535 Fax (508) 832-3393

September 19, 2005

Dan Lewis
Lamoureux Pagano
14 East Worcester Street
Worcester, MA 01604

Re: Lateral Force Resisting System for the Proposed Renovations to 89 Shrewsbury Street, Worcester, MA

Dear Mr. Lewis:

The following is a report reviewing the proposed renovations at 89 Shrewsbury Street and the structural implications of these changes to the lateral force resisting system as per 780 CMR, the sixth edition of the Massachusetts State Building Code.

The existing building is constructed of brick bearing walls and wood and steel framing. The structure has been used as a factory and industrial building in the past. It is proposed that the building be converted to a restaurant and businesses.

According to 780 CMR 3403.1, buildings for factory and industrial have a Hazard Index of 3 while restaurants have a Hazard Index of 5. The change of use creating a hazard index greater than 4 due to the restaurant, along with the occupancy increase and comparative cost of the alterations with the structure's value, classifies 89 Shrewsbury Street as a Seismic Hazard Category 3 per section 3408.5. Structures in this category must comply with the regulations specified in 780 CMR 3408.5.4.5. This section of the code states that the building shall be in full compliance with the earthquake resistance requirements in 780 CMR 1612.0 except as provided in 780 CMR 3408.5.4.6 and 3408.6.4. These sections modify the maximum lateral earthquake forces indicated in section 1612.0 and require attachments for existing masonry walls. This includes tying the existing masonry walls to the roof and floor diaphragms, and removing or bracing any existing masonry parapets.

Without the addition of the restaurant, the Hazard Index for the proposed business use would be 2. This change of use to business, along with the comparative cost of the alternations with the structure's value, classifies 89 Shrewsbury Street as a Seismic Hazard Category 2. 780 CMR requires structures in this category to have all existing masonry walls tied to floors and roofs as indicated above but with no modifications to the lateral load resisting system of the building (no masonry shear walls removed), no structural upgrade to the lateral system would be required.

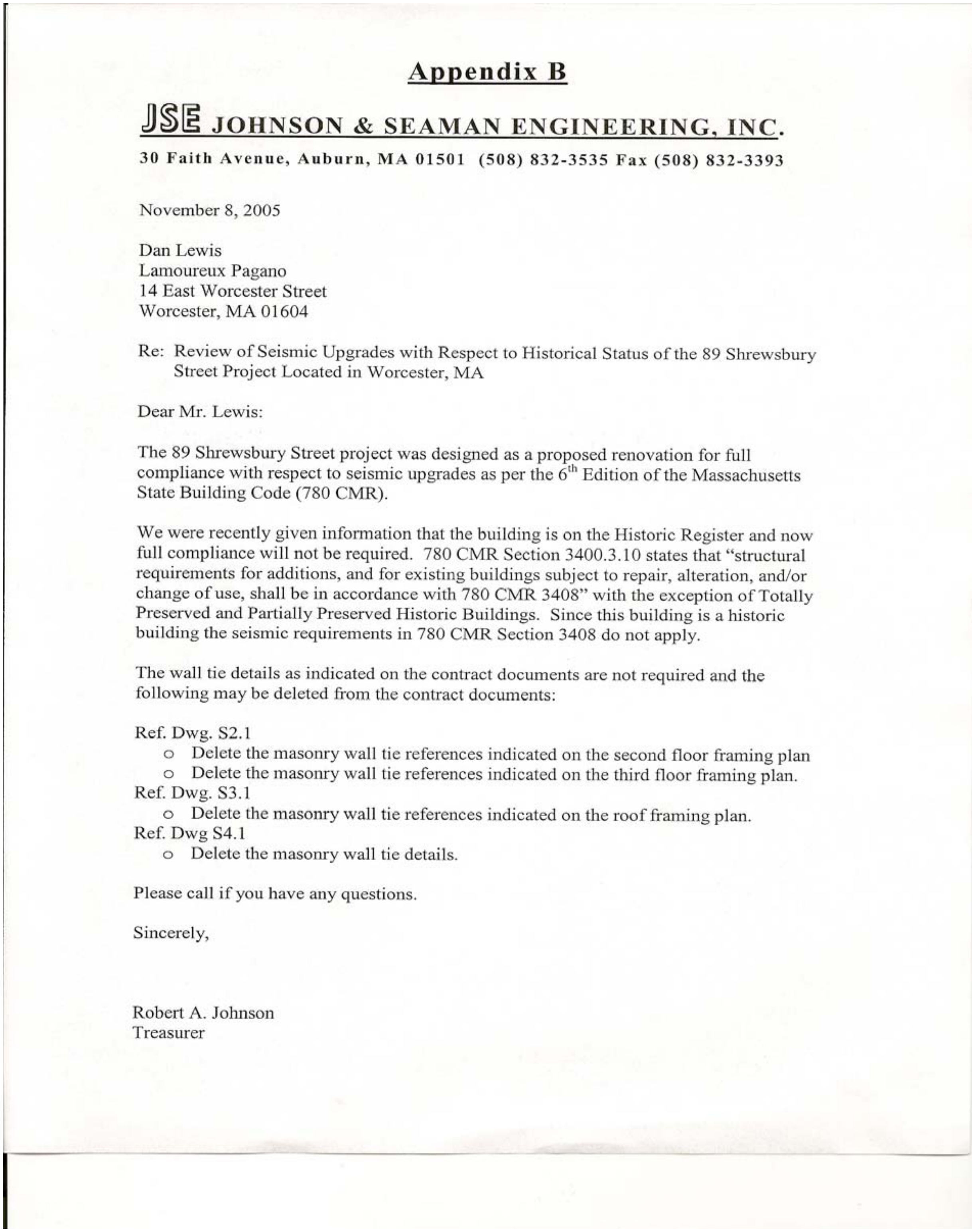
The requirements of 780 CMR indicated above for the change in use to a restaurant and business would require a full structural upgrade to the lateral force resisting system with more stringent criteria. This would include the construction of new shear walls and/or steel bracing at each level of the structure.

Please call if you have any questions.

Sincerely,

Robert A. Johnson
Treasurer

APPENDIX B



Appendix B

JSE JOHNSON & SEAMAN ENGINEERING, INC.
30 Faith Avenue, Auburn, MA 01501 (508) 832-3535 Fax (508) 832-3393

November 8, 2005

Dan Lewis
Lamoureux Pagano
14 East Worcester Street
Worcester, MA 01604

Re: Review of Seismic Upgrades with Respect to Historical Status of the 89 Shrewsbury Street Project Located in Worcester, MA

Dear Mr. Lewis:

The 89 Shrewsbury Street project was designed as a proposed renovation for full compliance with respect to seismic upgrades as per the 6th Edition of the Massachusetts State Building Code (780 CMR).

We were recently given information that the building is on the Historic Register and now full compliance will not be required. 780 CMR Section 3400.3.10 states that "structural requirements for additions, and for existing buildings subject to repair, alteration, and/or change of use, shall be in accordance with 780 CMR 3408" with the exception of Totally Preserved and Partially Preserved Historic Buildings. Since this building is a historic building the seismic requirements in 780 CMR Section 3408 do not apply.

The wall tie details as indicated on the contract documents are not required and the following may be deleted from the contract documents:

Ref. Dwg. S2.1
- Delete the masonry wall tie references indicated on the second floor framing plan
- Delete the masonry wall tie references indicated on the third floor framing plan.

Ref. Dwg. S3.1
- Delete the masonry wall tie references indicated on the roof framing plan.

Ref. Dwg S4.1
- Delete the masonry wall tie details.

Please call if you have any questions.

Sincerely,

Robert A. Johnson
Treasurer

APPENDIX C

FINITE ELEMENT MODELING OF WOOD DIAPHRAGMS

By Robert H. Falk[1] and Rafii Y. Itani,[2] Member, ASCE

ABSTRACT: This report describes a two-dimensional finite element model for analyzing vertical and horizontal wood diaphragms. Central to the development of this model is the formulation of a nonlinear finite element that accounts for the distribution and stiffness of fasteners connecting the sheathing to the framing. Linked with conventional beam and plane stress elements, which represent diaphragm framing and sheathing, respectively, the resulting model can be used to analyze a variety of wood diaphragms (walls, floors, ceilings, etc.). Load-displacement results from experimentally tested diaphragms and model predictions were found to be in good agreement. Parametric studies with the model show that diaphragm stiffness is significantly affected by nail stiffness, nail spacing, and the use of blocking. At code allowable diaphragm shear load levels, a variation of 20% in nail stiffness resulted in a change in diaphragm stiffness of less than 10%. Nail spacing was shown to have a more dominant effect on diaphragm stiffness than nail stiffness.

INTRODUCTION

Diaphragms are important components of wood-framed buildings that are used to resist and transfer the lateral shear forces produced by wind or earthquakes. The analysis of these complex components has been oversimplified because of a lack of understanding of their static and dynamic behavior. Analytical research efforts have primarily concentrated on the modeling of wall diaphragm behavior. Few models have been used to address the behavior of other types of diaphragms, such as floors and ceilings.

The purpose of this paper is to describe a nonlinear finite element formulated to represent the distribution and stiffness of the nails that secure sheathing to framing in a wood diaphragm. When linked with conventional beam and plane stress elements, which represent diaphragm framing and sheathing, respectively, the resulting model can be used to analyze a variety of diaphragms (walls, floors, and ceilings) with different geometry and loading arrangements.

We also present the results of studies performed with the developed model to determine the effect of varying input parameters—naiþroperties, nail spacing, and the use of blocking—on floor diaphragms.

RELATED RESEARCH

Though research on wood diaphragms dates back to 1927 (Peterson 1983), most of the research until the 1960s was experimental in nature and focused

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[2]Prof., Civ. and Environ. Engrg., Washington State University, Pullman, WA 99164-2914.

Note. Discussion open until August 1, 1989. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on January 15, 1988. This paper is part of the *Journal of Structural Engineering*, Vol. 115, No. 3, March, 1989. ASCE, Paper No. 23247.

89 SHREWSBURY ST. LPA 1/29/08
PG ER.1

(E) RB-1
CHECK EXIST. $9\frac{1}{4}'' \times 11\frac{3}{4}$ ROOF BEAM CAP.

