RETROFITTING OF R/C STRUCTURES ON GRAVITY COLUMNS USING INVERTED-Y STEEL BRACINGS

By

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Abstract

It is common practice in Puerto Rico to build elevated reinforced concrete frame and shear wall structures in hillsides and hilly terrains by supporting them on gravity columns. Recent studies have indicated the vulnerability of these structures to seismic events, especially when the loading is amplified to account for the site topography. It is an objective of this research to examine the use of inverted-Y steel bracing system as a retrofitting measure. The system performance is evaluated against shear wall retrofits which is the only option currently used in the Island. The building prototypes are retrofitted and numerically analyzed in order to develop design guidelines. The size and placement of retrofits were found to be a function of geometric and topographic properties such as the building footprints, column heights, number of bays, and whether or not the earthquake is amplified. The inverted-Y retrofit is proven to be the more cost effective of the two options and is recommended as such. Details on the acceptable retrofitting configurations are provided as well as the built-up sections that may be used for shear links in an inverted-Y system. In the aftermath of an earthquake, it is only the shear links that will most likely need replacing. Easy reparability is an added cost benefit of these systems over the shear wall retrofits.

Resumen

En Puerto Rico es una práctica común construir estructuras elevadas de hormigón armado y paredes cortantes apoyadas sobre columnas diseñadas para cargas gravitacionales en laderas de montanas o en terreno accidentado. Estudios recientes demuestran la vulnerabilidad de estas estructuras a eventos sísmicos, especialmente cuando la carga es amplificada para tomar en cuenta la topografía del terreno. El objetivo de este estudio es examinar el del sistema de marcos de acero en forma de Y-invertida para reducir daños durante un evento sísmico. El sistema será evaluado contra la única opción utilizada actualmente en la isla que es la añadidura de paredes cortantes. Prototipos de las estructuras son analizados utilizando ambos sistemas para desarrollar unas guías de diseño. Se determino que el tamaño y la colocación de ambos sistemas son función de las propiedades geométricas de la estructura como por ejemplo: la huella de la estructura, altura de las columnas, numero de luces o si el terremoto esta amplificado. El sistema de marcos de acero demostró ser la solución más costo-efectiva y se recomienda la utilización de este. Se proveen los detalles de las configuraciones aceptables para el uso del sistema recomendado, como también de las secciones compuestas que pueden ser utilizadas como el enlace de cortante. Luego de un terremoto el único componente del sistema que probablemente necesite reemplazo es el enlace de cortante. La facilidad de reparación de este sistema presenta un beneficio adicional sobre la utilización de paredes de cortante.

To everyone who believed in me even in times when I doubted myself,

I could not have done it without you.

Acknowledgements

To everyone that contributed in one way or another with me during the research process and during my years in graduate studies: professors, colleagues, classmates, friends and family I would like to express my most sincere thanks.

I would like to specially acknowledge the contribution of my advisor Dr. Ali Saffar and thank him for his support and guidance during the research process. Leading by example he inspired me to work with an elevated level of effort to complete this project. The lessons learned during this process will accompany me during the rest of my career. My sincere gratitude also goes to the members of my graduate committee Dr. Luis Suárez and Dr. Ricardo Ramos for their time and effort during the research process and for the knowledge they bestowed in me as a student.

The moral and intellectual support received from colleagues, classmates, friends and family members was instrumental for me as a graduate student. Thanks to all of them the journey through graduate school was a much more gratifying experience.

Also I would like to show my appreciation to two institutions that assisted me through graduate school: the Louis Stokes Association for Minority Participation and the Mid America Earthquake Center.

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1 Introduction

1.1 Justification

A common mode of construction in Puerto Rico is to build reinforced concrete frame and shear wall structures in hillside and hilly terrain by supporting them on gravity columns (Figure 1.1). Studies on the behavior of seismic waves that arrive at hills or escarpment have shown that the site topography can amplify the ground acceleration by as much as 235 percent (Arroyo, 2001). Field observations in the aftermath of recent earthquakes in Italy, China, and Pakistan have provided clear examples to further such arguments (Sano and Pugliese, 1999). However, at the time of this writing, regulations in the US and Puerto Rico do not include topographic amplification factors for seismic design.

In a recent study by Vázquez (2002), prototypes were selected from a survey of residential Houses on the western side of the Island. After analyzing the structures without applying the amplification factor determined by Arroyo, eleven out of twelve structures selected had shown extensive damage or collapse. It is highly unlikely that any of those structures will survive if a significant amplification factor was included in the analysis.

Figure 1.1 - Examples of concrete frame structures supported on gravity columns in Puerto Rico

The last major earthquake to hit Puerto Rico was the 1918 earthquake in Mayagüez measuring 7.5 on the Richter scale. If an earthquake of that magnitude was to occur again today, most structures on gravity columns will be subject to failure, accounting for considerable loss of life and properties.

1.2 Previous Works

In the thesis "Numerical Study of the Amplification of the Seismic Ground Acceleration Due to Local Topography", Arroyo (2001) studied the effect of hilly or escarped topography on the amplification of seismic waves based on a peak acceleration comparison. She developed a series of equations to relate the amplification factor to the topography as well to the location of the structure along the hill or escarpment. Arroyo concluded from two dimensional nonlinear analyses using the Finite Element Method that the amplification factor varies from a range of 1 to 2.35.

In the dissertation "Seismic Behavior and Retrofitting of Hillside and Hilly Terrain R/C Houses Raised on Gravity Columns", Vázquez (2002) utilized the results found by Arroyo (2001) to analyze their effect on actual residential structures in Puerto Rico. A field survey was conducted in five municipalities and a total of 24 residences were evaluated and measured. The parameters considered were the height of the columns, the cross-sectional properties, the bay length of beams, the steel reinforcement and number of stories. Vázquez analyzed two dimensional frames from the surveyed homes. In his analysis he utilized an earthquake created for the Mayagüez zone, this earthquake record was amplified by a factor of two considering the amplification factor developed by Arroyo. All residences studied collapsed when subjected to earthquake records with and without the topographic amplification factor. This concludes that there is a need for a retrofitting system for existing structures.

The proposed retrofitting scheme is the inverted-Y steel bracing systems. With this system a steel frame is placed inside of the existing reinforced concrete frame and they are connected utilizing stud bolts and mortar in a manner similar to composite steel and concrete beams. The inverted Y consists of two diagonal bracing members and a shear link. When retrofit is needed in existing R/C frame structures, this system possesses many advantages. High strength and stiffness is provided, openings can be made without losing seismic capacity, and the increase of mass associated with the retrofitting is comparatively small and therefore retrofitting of the foundation can be minimized. Most steel frame retrofitting only increase the stiffness of the structure, but in the Y-shaped brace a shear plate is utilized to dissipate energy due to inelastic deformation, thus increasing the ductility of the structure. In an experimental study conducted by Okada et al. (1995) the retrofitted specimens exhibited improved response in terms of strength, stiffness, and energy dissipation. The ultimate capacity of the braced frame was approximately five times that of the bare frame. Energy dissipation was eight times as much in the braced frame than in the without bracing and the contribution of the shear panel to the total energy dissipation was approximately 75% of the total energy dissipation.

1.3 Objective

The objective of this study is to determine whether the inverted Y-shape bracing system is a viable form of retrofitting for the residential structures commonly found on hillsides or hilly terrain in Puerto Rico. Computer structural analysis will be conducted on the structures previously studied by Vázquez, using the retrofitting system tested by Okada et al. (1995) to determine the levels of improvements in strength, stiffness and energy dissipation. The structural performance of the bracing system will be compared to that of shear walls which is a more traditional way of improving seismic response of R/C frame structures. A construction cost estimate of both the bracing system and structural walls will be conducted in order to conduct a cost-benefit analysis and determine which retrofitting system is better suited for the structures and conditions commonly found in Puerto Rico.

1.4 Methodology

To determine the effectiveness of the inverted Y-shape retrofitting system a computer Model of a two dimensional frame will be developed. The bare frame will be tested with a sinusoidal lateral load and the displacement will be recorded. Next the bracing system will be installed into the reinforced concrete frame and the results will be compared. For the design of the shear panel, an integral part of the system, it is preferable to have a program that can incorporate a shear link element in design, which is an element that can yield in shear while remaining elastic in bending. The chosen program, SAP2000, met the requirements needed for this analysis.

The structures analyzed by Vázquez (2002) will be reanalyzed utilizing the program SAP 2000 and the results will be compared. Then the R/C frames will be retrofitted with the inverted Y-shape steel frame and reanalyzed. Based on the results from the study by Okada et al., (1995), it is expected that the use of bracing as a retrofitting measure will present the most improvement in seismic performance. However, the improvements will also be measured in terms of some of the parameters identified by Vázquez, such as column length, number of stories and cross section. This will help to identify scenarios under which this retrofitting scheme is less successful and perhaps a different alternative should be taken. In this study, the alternative to the steel bracing system is the use of concrete shear walls. The analyses will be performed with shear walls in place of the steel bracing and the results will be compared.

Once the structural analyses are completed, a cost estimate study between the two systems (steel bracing and structural walls) will be conducted. An advantage of the inverted Y-shape steel bracing is that it can be constructed offsite and installed in a relatively short amount of time in the residential structure. Another characteristic that can lower cost is the light weight of the system when compared to shear walls. Because of the size of the concrete shear wall compared to the steel frame, it is highly likely that the foundation must be significantly altered to support the new wall. Shear walls also require that a temporary wooden form be constructed onsite, which may be difficult to install in a hillside situation. All of these factors must be taken into account when determining the cost of a shear wall.

2 Inverted-Y Braced Steel System

2.1 Introduction

In this study, the inverted-Y steel bracing system is utilized to retrofit concrete structures that were not originally designed for seismic loading. Figure 2.1 shows an example of such a system. The main component in an inverted-Y bracing is the shear link at the top which dissipates seismic energy by undergoing inelastic deformation. This will allow the deformations in the connecting braces to stay in the elastic range.

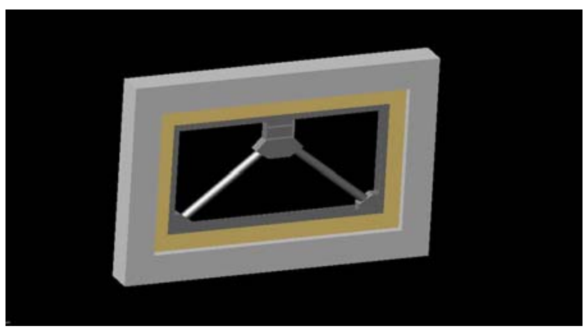


Figure 2.1 - Concrete Frame with Inverted-Y Bracing Steel System

In "Experimental Study of 1/10 Scaled R/C Frames Retrofitted with Steel Framed Y-Shaped Bracing System" (Okada et. al., 1995), one tenth scaled versions of reinforced concrete frames were retrofitted with the inverted-Y steel braced systems and tested for lateral seismic loads. In our current study a computer Model of a full size reinforced concrete frame utilizing this system will be created, utilizing the SAP2000 program with the analytical shear link Model. The goal is to have a computer Model that will simulate the behavior and duplicate the results of the experimental tests. Once the system is developed for the two dimensional frame, it will be utilized to retrofit a series of thee-dimensional reinforced concrete frame prototypes created with the information from the survey conducted by Vázquez(2001) and also in Models of the actual surveyed homes.

2.2 Shear Link Performance

A shear link is a structural element that is expected to yield in shear while remaining elastic in bending. They are commonly employed in eccentrically braced frames. A shear link could have any cross-section but it is usually I-shaped. The web of the I-shape section resists most of the shear forces while the flanges resist the bending moments.

In early designs, shear links were placed at the floor level. Figure 2.2 shows examples of this. Horizontal shear links depicted in white are part of the floor beams, placed either in the middle or at the end. In the case of a strong earthquake, the shear link is expected to absorb the energy through shear deformations. While this is beneficial for the overall structural integrity and the safety of the occupants, the floor system may be severely damaged. Figure 2.3 shows a schematic drawing of the structure deformed due to lateral load. Notice that most of the deformation is located in the shear link at floor level. In such cases, repairing the damage to the floors could be very expensive.

The vertical shear link in an inverted-Y braced system provides all the advantages of a horizontal link without the extensive damage to the floor systems. They are also easily and inexpensively replaced. Figure 2.4 shows an example of the system with the corresponding deformation shape. Again the shear links experience drastic amount of deformations. However, because of their positions the damage does not extend to the beams or columns. The damage to this structure could be repaired much faster and at a lower cost than the structure with horizontal shear links.

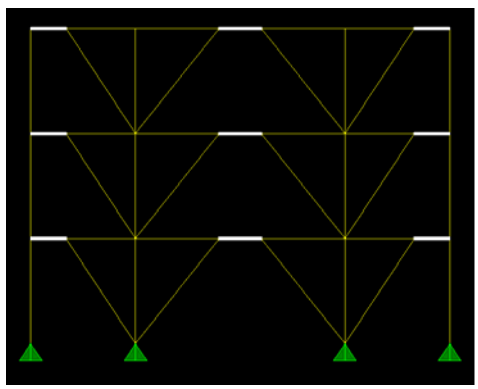


Figure 2.2 - Two dimensional frame with horizontal shear links

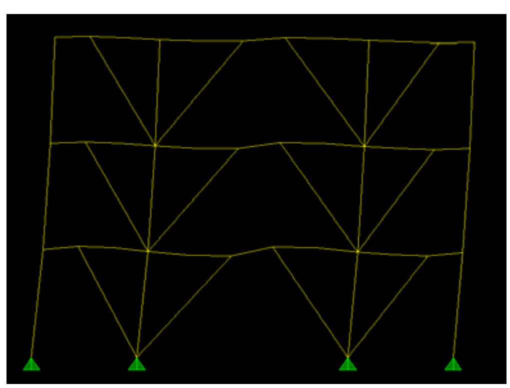


Figure 2.3 - Deformation Diagram. 2D Frame with horizontal shear links

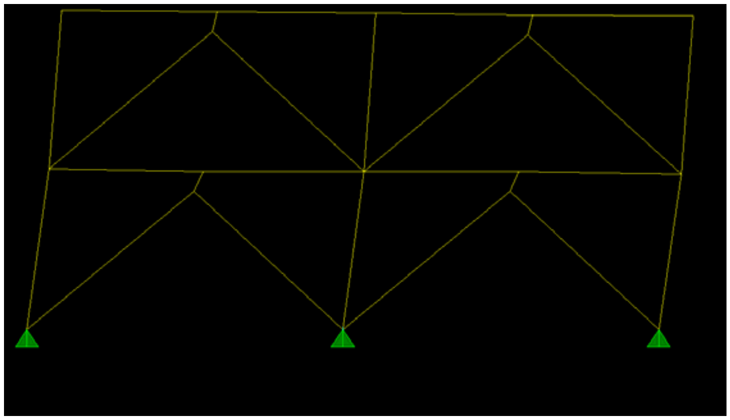


Figure 2.4 - Deformation diagram. 2D frame with vertical shear links

2.3 Model Calibration

To Model the shear link, the force-deformation relationships must be formulated. Figure 2.5 presents multi-linear behavioral patterns investigated by Perera et al. (2004). As shown, there are two types of behavioral equations utilized for shear links, one for shear and the other for flexural behavior. The control parameters in these graphs are calculated in accordance with the formulas listed in Table 2.1.

Figure 2.6 shows the dimensions for one of the shear links utilized during the course of this study. This particular shear link is made using A992 W8x10 steel section. Using the material properties given and cross-section geometry:

$$V_y = 38.7 \text{ kip}$$

$$M_y = 443.5 \text{ kip-in}$$

$$K_{V1} = 1245 \text{ kip/in}$$

$$K_{M1} = 446600 \text{ kip-in}$$

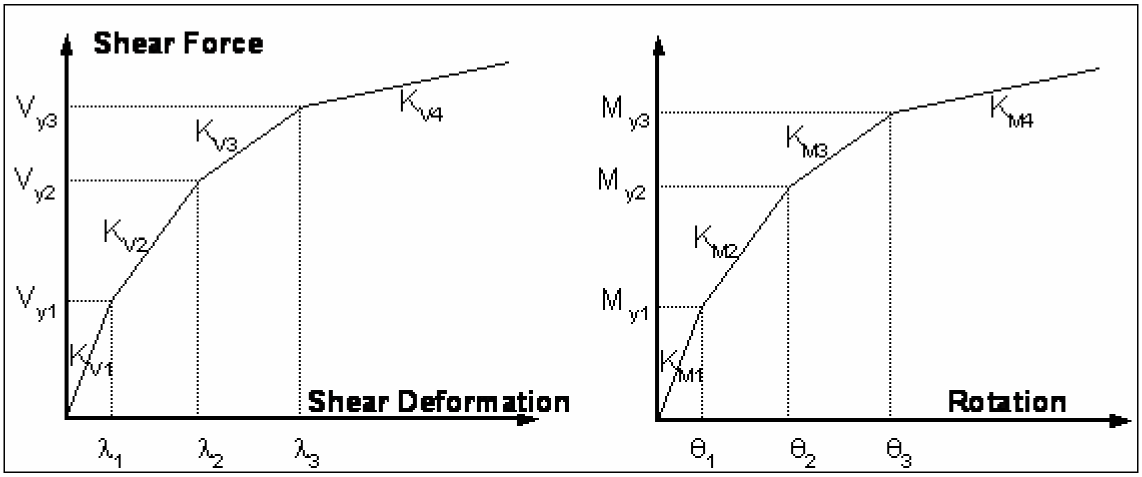


Figure 2.5 - Shear link multi-linear behavior

Table 2.1 - Shear and Moment Equations (Perera et al, 2004)

Shear Forces	**Moments**	**Shear Constants**	**Moment Constants**
$V_y = \tau_y (d - t_f) t_w$	$M_y = \sigma_y S$	$K_{V1} = \dfrac{GA_{web}}{e}$	$K_{M1} = \dfrac{6EI}{e}$
$V_{y1} = 1.0V_y$	$M_{y1} = 1.00M_y$	$K_{V2} = 0.100K_{V1}$	$K_{M2} = 0.030K_{M1}$
$V_{y2} = 1.5V_y$	$M_{y2} = 1.03M_y$	$K_{V3} = 0.030K_{V1}$	$K_{M3} = 0.015K_{M1}$
$V_{y3} = 2.0V_y$	$M_{y3} = 1.06M_y$	$K_{V4} = 0.007K_{V1}$	$K_{M4} = 0.002K_{M1}$
Comment: e = shear link lenght			

Table 2.2 lists all the control points for the shear specimen of Figure 2.5. Using theses values as basic input, a Model of the shear link was created and tested in SAP2000. The hysteretic diagram shown in Figure 2.7 was obtained by applying a sinusoidal load to the link Model.

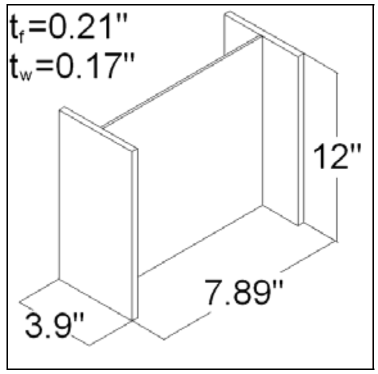


Figure 2.6 - W8x10 steel section utilized as a shear link

Table 2.2 - Shear and Moment Values

Shear Forces (kip)	Moments (kip-in)	Shear Constants (kip/in)	Shear Deformation (in)	Moment Constants (kip-in)	Rotation (radians)
V_{y1} = 38.7	M_{y1} = 444	K_{V2} = 125	γ_1 = 0.031	K_{M2} = 13398	$\theta_1 = 9.93\text{x}10^{-4}$
V_{y2} = 58.0	M_{y2} = 457	K_{V3} = 37.7	γ_2 = 0.186	K_{M3} = 6699	$\theta_2 = 1.99\text{x}10^{-3}$
V_{y3} = 77.4	M_{y3} = 470	K_{V4} = 8.72	γ_3 = 0.704	K_{M4} = 893.2	$\theta_3 = 1.89\text{x}10^{-2}$

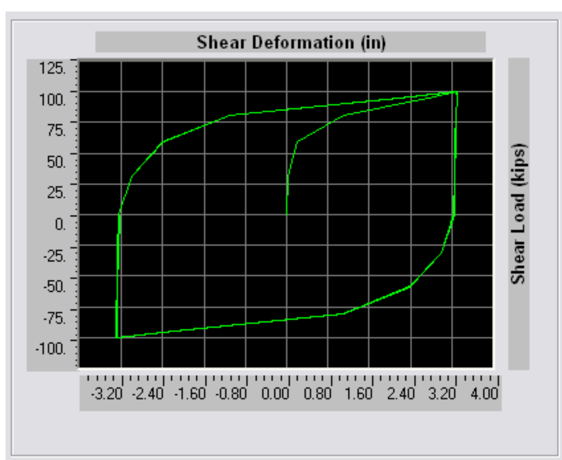


Figure 2.7 - Shear link hysteretic diagram

2.4 Reinforced Concrete Frame Modeling

A reinforced concrete frame will be analyzed as a control Model in order to determine the improvement that the bracing system provides. This concrete frame consists of a pair of twelve feet tall columns that have a square cross-section of ten inches on each side and are reinforced using four #5 steel bars. Nonlinear links were placed at the ends of the columns to account for the additional deformations that will occur if the columns reach their ultimate moment capacities. These links were programmed utilizing a moment-curvature curve developed for this reinforced concrete cross-section.

To take into account the concrete degradations when subjected to many cycles of load reversals, the nonlinear links are made capable of representing the degradation Model developed by Takeda (Takeda, 1970). According to this Model, the stiffness of the element is reduced every time the element experiences a load reversal. Figure 2.8 presents the moment-curvature relationship for a column cross section. Figure 2.9 shows how this is modeled in the SAP2000 computer program without degradation. Figure 2.10 presents the element under identical load but in this case the nonlinear links are capable of degradation, and the reduction in stiffness can be seen in the diagram every time there is a load reversal.

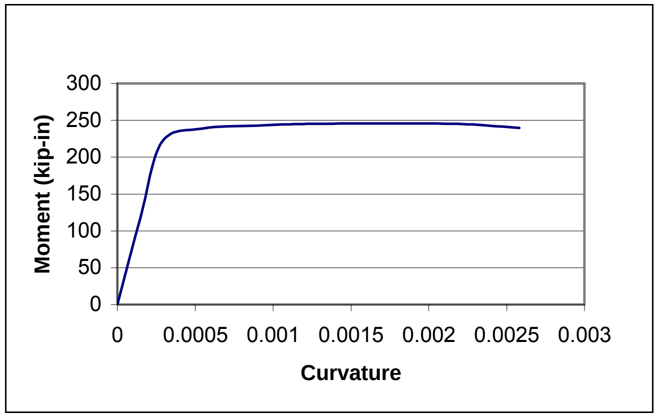


Figure 2.8 - Moment-curvature relationship for columns in Model frame

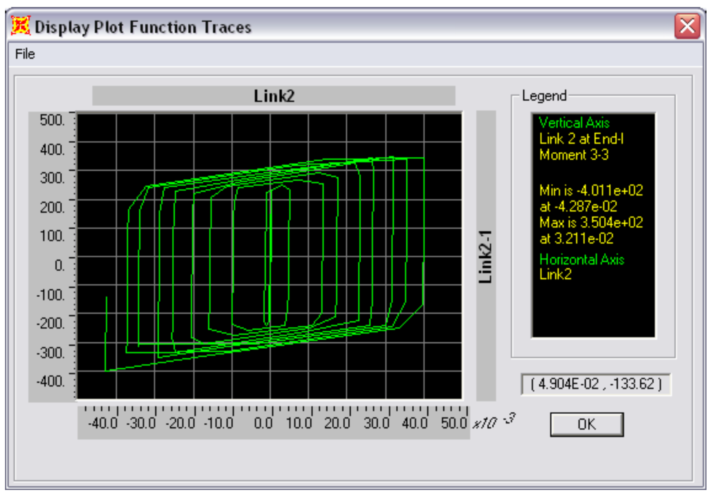


Figure 2.9 - Moment-curvature link Model without degradation

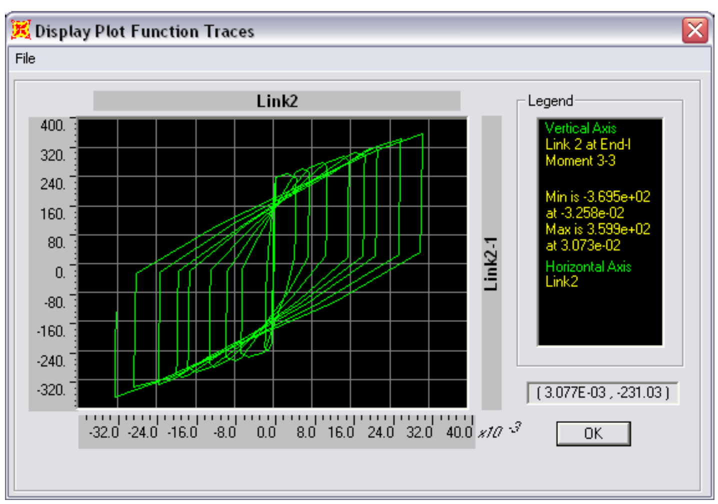


Figure 2.10 - Moment-curvature link Model with degradation

The frame was tested with an incrementing lateral sinusoidal load until failure is reached when the nonlinear links can no longer resist the load applied and the lateral deflection increases considerably. Figure 2.11 presents the lateral deflection experienced by the frame due to the applied load versus the shear force at the base. From the diagram, we can see that when the load reaches approximately ten kips the deflection starts to increase dramatically, meaning that the columns have failed. The maximum displacement reached is approximately 3.5 inches when the load reaches its maximum value of 15 kip.