L = 21'

$w = 19.5'/2 \times 50\,PSF = 488\ \#/'$

R = 5124
M = 26901
A = 85.40
S = 222.62
I = 1453

Fb = 1450	USE HIGHER
Fv = 90 × 2/3 = 60	VALUES W/O
E = 1,400,000	FACTORS

$9\frac{1}{4} \times 11\frac{3}{4}$ $A = 9.25 \times 11.75 = 108.7\ in^2$

$S_x = 9.25 \times 11.75^2/6 = 213\ in^3$

$I_x = 9.25 \times 11.75^3/12 = 1250\ in^4$

$w = 19.5'/2 \times 45\,PSF = 439\ \#/'$

R = 4610
M = 24200
A = 76.82
S = 200.27
I = 1307

$\boxed{\text{TOT. CAP.} = 40 \text{ TO } 45\,PSF}$

(E) RB-2
CHECK EXIST. $7\frac{1}{2}'' \times 11''$ $A = 7.5'' \times 11'' = 82.5\ in^2$

$S_x = 7.5'' \times 11^2/6 = 151\ in^3$

L = 19'

$I_x = 7.5 \times 11^3/12 = 832\ in^4$

$w = 19.5'/2 \times 45\,PSF$
$= 439\ \#/'$

R = 4171
M = 19810
A = 69.50
S = 163.94
I = 768

$w = 19.5'/2 \times 40\,PSF$
$= 390\ \#/'$

R = 3705
M = 17600
A = 61.75
S = 145.64
I = 860

$\boxed{\text{TOT. CAP.} = 40 \text{ TO } 45\,PSF}$

89 SHREWSBURY ST. | LPA | 2/8/08

(SECOND FLOOR)
(THIRD FLOOR – SAME)

2B-1

(E) 2B-1 (E) 3B-18

CHECK EXIST TRIPLE 7"x15" $L = 20.5'$

$w = 19'/2 \times 60$ PSF $= 570$ #/' (CAP = 60 PSF x 3 = 180 PSF)

TOTAL LOAD

7"x15" (ONE)

$A = 7 \times 15 = 105\ in^2$

$Sx = 7 \times 15^2/6 = 262.5\ in^3$

$Ix = 7 \times 15^3/12 = 1{,}968.75\ in^4$

R = 5842.5
M = 29,943
A = 93.38
S = 247.81
I = 1578.41

TOTAL CAPACITY = 180 PSF

(E) 2B-2 (E) 3B-18

CHECK EXIST. DOUBLE 9" x15"

$L = 18'$

$w = 19.5'/2 \times 100$ PSF $= 975$ #/' (CAP. = 100 PSF x 2 = 200 PSF)

9.0"x 15"

$A = 9 \times 15 = 135\ in^2$

$Sx = 9 \times 15^2/6 = 337.5\ in^3$

$Ix = 9 \times 15^3/12 = 2531$

R = 8775
M = 39488
A = 146.25
S = 326.79
I = 1827.71

TOTAL CAPACITY = 200 PSF

(E) 2B-3 (E) 3B-19

CHECK EXIST. 11"x15"

$L = 16'$

$w = 19.5'/2 \times 155$ PSF $= 1511.25$

11"x15"

$A = 11 \times 15 = 165\ in^2$

$Sx = 11 \times 15^2/6 = 412.5\ in^3$

$Ix = 11 \times 15^3/12 = 3093.75\ in^4$

R = 12090
M = 48360
A = 201.5
S = 400.22
I = 1989.67

TOTAL CAPACITY = 155 PSF

89 SHREWSBURY ST. LPA 2/8/08
2B-

(E) 2B-10

CHECK EXIST. WF15x50.5

$L = 16'$

$W = 11'/2 \times 600\ PSF = 3{,}300\ \#/' = 3.3\ K/'$

WF15x50.5

$A = 14.66\ in^2$

$S_x = 74.7\ in^3$

$I_x = 555.8\ in^4$

R = 26.4
M = 105.6
A = 2.2
S = 70.4
I = 209.73

TOTAL CAPACITY = 600 PSF

(E) 2B-11

CHECK EXIST. WF 15x50.5

$L = 19'$

$W = 20'/2 \times 240\ PSF = 2.4\ K/'$

WF15x50.5

$A = 14.66$

$S_x = 74.7$

$I_x = 555.8$

R = 22.8
M = 108.3
A = 1.9
S = 72.2
I = 255.44

TOTAL CAPACITY = 240 PSF

(E) 2B-12

CHECK EXIST. WF 15x50.5

$L = 19'$

$W = 12'/2 \times 400\ PSF = 2.4\ K/'$

WF15x50.5

$A = 14.66$

$S_x = 74.7$

$I_x = 555.8$

R = 22.8
M = 108.3
A = 1.9
S = 72.2
I = 255.44

TOTAL CAPACITY = 400 PSF

TOPS 35500

89 SHREWSBURY ST. LPA 2/9/08

LAT 1

LATERAL SYSTEM:

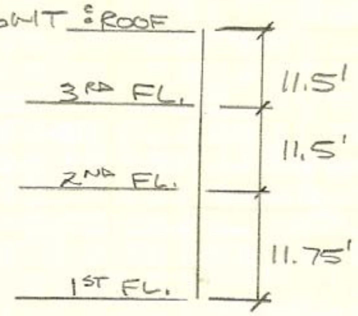
DESIGN OF LATERAL SYSTEM WILL CONSIST OF THE DISTRIBUTION OF THE LATERAL FORCES TO NEW VERTICAL STEEL BRACED BAYS, THE FLOOR AND ROOF DIAPHRAGMS ARE CONSIDERED FLEXIBLE AND THE LATERAL FORCES WILL BE DISTRIBUTED BASED ON TRIBUTARY AREA:

SEISMIC LOADS:

BUILDING AREA:

$\frac{1}{2} \times 148' \times 34' + \frac{1}{2} \times 148' \times 32 + 148' \times 25' = 8584$ S.F.

BUILDING HEIGHT: ROOF



FIND WEIGHT OF EXISTING STRUCTURE:

ROOF DEAD LOAD:

$8584 \text{ S.F.} \times 20 \text{ PSF} = 171680^{\#} = 171.7^{K}$

ROOF SNOW LOAD: (NO DRIFTING)

$8584 \text{ S.F.} \times 35 \text{ PSF} = 300440^{\#} = 300.44^{K}$

THIRD FLOOR DEAD LOAD:

$8584 \text{ S.F} \times 20 \text{PSF} = 171680^{\#} = 171.7^{K}$

SECOND FLOOR DEAD LOAD:

$8584 \text{ S.F.} \times 20 \text{ PSF} = 171680^{\#} = 171.7^{K}$

89 SHREWSBURY ST. LPA 2/9/08

LAT 2

WALL WEIGHTS: (SOLID EXTERIOR WALLS)
IGNORE INTERIOR WALLS.

THIRD FLOOR TO ROOF: 12" SOLID BRICK

$11.5' \times [25' + 152' + 152' + 91'] \times 120$ PSF

$= 579{,}600^{\#} = 579.6^{K}$

SECOND FLOOR TO THIRD FLOOR: 12" SOLID BRICK

$11.5' \times [25' + 152' + 152' + 91'] \times 120$ PSF

$= 579{,}600^{\#} = 579.6^{K}$

FIRST FLOOR TO SECOND FLOOR: 16" SOLID BRICK

$11.75' \times [25' + 152' + 152' + 91'] \times 160$ PSF

$= 789{,}600^{\#} = 789.6^{K}$

SEISMIC CRITERIA: (MA CODE - 6TH EDITION)

SECTION 1612.2.3 - A_v; $A_a = .12g$

TABLE 1612.2.5 - SEISMIC HAZARD EXPOSURE GROUP II (USE GROUP A - 1ST FL)

TABLE 3403 - HAZARD INDEX - 5

SECTION 1612.2.7 - SEISMIC PERFORMANCE CATEGORY - C

TABLE 1612.4.1 - SITE COEFFICIENT

- ASSUMED S_2 - $S = 1.2$

TABLE 1612.4.4 - LOAD BEARING WALL SYSTEM (CONCENTRICALLY BRACED FRAMES)
RESP. MOD. FACTOR $R = 3.5$ DEFL. AMP FACTOR $C_d = 3.5$

SECTION 1612.4.7.2.7 - FOR DIAPHRAGM DESIGN.

* DESIGN CONCEPT - COMPLETELY NEW LATERAL SYSTEM. ACCORDING TO TABLE 3408.2 - STEEL SYSTEM - STEEL BRACED FRAMES W/O GRAVITY LOADS IN BRACES
$R = 5.0$ $C_d = 4.5$ (USE $R = 3.5$, $C_d = 3.5$ - SEE ABOVE)

89 SHREWSBURY ST. LPA 2/9/08
LAT 3

SECTION 1612.5.1
SEISMIC BASE SHEAR:

$V = C_S \times W$

$W = \underset{\text{ROOF D.L.}}{171.7^K} + \underset{\text{½ ROOF SNOW}}{300.44^K/2} + \underset{\text{3RD FL D.L.}}{171.7^K} + \underset{\text{2ND FL D.L.}}{171.7^K}$

$+ \underset{\text{(3RD FL WALLS)}}{579.6^K} + \underset{\text{(2ND FL. WALLS)}}{579.6^K} + \underset{\text{(1ST FL. WALLS) *HALF}}{789.6^K/2}$

$= 2219.32^K \approx 2220$ KIPS

SECTION 1612.5.1.1 – C_S

$C_S = \dfrac{1.2 \times A_V \times S}{R \times T^{2/3}} = \dfrac{1.2 \times 0.12 \times 1.2}{3.5 \times (.4576)^{2/3}} = 0.0831$

$A_V = 0.12 \quad S = 1.2 \quad R = 3.5$

$C_a = 1.6$

SECTION 1612.5.1.2 – T_a

$T_a = C_T \times H_n^{3/4} \quad C_T = 0.02$

$T_a = 0.02 \times 34.75^{3/4} = 0.286$

$T = 0.286 \times 1.6 = .4576$

$C_S \leq 2.5 \times .12 / 3.5 = 0.0857$

$V = C_S \times W = 0.0831 \times 2220^K = 184.48^K$

COMPARE WIND LOAD: (WORSE CASE)

$152' \times \left[11.5' + 11.5' + 11.75/2\right] \times 17 \text{ PSF} = 74.6^K$

SEISMIC
CONTROLS

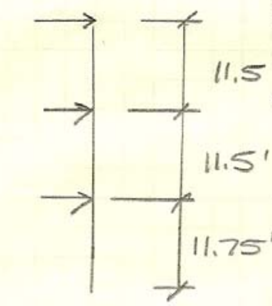
ADD ADDITIONAL 5% TO V TO
APPROXIMATE TORSION. (SECTION 1612.5.3.1)

$V = 184.48^K \times 1.05 = 193.7^K \approx 194^K$

89 SHREWSBURY ST. LPA 2/9/08

LAT 4

VERTICAL DISTRIBUTION OF FORCES:
SECTION 1612.5.2



$F_{ROOF} = .44 \times 194^K = 85.4^K$

$F_{3RD FL} = .36 \times 194^K = 69.84^K$

$K = 1.$ (PERIOD ≤ 0.5)

$F_{2ND FL} = .21 \times 194^K = 40.74^K$

$F_x = C_{vx} \cdot V$

$$C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$W_{2ND FL} = \underset{\text{1/2 2ND FL WALLS}}{579.6^K/2} + \underset{\text{1/2 1ST FL. WALLS}}{789.6^K/2} + \underset{\text{2ND FL D.L.}}{171.7^K}$

$= 856.3^K$

$W_{3RD FL} = \underset{\text{1/2 2ND FL. WALLS}}{579.6^K/2} + \underset{\text{1/2 3RD FL. WALLS}}{579.6^K/2} + \underset{\text{3RD FL. D.L.}}{171.7^K}$

$= 751.3^K$

$W_{ROOF} = \underset{\text{1/2 3RD FL. WALLS}}{579.6^K/2} + \underset{\text{ROOF D.L}}{171.7^K} + \underset{\text{(1/2 SNOW)}}{300.44^K/2}$

$= 611.72^K$

$$C_{vx}(ROOF) = \frac{611.72^K \times 34.75}{611.72 \times 34.75 + 751.3 \times 23.25 + 856.3 \times 11.75}$$

$= .44$

$$C_{vx}(3RD FL) = \frac{751.3 \times 23.25}{611.72 \times 34.75 + 751.3 \times 23.25 + 856.3 \times 11.75}$$

$= .36$

$$C_{vx}(2ND FL) = \frac{856.3 \times 11.75}{611.72 \times 34.75 + 751.3 \times 23.25 + 856.3 \times 11.75}$$

$= .21$

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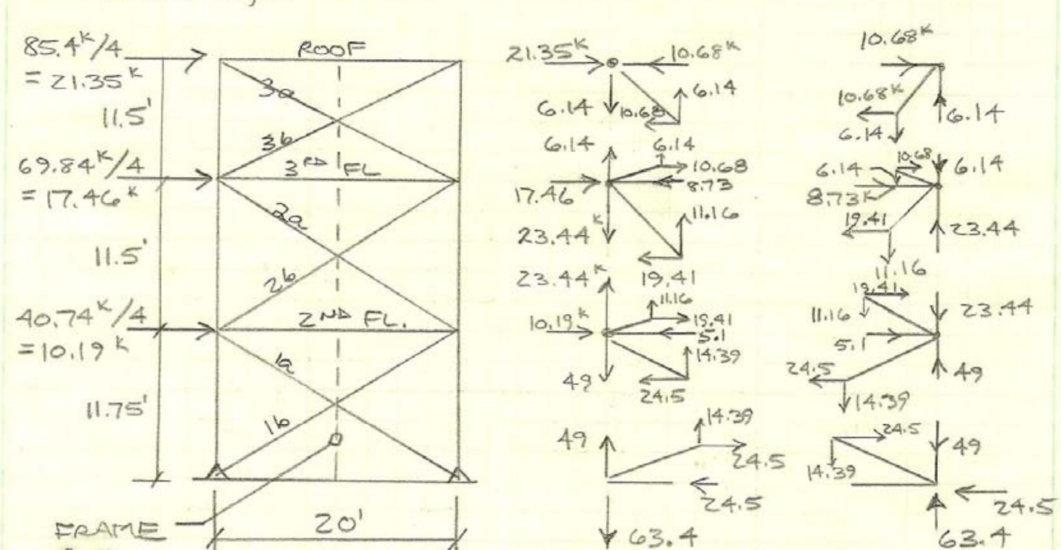
LAT 5

DESIGN OF BRACED FRAMES:

BRACED FRAME LINES A, B, C, D (TWO CONDITIONS) - APPLY LATERAL LOAD EQUALLY.