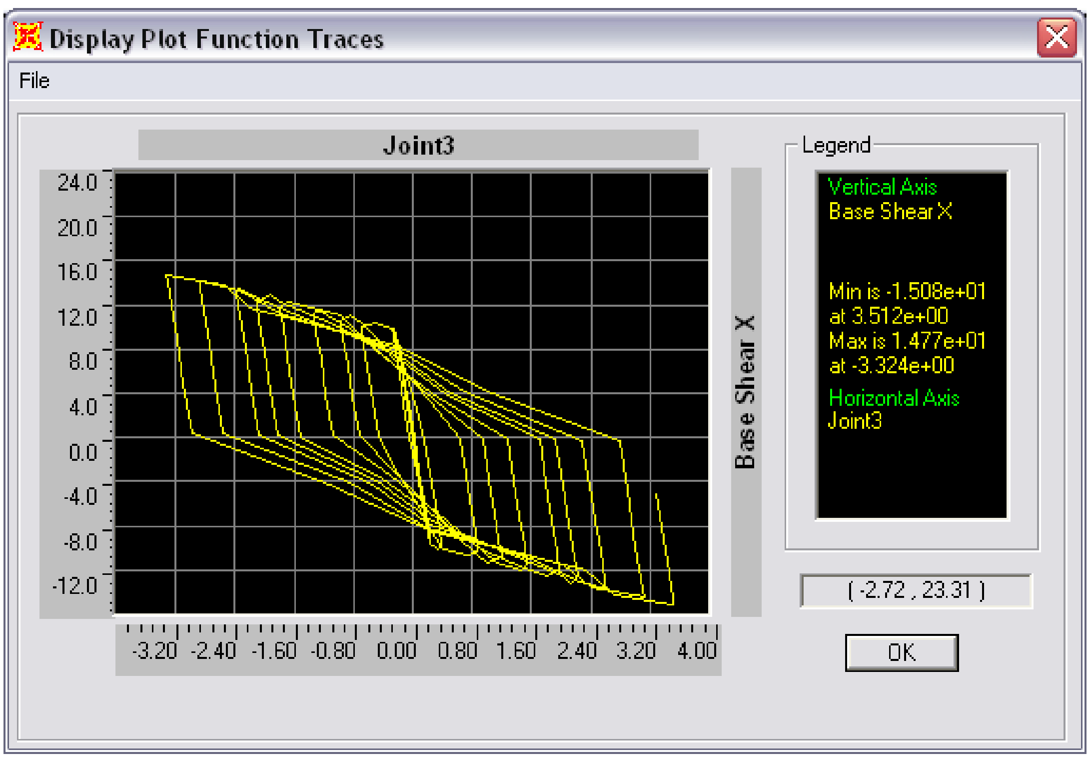


Figure 2.11 - Base shear force vs. lateral displacements – R/C Frame

2.5 Inverted-Y Steel Bracing System Modeling

The reinforced concrete frame Model in Section 2.4 will now be retrofitted with the Inverted-Y Steel Bracing System. The SAP2000 model is discussed in Appendix A and shown in Figure 2.12. The shear link is a 12-in long A992 W8x10 steel section. The diagonal bracing members are A501 HSS6.625 round steel tubes. The retrofitted system is tested in a similar manner to the original system, with incremental loading until the failure is achieved. It is important to note that under the load causing the maximum displacement of 0.018 inch in the original structure, the retrofitted system was still undergoing elastic deformations.

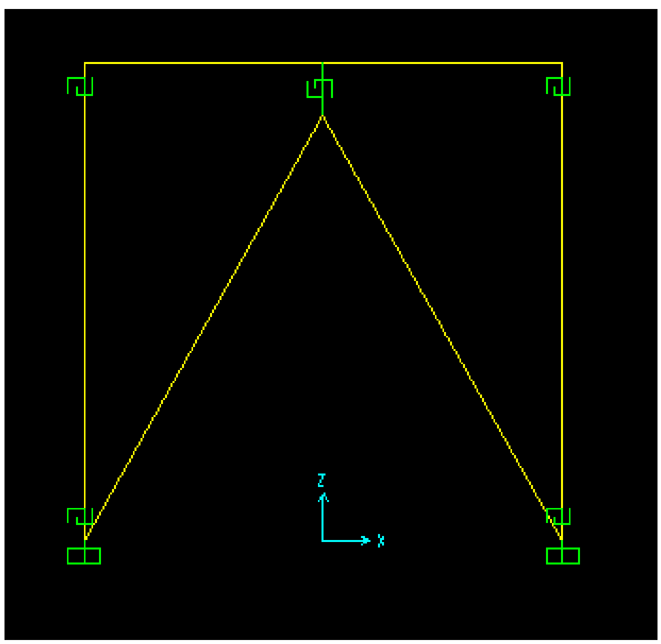


Figure 2.12 – SAP2000 Model for inverted-Y bracing system

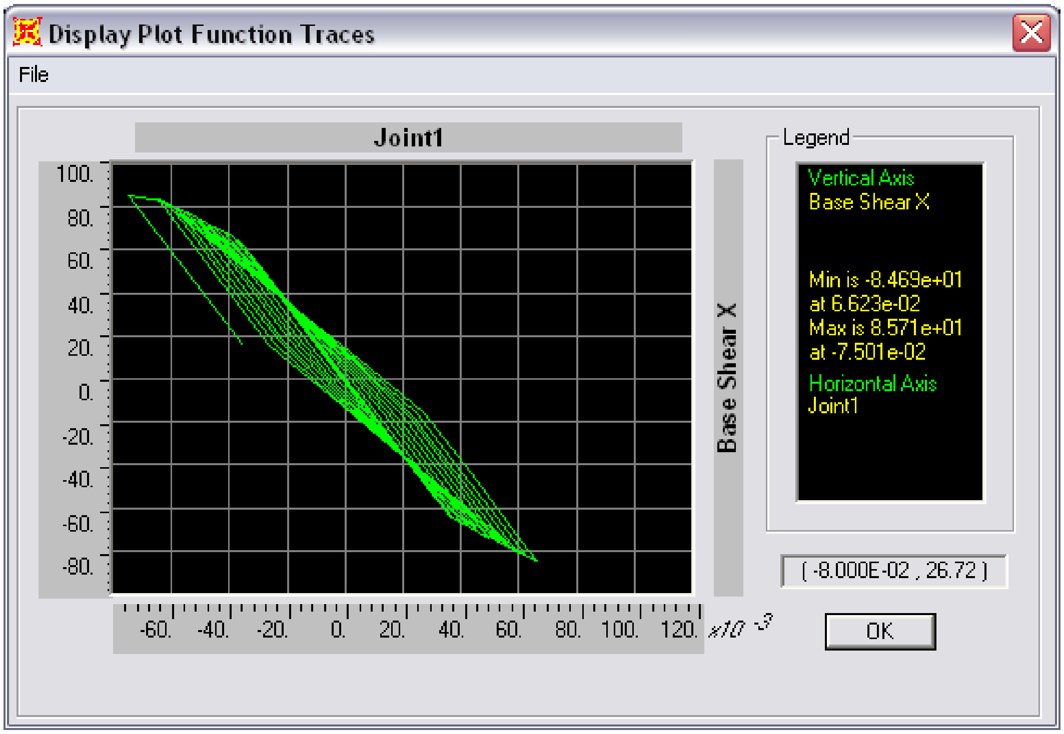


Figure 2.13 - Displacement versus Shear Load for the Inverted-Y Steel Bracing System Loaded to 85 kips

From the shear load versus displacement plot in Figure 2.13, one can see that the frame has displaced 0.075 inch under a lateral load of 85 kips. Even with this small lateral displacement, the system enters into a nonlinear pattern because the shear link quickly deforms in a plastic manner and absorbs energy from the lateral motion. Other components remain in the elastic range. Figure 2.14 shows the same plot but with the system loaded to 145 kips. In this case the maximum lateral displacement value is 1.33 inches.

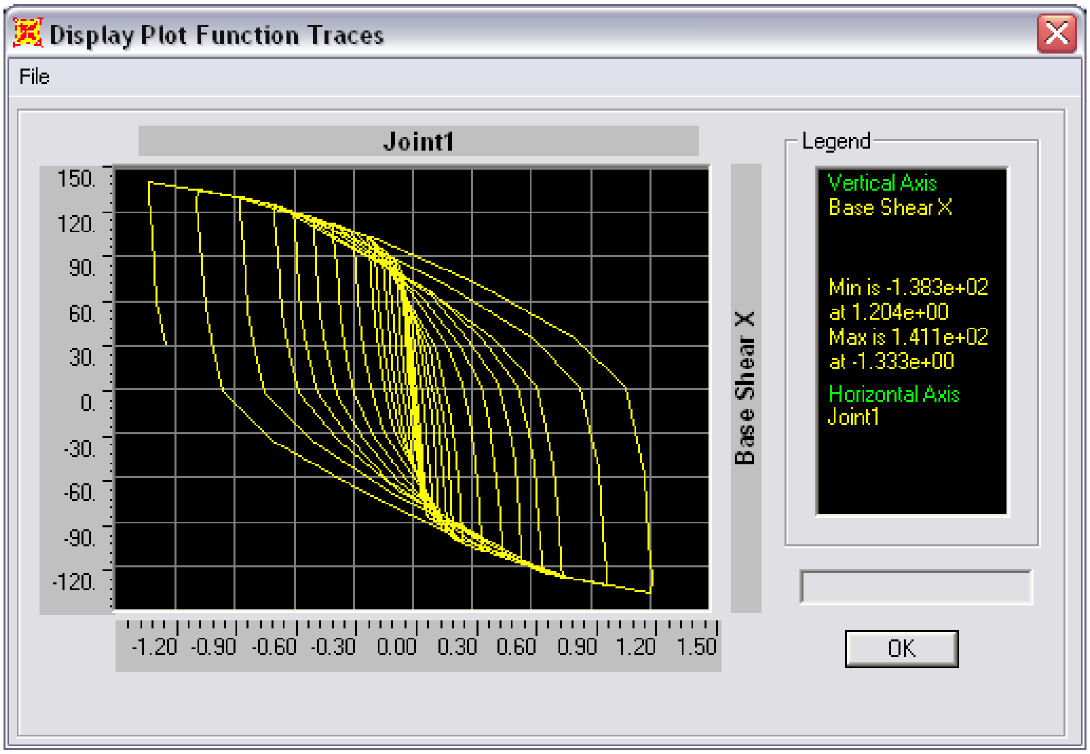


Figure 2.14 - Displacement versus shear load for the Inverted-Y steel bracing system loaded to 145 kips

Figure 2.15 shows the shear force in the link versus the lateral deformation experienced during the loading cycles. The shear link alone resists more than 80 kips of lateral shear force while it deforms plastically. The shear link deformation is almost equal to the lateral displacement of the frame. The performance accentuates the property of the inverted-Y steel bracing system to effectively increase the stiffness of the retrofitted structure while at the same time dissipating energy through plastic deformation in the shear link.

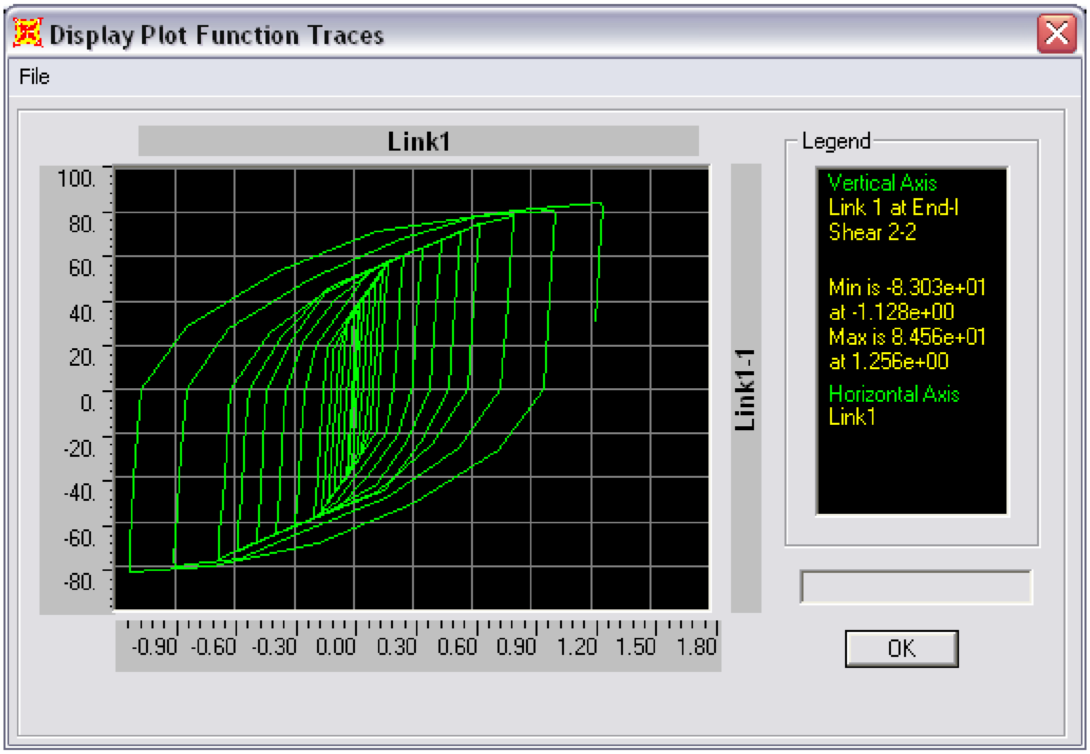


Figure 2.15 - Lateral deformation vs. shear force in the shear link – Frame loaded to 100 kips

3 Prototypes

3.1 Introduction

Many hillside Houses in Puerto Rico are built on platforms supported by gravity columns. Past studies have found these Houses to be most vulnerable to seismic activities and in need of retrofit. The irregular geometries of these structures are often a function of site topographies, although in some cases flood concerns may also be a contributing factor. Many are more than thirty years old and built at a time when seismic provisions were not in effect in Puerto Rico.

The selection of the prototypes analyzed in this Chapter is based on the field data reported by Vázquez in a previous study (Vázquez, 2002). A total of 24 Houses were surveyed by Vázquez around Puerto Rico, from towns containing hilly or escarped terrain like Jayuya, Cabo Rojo, Hormigueros, Yauco and Arecibo. All relevant structural data were recorded including column heights, bay lengths, cross-sectional dimensions and steel reinforcements. A set of prototype Models are created to emulate such characteristics. In addition to these prototypes, some of the actual Houses surveyed are also analyzed.

3.2 Prototype and House Models

The effectiveness of the inverted Y-bracing as a retrofitting scheme has been established in previous studies (Badoux et. al., 1987, Badoux et. al., 1990, Ghoborah et. al., 2001). Although two dimensional computer Models are often sufficient for investigating these systems, the irregular geometries of hillside Houses necessitated the use of three dimensional Models. Figures 3.1 through 3.6 represent 3D Models of the selected prototypes. Included in these Figures are the cross-sectional properties.

The prototypes are rectangular platforms spanning over two to four bays similar to that in typical single family Houses supported by gravity columns. The prototype Models are divided into three main groups depending on the bay lengths and the heights of the gravity columns. The classifications are based on the field data and were previously established by Vázquez (2002). Prototype I Models use columns that are 10 ft long and spaced 12 ft on center. The bay lengths are similar for Prototype II Models; only columns are longer at 15 feet. Prototype III Model has the longest bays, at 16 ft, and the maximum column heights, also at 16 feet. Additionally, three surveyed Houses were selected at random to be analyzed and retrofitted. The dimensions and cross-sections of the House Models can be seen in Figures 3.7 through 3.9. The alpha-numeric tags in these Figures are self explanatory. For example, prototype one with three bays is P1-3B and House Model one is H1.

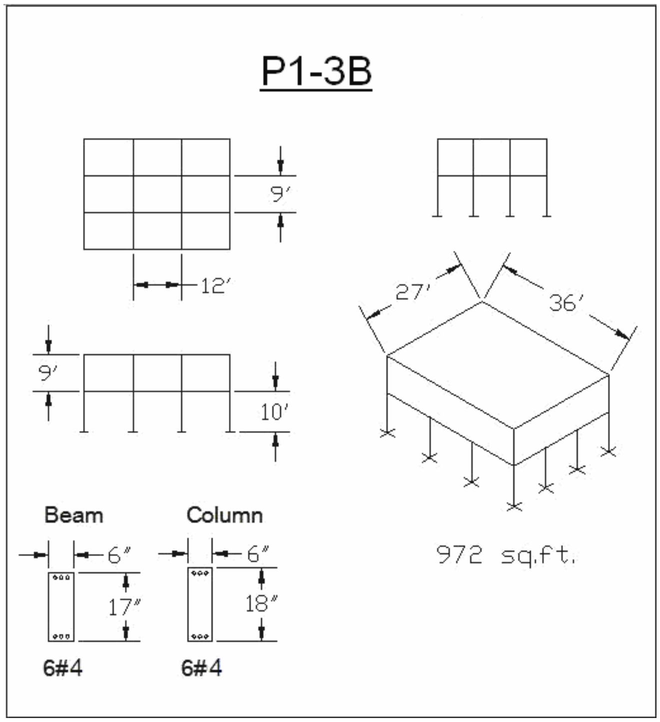


Figure 3.1 - P1-3B: Prototype I Model with three bays

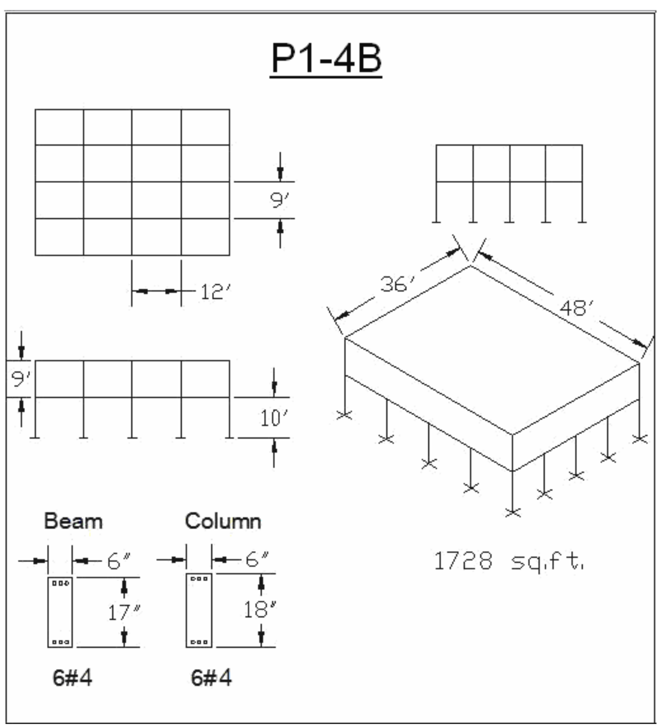


Figure 3.2 - P1-4B: Prototype I Model with four bays

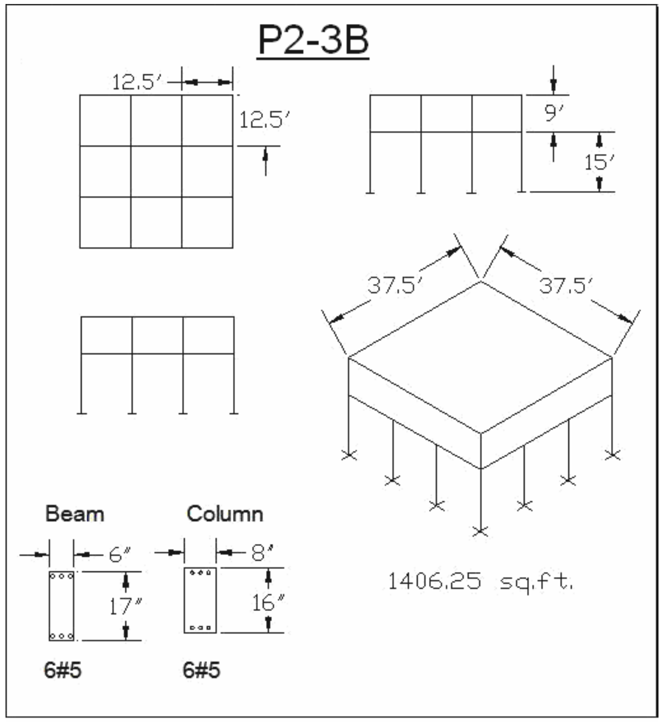


Figure 3.3 - Prototype II Model with three bays

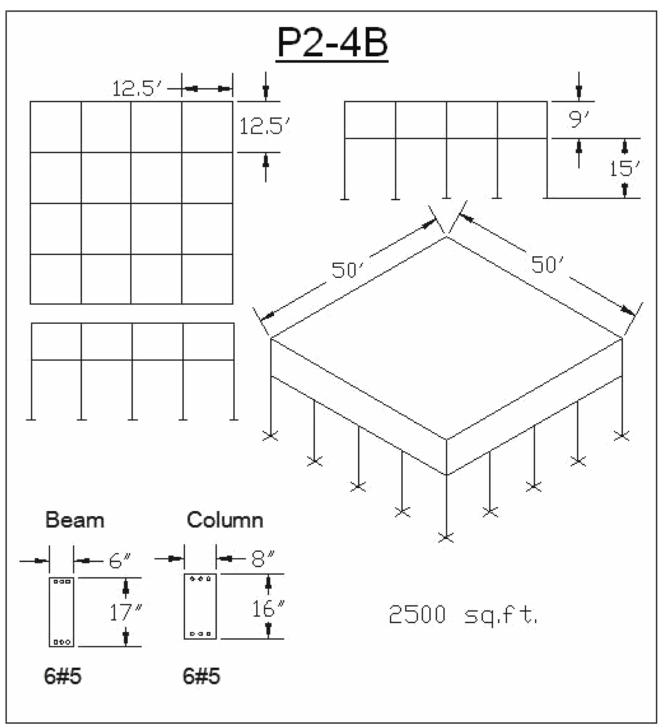


Figure 3.4 - Prototype II Model with four bays

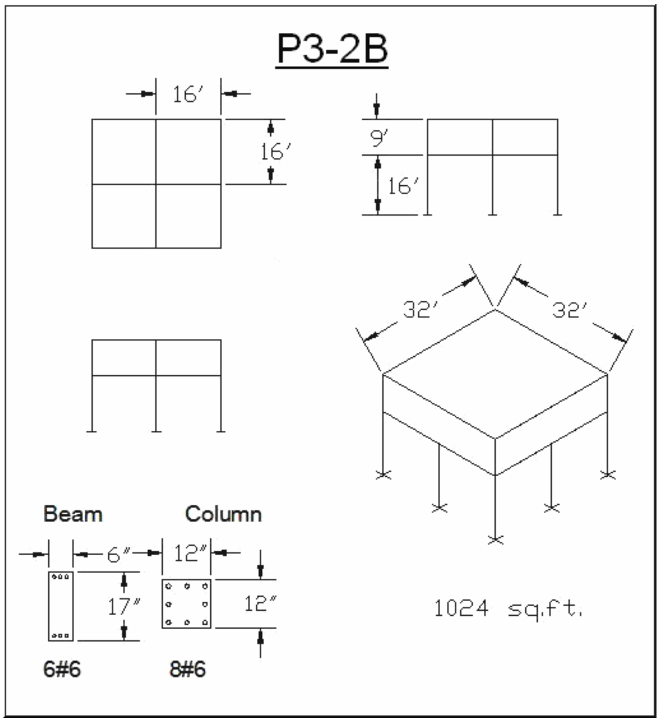


Figure 3.5 - Prototype III Model with two bays

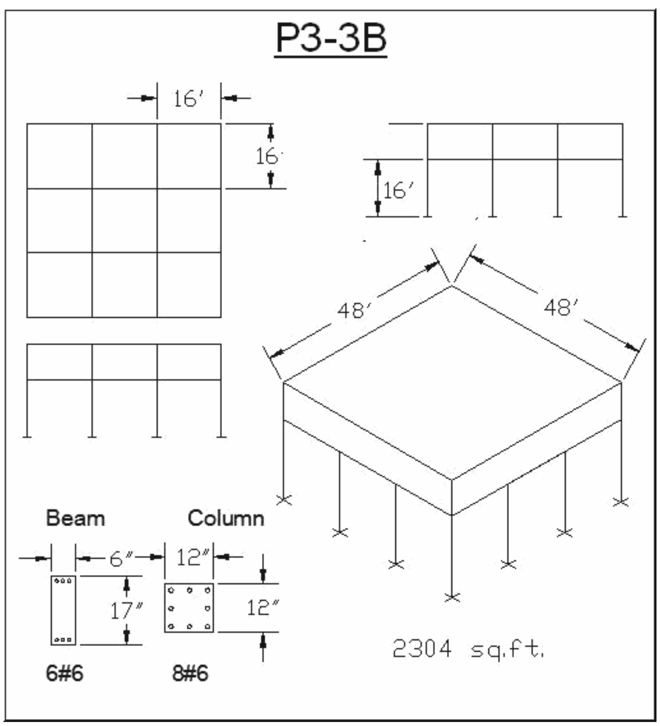


Figure 3.6 - Prototype III Model with three bays

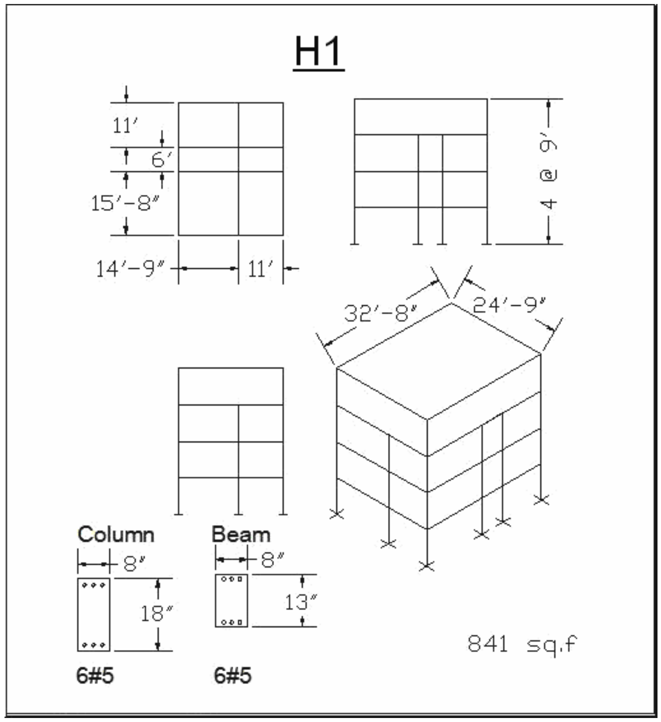


Figure 3.7 - House Model 1

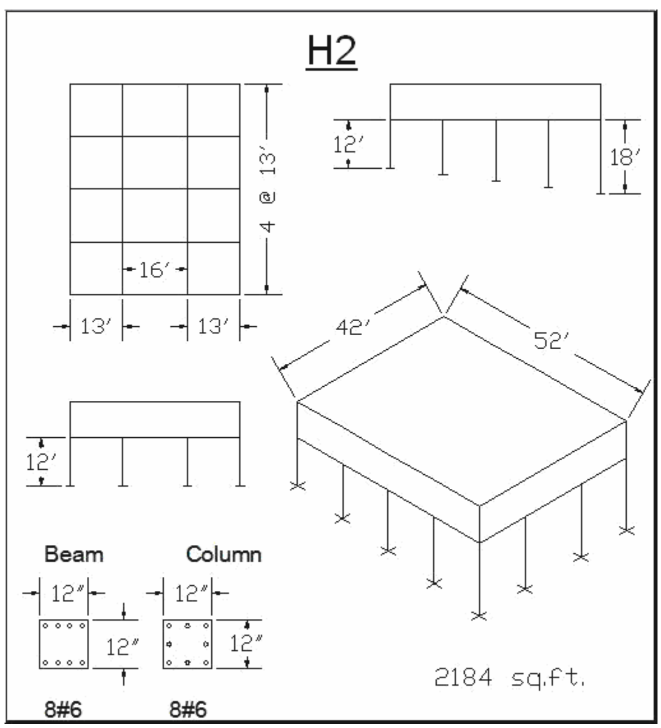


Figure 3.8 - House Model 2

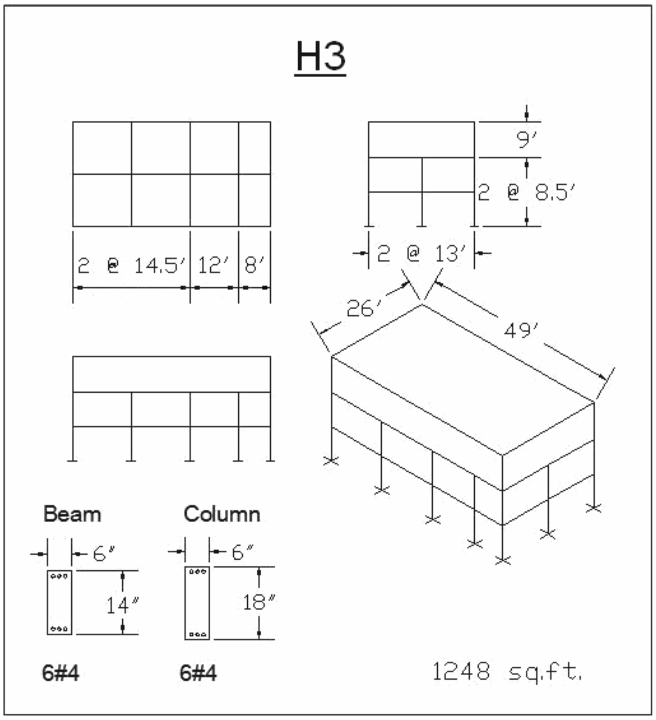


Figure 3.9 - House Model 3

3.3 Earthquakes

To test the structural integrity of the structures, four earthquakes were selected. The Models will be subjected to these earthquakes, with and without amplification to emulate the hillside conditions. As in previous studies, these prototypes are expected to experience extensive damage or collapse. The selected earthquake records vary from the synthetic Model for the Mayagüez area to actual data from earthquakes in California, Japan and the former Soviet Union.

- Mayagüez – The Mayagüez earthquake is a computer generated acceleration record utilized for analysis of the west coast of Puerto Rico. (Irizarry, 1999) No actual earthquake has been measured since the last strong magnitude earthquake occurred at the beginning of the 20th century. The peak ground acceleration is 0.46g.

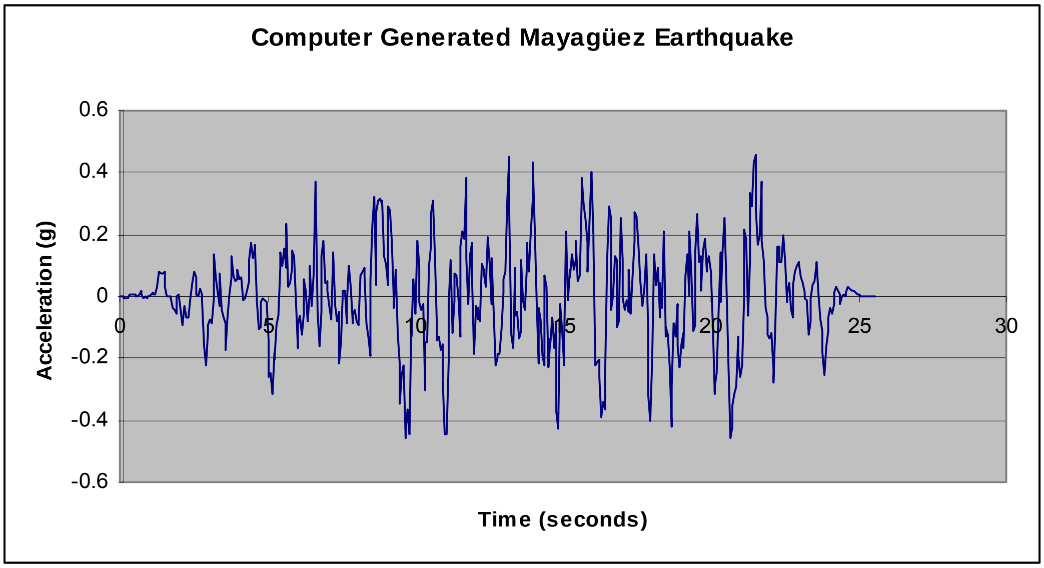


Figure 3.10 - Mayagüez Earthquake Accelerogram

- Northridge – The 1994 Northridge earthquake occurred on January 17, 1994 in the city of Los Angeles, California. The earthquake had a "moderate" magnitude of 6.7, but the ground acceleration was the highest ever instrumentally recorded in an urban area in North America, and it proved to be the most costly earthquake in United States history. Damage occurred up to 125 km (85 mi) away, with the most damage in the west San Fernando Valley, the city of Santa Monica, and Simi Valley. Fifty-one people were killed, and 9000 were seriously injured. Post-quake investigations revealed that some structural specifications did not perform as well as expected. Because of this building codes were revised. Figure 3.11 shows the utilized accelerogram, it was measured at the White Oak Covenant Church in Northridge, California. For this record the peak ground acceleration was 0.43g.

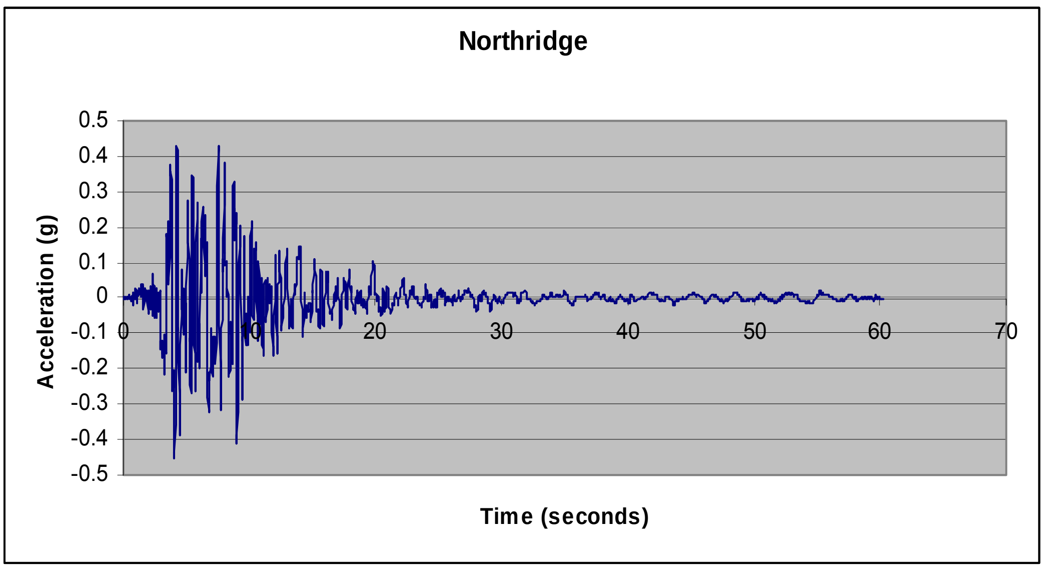


Figure 3.11 - Northridge Earthquake Accelerogram

- Kobe – The Great Hanshin Earthquake Disaster, or Kobe earthquake as it is more commonly known overseas, was an earthquake in Japan that measured 7.2 on the Richter scale. It occurred on January 17, 1995 in the southern part of Hyogo Prefecture and lasted for approximately 20 seconds. Approximately

6,200 people, mainly in Kobe, lost their lives, being that Kobe was the closest major city to the epicenter of the earthquake. Additionally, it caused approximately ten trillion yen in damage, 2.5% of Japan's GDP at the time. It was the worst earthquake in Japan since the Great Kanto earthquake in 1923, which claimed 140,000 lives, and it is listed in the Guinness Book of Records as the "costliest natural disaster to befall any one country." Figure 3.12 shows the utilized accelerogram which was measured in the Nishi-Akashi station. For this record the peak ground acceleration was 0.51g.

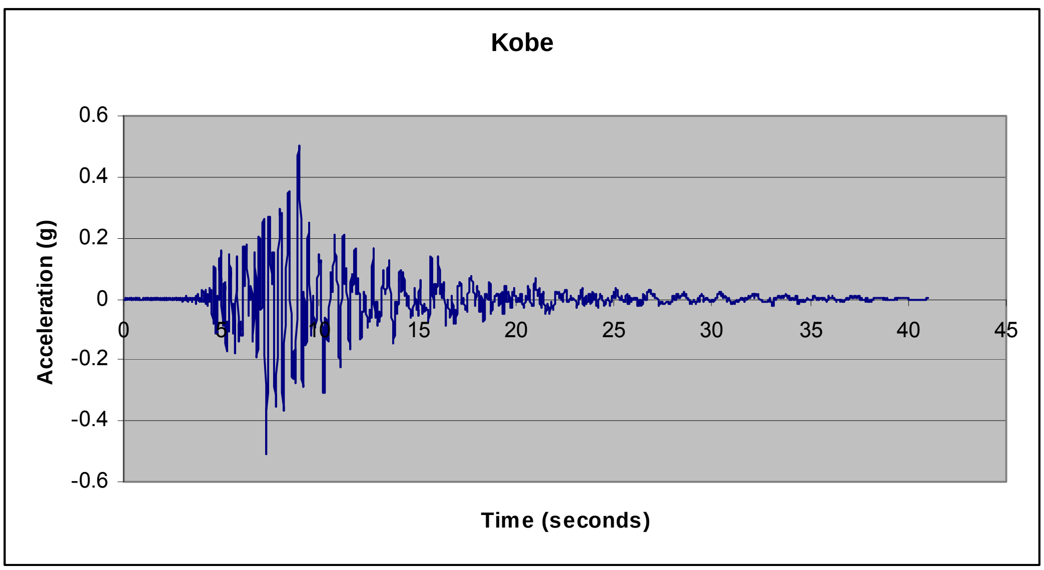


Figure 3.12 – Kobe Earthquake Accelerogram

- Uzbekistan – The Uzbekistan earthquake occurred in May 17, 1976 at the city of Gazli, a region not known for strong seismic activity. It measured 7.0 on the Richter scale. This earthquake was one of three major earthquakes occurring from 1976 to 1984. According to the Russian newspaper Izvestia, oil drilling in Gazli, Uzbekistan triggered quakes in 1976 and 1984. The oil drilling companies, an international consortium including Exxon, Texaco, Marathon, McDermott, Mobil, Shell, Mitsui and Mitsubishi have so far refused to guarantee that their operations can survive large earthquakes. The accelerogram utilized, seen in Figure3.13, was measured in the Karakyr station, its peak ground acceleration is 0.7g.

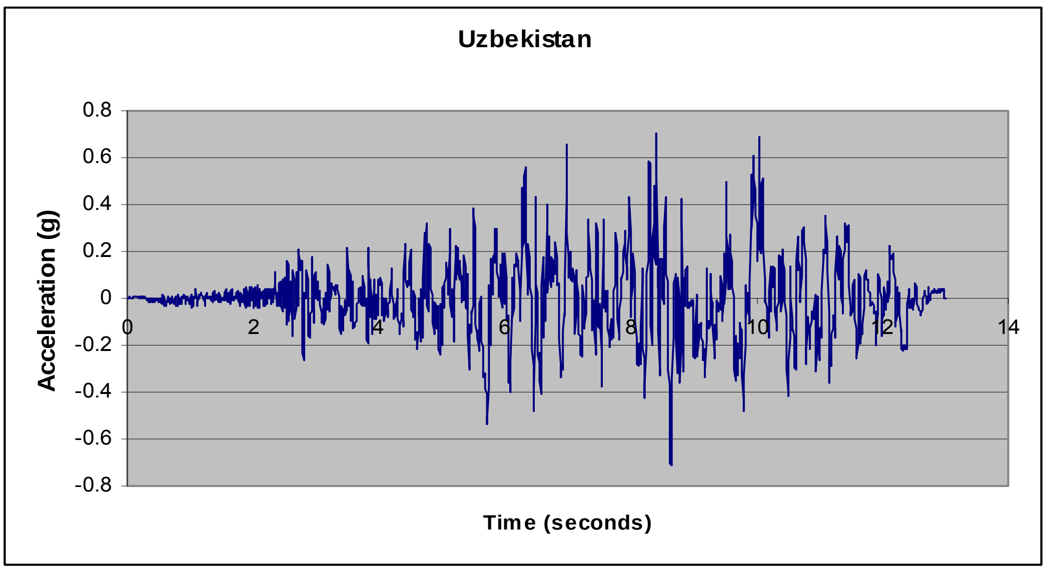


Figure 3.13 - Uzbekistan Earthquake Accelerogram

3.4 Damage Estimation

The damages to the structures are determined based on the maximum drift. The two definitions of drift utilized are the total drift and the inter-story drift.

Total drift can be calculated as the lateral displacement divided by the height of the column. Inter-story drift can be calculated as the inter-story displacement divided by the height of the story.

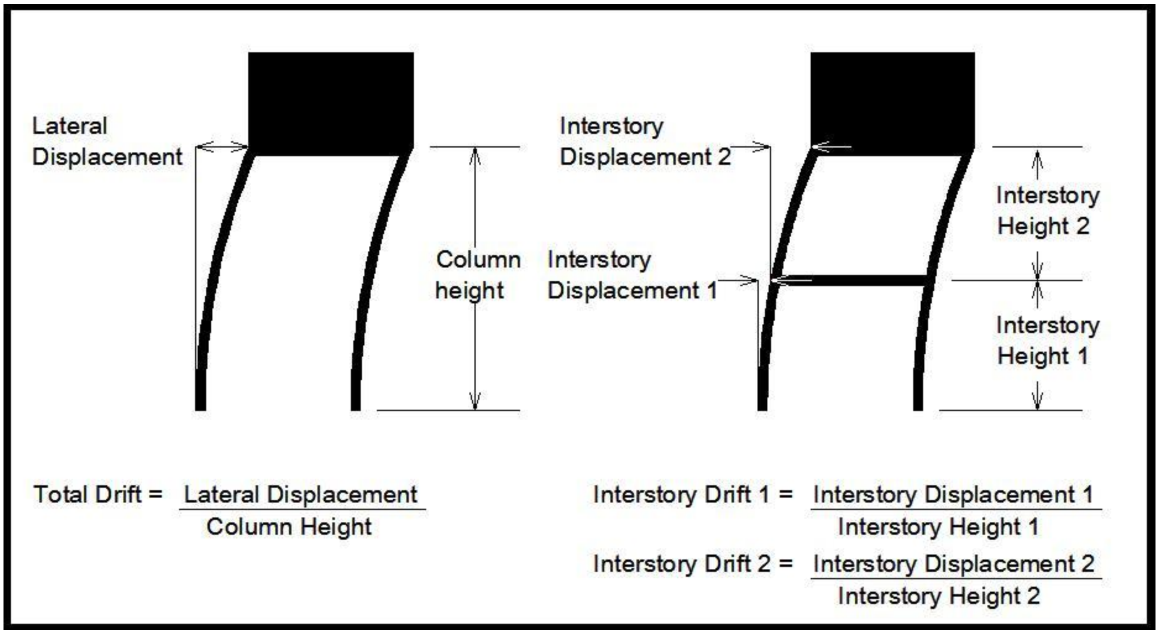


Figure 3.14 - Total and inter-story drifts

Once the drift is determined for each structure, a damage state may be assigned as None, Slight, Moderate, Extensive or Complete. These damage states are in accordance with the HAZUS-MH definitions for moment resisting concrete frames (HAZUS, 2003). Taken directly from the HAZUS User Guide:

- "Slight Structural Damage: Flexural or shear type hairline cracks in some beams and columns near joints or within joints."
- "Moderate Structural Damage: Most beams and columns exhibit hairline cracks. In ductile frames some of the frame elements have reached yield

capacity indicated by larger flexural cracks and some concrete spalling. Non-ductile frames may exhibit larger shear cracks and spalling."

- "Extensive Structural Damage: Some of the frame elements have reached their ultimate capacity indicated in ductile frames by large flexural cracks, spalled concrete and buckled main reinforcement; non-ductile frame elements may have suffered shear failures or bond failures at reinforcement splices, or broken ties or buckled main reinforcement in columns which may result in partial collapse."
- "Complete Structural Damage: Structure is collapsed or in imminent danger of collapse due to brittle failure of non-ductile frame elements or loss of frame stability. Approximately 13% (low-rise), 10% (mid-rise) or 5% (high-rise) of the total area of the buildings with complete damage is expected to be collapsed."

The structures analyzed fall under the category of pre-code structures which are structures that are not seismically designed. Another reason the structures are classified as pre-code is that given the amount of research made in the amplification of seismic waves in hilly or escarped terrain, it is possible that in the future the codes are amended to include this amplification factor in the design of future structures. The drift values for the structural damage states as given by HAZUS are listed in Table 3.1.

Table 3.1 - Damage Based on Drift

Drift Value	Damage State
0 - 0.004	No Damage
0.004 – 0.006	Slight
0.006 – 0.016	Moderate
0.016 – 0.04	Extensive
More than 0.04	Complete

3.5 Structural Analysis

A three-dimensional nonlinear modal analysis was performed on the structures utilizing the structural analysis program SAP2000. The three dimensional analysis allows the result to include torsional modes of deformation that could not be appreciated with only a two-dimensional analysis. This is essential information when evaluating the most effective retrofitting measures. The building Models were subjected to the acceleration record of the previously mentioned earthquakes (Mayagüez, Northridge, Kobe and Uzbekistan) under normal circumstances and also with the acceleration multiplied by a factor of two.

The structures are residential Houses supported on slender reinforced concrete gravity columns. The roof and floor slab are five inch thick, the exterior walls consist of six inch masonry blocks with covering and the interior walls consist of four inch masonry blocks also with covering. The maximum lateral displacement of every case analyzed is measured at the top of the column which is also considered the floor slab. This is because the retrofitting system that will be tested will reinforce the slender columns and it is at this location that we can detect the reduction in deflections that the retrofitting systems will provide.

The plots in Figures 3.15 through 3.20 present the maximum total drift values of the prototype Models for each of the earthquake cases. In each case, the drifts for earthquakes without amplifications are presented in both the x and the y horizontal directions. For amplified earthquakes the drift values are represented by symbols designated as X-A and Y-A. In the plots, the drift values that fall in the yellow area represent extensive damage, and values in the red area represent collapse in the structure. For the Model House plots the values in the green area represent a moderate level of damage. Some drift values were extremely high meaning that the structure collapsed. There were cases not represented in the following plots that resulted in drift values higher than 0.2.