½ LOAD TO BE APPLIED TO EACH SIDE

FRAME A, D



B, C HAS MIDDLE COLUMN (NOT USED FOR LATERAL)

$\frac{10.68^K}{20} = \frac{F_{Y_1}}{11.5}$ $F_{Y_1} = 6.14^K$

$\frac{19.41}{20} = \frac{F_{Y_2}}{11.5}$ $F_{Y_2} = 11.16$

$\frac{24.5}{20} = \frac{F_{Y_3}}{11.75}$ $F_{Y_3} = 14.39$

FORCES (BRACES)

3a, 3b T=C
$\sqrt{6.14^2 + 10.68^2} = 12.3^K$

2a, 2b T=C
$\sqrt{11.16^2 + 19.41^2} = 22.4^K$

1a, 1b T=C
$\sqrt{14.39^2 + 24.5^2} = 28.4^K$

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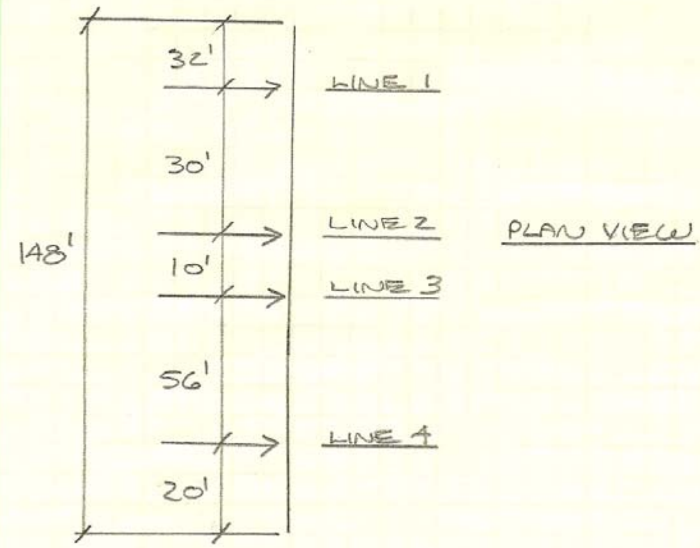
LAT 6

OPPOSITE DIRECTION:

DIVIDE LATERAL LOAD ON TRIBUTARY AREA. (FLEXIBLE DIAPHRAGM)



ROOF LEVEL:

$\text{FORCE (LINE 1)} = 85.4^{K} \times \left(\frac{32+15}{148}\right) = 27.12^{K}$

$\text{FORCE (LINE 2)} = 85.4^{K} \times (20/148) = 11.54^{K}$

$\text{FORCE (LINE 3)} = 85.4^{K} \times (33/148) = 19.04^{K}$

$\text{FORCE (LINE 4)} = 85.4^{K} \times (48/148) = 27.7^{K}$

THIRD FLOOR:

$\text{FORCE (LINE 1)} = 69.84^{K} \times (47/148) = 22.18^{K}$

$\text{FORCE (LINE 2)} = 69.84^{K} \times (20/148) = 9.44^{K}$

$\text{FORCE (LINE 3)} = 69.84^{K} \times (33/148) = 15.57^{K}$

$\text{FORCE (LINE 4)} = 69.84^{K} \times (48/148) = 22.65^{K}$

SECOND FLOOR:

$\text{FORCE (LINE 1)} = 40.74^{K} \times (47/148) = 12.94^{K}$

$\text{FORCE (LINE 2)} = 40.74^{K} \times (20/148) = 5.51^{K}$

$\text{FORCE (LINE 3)} = 40.74^{K} \times (33/148) = 9.08^{K}$

$\text{FORCE (LINE 4)} = 40.74^{K} \times (48/148) = 13.21^{K}$

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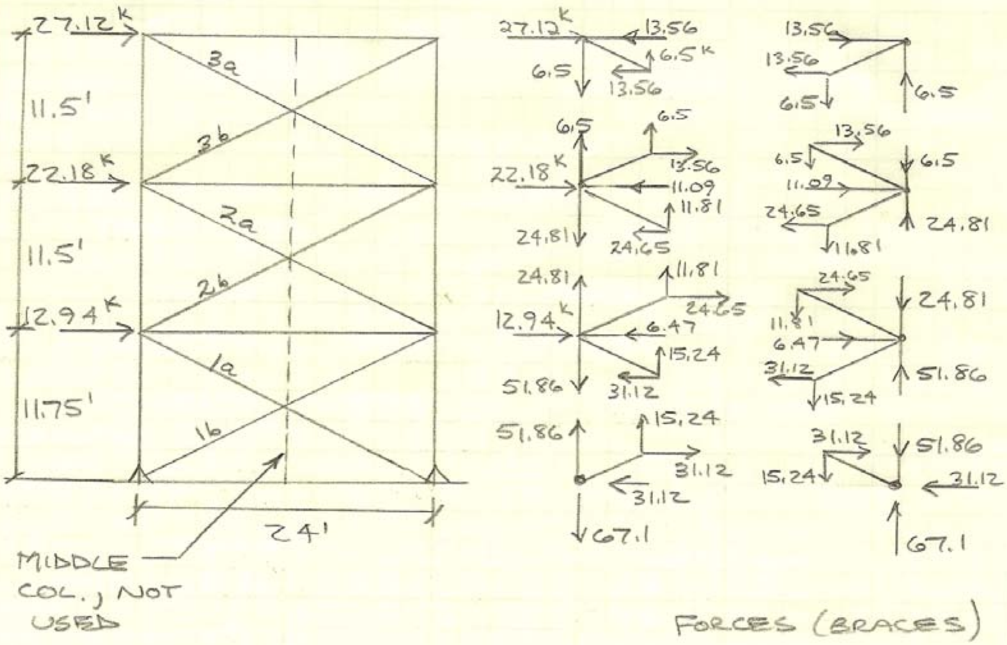
LAT 7

BRACED BAY - LINE 1



FORCES (BRACES)

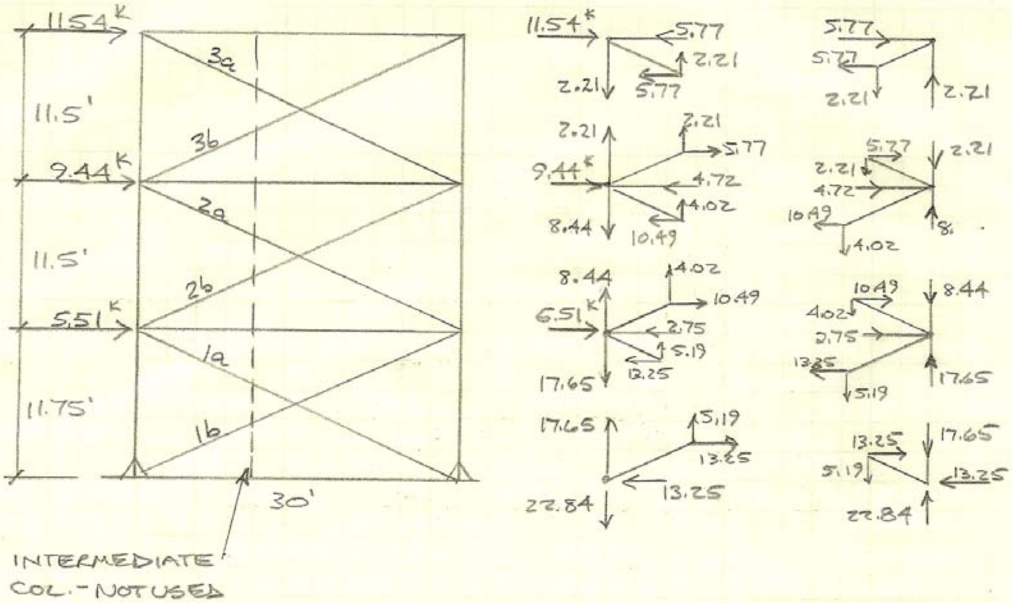
$\frac{13.56}{24} = \frac{F_{Y1}}{11.5}$ $F_{Y1} = 6.5^K$ 3a, 3b T=C $\sqrt{6.5^2 + 13.56^2} = 15.04^K$

$\frac{24.65}{24} = \frac{F_{Y2}}{11.5}$ $F_{Y2} = 11.81^K$ 2a, 2b T=C $\sqrt{11.81^2 + 24.65^2} = 27.3^K$

$\frac{31.12}{24} = \frac{F_{Y3}}{11.75}$ $F_{Y3} = 15.24^K$ 1a, 1b T=C $\sqrt{15.24^2 + 31.12^2} = 34.65^K$

89 SHREWSBURY ST. LPA 2/9/08 LAT 8

BRACED BAY – LINE 2



FORCES (BRACES)

$\frac{5.77}{30} = \frac{F_{Y_1}}{11.5}$ $F_{Y_1} = 2.21$ 3a, 3b T=C $\sqrt{5.77^2 + 2.21^2} = 6.18^k$

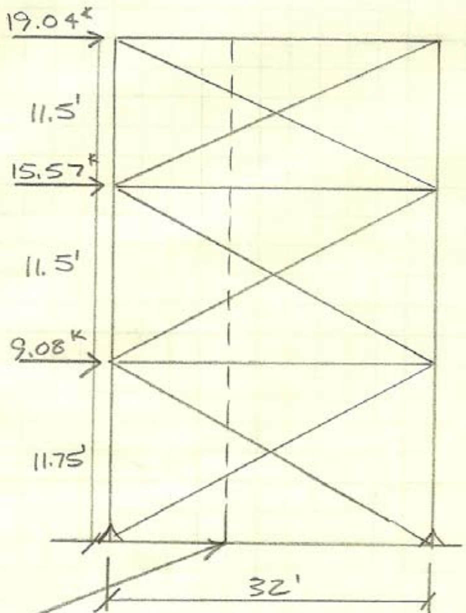
$\frac{10.49}{30} = \frac{F_{Y_2}}{11.5}$ $F_{Y_2} = 4.02$ 2a, 2b T=C $\sqrt{4.02^2 + 10.49^2} = 11.23^k$