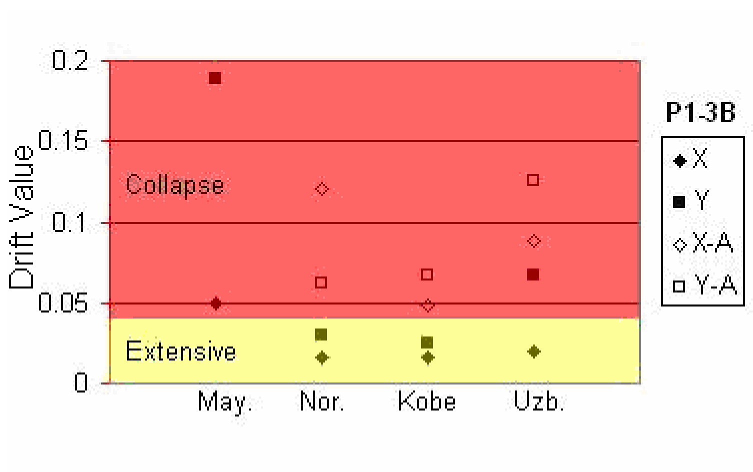


Figure 3.15 - Drift: Prototype I Model with three bays (P1-3B)

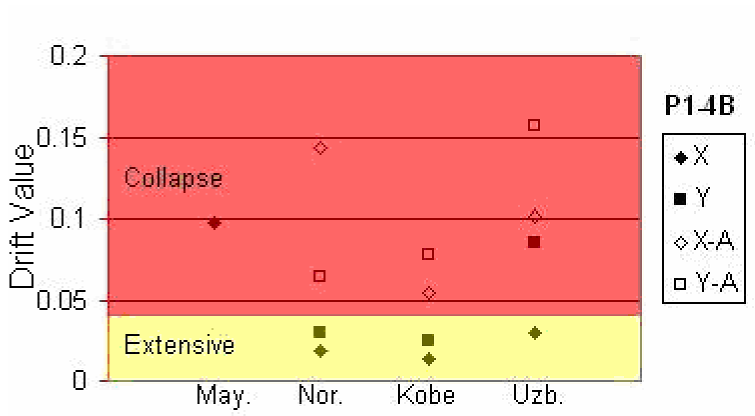


Figure 3.16 - Drift: Prototype I Model with four bays (P1-4B)

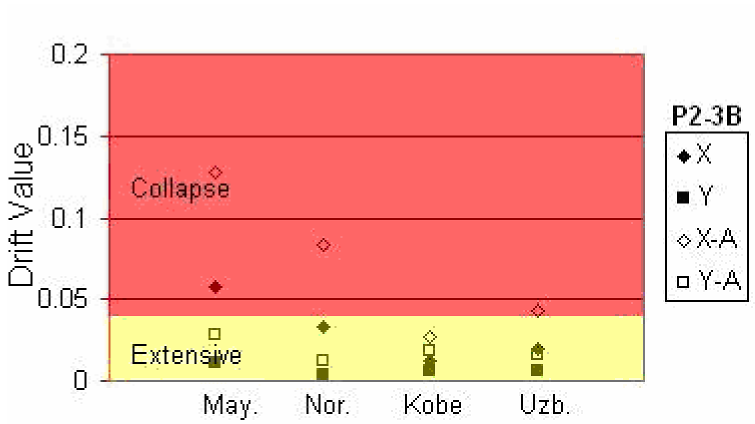


Figure 3.17 - Drift: Prototype II Model with three bays (P2-3B)

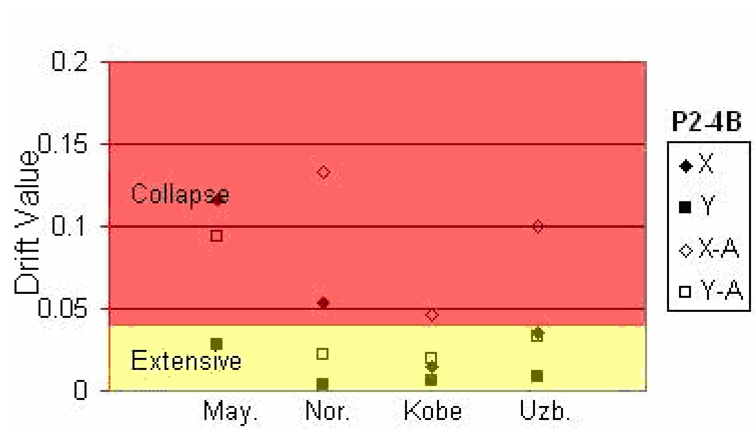


Figure 3.18 - Drift: Prototype II Model with four bays (P2-4B)

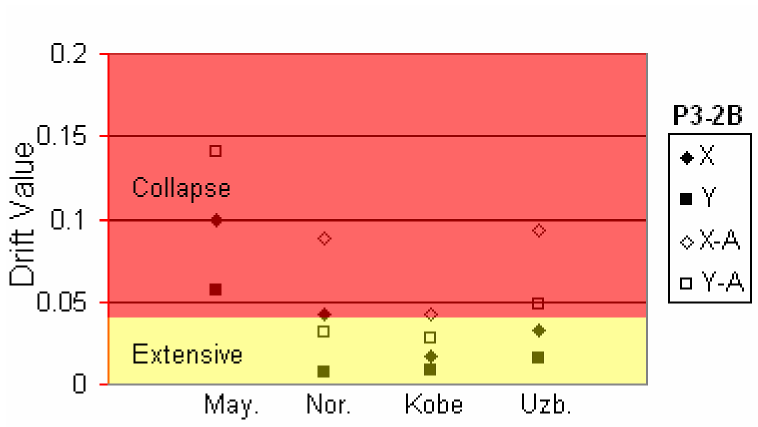


Figure 3.19 - Drift: Prototype III Model with two bays (P3-2B)

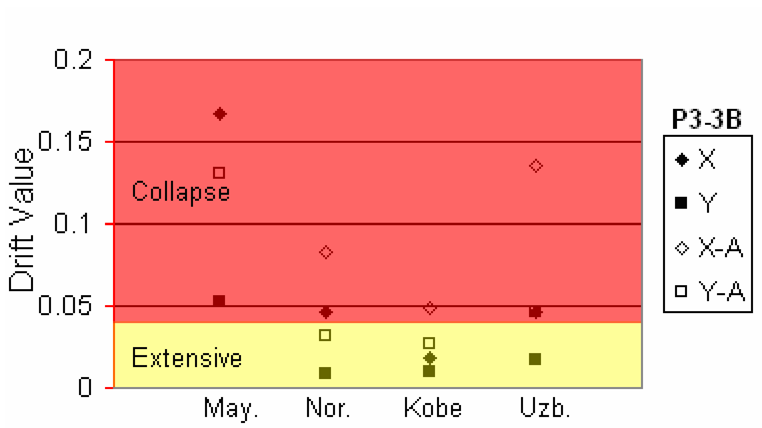


Figure 3.20 - Drift: Prototype III Model with three bays (P3-3B)

House Model 1 has columns that are twenty-seven feet tall with beams located every nine feet of elevation in order to provide additional lateral support. At these elevations the maximum drift was measured utilizing the inter-story drift formula and it was observed that the magnitude of the drift did not vary linearly from ground level to the floor slab. Instead the structure always experienced higher drift values at nine feet of elevation. Figure 3.21 presents the corresponding drift values for House Model 1.

The heights of the columns for House Model 2 vary linearly from twelve to eighteen feet. The drift was measured at every column and it was observed that the column with a height of twelve feet always presented the higher drift values. Figure 3.22 present the corresponding drift values for House Model 2. House Model three has seventeen feet tall columns with a beam at mid-height. Similar to House Model 1, the drift did not vary linearly from ground level to floor slab. Instead the higher drift values were measured from the mid-height beam to the floor slab. The values were calculated utilizing the inter-story drift formula and they are presented in Figure 3.23.

Prototype Models 1 and 2 suffered extensive damage and collapse when subjected to earthquakes without amplification and they collapsed in all cases when the amplification factor was utilized. Prototype Model 3 suffered extensive damage in cases without the amplification factor and collapsed when the factor was applied. In general, the House Models sustained less damage than the prototypes; however in all cases retrofitting is needed to prevent extensive damage in earthquakes without amplification and to prevent collapse in amplified earthquakes.

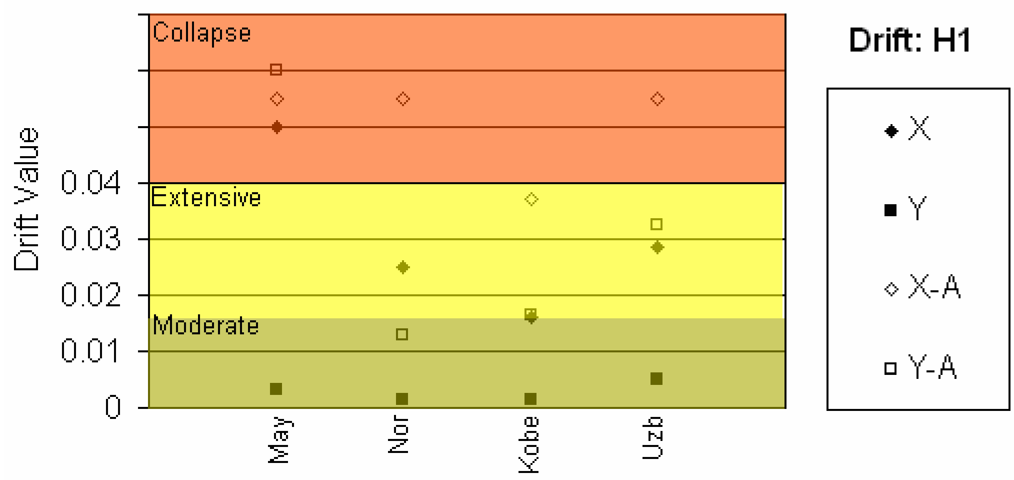


Figure 3.21 - Drift: House Model 1

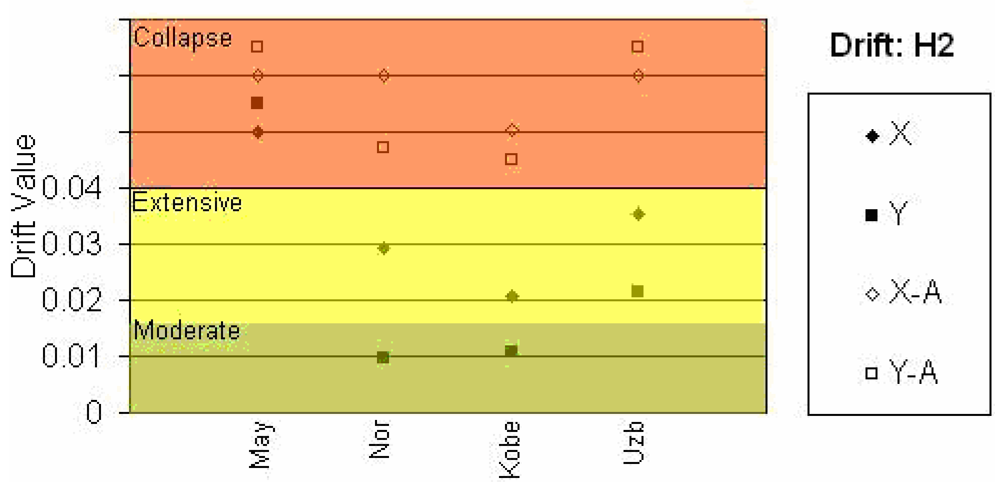


Figure 3.22 - Drift: House Model 2

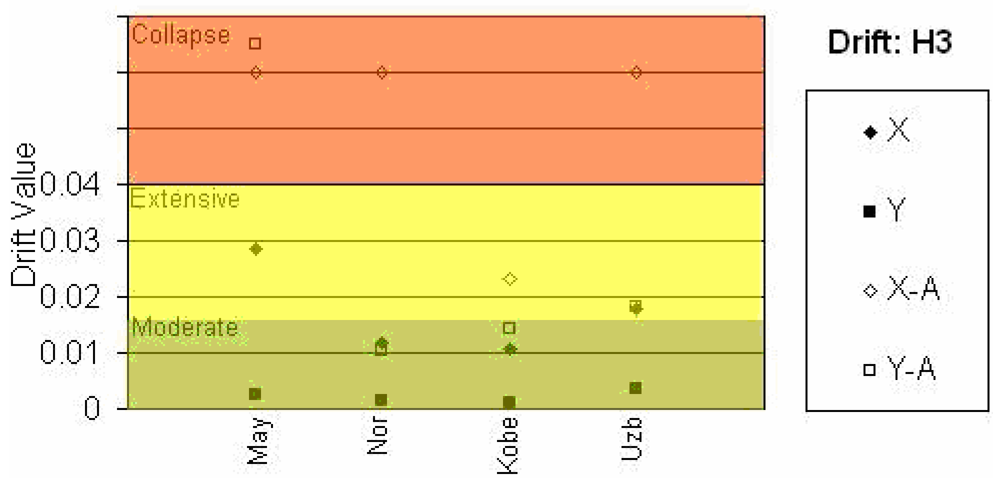


Figure 3.23 - Interstory Drift: House Model 3

3.6 Summary of Damage States

Table 3.2 presents the damage level experienced by the Models when subjected to the different earthquake records. The prototype Model experience collapse in exactly half of the cases when subjected to earthquake records without acceleration and with acceleration applied, 22 out of 24 cases showed collapse. Only two out of three of the House Models showed collapse in cases without amplification but all experienced collapse when amplification was applied.

Table 3.2 – Summary of Damage Levels

Building ID / Earthquake Record	Mayagüez	Northridge	Kobe	Uzbekistan	Mayagüez-Amplified	Northridge-Amplified	Kobe-Amplified	Uzbekistan-Amplified
P1-3B	C	E	E	C	C	C	C	C
P1-4B	C	E	E	C	C	C	C	C
P2-3B	C	E	E	E	C	C	E	C
P2-4B	C	C	E	E	C	C	C	C
P3-2B	C	C	E	E	C	C	C	C
P3-3B	C	C	E	C	C	C	E	C
H1	C	E	E	E	C	C	E	E
H2	C	E	E	E	C	C	C	C
H3	E	M	M	E	C	C	E	C

4 Retrofitting with Inverted-Y Steel Bracing

4.1 Introduction

Of all the prototypes analyzed in the previous Chapter, none survived an amplified earthquake and all suffered extensive damages when subject to earthquakes without amplification. In this Chapter, the inverted-Y steel bracing system will be implemented as a retrofitting measure to improve seismic performance. The retrofitted structures will be subjected to the same analysis as the originals and the drift results and damage levels will be presented. The goal is to select a bracing configuration for each structure that prevents the collapse of the structure and, if possible, reduces the damage level from extensive to moderate or less.

4.2 System Description

The components of the inverted-Y steel framing system that will be utilized were chosen so that they can be effective for all the structures that have already been analyzed without modifications. If the bracing was going to be applied to a new and expensive structure, the components would be chosen in a way that would maximize the effectiveness of the system. The Models studied are single family dwellings, owned by people with moderate or low incomes.

Selecting a different system for each House based on a detailed structural analysis may not be feasible. Instead a system will be chosen that proves effective for all the tested structures. It will be tested in a set on the prototype Models that were analyzed in the previous Chapter. From the retrofitted analysis results, a set of recommendations will be made on the number and placement of braces based on the geometric properties of a structure. The idea being that any contractor, following the recommendations, will be able to select an effective retrofitting scheme for the structures without having to carry out any extensive structural analysis.

Figure 4.1 shows the inverted-Y system analyzed in Chapter 2. The shear link utilized is made of a W8x10 steel section with a longitude of 12 inches. The steel framing members consist of C7x12.25 sections with stud bolts welded to their surface. The function of the C section is to have a surface to attach the other steel components and help transfer shear from the concrete frame to the shear link. The diagonal bracing elements connecting the shear link to the base of the steel frame are round A501 HSS6.625 sections. The stud bolts utilized have a diameter of 3/4-in with a longitude of 5 inches. They are spaced at intervals of 6 inches and are welded to the steel frame and also placed inside the existing concrete frame. The gap between the stud bolts is filled with mortar, creating a cross section of 6-in x 6-in of mortar with the stud bolts inside. The mortar along with the stud bolts transfers the shear force from the concrete frame to the shear link.

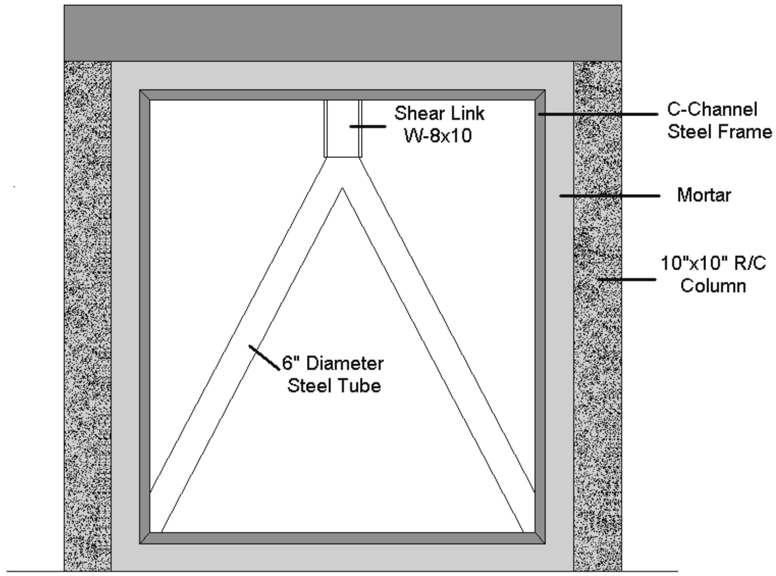
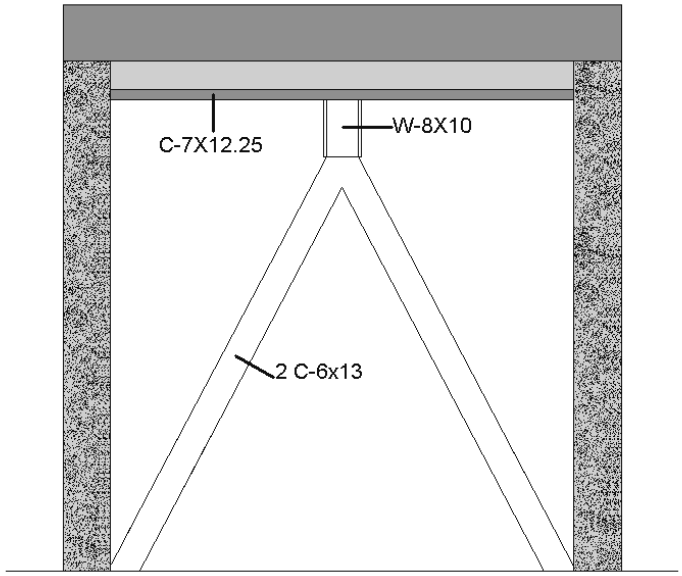
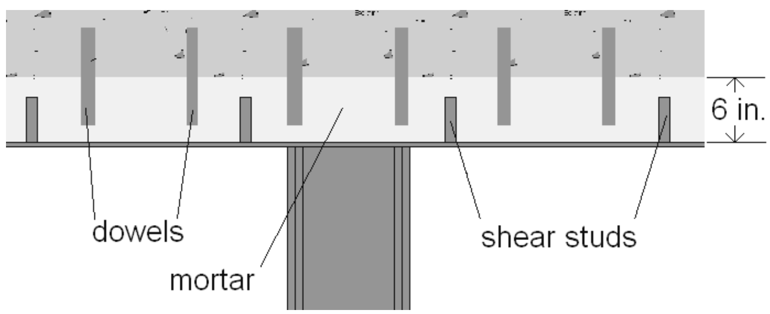


Figure 4.1 - Steel frame retrofit with inverted–Y bracing

The configuration in Figure 4.1 was proven effective in the study by Okada et al. (1999). However, a simpler configuration used by Perera et al. (2004) in their experimental studies on multi-story buildings is the preferred solution. Figure 4.2 illustrates the basic idea where the C channel steel frame is replaced by the top beam connected to the shear link. This is justified by the fact that the shear force is effectively transferred from top beam through the stud bolts and mortar down to the shear link. The A992 W8x10 shear link previously discussed in Chapter 3 will be utilized again in the proposed scheme. Instead of the round tubes, however, double C sections are a more practical choice for diagonal bracing members. This will result in simpler brace-to-link connections (Figure 4.3).



(a) Front view



(b) Connecting the beams

Figure 4.2 - Proposed retrofit using inverted-Y braces

Figure 4.4 shows the details for shear link assembly. The weld sizes noted in this Figure were later verified in Section 4.7. For the mechanism to function properly, the shear demands transferred from the steel beam cannot exceed the capacity of the beam-to-shear link connection. Figure 4.4 shows the weld length available to the connection. Included in this Figure are the flange cover plates that are required in order to meet shear demands reported in Section 4.7(Sarraf et. al., 1998). The minimum weld size may be determined from (AISC, 2005):

$$\boldsymbol{D} = \frac{0.75\boldsymbol{V_u}}{\boldsymbol{(}1.392\boldsymbol{)}\left[2\boldsymbol{(}\boldsymbol{L}_1 + \boldsymbol{L}_2 + \boldsymbol{b_f} - \boldsymbol{t_w}\boldsymbol{)}\right]} \tag{4.1}$$

where D is the number of sixteenths of an inch in weld size. V_u is the shear demand. As usual, b_f is the flange width and t_w is the thickness of the web. The parameters L_1 and L_2 are as depicted in Figure 4.4. In Equation 4.1, the factor 0.75 is used to take the resistance factor out of the weld capacity equation.