$\frac{13.25}{30} = \frac{F_{Y_3}}{11.75}$ $F_{Y_3} = 5.19$ 1a, 1b T=C $\sqrt{5.19^2 + 13.25^2} = 14.23^k$

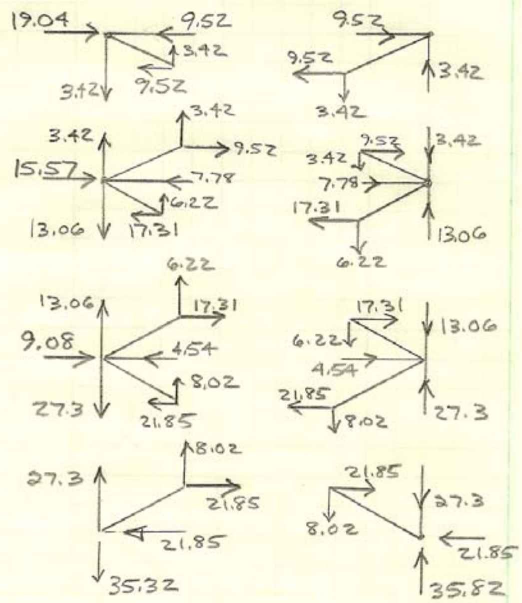
89 SHREWSBURY ST. LPA 2/9/08

LAT 9

BRACED BAY – LINE 3



INTERMEDIATE COLUMN – NOT USED



$$\frac{9.52}{32} = \frac{F_{y1}}{11.5} \qquad F_{y1} = 3.42$$

$$\frac{17.31}{32} = \frac{F_{y2}}{11.5} \qquad F_{y2} = 6.22$$

$$\frac{21.85}{32} = \frac{F_{y3}}{11.75} \qquad F_{y3} = 8.02$$

FORCES (BRACES)

3a, 3b T = C

$$\sqrt{9.52^2 + 3.42^2} = 10.12^k$$

2a, 2b T = C

$$\sqrt{6.22^2 + 17.31^2} = 18.39^k$$

1a, 1b T = C

$$\sqrt{8.02^2 + 21.85^2} = 23.28^k$$

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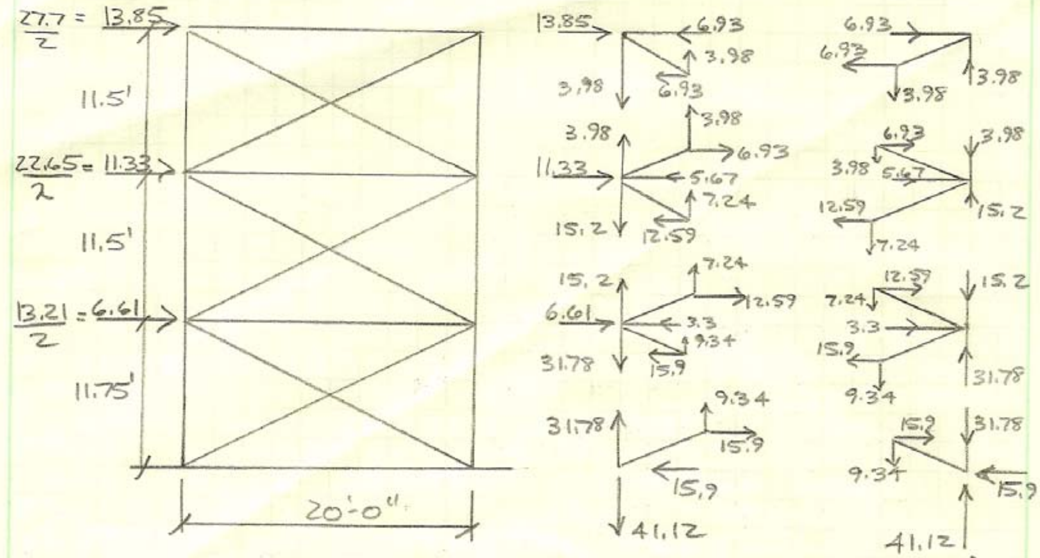
LAT 10

BRACED BAY - LINE 4 (2 BAYS - SAME DIM'S)



FORCES (BRACES)

$$\frac{6.93}{20} = \frac{F_{y_1}}{11.5} \quad F_{y_1} = 3.98 \qquad 3a, 3b \quad T = C \qquad \sqrt{3.98^2 + 6.93^2} = 7.99^K$$

$$\frac{12.59}{20} = \frac{F_{y_2}}{11.5} \quad F_{y_2} = 7.24 \qquad 2a, 2b \quad T = C = 14.52^K \qquad \sqrt{7.24^2 + 12.59^2}$$

$$\frac{15.90}{20} = \frac{F_{y_3}}{11.75} \quad F_{y_3} = 9.34 \qquad 1a, 1b \quad T = C = 18.44^K \qquad \sqrt{9.34^2 + 15.9^2}$$

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LAT 11

APPLY LATERAL LOADS TO 3-d MODEL:

FROM PG 4, LATERAL LOADS ARE

ROOF - 85.4 KIPS

3RD FL - 69.84 KIPS

2ND FL - 40.74 KIPS.

APPLY LOADS IN Z DIRECTIONS; (GLOBAL "Z")

IN 89' DIMENSION;

APPLY MEMBER LOADS IN "Z" DIRECTION:

ROOF MEMBERS 401A, 401B, 401C, 402A, 402B
403A, 403B, 404A, 404B, 404C

LOAD = 85.4 KIPS/89' = .96 K/'

3RD FL (MEMBERS IN 300 SERIES)

LOAD = 69.84 KIPS/89' = .785 K/'

2ND FL (MEMBERS IN 200 SERIES)

LOAD = 40.74 KIPS/89' = .458 K/'

IN 145' DIMENSION;

ROOF MEMBERS 405, 414, 422, 429, 434
439, 444, 449, 454, 459, 464

LOAD = 85.4 K/145' = .589 K/'

3RD FL (MEMBERS IN 300 SERIES)

LOAD = 69.84 K/145' = .482 K/'

2ND FL (MEMBERS IN 200 SERIES)

LOAD = 40.74 KIPS/145' = .281 K/'

TOPS 35500

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ANALYSIS OF SMALL SEGMENT OF HORIZONTAL DIAPHRAGM – PHASE 1 AND PHASE 2

PHASE 1 AREA = 9'-6" x 12'-0"

PHASE 2 AREA = 20'-0" x 20'-0"

CASE STUDY WILL OCCUR @ 3RD FLOOR

FROM LATERAL CALCULATION PG 4;

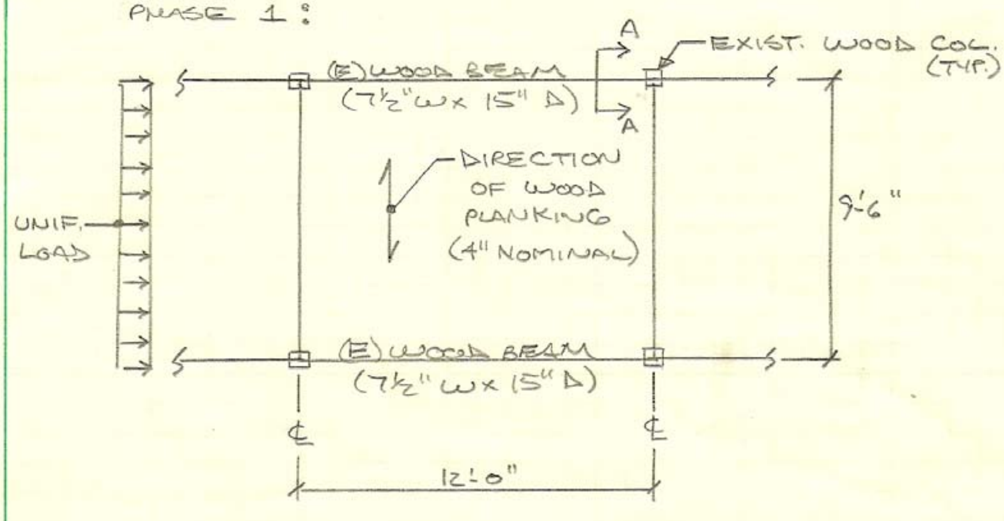
$W_{3RD\ FLOOR} = 751.3$ KIPS

AREA OF DIAPHRAGM = 8584 S.F.

LOAD/S.F. = 751.3 KIPS / 8584 S.F.

= 87.5 #/S.F.

PHASE 1:



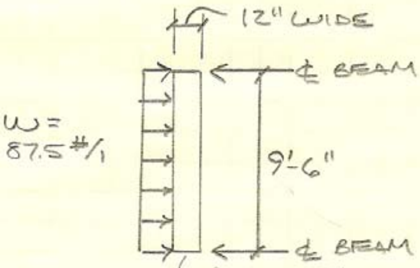
PARTIAL 3RD FL. FRAMING PLAN

89 SHREWSBURY ST.	LPA	2/21/08

PHASE 1 CONT. :

ASSUME PLANKS - 4"x12" (NOMINAL)
- (USE 3 3/4 x 12" ACTUAL)

ANALYZE PLANK SPANNING HORIZ.
W/ 9'-6" SPAN

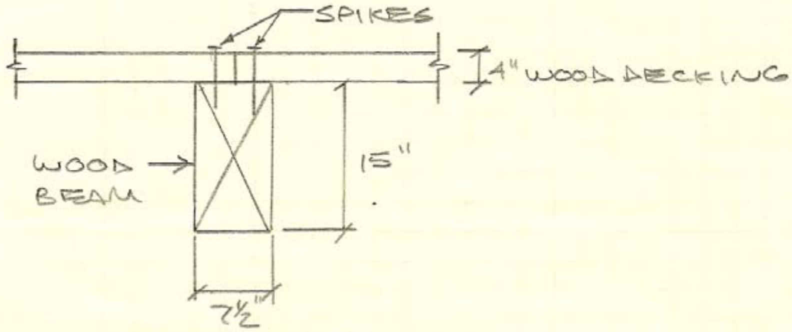


PLAN VIEW OF PLANK

VERIFY PLANK:
$A = 3.75" \times 12" = 45 \text{ in}^2$
$S_x = 3.75 \times 12^2/6 = 90 \text{ in}^3$
$I_x = 3.75 \times 12^3/12 = 540 \text{ in}^4$

BEAM 900 PROGRAM:
R = 415.62
M = 987.10
A = 6.92
S = 8.16
I = 24.11

FROM RESULTS OF PROGRAM, PLANKS CAN SPAN BETWEEN WOOD BEAMS TO DISTRIBUTE HORIZ. LOAD.



SECTION A-A

IN ORDER FOR NEW BRACING TO ABSORB LATERAL LOADS, HORIZONTAL DIAPHRAGM MUST TRANSFER THE LOADS. FIND MAX. SPAN OF PLANKING.

$S_{x\,req} = \frac{M \times 12}{F_b}$ $M = w \cdot l^2/8$

$90 = \frac{M \times 12}{1450}$ $M = 10875 \text{ FT.LBS} = 87.5 \times (L)^2/8$

$L_{MAX} = 31.5'$

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VERIFY SHEAR & DEFLECTION W/BEAM 900:

$w = 87.5^{\#}/'$ $L = 31.5'$ $(26.5')$

$R = 1378$ (1159)
$M = 10852$ (7680)
$A = 22.96$ (19.32)
$S = 89.81$ (63.56)
$I = 879.07$ (523)

DEFLECTION TOO HIGH

$\boxed{\text{MAX PLANK SPAN} = 26.5' \text{ (3 BAYS +/-)}}$

CHECK CAPACITY OF EXISTING SPIKES:

$LOAD = 87.5^{\#}/' \times 9.5' = 831.5^{\#}$ @ BEAM

ASSUME 4 NAILS @ JOINT

$LOAD/NAIL = 831.5^{\#}/4 = 208^{\#}/NAIL.$

IF PLANKS SPAN 3 BAYS.

$LOAD = 87.5^{\#}/' \times (9.5 \times 3) = 2494^{\#}$

$LOAD/NAIL = 2494^{\#}/4 = 623^{\#}/NAIL$

FROM NDS:

TOPS 35500

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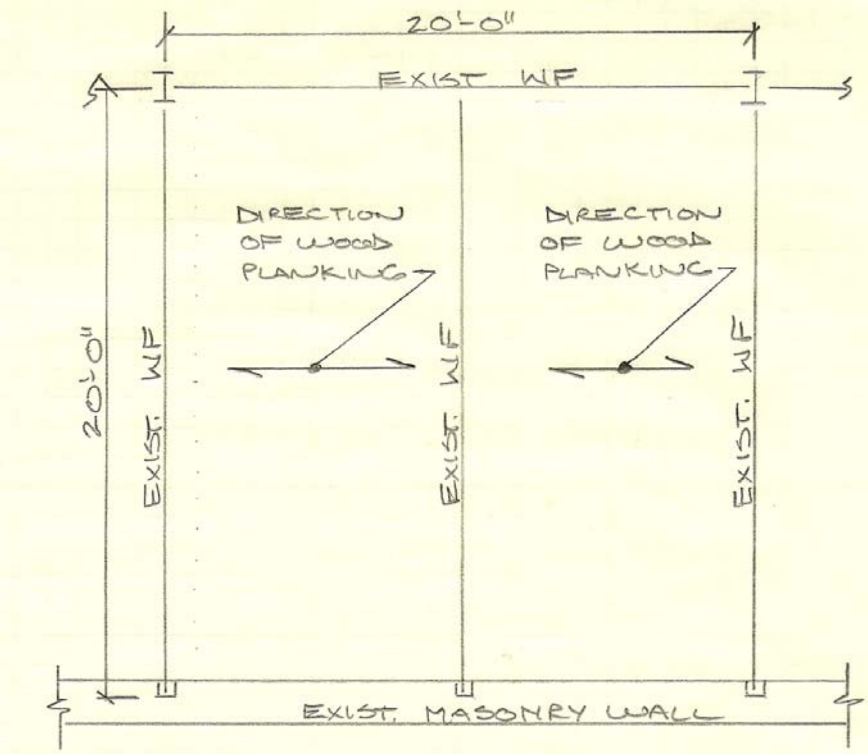
PHASE 2:



CHECK CAPACITY OF EXISTING SPIKES:

LOAD = 87.5#/' x 10'-0" = 875 # @ BEAM

ASSUME 4 NAILS @ JOINT

LOAD/NAIL = 875#/4 = 219#/NAIL

IF PLANKS SPAN 3 BAYS

LOAD = 87.5#/' x (10' x 3) = 2625#

LOAD/NAIL = 2625#/4 = 656#/NAIL

FROM NDS;

TOPS 35500

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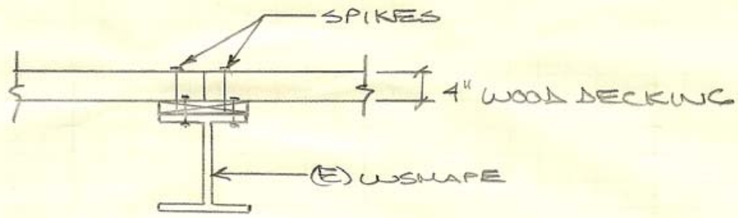

SECTION B-B

APPENDIX G

Member Section Forces (By Combination)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
2446	1	MB1	1	-27.844	0	0	0	0	0
2447			2	-27.844	0	0	0	0	0
2448			3	-27.844	0	0	0	0	0
2449			4	-27.844	0	0	0	0	0
2450			5	-27.844	0	0	0	0	0
2451	1	MB2	1	28.022	0	0	0	0	0
2452			2	28.022	0	0	0	0	0
2453			3	28.022	0	0	0	0	0
2454			4	28.022	0	0	0	0	0
2455			5	28.022	0	0	0	0	0
2456	1	MB201	1	-21.305	0	0	-.002	0	0
2457			2	-21.305	0	0	-.002	0	0
2458			3	-21.305	0	0	-.002	0	0
2459			4	-21.305	0	0	-.002	0	0
2460			5	-21.305	0	0	-.002	0	0
2461	1	MB202	1	21.996	0	0	0	0	0
2462			2	21.996	0	0	0	0	0
2463			3	21.996	0	0	0	0	0
2464			4	21.996	0	0	0	0	0
2465			5	21.996	0	0	0	0	0
2466	1	MB301	1	-11.855	0	0	0	0	0
2467			2	-11.855	0	0	0	0	0
2468			3	-11.855	0	0	0	0	0
2469			4	-11.855	0	0	0	0	0
2470			5	-11.855	0	0	0	0	0
2471	1	MB302	1	13.293	0	0	-.003	0	0
2472			2	13.293	0	0	-.003	0	0
2473			3	13.293	0	0	-.003	0	0
2474			4	13.293	0	0	-.003	0	0
2475			5	13.293	0	0	-.003	0	0
2476	1	MB3	1	-28.46	0	0	0	0	0
2477			2	-28.46	0	0	0	0	0
2478			3	-28.46	0	0	0	0	0
2479			4	-28.46	0	0	0	0	0
2480			5	-28.46	0	0	0	0	0
2481	1	MB4	1	28.781	0	0	0	0	0
2482			2	28.781	0	0	0	0	0
2483			3	28.781	0	0	0	0	0
2484			4	28.781	0	0	0	0	0
2485			5	28.781	0	0	0	0	0
2486	1	MB203	1	-20.549	0	0	.002	0	0
2487			2	-20.549	0	0	.002	0	0
2488			3	-20.549	0	0	.002	0	0
2489			4	-20.549	0	0	.002	0	0
2490			5	-20.549	0	0	.002	0	0
2491	1	MB204	1	21.339	0	0	0	0	0
2492			2	21.339	0	0	0	0	0
2493			3	21.339	0	0	0	0	0
2494			4	21.339	0	0	0	0	0
2495			5	21.339	0	0	0	0	0
2496	1	MB303	1	-11.606	0	0	0	0	0
2497			2	-11.606	0	0	0	0	0
2498			3	-11.606	0	0	0	0	0
2499			4	-11.606	0	0	0	0	0
2500			5	-11.606	0	0	0	0	0
2501	1	MB304	1	13.133	0	0	.003	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
2502			2	13.133	0	0	.003	0	0
2503			3	13.133	0	0	.003	0	0
2504			4	13.133	0	0	.003	0	0
2505			5	13.133	0	0	.003	0	0
2506	1	MB5	1	-23.343	0	0	0	0	0
2507			2	-23.343	0	0	0	0	0
2508			3	-23.343	0	0	0	0	0
2509			4	-23.343	0	0	0	0	0
2510			5	-23.343	0	0	0	0	0
2511	1	MB6	1	15.988	0	0	0	0	0
2512			2	15.988	0	0	0	0	0
2513			3	15.988	0	0	0	0	0
2514			4	15.988	0	0	0	0	0
2515			5	15.988	0	0	0	0	0
2516	1	MB205	1	-15.171	0	0	0	0	0
2517			2	-15.171	0	0	0	0	0
2518			3	-15.171	0	0	0	0	0
2519			4	-15.171	0	0	0	0	0
2520			5	-15.171	0	0	0	0	0
2521	1	MB206	1	14.314	0	0	0	0	0
2522			2	14.314	0	0	0	0	0
2523			3	14.314	0	0	0	0	0
2524			4	14.314	0	0	0	0	0
2525			5	14.314	0	0	0	0	0
2526	1	MB305	1	-6.64	0	0	0	0	0
2527			2	-6.64	0	0	0	0	0
2528			3	-6.64	0	0	0	0	0
2529			4	-6.64	0	0	0	0	0
2530			5	-6.64	0	0	0	0	0
2531	1	MB306	1	8.616	0	0	0	0	0
2532			2	8.616	0	0	0	0	0
2533			3	8.616	0	0	0	0	0
2534			4	8.616	0	0	0	0	0
2535			5	8.616	0	0	0	0	0
2536	1	MB7	1	-20.87	0	0	0	0	0
2537			2	-20.87	0	0	0	0	0
2538			3	-20.87	0	0	0	0	0
2539			4	-20.87	0	0	0	0	0
2540			5	-20.87	0	0	0	0	0
2541	1	MB8	1	14.378	0	0	0	0	0
2542			2	14.378	0	0	0	0	0
2543			3	14.378	0	0	0	0	0
2544			4	14.378	0	0	0	0	0
2545			5	14.378	0	0	0	0	0
2546	1	MB207	1	-16.733	0	0	0	0	0
2547			2	-16.733	0	0	0	0	0
2548			3	-16.733	0	0	0	0	0
2549			4	-16.733	0	0	0	0	0
2550			5	-16.733	0	0	0	0	0
2551	1	MB208	1	16.053	0	0	0	0	0
2552			2	16.053	0	0	0	0	0
2553			3	16.053	0	0	0	0	0
2554			4	16.053	0	0	0	0	0
2555			5	16.053	0	0	0	0	0
2556	1	MB307	1	-7.195	0	0	0	0	0
2557			2	-7.195	0	0	0	0	0
2558			3	-7.195	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-.
2559			4	-7.195	0	0	0	0	0
2560			5	-7.195	0	0	0	0	0
2561	1	MB308	1	9.769	0	0	0	0	0
2562			2	9.769	0	0	0	0	0
2563			3	9.769	0	0	0	0	0
2564			4	9.769	0	0	0	0	0
2565			5	9.769	0	0	0	0	0
2566	1	MB9	1	-1.691	0	0	0	0	0
2567			2	-1.691	0	0	0	0	0
2568			3	-1.691	0	0	0	0	0
2569			4	-1.691	0	0	0	0	0
2570			5	-1.691	0	0	0	0	0
2571	1	MB10	1	-2.096	0	0	0	0	0
2572			2	-2.096	0	0	0	0	0
2573			3	-2.096	0	0	0	0	0
2574			4	-2.096	0	0	0	0	0
2575			5	-2.096	0	0	0	0	0
2576	1	MB209	1	.383	0	0	.003	0	0
2577			2	.383	0	0	.003	0	0
2578			3	.383	0	0	.003	0	0
2579			4	.383	0	0	.003	0	0
2580			5	.383	0	0	.003	0	0
2581	1	MB210	1	-2.38	0	0	.006	0	0
2582			2	-2.38	0	0	.006	0	0
2583			3	-2.38	0	0	.006	0	0
2584			4	-2.38	0	0	.006	0	0
2585			5	-2.38	0	0	.006	0	0
2586	1	MB309	1	1.543	0	0	.006	0	0
2587			2	1.543	0	0	.006	0	0
2588			3	1.543	0	0	.006	0	0
2589			4	1.543	0	0	.006	0	0
2590			5	1.543	0	0	.006	0	0
2591	1	MB310	1	-2.5	0	0	.003	0	0
2592			2	-2.5	0	0	.003	0	0
2593			3	-2.5	0	0	.003	0	0
2594			4	-2.5	0	0	.003	0	0
2595			5	-2.5	0	0	.003	0	0
2596	1	MB11	1	-3.188	0	0	0	0	0
2597			2	-3.188	0	0	0	0	0
2598			3	-3.188	0	0	0	0	0
2599			4	-3.188	0	0	0	0	0
2600			5	-3.188	0	0	0	0	0
2601	1	MB12	1	.107	0	0	0	0	0
2602			2	.107	0	0	0	0	0
2603			3	.107	0	0	0	0	0
2604			4	.107	0	0	0	0	0
2605			5	.107	0	0	0	0	0
2606	1	MB211	1	-3.654	0	0	-.007	0	0
2607			2	-3.654	0	0	-.007	0	0
2608			3	-3.654	0	0	-.007	0	0
2609			4	-3.654	0	0	-.007	0	0
2610			5	-3.654	0	0	-.007	0	0
2611	1	MB212	1	1.968	0	0	-.004	0	0
2612			2	1.968	0	0	-.004	0	0
2613			3	1.968	0	0	-.004	0	0
2614			4	1.968	0	0	-.004	0	0
2615			5	1.968	0	0	-.004	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
2616	1	MB311	1	-3.322	0	0	-.003	0	0
2617			2	-3.322	0	0	-.003	0	0
2618			3	-3.322	0	0	-.003	0	0
2619			4	-3.322	0	0	-.003	0	0
2620			5	-3.322	0	0	-.003	0	0
2621	1	MB312	1	2.397	0	0	-.006	0	0
2622			2	2.397	0	0	-.006	0	0
2623			3	2.397	0	0	-.006	0	0
2624			4	2.397	0	0	-.006	0	0
2625			5	2.397	0	0	-.006	0	0