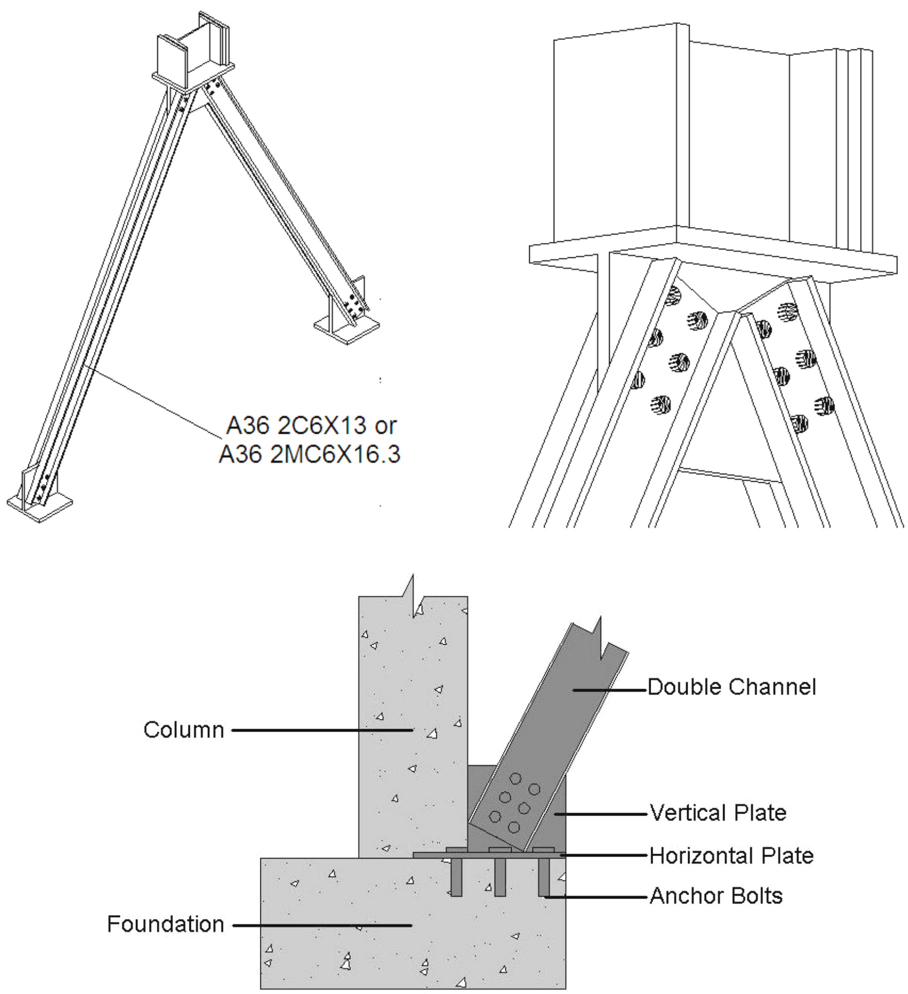


Figure 4.3 - Connection details for diagonal braces

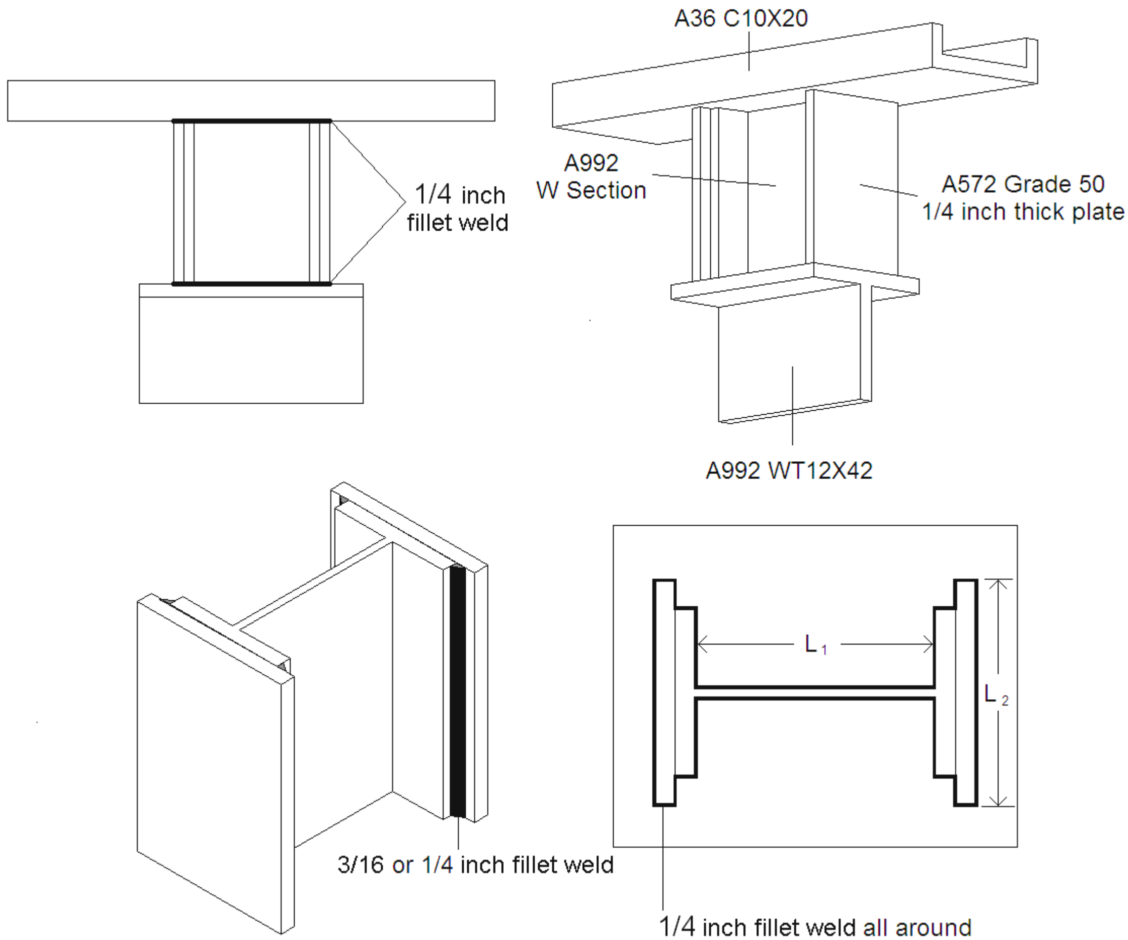


Figure 4.4 - Shear link connection details

The minimum thickness for the flange cover plates is determined based on the connecting element rupture strength requirements (AISC, 2005):

$$t_{cp} = \frac{V_u - 0.6F_u\left[L_2 t_w + 2(b_f - t_w)t_f\right]}{1.2F_u L_1} \qquad (4.2)$$

where t_{cp} is the thickness of the cover plate and t_f is the thickness of the flange. All other parameters are as previously defined. The flange is continuously welded to the cover plate on both edges. Given the length of the shear link, this will give a weld length of 48 inches for the two flanges. The minimum weld size for flange to cover plate connection may then be determined from:

$$\boldsymbol{D} = \frac{0.75\boldsymbol{V_u}}{1.392(48)} \qquad (4.3)$$

Using the above equation, 3/8-in weld size is adequate for shear demands of up to 267 kip. Additionally, 1/4-in weld size can manage up to 356 kips.

An advantage that the Perera scheme has over the Okada scheme is that most of the assembly can be done beforehand. The stud bolts, shear links, and the connecting steel plate can be shop welded to the C sections. The bolt holes in the connecting steel plate may also be predrilled or punched depending on the slope of the diagonal. The in-site preparations are limited to drilling holes in concrete to insert the anchor bolts, and pouring mortar once the metal beam is in placed.

The connection details for the diagonal braces are shown in Figure 4.3. The size and number of the bolts are determined based on the capacity demands on braces and are discussed in Section 4.7. When making the connections at the foundation, steel bearing plates of adequate width and thickness are installed using anchor bolts. The plate is shop welded to another plate to form a tee. The double channel braces are bolted to the stem.

4.3 General Procedure

The prototype Models serve the purpose of testing the performance of the Inverted-Y steel bracing systems in a variety of different scenarios. By changing column heights from 10 to 16 feet we can study the effect of the bracing system on numerous geometrical configurations including bay lengths and number of bays. If the bracing proves effective in enhancing seismic performance in the prototype Models, it can safely be designated as an effective system for most reinforced concrete structures supported on gravity columns. The amplified earthquakes take into account the possibility of uneven or hilly terrain, bad soil conditions, or a combination of these factors.

The main goal of the Inverted-Y bracing system is to prevent the collapse of the structures under any condition. If we can reduce the level of damage from collapse to extensive, we can give the occupants the chance to survive the earthquake. With life safety criteria met, the second goal will be to save the structure. Bracing configurations to reduce the damage level from extensive to moderate or less will be investigated. With this reduction in damage levels, the immediate occupancy issues may be addressed.

The unretrofitted prototype structures suffered extensive damages when loaded in either direction. Therefore, the retrofitting scheme will utilize at least one inverted-Y bracing in each direction. Initially, the A992 W8x10 shear links are used. If the damages are reduced to moderate or less, the configuration is marked as effective. If the damage state is extensive or total, the shear link section will be changed to one with a higher shear capacity. If the change in shear link does not reduce the damages to the desired level, the number of inverted-Y braces is increased. The process continues until the configuration meeting the requirements is found.

4.4 Prototype I Models

The prototype I Models have 12 ft long columns spanned 12 ft on center. For the first trial, the prototype P1-3B will be tested with two inverted-Y braces placed in an L configuration at the center of the structure. This is shown in Figure 4.5. The prototype will be retrofitted with different shear links to determine how an increase in shear capacity of the link will improve the performance of the structure and reduce the total number of inverted-Y braces needed to reach the desired damage state.

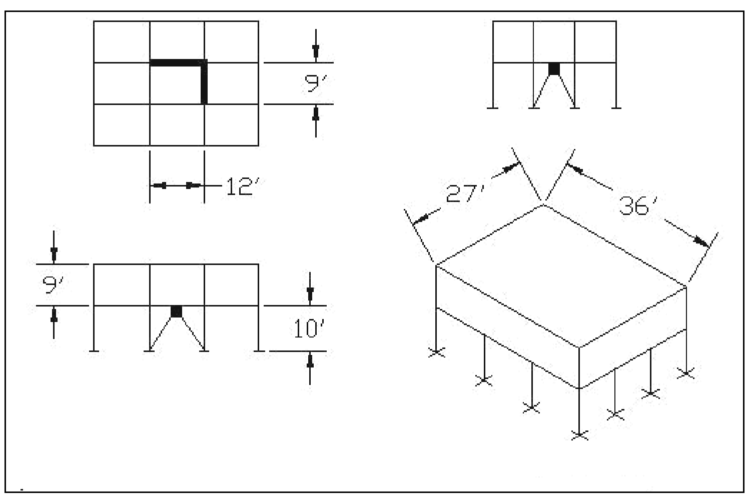


Figure 4.5 - P1-3B prototype with two inverted-Y braces

Figure 4.6 depicts the changes in performances following the retrofits. Four design earthquakes are considered in both horizontal directions, resulting in eight cases. The earthquakes are not amplified. The drift values are presented for unretrofitted system as well as for the systems retrofitted with two different shear link sizes. After retrofitting the goal is to reach a moderate level of damage or less. In the graph this level is represented by the yellow line, once every point in the graph is below this level, the configuration will be considered successful. The red line represents the limit between the extensive level of damage and collapse.

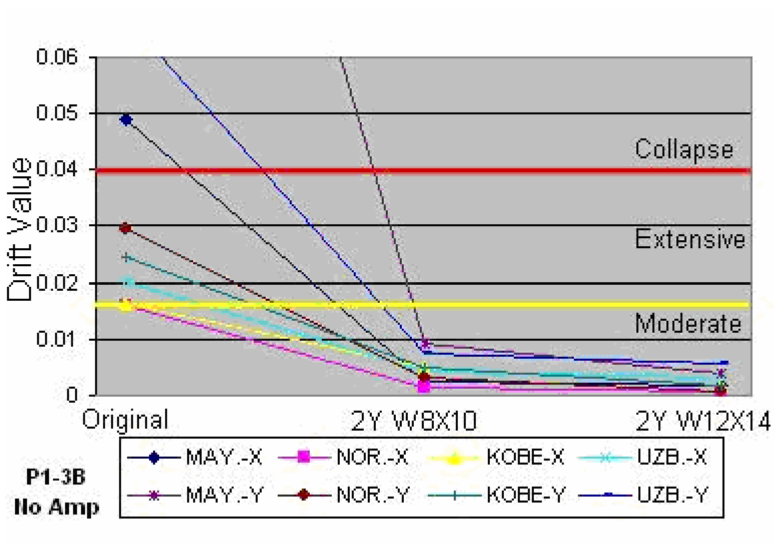


Figure 4.6 Drift - P1-3B prototype with two inverted-Y braces – No amplification

From Figure 4.6, we can see that when subjected to the Mayagüez or Uzbekistan earthquake, prototype P1-3B collapsed without the use of retrofits. With the two inverted-Y braces and W8x10 shear links, the damage is reduced to a satisfactory level. The situation is very different when the earthquakes are amplified. This case is shown in Figure 4.7. While utilizing the W8x10 section as a shear link, the structure is still in danger of collapsing. Since the goal is to reduce the damage to moderate, section W12x24 is utilized as a shear link in the second trial. Although there are some improvements, extensive damage states are still reported in most cases.

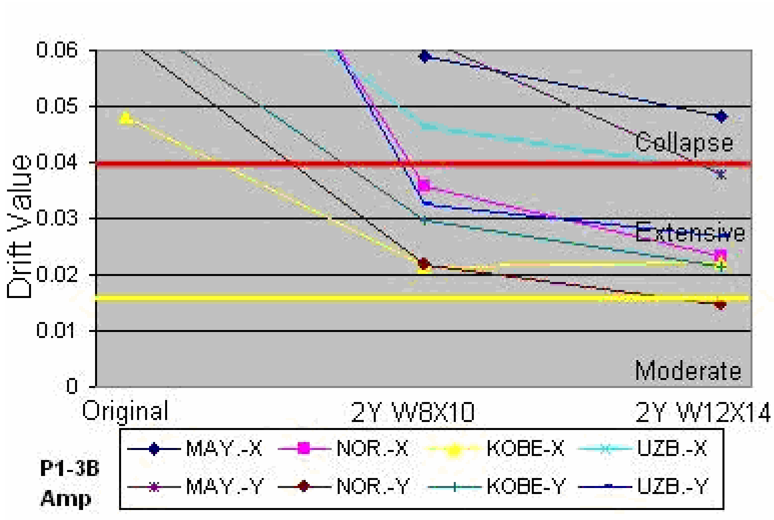


Figure 4.7 - Drift: P1-3B prototype with two inverted-Y braces –Amplified earthquakes

For P1-3B prototypes subject to amplified earthquakes, highest drift values were from exterior columns, specifically those in the corners. Figure 4.8 describes the behavior of P1-3B prototype with two inverted-Y braces. The four nodes in the floor plan of the structure are represented in the graph by their respective colors. The graph shows the displacement of each node in the X direction during a period of four seconds that the structure was subjected to the Kobe earthquake. The yellow node presents the highest displacement values; this is the node that is furthest away from the braces. The green node has an almost identical behavior to the yellow node but in a lesser magnitude. The red node is located right on top of the bracing members; this node shows the least displacement. The displacement of the blue node has a magnitude similar to that of the green node but is in the opposite direction.

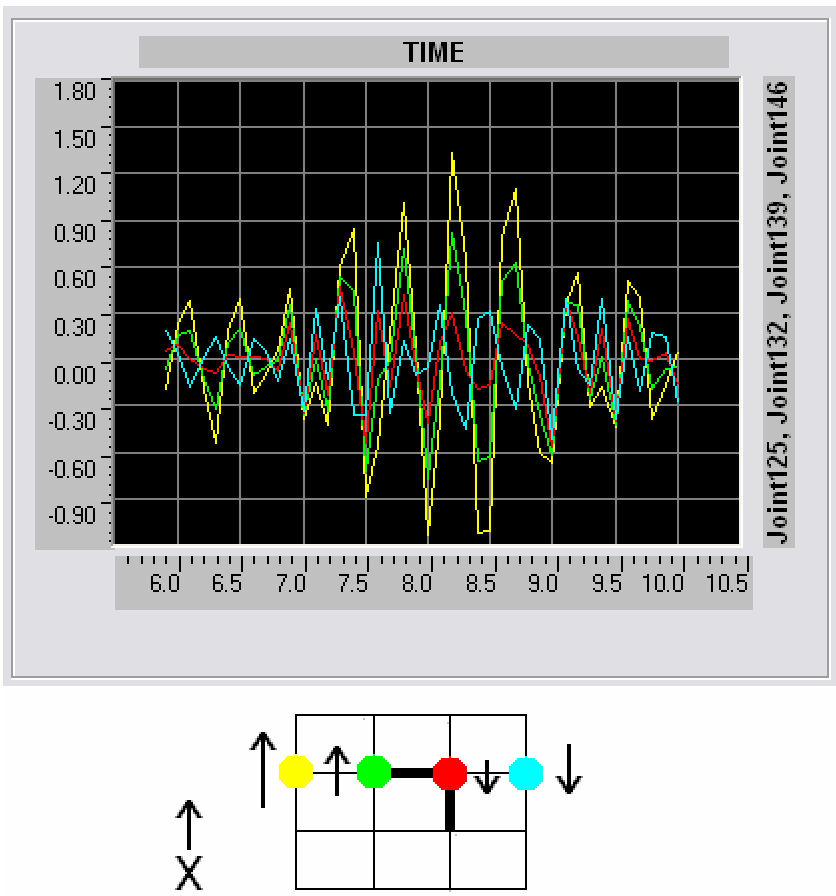


Figure 4.8 – Displacements: P1-3B prototype with two inverted-Y braces

A schematic contour of the displacement magnitudes experienced by the structure is shown below the graph in Figure 4.8. It demonstrates a rotational movement of the structure around the bracing elements. An increase in shear capacity of the braces is not able to significantly alter the rotational behavior. The columns that suffer extensive damages are located too far from the braces to be affected in a positive manner by an increase in the capacity of the braces. This configuration fails to improve the damage state when subject to amplified earthquakes.

For the next retrofitting configuration, two inverted-Y braces are located in the X direction to try to minimize the rotational effect. The two braces will be located in the exterior columns as shown in Figure 4.9. Figure 4.10 presents the results of the analysis when subject to amplified earthquakes. With the use of A992 W12x14 steel sections as shear links, the damages are reduced to desired levels in all cases. However, the use of A992 W8x10 steel sections as shear links is problematic in some cases. The recommended configuration for P1-3B prototypes in the case of amplified earthquakes is three inverted-Y braces with W12x14steel section as a shear link.

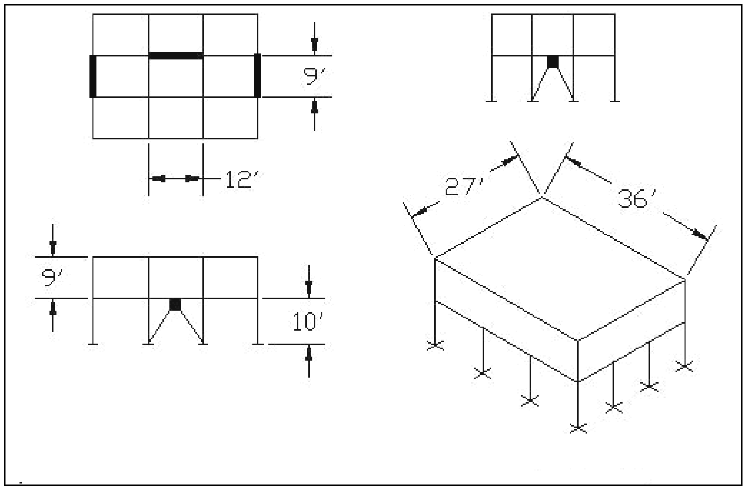


Figure 4.9 - P1-3B prototype with three inverted-Y braces

The most effective retrofitting schemes for P1-4B prototypes were selected in a similar manner. Figure 4.11 shows the configuration with two inverted-Y braces. It was proven effective in cases without earthquake amplification and when using W12x14 section as a shear link. This is shown in Figures 4.12. For cases with earthquake amplifications, a retrofitting scheme with three inverted-Y braces (Figure 4.13) and using W16x26 section for shear links is recommended. Figure 4.14 shows the associated damage states.

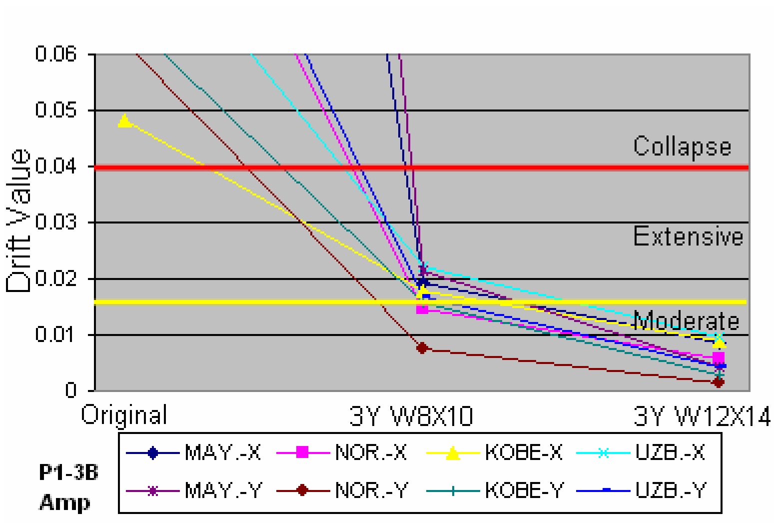


Figure 4.10 - Drift: P1-3B prototype with three inverted-Y braces – Amplified earthquakes

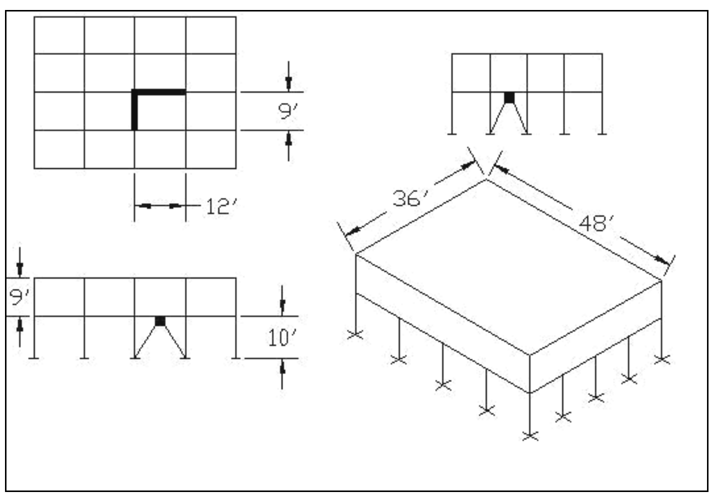


Figure 4.11 - P1-4B prototype with two inverted-Y braces

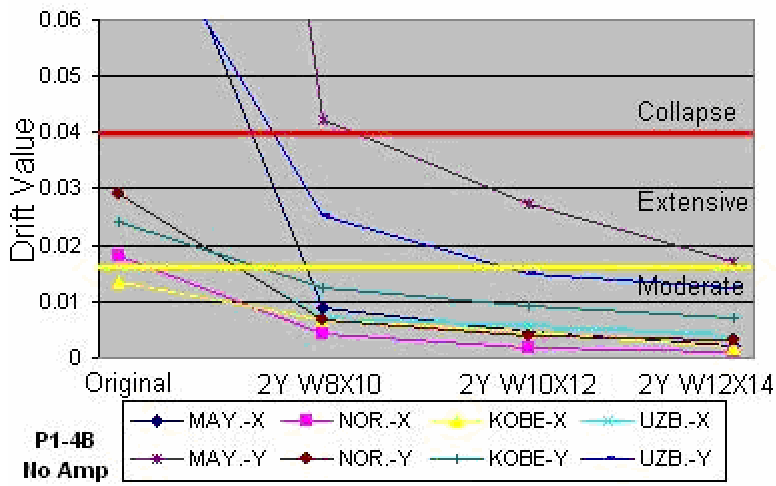


Figure 4.12 - Drift: P1-4B prototype with two inverted-Y braces – No amplification

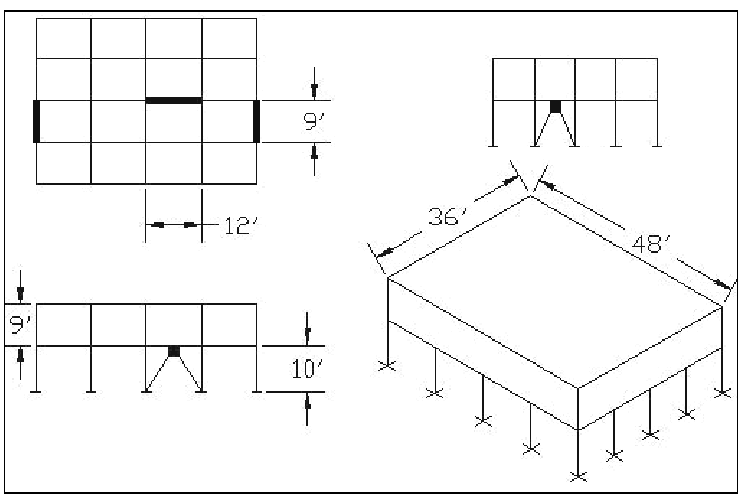


Figure 4.13 - P1-4B prototype with three inverted-Y braces

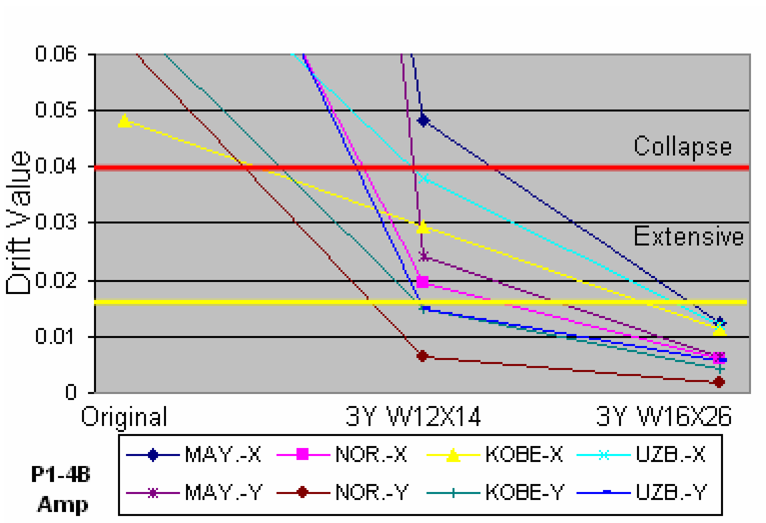


Figure 4.14 - Drift: P1-4B Prototype with Three Inverted-Y braces – Amplified earthquake

4.5 Prototype II Models

The prototype II Models have 15 ft long columns spanned 12.5 ft on center. For the first trial, P2-3B prototypes are tested with two inverted-Y braces placed in an L configuration at the center of the structure (Figure 4.15). It was proven effective in cases without earthquake amplifications and when using W10x12 section as a shear link. This is shown in Figure 4.16. For cases with earthquake amplifications, a retrofitting scheme with three inverted-Y braces (Figure 4.17) and using W12x14 section for shear links is recommended. Figure 4.18 shows the associated damage states.

The most effective retrofitting schemes for P2-4B prototypes were selected in a similar manner. Figure 4.19 shows the configuration with two inverted-Y braces. It was proven effective in cases without earthquake amplifications and when using W12x14 section as a shear link. This is shown in Figures 4.20. For cases with earthquake amplifications, a retrofitting scheme with three inverted-Y braces (Figure 4.21) and using W16x26 section for shear links is recommended. Figure 4.22 shows the associated damage states.

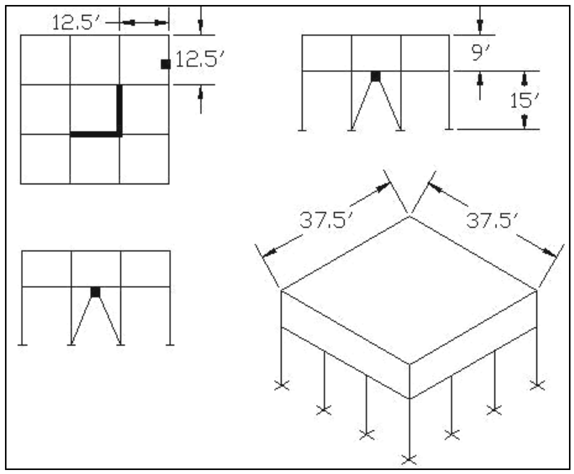


Figure 4.15 - P2-3B Prototype with two inverted-Y braces

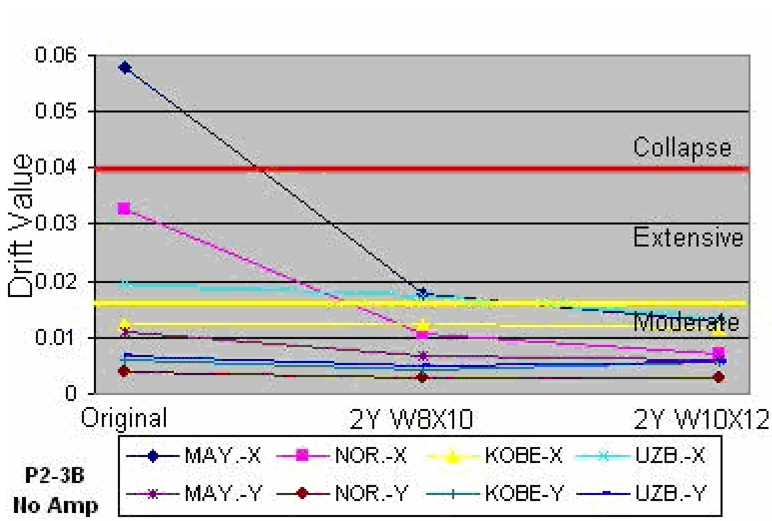


Figure 4.16 Drift: P2-3B prototype with two inverted-Y braces – No amplification

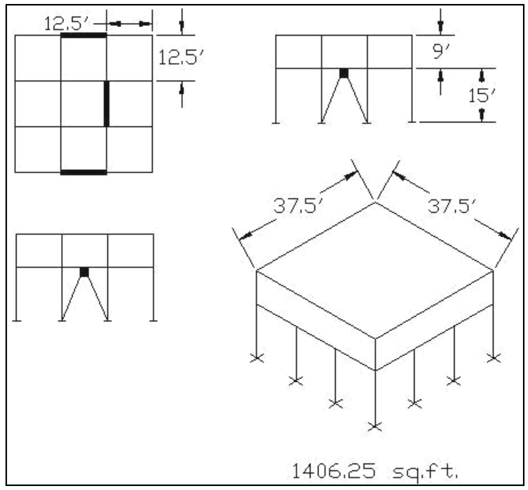


Figure 4.17 - P2-3B prototype with three inverted-Y braces

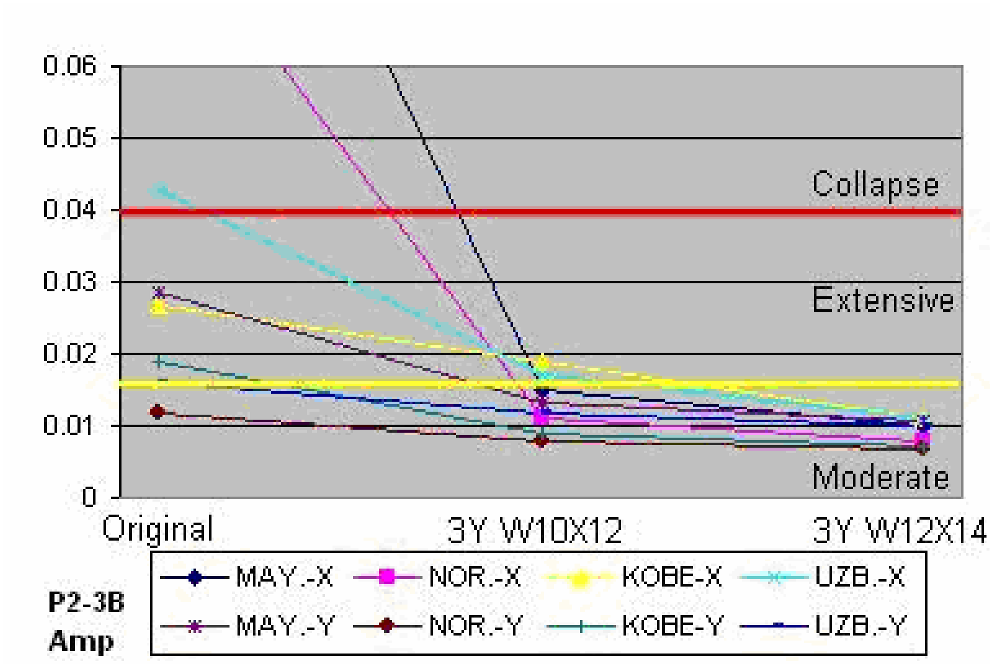


Figure 4.18 - Drift: P2-3B prototype with three inverted-Y braces – Amplified earthquakes

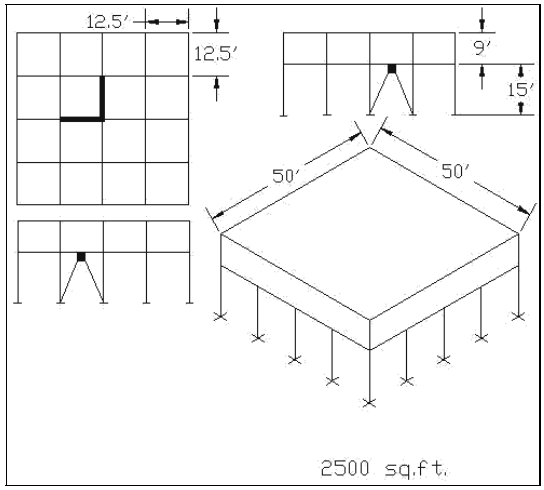


Figure 4.19 - P2-4B prototype with two inverted-Y braces

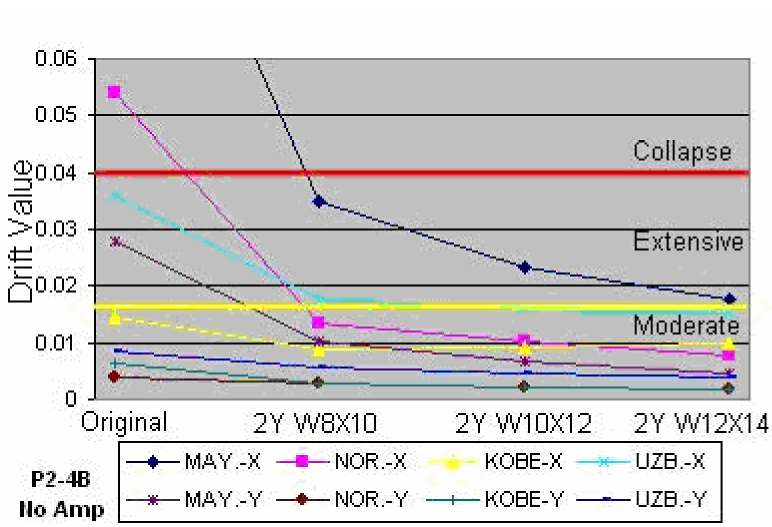


Figure 4.20 - Drift: P2-4B prototype with two inverted-Y braces – No amplification

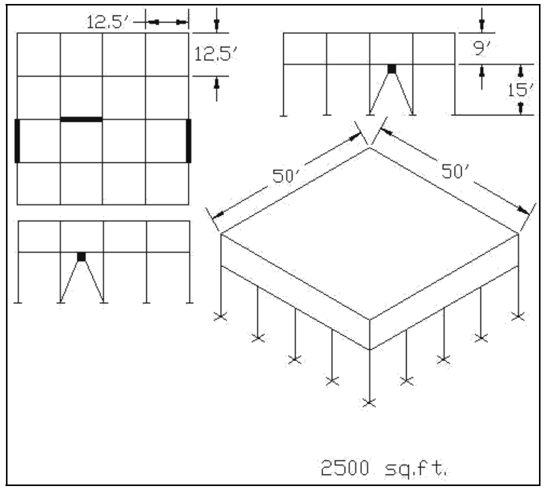


Figure 4.21 - P2-4B prototype with three inverted-Y braces

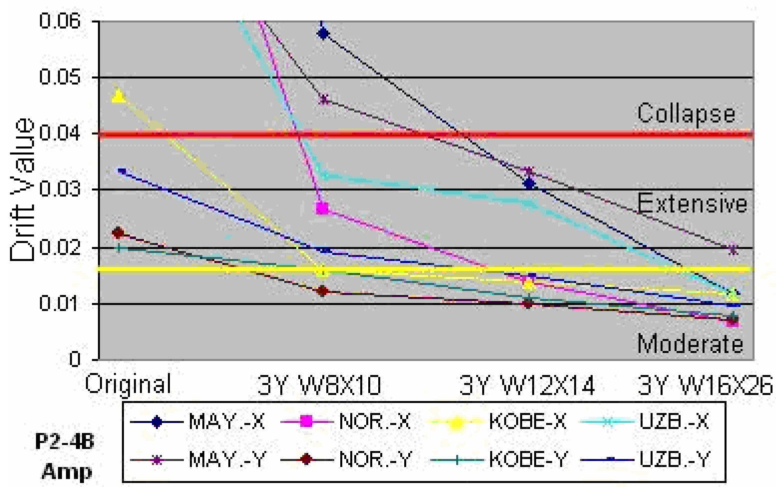


Figure 4.22 – Drift: P2-4B prototype with three inverted-Y braces – Amplified earthquakes

4.6 Prototype III Models

The prototype three Models have 16 ft long columns spanned 16 ft on center. For the first trial, P3-32B prototypes are tested with two inverted-Y braces placed in an L configuration shown in Figure 4.23. It was proven effective in all cases when using W10x12 section as a shear link. The corresponding damage states with and without earthquake amplifications are shown in Figures 4.24 and 4.25, respectively.

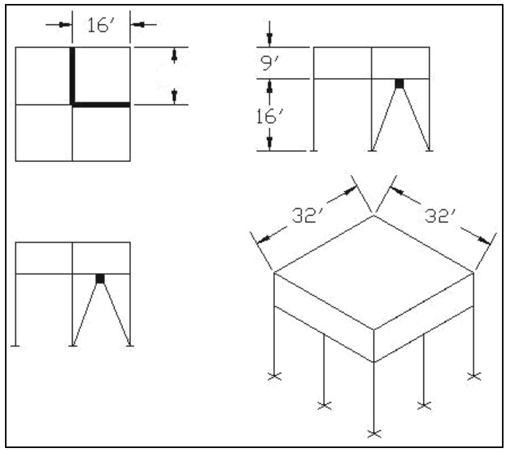


Figure 4.23 - P3-2B prototype with two inverted-Y braces

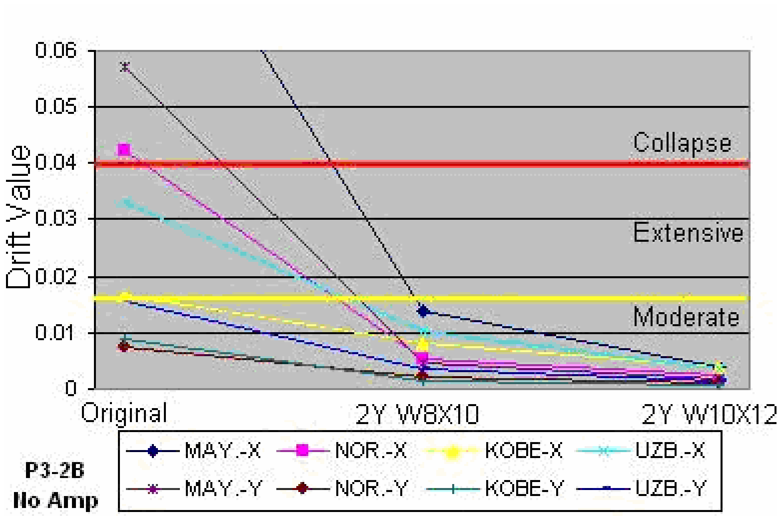


Figure 4.24 - Drift: P3-2B prototype with two inverted-Y braces – No amplification

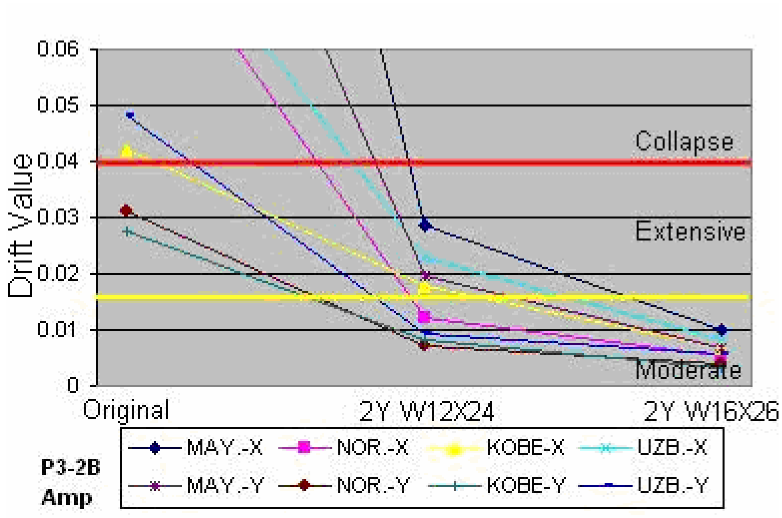


Figure 4.25 - Drift: P3-2B prototype with two inverted-Y braces – Amplified earthquakes

The most effective retrofitting schemes for P3-3B prototypes were selected in a similar manner. Figure 4.26 shows the configuration with three inverted-Y braces. It was proven effective in cases without earthquake amplifications and when using W10x12 section as a shear link. This is shown in Figures 4.27. For cases with earthquake amplifications, a retrofitting scheme with four inverted-Y braces (Figure 4.28) and using W16x26 section for shear links is recommended. Figure 4.29 shows the associated damage states

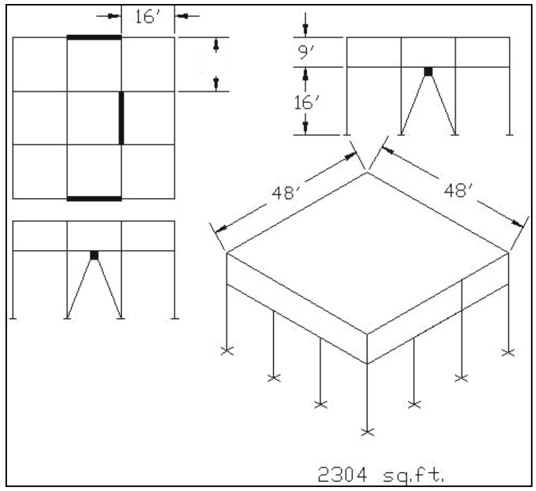


Figure 4.26 - P3-3B prototype with three inverted-Y braces

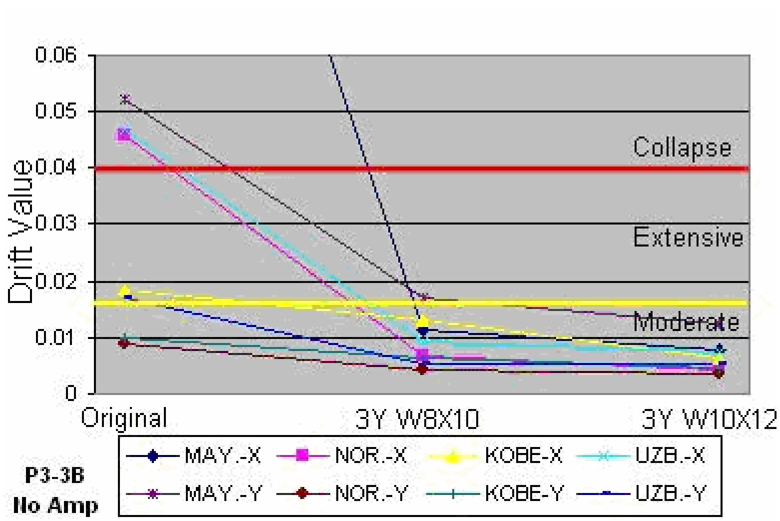


Figure 4.27 - Drift: P3-3B prototype with two inverted-Y braces – No amplification

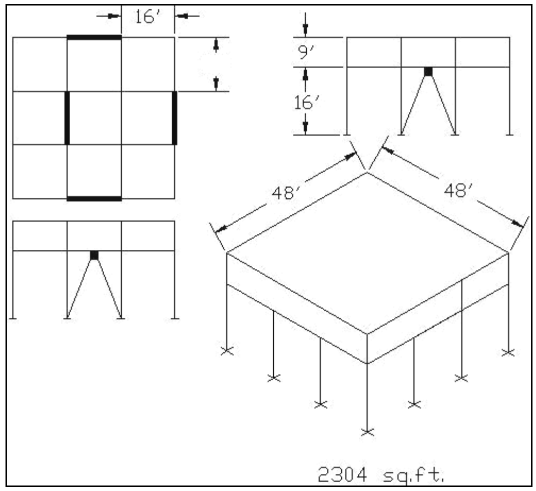


Figure 4.28 - P3-3B Prototype with Four inverted-Y braces

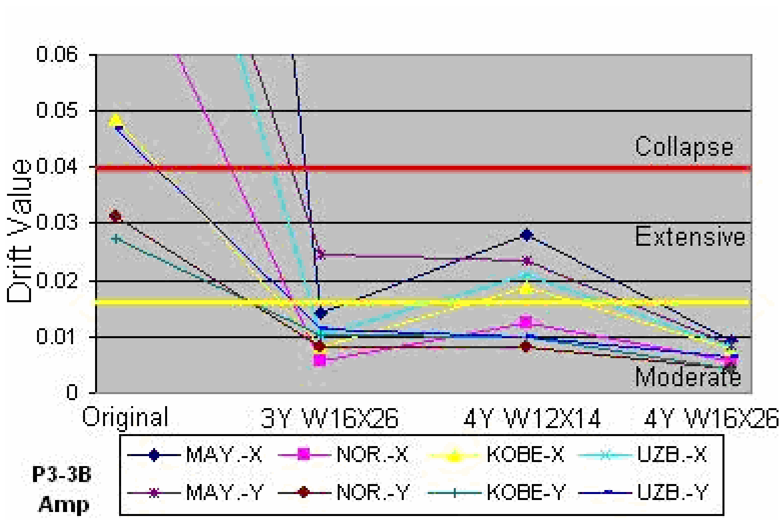


Figure 4.29 - Drift: P3-3B prototype with four inverted-Y braces – Amplified earthquakes

4.7 Retrofitting Recommendations

The structural recommendations given in Table 4.1 are based on the successful retrofitting schemes for prototype Models. The selection is influenced by geometrical properties like the column height, number of bays, and square footage. To use this table, first select the maximum number of bays followed by the area of the House. Then, depending on whether or not the earthquake is amplified, select the appropriate scheme. Listed are the number of Y-braces for each scheme and the cross section of the shear link. The recommended schemes are conservative.

The shear link assembly for each of the recommended design cases are built in accordance with the structural details provided in Figure 4.4. Table 4.2 presents a summary of the maximum shear demands, V_u, for each assembly based on our analysis of building prototypes. Included are the sizes for flange cover plates determined by substituting for V_u in Equations 4.2 and 4.3. Equation 4.1 is used to verify the choice of the 1/4-in fillet weld size in Figure 4.4. The fillet weld sizes for flange cover plates are based on the discussions in Section 4.4.

Table 4.1 - Retrofitting scheme recommendations

Number of Bays	Area (ft^2)	Scheme (No Amplifications)	Scheme (Amplified Earthquakes)
2	up to 1000	2Y (W8X10)	2Y (W16X26)
3	up to 1400	2Y (W10X12)	3Y (W12X14)
	1400-2300	3Y (W10X12)	4Y(W16X26)
4	up to 1700	2Y (W12X14)	3Y(W16X26)
	1700-2500	3Y (W10X12)	4Y(W16X26)

Table 4.2 – Shear link assembly data

Shear Link	Shear Force Demand (kips)	Shear Rupture strength of core member (kips)	Flange Cover Plate		Shear Rupture strength of assembly (kips)	Shear Capacity of ¼-in. fillet weld (kips)
			Dimensions (in)	Weld Size (in)		
W8X10	92	84.5	0.25 x 12 x 6	3/16	174.6	261.2
W10X12	122	101.4	0.25 x 12 x 6	3/16	191.4	285.3
W12X14 (No Amplifications)	148	119.6	0.25 x 12 x 6	3/16	209.6	315.1
W12X14 (Amplified Earthquakes)	134	119.6	0.25 x 12 x 6	3/16	209.6	315.1
W16X26	278	221.2	0.25 x 12 x 8	1/4	333.7	419.5

The bracing configurations must be placed as symmetrically as possible to prevent torsional loading. Some basic schemes were discussed in Sections 4.4 to 4.6. For the L-shaped configurations with two braces, the placement of the "L" should be close to the center of the House. If this is not possible, a three brace configuration must be utilized. The distance between the center of the House and the location of the Y-brace should not exceed 1/6 of the longitudinal dimensions of the House.

All diagonal braces are A36 2C6X13 sections except for W16x26 shear links where A36 2MC6x16.3 is used. Table 4.3 lists the maximum demands on this section for each of the predefined shear link assemblies. The number of bolts in each case is calculated assuming 1-in diameter A490-N sizes. Connection details are shown in Figure 4.3.

Table 4.3 – Diagonal brace data

Shear link	Cross section for diagonal braces (A36)	Axial demand (kips)	Number of 1-in diameter A490-N bolts	Shear capacity of the bolts (kips)
W8x10	2C6X13	103	4	283
W10x12	2C6X13	260.3	4	283
W12x14	2C6X13	206	4	283
W16x26	2MC6X16.3	344	6	424

4.8 Model House Calibrations

To test the validity of the design recommendations outlined in the previous section, the three Model Houses are retrofitted and analyzed accordingly. The retrofitting details are as follows.