Member Section Forces (By Combination)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
2806	1	MB17	1	-14.316	0	0	0	0	0
2807			2	-14.316	0	0	0	0	0
2808			3	-14.316	0	0	0	0	0
2809			4	-14.316	0	0	0	0	0
2810			5	-14.316	0	0	0	0	0
2811	1	MB18	1	23.643	0	0	0	0	0
2812			2	23.643	0	0	0	0	0
2813			3	23.643	0	0	0	0	0
2814			4	23.643	0	0	0	0	0
2815			5	23.643	0	0	0	0	0
2816	1	MB217	1	-12.229	0	0	0	0	0
2817			2	-12.229	0	0	0	0	0
2818			3	-12.229	0	0	0	0	0
2819			4	-12.229	0	0	0	0	0
2820			5	-12.229	0	0	0	0	0
2821	1	MB218	1	16.541	0	0	0	0	0
2822			2	16.541	0	0	0	0	0
2823			3	16.541	0	0	0	0	0
2824			4	16.541	0	0	0	0	0
2825			5	16.541	0	0	0	0	0
2826	1	MB317	1	-6.328	0	0	0	0	0
2827			2	-6.328	0	0	0	0	0
2828			3	-6.328	0	0	0	0	0
2829			4	-6.328	0	0	0	0	0
2830			5	-6.328	0	0	0	0	0
2831	1	MB318	1	8.416	0	0	0	0	0
2832			2	8.416	0	0	0	0	0
2833			3	8.416	0	0	0	0	0
2834			4	8.416	0	0	0	0	0
2835			5	8.416	0	0	0	0	0
2836	1	MB19	1	-12.539	0	0	0	0	0
2837			2	-12.539	0	0	0	0	0
2838			3	-12.539	0	0	0	0	0
2839			4	-12.539	0	0	0	0	0
2840			5	-12.539	0	0	0	0	0
2841	1	MB20	1	22.009	0	0	0	0	0
2842			2	22.009	0	0	0	0	0
2843			3	22.009	0	0	0	0	0
2844			4	22.009	0	0	0	0	0
2845			5	22.009	0	0	0	0	0
2846	1	MB219	1	-12.663	0	0	0	0	0
2847			2	-12.663	0	0	0	0	0
2848			3	-12.663	0	0	0	0	0
2849			4	-12.663	0	0	0	0	0
2850			5	-12.663	0	0	0	0	0
2851	1	MB220	1	17.225	0	0	0	0	0
2852			2	17.225	0	0	0	0	0
2853			3	17.225	0	0	0	0	0
2854			4	17.225	0	0	0	0	0
2855			5	17.225	0	0	0	0	0
2856	1	MB319	1	-6.494	0	0	0	0	0
2857			2	-6.494	0	0	0	0	0
2858			3	-6.494	0	0	0	0	0
2859			4	-6.494	0	0	0	0	0
2860			5	-6.494	0	0	0	0	0
2861	1	MB320	1	8.712	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
2862			2	8.712	0	0	0	0	0
2863			3	8.712	0	0	0	0	0
2864			4	8.712	0	0	0	0	0
2865			5	8.712	0	0	0	0	0
2866	1	MB21	1	-.831	0	0	0	0	0
2867			2	-.831	0	0	0	0	0
2868			3	-.831	0	0	0	0	0
2869			4	-.831	0	0	0	0	0
2870			5	-.831	0	0	0	0	0
2871	1	MB22	1	.949	0	0	0	0	0
2872			2	.949	0	0	0	0	0
2873			3	.949	0	0	0	0	0
2874			4	.949	0	0	0	0	0
2875			5	.949	0	0	0	0	0
2876	1	MB221	1	.415	0	0	.009	0	0
2877			2	.415	0	0	.009	0	0
2878			3	.415	0	0	.009	0	0
2879			4	.415	0	0	.009	0	0
2880			5	.415	0	0	.009	0	0
2881	1	MB222	1	1.376	0	0	0	0	0
2882			2	1.376	0	0	0	0	0
2883			3	1.376	0	0	0	0	0
2884			4	1.376	0	0	0	0	0
2885			5	1.376	0	0	0	0	0
2886	1	MB321	1	1.464	0	0	.011	0	0
2887			2	1.464	0	0	.011	0	0
2888			3	1.464	0	0	.011	0	0
2889			4	1.464	0	0	.011	0	0
2890			5	1.464	0	0	.011	0	0
2891	1	MB322	1	2.551	0	0	-.006	0	0
2892			2	2.551	0	0	-.006	0	0
2893			3	2.551	0	0	-.006	0	0
2894			4	2.551	0	0	-.006	0	0
2895			5	2.551	0	0	-.006	0	0
2896	1	MB23	1	-.262	0	0	0	0	0
2897			2	-.262	0	0	0	0	0
2898			3	-.262	0	0	0	0	0
2899			4	-.262	0	0	0	0	0
2900			5	-.262	0	0	0	0	0
2901	1	MB24	1	.792	0	0	0	0	0
2902			2	.792	0	0	0	0	0
2903			3	.792	0	0	0	0	0
2904			4	.792	0	0	0	0	0
2905			5	.792	0	0	0	0	0
2906	1	MB223	1	-.48	0	0	.002	0	0
2907			2	-.48	0	0	.002	0	0
2908			3	-.48	0	0	.002	0	0
2909			4	-.48	0	0	.002	0	0
2910			5	-.48	0	0	.002	0	0
2911	1	MB224	1	.579	0	0	0	0	0
2912			2	.579	0	0	0	0	0
2913			3	.579	0	0	0	0	0
2914			4	.579	0	0	0	0	0
2915			5	.579	0	0	0	0	0
2916	1	MB323	1	-1.516	0	0	0	0	0
2917			2	-1.516	0	0	0	0	0
2918			3	-1.516	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
2919			4	-1.516	0	0	0	0	0
2920			5	-1.516	0	0	0	0	0
2921	1	MB324	1	-.055	0	0	.002	0	0
2922			2	-.055	0	0	.002	0	0
2923			3	-.055	0	0	.002	0	0
2924			4	-.055	0	0	.002	0	0
2925			5	-.055	0	0	.002	0	0
2926	1	MB25	1	-6.304	0	0	0	0	0
2927			2	-6.304	0	0	0	0	0
2928			3	-6.304	0	0	0	0	0
2929			4	-6.304	0	0	0	0	0
2930			5	-6.304	0	0	0	0	0
2931	1	MB26	1	-5.425	0	0	0	0	0
2932			2	-5.425	0	0	0	0	0
2933			3	-5.425	0	0	0	0	0
2934			4	-5.425	0	0	0	0	0
2935			5	-5.425	0	0	0	0	0
2936	1	MB225	1	-1.94	0	0	.002	0	0
2937			2	-1.94	0	0	.002	0	0
2938			3	-1.94	0	0	.002	0	0
2939			4	-1.94	0	0	.002	0	0
2940			5	-1.94	0	0	.002	0	0
2941	1	MB226	1	-1.777	0	0	-.004	0	0
2942			2	-1.777	0	0	-.004	0	0
2943			3	-1.777	0	0	-.004	0	0
2944			4	-1.777	0	0	-.004	0	0
2945			5	-1.777	0	0	-.004	0	0
2946	1	MB325	1	-1.414	0	0	-.002	0	0
2947			2	-1.414	0	0	-.002	0	0
2948			3	-1.414	0	0	-.002	0	0
2949			4	-1.414	0	0	-.002	0	0
2950			5	-1.414	0	0	-.002	0	0
2951	1	MB326	1	-.588	0	0	0	0	0
2952			2	-.588	0	0	0	0	0
2953			3	-.588	0	0	0	0	0
2954			4	-.588	0	0	0	0	0
2955			5	-.588	0	0	0	0	0
2956	1	MB27	1	-.489	0	0	0	0	0
2957			2	-.489	0	0	0	0	0
2958			3	-.489	0	0	0	0	0
2959			4	-.489	0	0	0	0	0
2960			5	-.489	0	0	0	0	0
2961	1	MB28	1	-3.626	0	0	0	0	0
2962			2	-3.626	0	0	0	0	0
2963			3	-3.626	0	0	0	0	0
2964			4	-3.626	0	0	0	0	0
2965			5	-3.626	0	0	0	0	0
2966	1	MB227	1	1.139	0	0	-.006	0	0
2967			2	1.139	0	0	-.006	0	0
2968			3	1.139	0	0	-.006	0	0
2969			4	1.139	0	0	-.006	0	0
2970			5	1.139	0	0	-.006	0	0
2971	1	MB228	1	-3.217	0	0	-.002	0	0
2972			2	-3.217	0	0	-.002	0	0
2973			3	-3.217	0	0	-.002	0	0
2974			4	-3.217	0	0	-.002	0	0
2975			5	-3.217	0	0	-.002	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
2976	1	MB327	1	2.564	0	0	-.002	0	0
2977			2	2.564	0	0	-.002	0	0
2978			3	2.564	0	0	-.002	0	0
2979			4	2.564	0	0	-.002	0	0
2980			5	2.564	0	0	-.002	0	0
2981	1	MB328	1	-2.729	0	0	-.006	0	0
2982			2	-2.729	0	0	-.006	0	0
2983			3	-2.729	0	0	-.006	0	0
2984			4	-2.729	0	0	-.006	0	0
2985			5	-2.729	0	0	-.006	0	0
2986	1	MB29	1	-.614	0	0	0	0	0
2987			2	-.614	0	0	0	0	0
2988			3	-.614	0	0	0	0	0
2989			4	-.614	0	0	0	0	0
2990			5	-.614	0	0	0	0	0
2991	1	MB30	1	.287	0	0	0	0	0
2992			2	.287	0	0	0	0	0
2993			3	.287	0	0	0	0	0
2994			4	.287	0	0	0	0	0
2995			5	.287	0	0	0	0	0
2996	1	MB229	1	-.817	0	0	.003	0	0
2997			2	-.817	0	0	.003	0	0
2998			3	-.817	0	0	.003	0	0
2999			4	-.817	0	0	.003	0	0
3000			5	-.817	0	0	.003	0	0
3001	1	MB230	1	.648	0	0	-.002	0	0
3002			2	.648	0	0	-.002	0	0
3003			3	.648	0	0	-.002	0	0
3004			4	.648	0	0	-.002	0	0
3005			5	.648	0	0	-.002	0	0
3006	1	MB329	1	-.597	0	0	.002	0	0
3007			2	-.597	0	0	.002	0	0
3008			3	-.597	0	0	.002	0	0
3009			4	-.597	0	0	.002	0	0
3010			5	-.597	0	0	.002	0	0
3011	1	MB330	1	.647	0	0	-.001	0	0
3012			2	.647	0	0	-.001	0	0
3013			3	.647	0	0	-.001	0	0
3014			4	.647	0	0	-.001	0	0
3015			5	.647	0	0	-.001	0	0
3016	1	MB31	1	-.146	0	0	0	0	0
3017			2	-.146	0	0	0	0	0
3018			3	-.146	0	0	0	0	0
3019			4	-.146	0	0	0	0	0
3020			5	-.146	0	0	0	0	0
3021	1	MB32	1	.575	0	0	0	0	0
3022			2	.575	0	0	0	0	0
3023			3	.575	0	0	0	0	0
3024			4	.575	0	0	0	0	0
3025			5	.575	0	0	0	0	0
3026	1	MB231	1	-.676	0	0	-.012	0	0
3027			2	-.676	0	0	-.012	0	0
3028			3	-.676	0	0	-.012	0	0
3029			4	-.676	0	0	-.012	0	0
3030			5	-.676	0	0	-.012	0	0
3031	1	MB232	1	.93	0	0	-.002	0	0
3032			2	.93	0	0	-.002	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
3033			3	.93	0	0	-.002	0	0
3034			4	.93	0	0	-.002	0	0
3035			5	.93	0	0	-.002	0	0
3036	1	MB331	1	-.928	0	0	-.001	0	0
3037			2	-.928	0	0	-.001	0	0
3038			3	-.928	0	0	-.001	0	0
3039			4	-.928	0	0	-.001	0	0
3040			5	-.928	0	0	-.001	0	0
3041	1	MB332	1	.544	0	0	-.011	0	0
3042			2	.544	0	0	-.011	0	0
3043			3	.544	0	0	-.011	0	0
3044			4	.544	0	0	-.011	0	0
3045			5	.544	0	0	-.011	0	0

Member Section Forces (By Combination)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
5491	2	MB1	1	-.95	0	0	0	0	0
5492			2	-.95	0	0	0	0	0
5493			3	-.95	0	0	0	0	0
5494			4	-.95	0	0	0	0	0
5495			5	-.95	0	0	0	0	0
5496	2	MB2	1	-.419	0	0	0	0	0
5497			2	-.419	0	0	0	0	0
5498			3	-.419	0	0	0	0	0
5499			4	-.419	0	0	0	0	0
5500			5	-.419	0	0	0	0	0
5501	2	MB201	1	-.208	0	0	0	0	0
5502			2	-.208	0	0	0	0	0
5503			3	-.208	0	0	0	0	0
5504			4	-.208	0	0	0	0	0
5505			5	-.208	0	0	0	0	0
5506	2	MB202	1	-.487	0	0	0	0	0
5507			2	-.487	0	0	0	0	0
5508			3	-.487	0	0	0	0	0
5509			4	-.487	0	0	0	0	0
5510			5	-.487	0	0	0	0	0
5511	2	MB301	1	-.051	0	0	0	0	0
5512			2	-.051	0	0	0	0	0
5513			3	-.051	0	0	0	0	0
5514			4	-.051	0	0	0	0	0
5515			5	-.051	0	0	0	0	0
5516	2	MB302	1	-.436	0	0	0	0	0
5517			2	-.436	0	0	0	0	0
5518			3	-.436	0	0	0	0	0
5519			4	-.436	0	0	0	0	0
5520			5	-.436	0	0	0	0	0
5521	2	MB3	1	.63	0	0	0	0	0
5522			2	.63	0	0	0	0	0
5523			3	.63	0	0	0	0	0
5524			4	.63	0	0	0	0	0
5525			5	.63	0	0	0	0	0
5526	2	MB4	1	.63	0	0	0	0	0
5527			2	.63	0	0	0	0	0
5528			3	.63	0	0	0	0	0
5529			4	.63	0	0	0	0	0
5530			5	.63	0	0	0	0	0
5531	2	MB203	1	-.397	0	0	0	0	0
5532			2	-.397	0	0	0	0	0
5533			3	-.397	0	0	0	0	0
5534			4	-.397	0	0	0	0	0
5535			5	-.397	0	0	0	0	0
5536	2	MB204	1	1.015	0	0	0	0	0
5537			2	1.015	0	0	0	0	0
5538			3	1.015	0	0	0	0	0
5539			4	1.015	0	0	0	0	0
5540			5	1.015	0	0	0	0	0
5541	2	MB303	1	-.594	0	0	0	0	0
5542			2	-.594	0	0	0	0	0
5543			3	-.594	0	0	0	0	0
5544			4	-.594	0	0	0	0	0
5545			5	-.594	0	0	0	0	0
5546	2	MB304	1	.949	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
5547			2	.949	0	0	0	0	0
5548			3	.949	0	0	0	0	0
5549			4	.949	0	0	0	0	0
5550			5	.949	0	0	0	0	0
5551	2	MB5	1	-1.958	0	0	0	0	0
5552			2	-1.958	0	0	0	0	0
5553			3	-1.958	0	0	0	0	0
5554			4	-1.958	0	0	0	0	0
5555			5	-1.958	0	0	0	0	0
5556	2	MB6	1	-4.339	0	0	0	0	0
5557			2	-4.339	0	0	0	0	0
5558			3	-4.339	0	0	0	0	0
5559			4	-4.339	0	0	0	0	0
5560			5	-4.339	0	0	0	0	0
5561	2	MB205	1	.673	0	0	0	0	0
5562			2	.673	0	0	0	0	0
5563			3	.673	0	0	0	0	0
5564			4	.673	0	0	0	0	0
5565			5	.673	0	0	0	0	0
5566	2	MB206	1	-3.447	0	0	0	0	0
5567			2	-3.447	0	0	0	0	0
5568			3	-3.447	0	0	0	0	0
5569			4	-3.447	0	0	0	0	0
5570			5	-3.447	0	0	0	0	0
5571	2	MB305	1	1.242	0	0	0	0	0
5572			2	1.242	0	0	0	0	0
5573			3	1.242	0	0	0	0	0
5574			4	1.242	0	0	0	0	0
5575			5	1.242	0	0	0	0	0
5576	2	MB306	1	-2.44	0	0	0	0	0
5577			2	-2.44	0	0	0	0	0
5578			3	-2.44	0	0	0	0	0
5579			4	-2.44	0	0	0	0	0
5580			5	-2.44	0	0	0	0	0
5581	2	MB7	1	-.055	0	0	0	0	0
5582			2	-.055	0	0	0	0	0
5583			3	-.055	0	0	0	0	0
5584			4	-.055	0	0	0	0	0
5585			5	-.055	0	0	0	0	0
5586	2	MB8	1	1.164	0	0	0	0	0
5587			2	1.164	0	0	0	0	0
5588			3	1.164	0	0	0	0	0
5589			4	1.164	0	0	0	0	0
5590			5	1.164	0	0	0	0	0
5591	2	MB207	1	-.402	0	0	0	0	0
5592			2	-.402	0	0	0	0	0
5593			3	-.402	0	0	0	0	0
5594			4	-.402	0	0	0	0	0
5595			5	-.402	0	0	0	0	0
5596	2	MB208	1	.783	0	0	0	0	0
5597			2	.783	0	0	0	0	0
5598			3	.783	0	0	0	0	0
5599			4	.783	0	0	0	0	0
5600			5	.783	0	0	0	0	0
5601	2	MB307	1	-.258	0	0	0	0	0
5602			2	-.258	0	0	0	0	0
5603			3	-.258	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
5604			4	-.258	0	0	0	0	0
5605			5	-.258	0	0	0	0	0
5606	2	MB308	1	.44	0	0	0	0	0
5607			2	.44	0	0	0	0	0
5608			3	.44	0	0	0	0	0
5609			4	.44	0	0	0	0	0
5610			5	.44	0	0	0	0	0
5611	2	MB9	1	-16.521	0	0	0	0	0
5612			2	-16.521	0	0	0	0	0
5613			3	-16.521	0	0	0	0	0
5614			4	-16.521	0	0	0	0	0
5615			5	-16.521	0	0	0	0	0
5616	2	MB10	1	14.775	0	0	0	0	0
5617			2	14.775	0	0	0	0	0
5618			3	14.775	0	0	0	0	0
5619			4	14.775	0	0	0	0	0
5620			5	14.775	0	0	0	0	0
5621	2	MB209	1	-12.007	0	0	0	0	0
5622			2	-12.007	0	0	0	0	0
5623			3	-12.007	0	0	0	0	0
5624			4	-12.007	0	0	0	0	0
5625			5	-12.007	0	0	0	0	0
5626	2	MB210	1	11.824	0	0	.001	0	0
5627			2	11.824	0	0	.001	0	0
5628			3	11.824	0	0	.001	0	0
5629			4	11.824	0	0	.001	0	0
5630			5	11.824	0	0	.001	0	0
5631	2	MB309	1	-5.944	0	0	.001	0	0
5632			2	-5.944	0	0	.001	0	0
5633			3	-5.944	0	0	.001	0	0
5634			4	-5.944	0	0	.001	0	0
5635			5	-5.944	0	0	.001	0	0
5636	2	MB310	1	6.696	0	0	0	0	0
5637			2	6.696	0	0	0	0	0
5638			3	6.696	0	0	0	0	0
5639			4	6.696	0	0	0	0	0
5640			5	6.696	0	0	0	0	0
5641	2	MB11	1	-14.172	0	0	0	0	0
5642			2	-14.172	0	0	0	0	0
5643			3	-14.172	0	0	0	0	0
5644			4	-14.172	0	0	0	0	0
5645			5	-14.172	0	0	0	0	0
5646	2	MB12	1	15.97	0	0	0	0	0
5647			2	15.97	0	0	0	0	0
5648			3	15.97	0	0	0	0	0
5649			4	15.97	0	0	0	0	0
5650			5	15.97	0	0	0	0	0
5651	2	MB211	1	-11.538	0	0	-.001	0	0
5652			2	-11.538	0	0	-.001	0	0
5653			3	-11.538	0	0	-.001	0	0
5654			4	-11.538	0	0	-.001	0	0
5655			5	-11.538	0	0	-.001	0	0
5656	2	MB212	1	12.321	0	0	0	0	0
5657			2	12.321	0	0	0	0	0
5658			3	12.321	0	0	0	0	0
5659			4	12.321	0	0	0	0	0
5660			5	12.321	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
5661	2	MB311	1	-6.503	0	0	0	0	0
5662			2	-6.503	0	0	0	0	0
5663			3	-6.503	0	0	0	0	0
5664			4	-6.503	0	0	0	0	0
5665			5	-6.503	0	0	0	0	0
5666	2	MB312	1	6.883	0	0	-.001	0	0
5667			2	6.883	0	0	-.001	0	0
5668			3	6.883	0	0	-.001	0	0
5669			4	6.883	0	0	-.001	0	0
5670			5	6.883	0	0	-.001	0	0