- House Model 1:
  - Number of Bays: 3
  - Floor area: 841 ft^2
  - Recommendation for regular earthquakes: 2Y W10X12 (Figure 4.30)
  - Recommendation for amplified earthquakes: 3Y W12x14 (Figure 4.31)
- House Model 2:
  - Number of Bays: 4
  - Floor area: 2184 ft^2
  - Recommendation for regular earthquakes: 3Y W10X12 (Figure 4.32)

  - Recommendation for amplified earthquakes: 4Y W16x26 (Figure 4.33)
- House Model 3:
  - Number of Bays: 4
  - Floor area: 1248 ft^2
  - Recommendation for regular earthquakes: 2Y W12X14 (Figure 4.38)
  - Recommendation for amplified earthquakes: 3Y W16x26 (Figure 4.39)

Following the recommendations, the damage states for House Models 1 and 2 were improved to the desired levels. For example, when testing the L-shaped configuration for the House Model 1, the improvements were only adequate when subject to the earthquake loading without amplifications (Figure 4.36). As expected, the effects were not as good for amplified loading (Figure 4.37). Similar conclusions were drawn for House Model 2. The relevant data are presented in Figures 4.38 and 4.39.

House Model 3 suffered the least amount of damage when analyzed without any modifications. Utilizing the scheme consisting of two sets of braces recommended for earthquakes without the amplification factor applied the damage was effectively reduced to a moderate level (Figure 4.40). When the Model was analyzed utilizing the amplified earthquake, it was noted that the configuration of two braces was also effective in this case. Even though the House has four spans, its square footage is relatively small and the configuration of two braces is located very close to the center of mass of the structure, making this configuration effective in both cases (Figure 4.41).

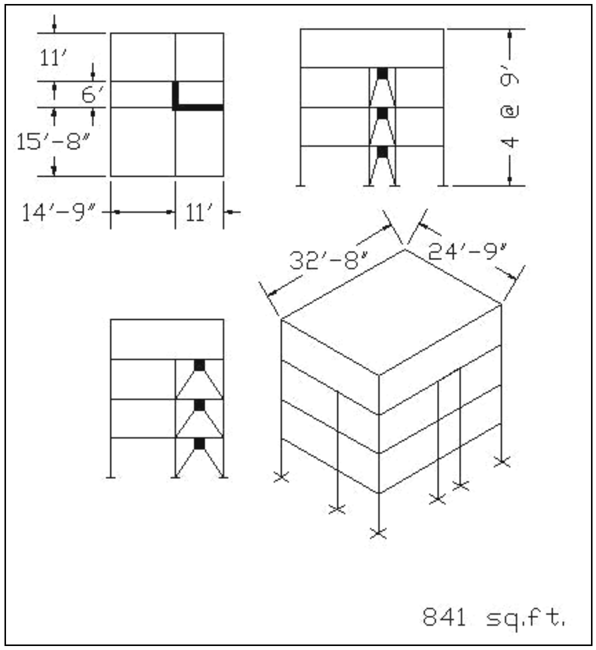


Figure 4.30 - House Model 1 retrofitted for earthquakes without amplification

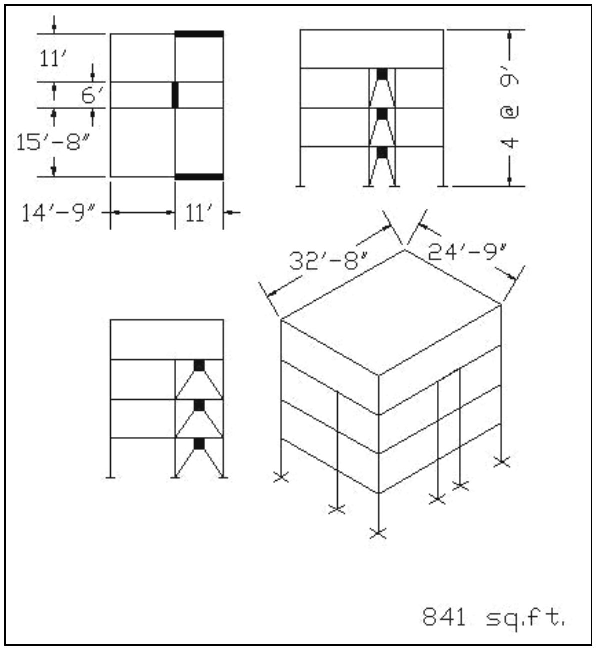


Figure 4.31 - House Model 1 retrofitted for amplified earthquakes

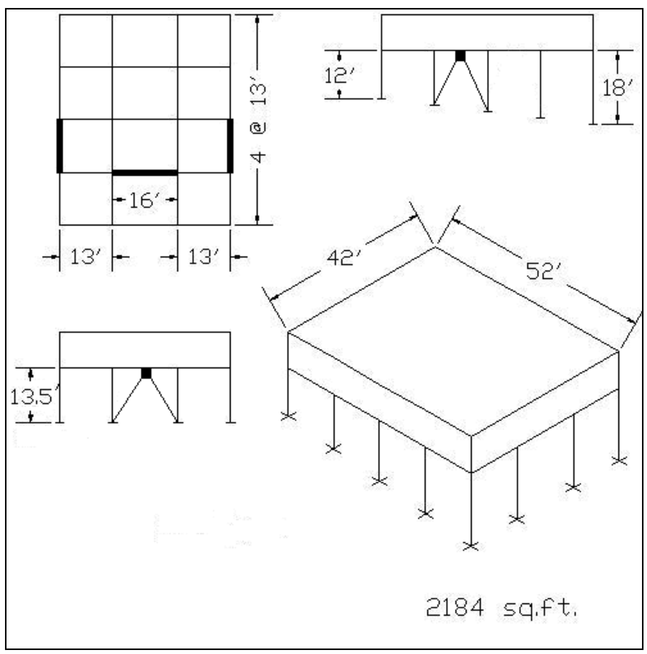


Figure 4.32 - House Model 2 retrofitted for earthquakes without amplification

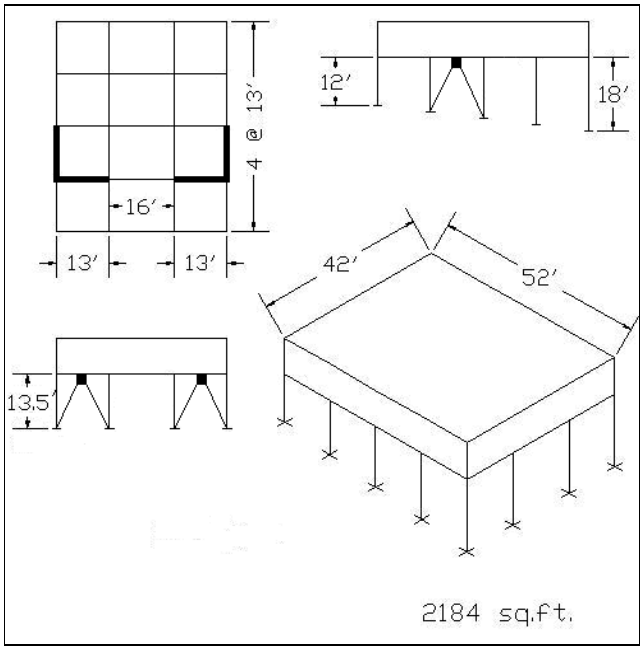


Figure 4.33 - House Model 2 retrofitted for amplified earthquakes

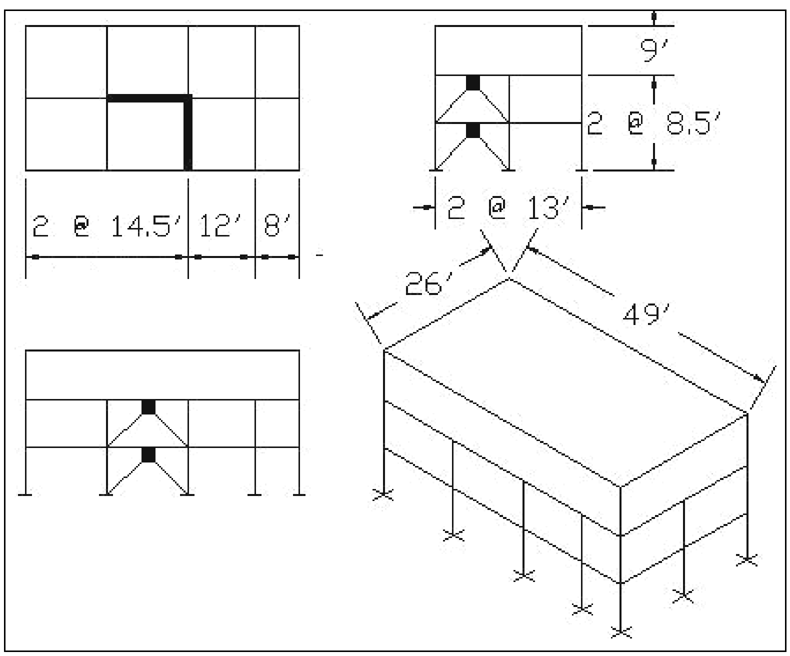


Figure 4.34 - House Model 3 retrofitted for earthquakes without amplification

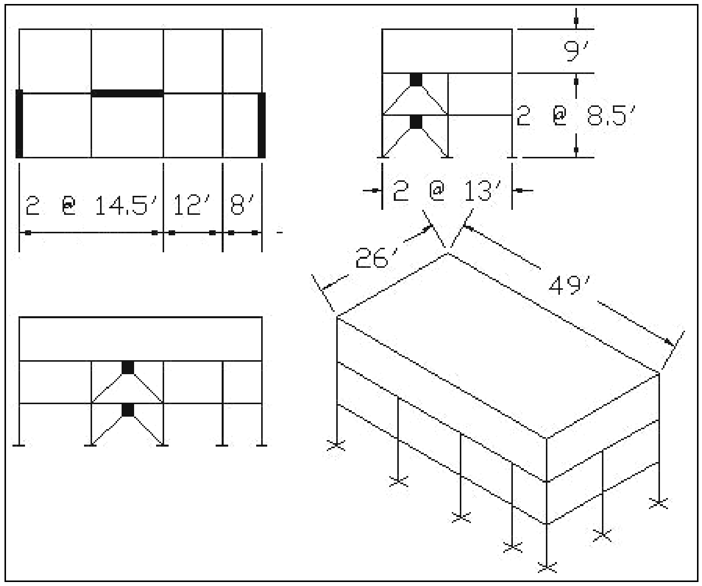


Figure 4.35 - House Model 3 retrofitted for amplified earthquakes

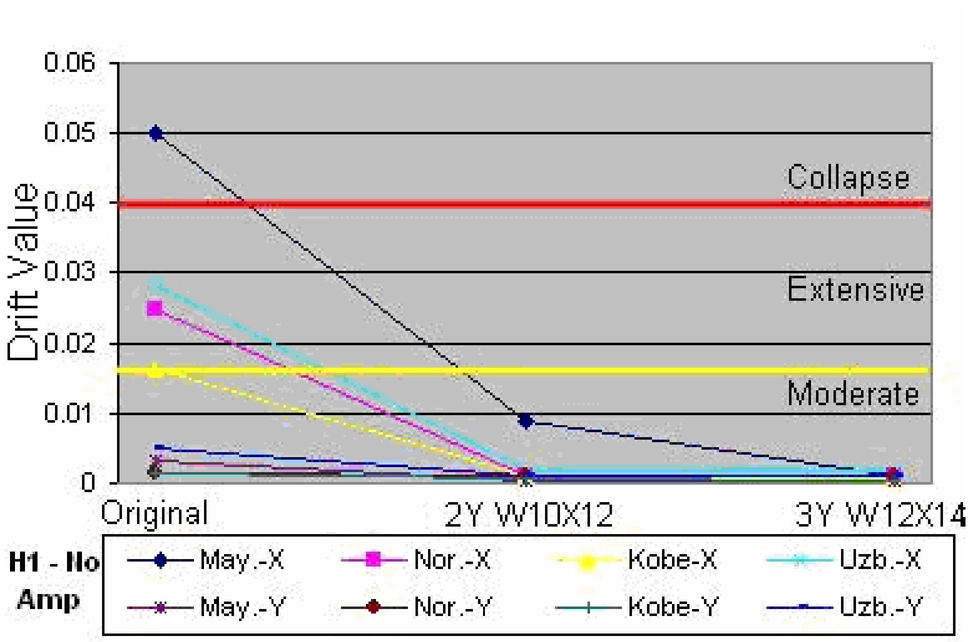


Figure 4.36 - Drift: House Model 1 – No amplifications

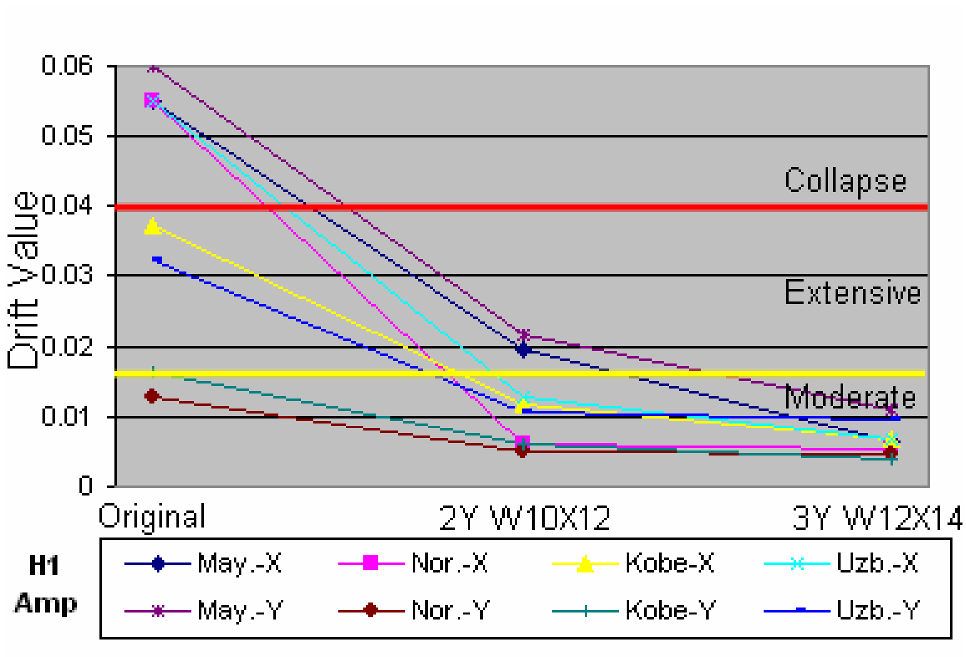


Figure 4.37 - Drift: House Model 1 - Amplified earthquakes

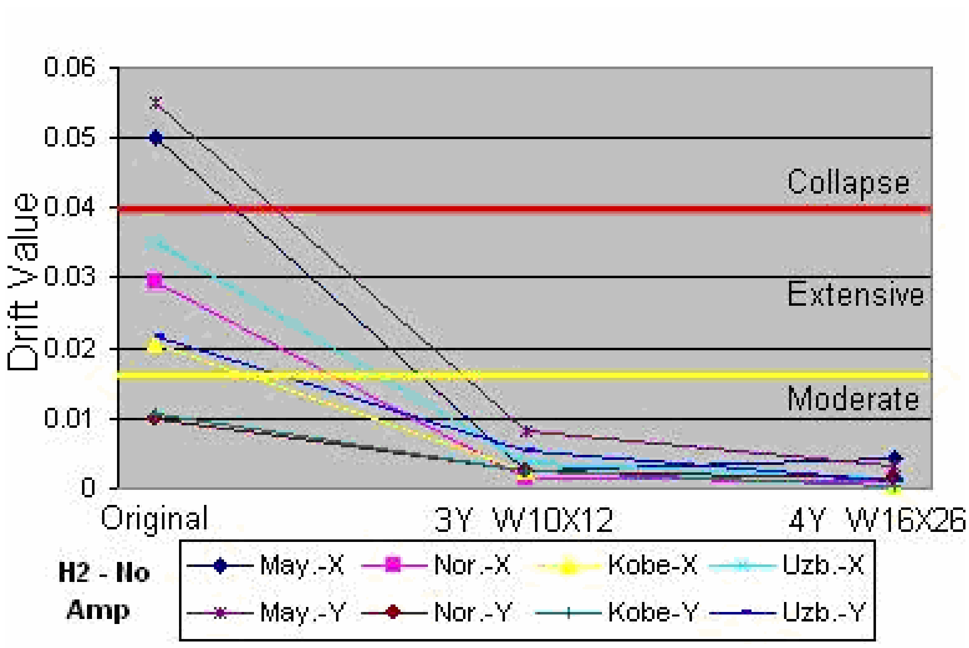


Figure 4.38 - Drift: House Model 2 – No amplifications

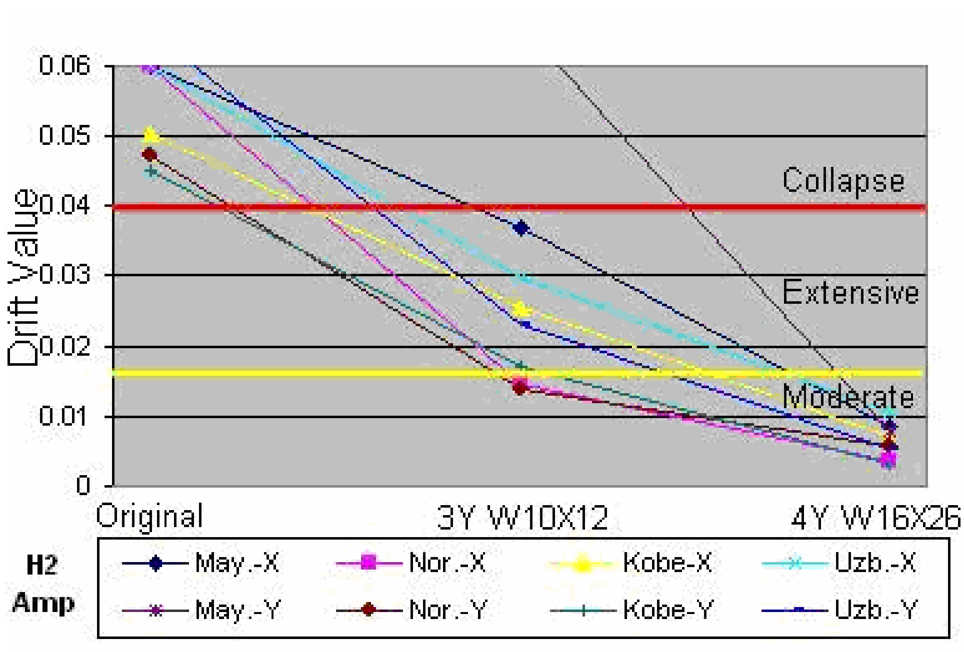


Figure 4.39 - Drift: House Model 2 - Amplified earthquakes

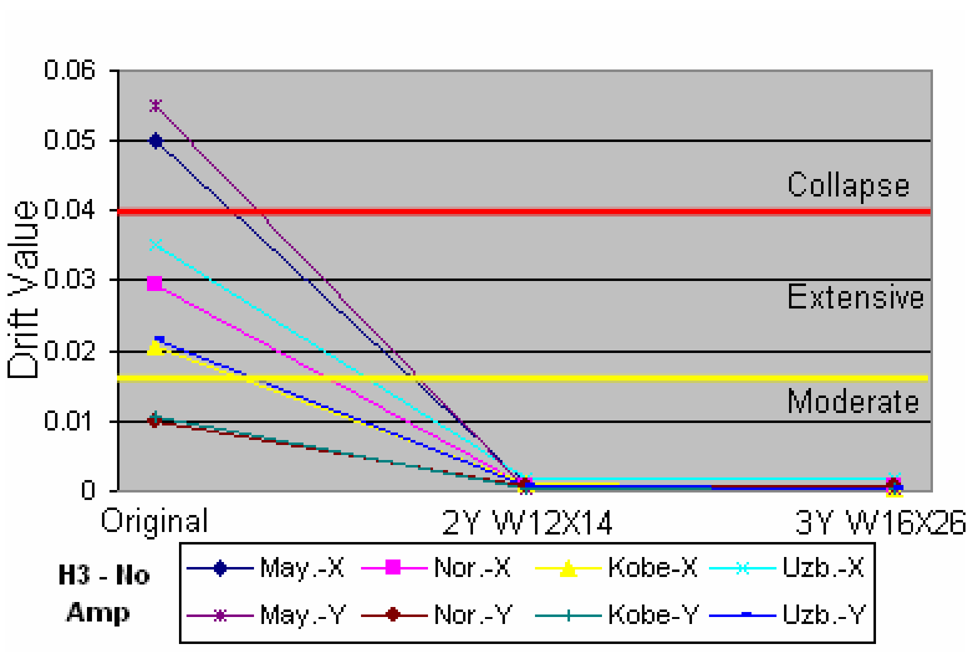


Figure 4.40 - Drift: House Model 3 – No amplifications

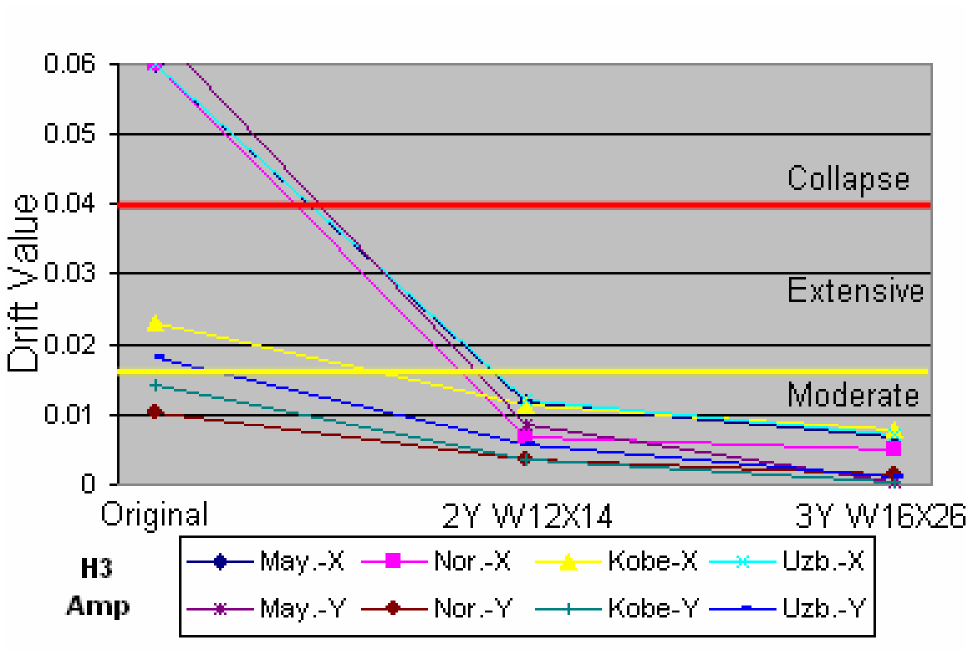


Figure 4.41 - Drift: House Model 3 - Amplified earthquakes

5 Use of Shear Walls

5.1 Introduction

In Puerto Rico, deficient reinforced concrete frame and shear wall structures are often retrofitted using concrete shear walls. It is therefore essential to evaluate their cost effectiveness in comparison with the proposed inverted-Y bracing scheme. The basic methodology discussed in the previous Chapter is followed. Both the thickness and the coverage area for the shear walls are varied. The configuration for which the damage state is reduced to moderate and better is marked as effective. To compare with the overall performance of the inverted-Y steel bracing systems, a set of recommendations on the use of shear walls is presented. Model Houses are analyzed to verify these recommendations.

5.2 Prototype I Model

An L-shaped configuration of two shear walls, 4 inches thick and with usual assortment of reinforcement bars, was tested for P1-3B prototypes. The configuration as shown in Figure 5.1 is identical to the one used when retrofitting with inverted-Y braces. The resulting damage states without earthquake amplifications are shown in Figure 5.2. The configuration is not effective if earthquakes are amplified even when the wall thickness is increased. To consider such a case, the three wall configuration of Figure 5.3 is used. Figure 5.4 shows the corresponding damage states. The retrofitting scheme of using two 4 inches thick shear walls in an L-shape configuration (Figure 5.5) was also effective for P1-4B prototypes with and without the amplification factor (Figure 5.6).