Member Section Forces (By Combination)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
5851	2	MB17	1	-.251	0	0	0	0	0
5852			2	-.251	0	0	0	0	0
5853			3	-.251	0	0	0	0	0
5854			4	-.251	0	0	0	0	0
5855			5	-.251	0	0	0	0	0
5856	2	MB18	1	-.567	0	0	0	0	0
5857			2	-.567	0	0	0	0	0
5858			3	-.567	0	0	0	0	0
5859			4	-.567	0	0	0	0	0
5860			5	-.567	0	0	0	0	0
5861	2	MB217	1	-.852	0	0	0	0	0
5862			2	-.852	0	0	0	0	0
5863			3	-.852	0	0	0	0	0
5864			4	-.852	0	0	0	0	0
5865			5	-.852	0	0	0	0	0
5866	2	MB218	1	.269	0	0	0	0	0
5867			2	.269	0	0	0	0	0
5868			3	.269	0	0	0	0	0
5869			4	.269	0	0	0	0	0
5870			5	.269	0	0	0	0	0
5871	2	MB317	1	-.773	0	0	0	0	0
5872			2	-.773	0	0	0	0	0
5873			3	-.773	0	0	0	0	0
5874			4	-.773	0	0	0	0	0
5875			5	-.773	0	0	0	0	0
5876	2	MB318	1	.62	0	0	0	0	0
5877			2	.62	0	0	0	0	0
5878			3	.62	0	0	0	0	0
5879			4	.62	0	0	0	0	0
5880			5	.62	0	0	0	0	0
5881	2	MB19	1	-.214	0	0	0	0	0
5882			2	-.214	0	0	0	0	0
5883			3	-.214	0	0	0	0	0
5884			4	-.214	0	0	0	0	0
5885			5	-.214	0	0	0	0	0
5886	2	MB20	1	.49	0	0	0	0	0
5887			2	.49	0	0	0	0	0
5888			3	.49	0	0	0	0	0
5889			4	.49	0	0	0	0	0
5890			5	.49	0	0	0	0	0
5891	2	MB219	1	-.012	0	0	0	0	0
5892			2	-.012	0	0	0	0	0
5893			3	-.012	0	0	0	0	0
5894			4	-.012	0	0	0	0	0
5895			5	-.012	0	0	0	0	0
5896	2	MB220	1	.171	0	0	0	0	0
5897			2	.171	0	0	0	0	0
5898			3	.171	0	0	0	0	0
5899			4	.171	0	0	0	0	0
5900			5	.171	0	0	0	0	0
5901	2	MB319	1	.061	0	0	0	0	0
5902			2	.061	0	0	0	0	0
5903			3	.061	0	0	0	0	0
5904			4	.061	0	0	0	0	0
5905			5	.061	0	0	0	0	0
5906	2	MB320	1	-.014	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
5907			2	-.014	0	0	0	0	0
5908			3	-.014	0	0	0	0	0
5909			4	-.014	0	0	0	0	0
5910			5	-.014	0	0	0	0	0
5911	2	MB21	1	-17.435	0	0	0	0	0
5912			2	-17.435	0	0	0	0	0
5913			3	-17.435	0	0	0	0	0
5914			4	-17.435	0	0	0	0	0
5915			5	-17.435	0	0	0	0	0
5916	2	MB22	1	14.345	0	0	0	0	0
5917			2	14.345	0	0	0	0	0
5918			3	14.345	0	0	0	0	0
5919			4	14.345	0	0	0	0	0
5920			5	14.345	0	0	0	0	0
5921	2	MB221	1	-10.761	0	0	0	0	0
5922			2	-10.761	0	0	0	0	0
5923			3	-10.761	0	0	0	0	0
5924			4	-10.761	0	0	0	0	0
5925			5	-10.761	0	0	0	0	0
5926	2	MB222	1	11.048	0	0	0	0	0
5927			2	11.048	0	0	0	0	0
5928			3	11.048	0	0	0	0	0
5929			4	11.048	0	0	0	0	0
5930			5	11.048	0	0	0	0	0
5931	2	MB321	1	-2.954	0	0	0	0	0
5932			2	-2.954	0	0	0	0	0
5933			3	-2.954	0	0	0	0	0
5934			4	-2.954	0	0	0	0	0
5935			5	-2.954	0	0	0	0	0
5936	2	MB322	1	5.094	0	0	-.004	0	0
5937			2	5.094	0	0	-.004	0	0
5938			3	5.094	0	0	-.004	0	0
5939			4	5.094	0	0	-.004	0	0
5940			5	5.094	0	0	-.004	0	0
5941	2	MB23	1	-14.097	0	0	0	0	0
5942			2	-14.097	0	0	0	0	0
5943			3	-14.097	0	0	0	0	0
5944			4	-14.097	0	0	0	0	0
5945			5	-14.097	0	0	0	0	0
5946	2	MB24	1	15.911	0	0	0	0	0
5947			2	15.911	0	0	0	0	0
5948			3	15.911	0	0	0	0	0
5949			4	15.911	0	0	0	0	0
5950			5	15.911	0	0	0	0	0
5951	2	MB223	1	-13.112	0	0	0	0	0
5952			2	-13.112	0	0	0	0	0
5953			3	-13.112	0	0	0	0	0
5954			4	-13.112	0	0	0	0	0
5955			5	-13.112	0	0	0	0	0
5956	2	MB224	1	12.608	0	0	0	0	0
5957			2	12.608	0	0	0	0	0
5958			3	12.608	0	0	0	0	0
5959			4	12.608	0	0	0	0	0
5960			5	12.608	0	0	0	0	0
5961	2	MB323	1	-9.431	0	0	0	0	0
5962			2	-9.431	0	0	0	0	0
5963			3	-9.431	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
5964			4	-9.431	0	0	0	0	0
5965			5	-9.431	0	0	0	0	0
5966	2	MB324	1	6.617	0	0	0	0	0
5967			2	6.617	0	0	0	0	0
5968			3	6.617	0	0	0	0	0
5969			4	6.617	0	0	0	0	0
5970			5	6.617	0	0	0	0	0
5971	2	MB25	1	-16.863	0	0	0	0	0
5972			2	-16.863	0	0	0	0	0
5973			3	-16.863	0	0	0	0	0
5974			4	-16.863	0	0	0	0	0
5975			5	-16.863	0	0	0	0	0
5976	2	MB26	1	14.288	0	0	0	0	0
5977			2	14.288	0	0	0	0	0
5978			3	14.288	0	0	0	0	0
5979			4	14.288	0	0	0	0	0
5980			5	14.288	0	0	0	0	0
5981	2	MB225	1	-11.884	0	0	0	0	0
5982			2	-11.884	0	0	0	0	0
5983			3	-11.884	0	0	0	0	0
5984			4	-11.884	0	0	0	0	0
5985			5	-11.884	0	0	0	0	0
5986	2	MB226	1	11.937	0	0	.001	0	0
5987			2	11.937	0	0	.001	0	0
5988			3	11.937	0	0	.001	0	0
5989			4	11.937	0	0	.001	0	0
5990			5	11.937	0	0	.001	0	0
5991	2	MB325	1	-5.316	0	0	0	0	0
5992			2	-5.316	0	0	0	0	0
5993			3	-5.316	0	0	0	0	0
5994			4	-5.316	0	0	0	0	0
5995			5	-5.316	0	0	0	0	0
5996	2	MB326	1	7.193	0	0	0	0	0
5997			2	7.193	0	0	0	0	0
5998			3	7.193	0	0	0	0	0
5999			4	7.193	0	0	0	0	0
6000			5	7.193	0	0	0	0	0
6001	2	MB27	1	-14.604	0	0	0	0	0
6002			2	-14.604	0	0	0	0	0
6003			3	-14.604	0	0	0	0	0
6004			4	-14.604	0	0	0	0	0
6005			5	-14.604	0	0	0	0	0
6006	2	MB28	1	16.017	0	0	0	0	0
6007			2	16.017	0	0	0	0	0
6008			3	16.017	0	0	0	0	0
6009			4	16.017	0	0	0	0	0
6010			5	16.017	0	0	0	0	0
6011	2	MB227	1	-14.238	0	0	-.002	0	0
6012			2	-14.238	0	0	-.002	0	0
6013			3	-14.238	0	0	-.002	0	0
6014			4	-14.238	0	0	-.002	0	0
6015			5	-14.238	0	0	-.002	0	0
6016	2	MB228	1	12.031	0	0	0	0	0
6017			2	12.031	0	0	0	0	0
6018			3	12.031	0	0	0	0	0
6019			4	12.031	0	0	0	0	0
6020			5	12.031	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
6021	2	MB327	1	-10.682	0	0	0	0	0
6022			2	-10.682	0	0	0	0	0
6023			3	-10.682	0	0	0	0	0
6024			4	-10.682	0	0	0	0	0
6025			5	-10.682	0	0	0	0	0
6026	2	MB328	1	6.046	0	0	-.002	0	0
6027			2	6.046	0	0	-.002	0	0
6028			3	6.046	0	0	-.002	0	0
6029			4	6.046	0	0	-.002	0	0
6030			5	6.046	0	0	-.002	0	0
6031	2	MB29	1	-19.092	0	0	0	0	0
6032			2	-19.092	0	0	0	0	0
6033			3	-19.092	0	0	0	0	0
6034			4	-19.092	0	0	0	0	0
6035			5	-19.092	0	0	0	0	0
6036	2	MB30	1	14.414	0	0	0	0	0
6037			2	14.414	0	0	0	0	0
6038			3	14.414	0	0	0	0	0
6039			4	14.414	0	0	0	0	0
6040			5	14.414	0	0	0	0	0
6041	2	MB229	1	-13.201	0	0	-.005	0	0
6042			2	-13.201	0	0	-.005	0	0
6043			3	-13.201	0	0	-.005	0	0
6044			4	-13.201	0	0	-.005	0	0
6045			5	-13.201	0	0	-.005	0	0
6046	2	MB230	1	12.775	0	0	-.009	0	0
6047			2	12.775	0	0	-.009	0	0
6048			3	12.775	0	0	-.009	0	0
6049			4	12.775	0	0	-.009	0	0
6050			5	12.775	0	0	-.009	0	0
6051	2	MB329	1	-6.287	0	0	-.008	0	0
6052			2	-6.287	0	0	-.008	0	0
6053			3	-6.287	0	0	-.008	0	0
6054			4	-6.287	0	0	-.008	0	0
6055			5	-6.287	0	0	-.008	0	0
6056	2	MB330	1	7.437	0	0	-.01	0	0
6057			2	7.437	0	0	-.01	0	0
6058			3	7.437	0	0	-.01	0	0
6059			4	7.437	0	0	-.01	0	0
6060			5	7.437	0	0	-.01	0	0
6061	2	MB31	1	-15.962	0	0	0	0	0
6062			2	-15.962	0	0	0	0	0
6063			3	-15.962	0	0	0	0	0
6064			4	-15.962	0	0	0	0	0
6065			5	-15.962	0	0	0	0	0
6066	2	MB32	1	18.948	0	0	0	0	0
6067			2	18.948	0	0	0	0	0
6068			3	18.948	0	0	0	0	0
6069			4	18.948	0	0	0	0	0
6070			5	18.948	0	0	0	0	0
6071	2	MB231	1	-15.151	0	0	-.002	0	0
6072			2	-15.151	0	0	-.002	0	0
6073			3	-15.151	0	0	-.002	0	0
6074			4	-15.151	0	0	-.002	0	0
6075			5	-15.151	0	0	-.002	0	0
6076	2	MB232	1	12.167	0	0	0	0	0
6077			2	12.167	0	0	0	0	0

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
6078			3	12.167	0	0	0	0	0
6079			4	12.167	0	0	0	0	0
6080			5	12.167	0	0	0	0	0
6081	2	MB331	1	-10.475	0	0	0	0	0
6082			2	-10.475	0	0	0	0	0
6083			3	-10.475	0	0	0	0	0
6084			4	-10.475	0	0	0	0	0
6085			5	-10.475	0	0	0	0	0
6086	2	MB332	1	4.9	0	0	-.002	0	0
6087			2	4.9	0	0	-.002	0	0
6088			3	4.9	0	0	-.002	0	0
6089			4	4.9	0	0	-.002	0	0
6090			5	4.9	0	0	-.002	0	0