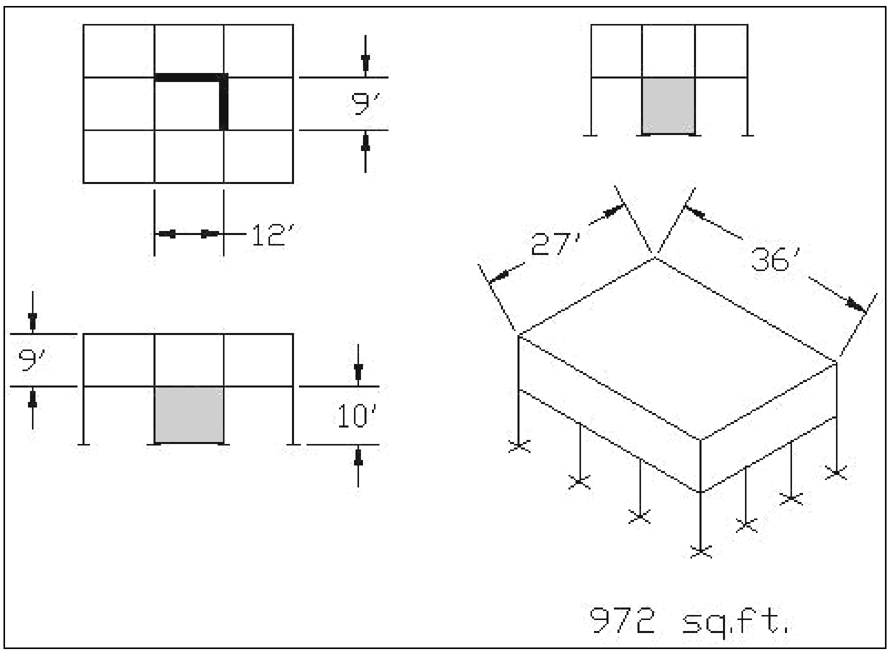


Figure 5.1 - P1-3B prototype with two shear walls

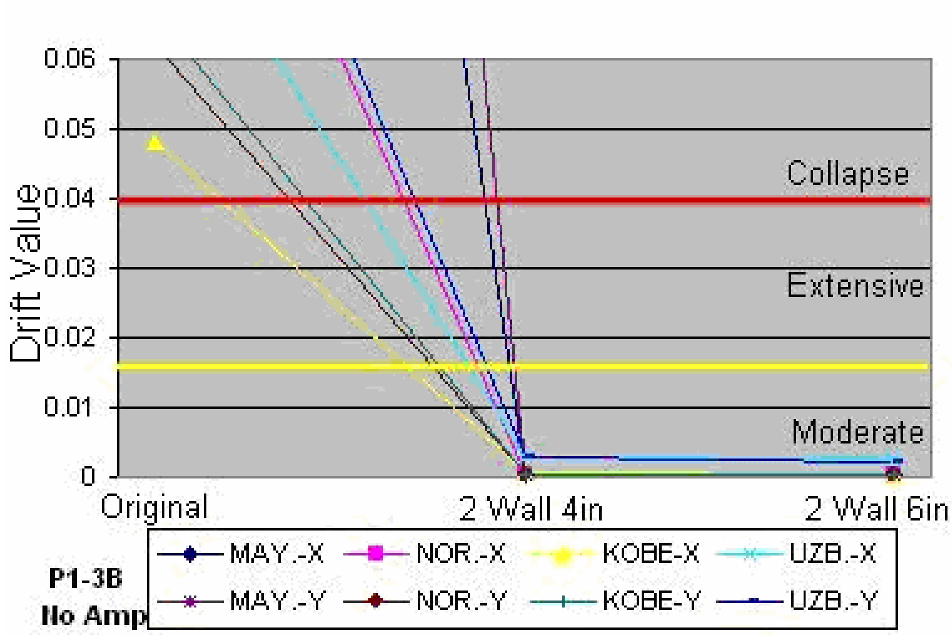


Figure 5.2 - Drift- P1-3B prototype with two shear walls - No Amplification

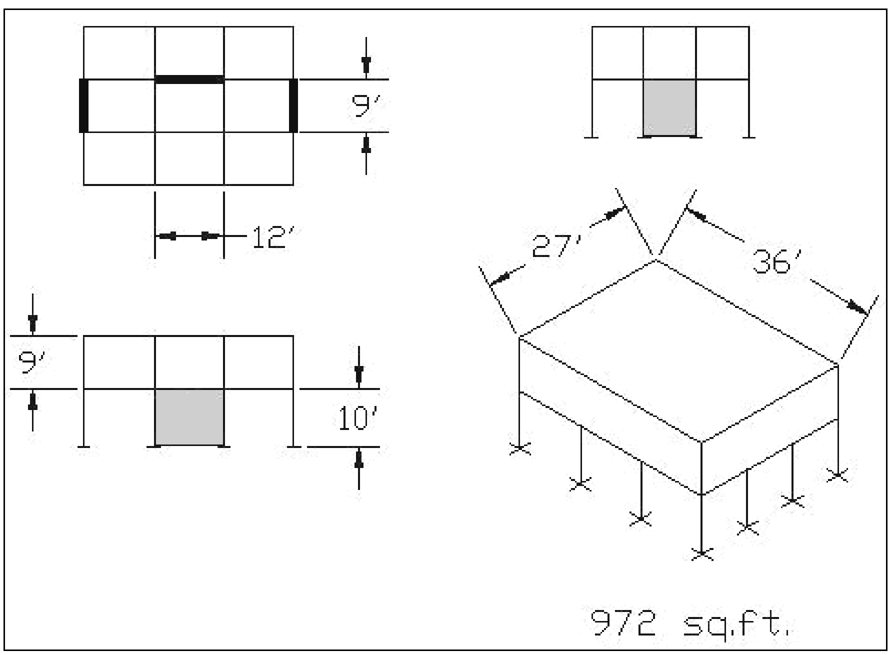


Figure 5.3 - P1-3B prototype with two shear walls

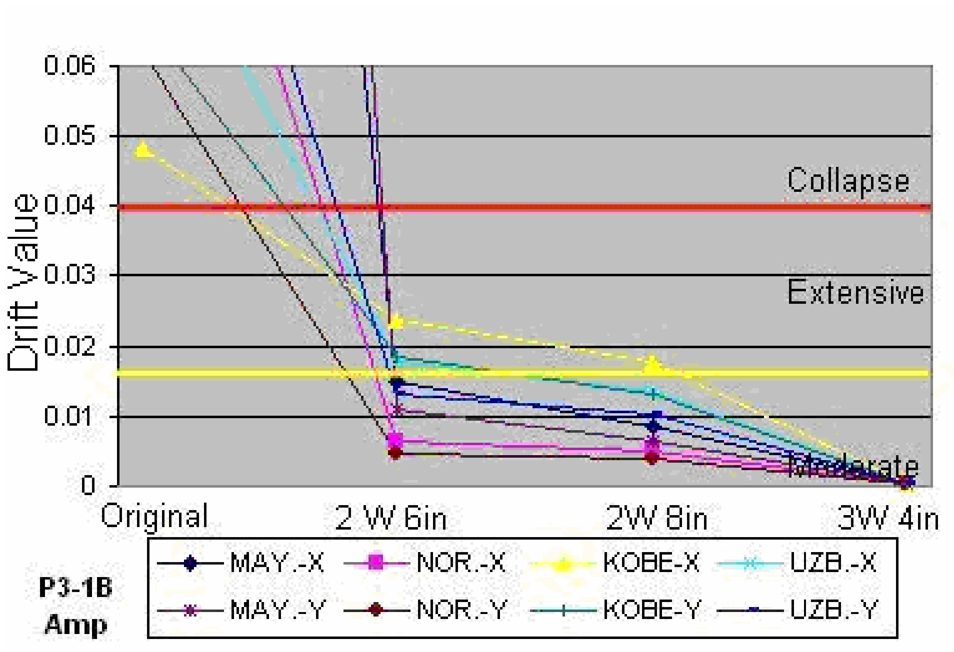


Figure 5.4 - Drift- P1-3B prototype with three shear walls – Amplified earthquakes

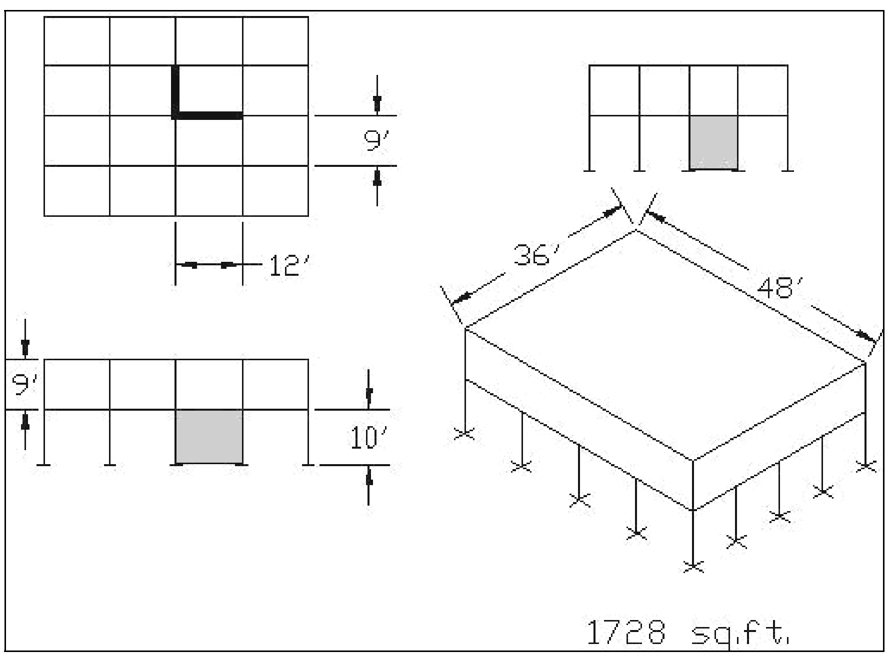


Figure 5.5 - P1-4B prototype with two shear walls

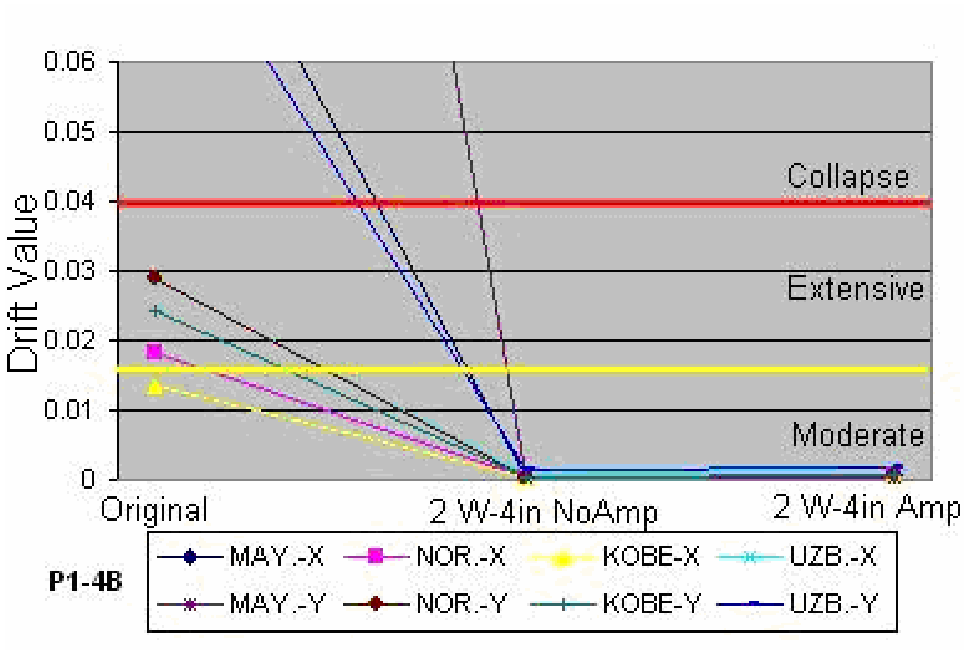


Figure 5.6 – Drift: P1-4B prototype with two Shear Walls

5.3 Prototype II Models

When retrofitting prototype II Models, L-shaped configurations using two shear walls were proven effective in most cases. Those configurations are shown in Figures 5.7 and 5.8. For P2-3B prototypes, the use of 4 inches thick shear walls provided the solution in all cases with and without earthquake amplifications (Figures 5.9 and 5.10). For P2-4B prototypes, 6-inches thick shear walls will work if earthquakes are not amplified. The three shear wall configuration in Figure 5.11 was proven effective for all P2-4B prototypes with and without earthquake amplifications and for wall thicknesses of only 4 inches. The corresponding damage states are shown in Figure 5.12.

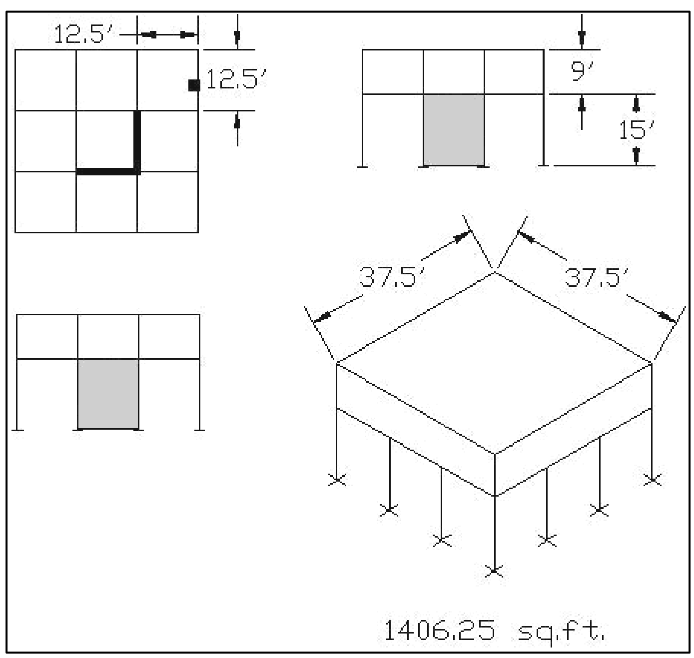


Figure 5.7 – P2-3B prototype with two shear walls

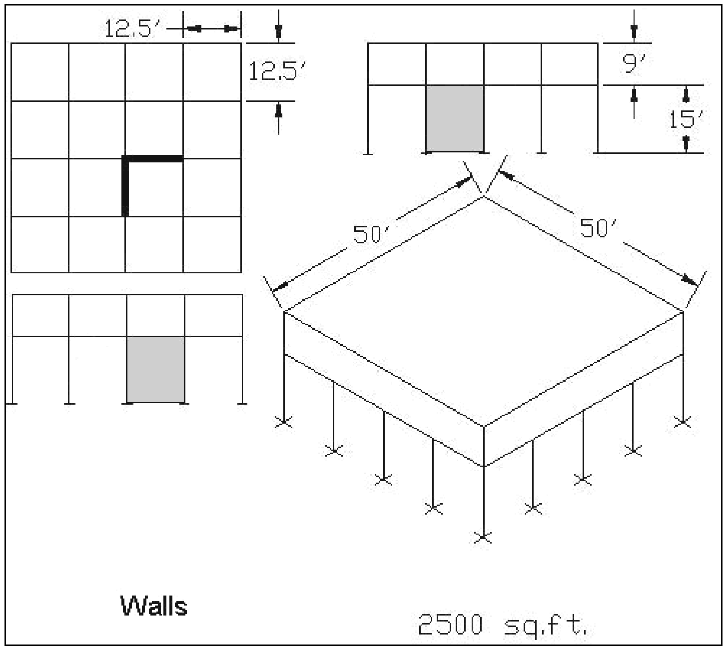


Figure 5.8 – P2-4B prototype with two shear walls

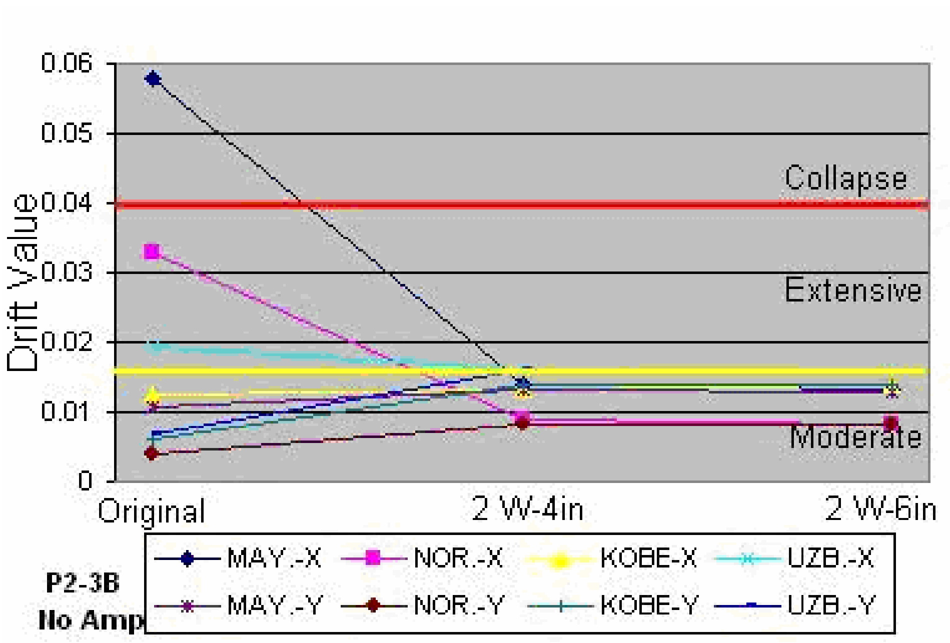


Figure 5.9 – Drift: P2-3B prototype with two Shear Walls – No amplifications

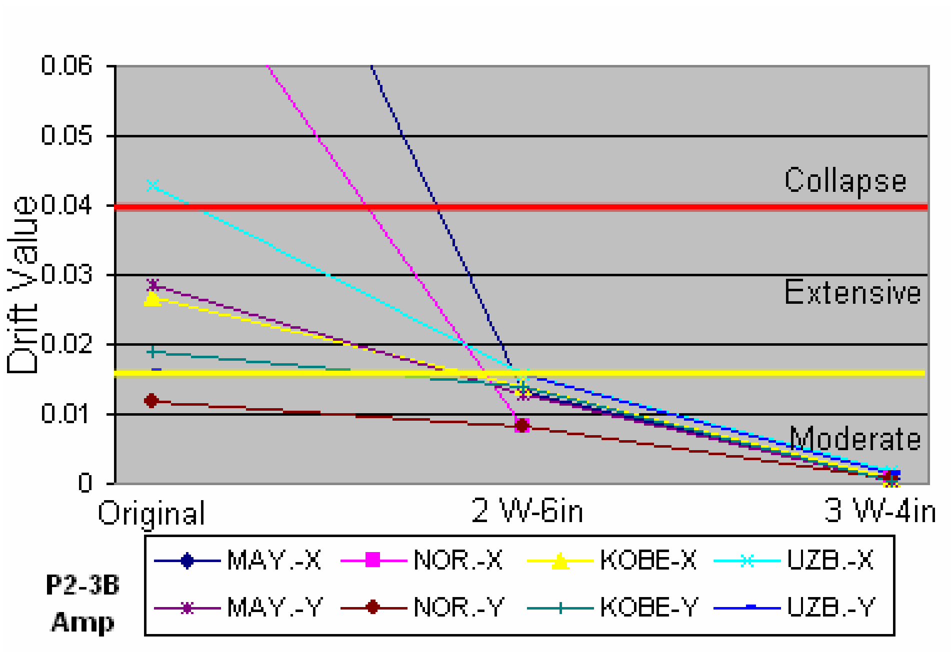


Figure 5.10 – Drift: P2-3B prototype with two Shear Walls – Amplified earthquakes

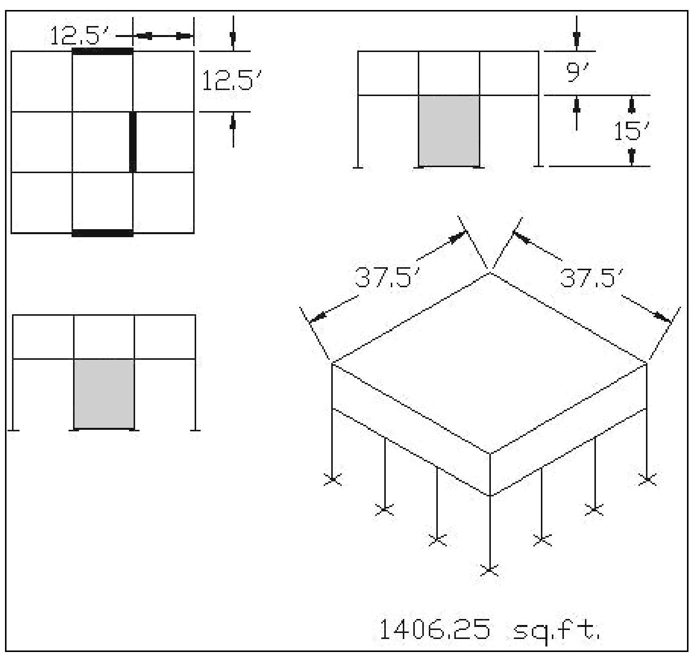


Figure 5.11 – P2-4B prototype with three shear walls

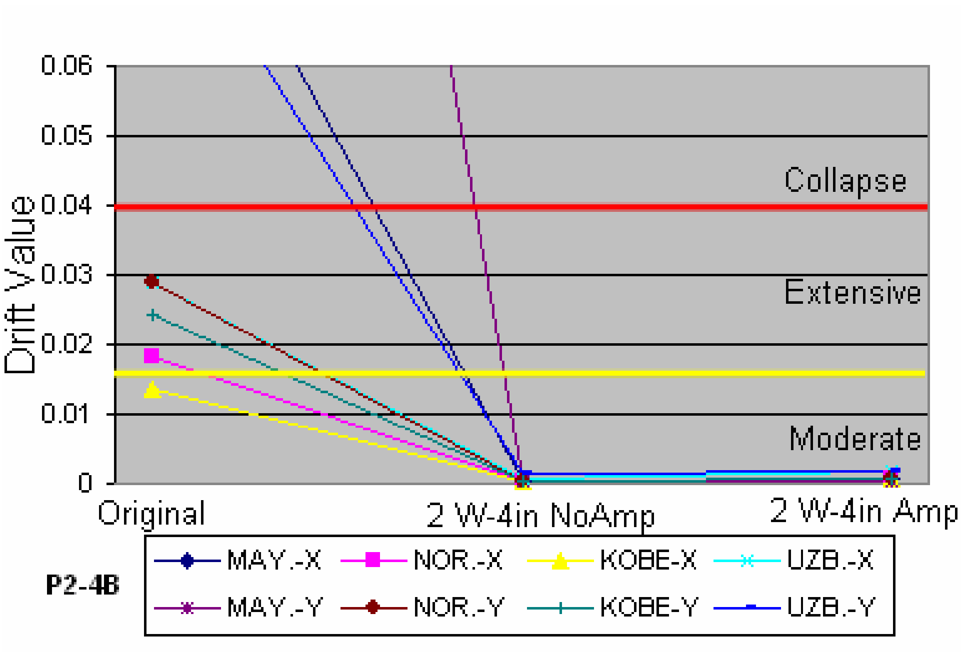


Figure 5.12 – Drift: P2-3B prototype with three Shear Walls

5.4 Prototype III Models

When retrofitting prototype III Models, L-shaped configurations using two shear walls were proven effective in all cases where earthquakes are not amplified. The configurations and the corresponding damage states are shown in Figures 5.13 and 5.14 for P3-2B prototypes and in Figures 5.15 and 5.16 for P3-3B prototypes. When earthquakes are amplified, the L-shaped configuration will still work for P3-2B prototypes (Figure 5.14). However, the three shear wall configuration in Figure 5.17 is required for P3-3B prototypes. The improvements are depicted in Figure 5.18.

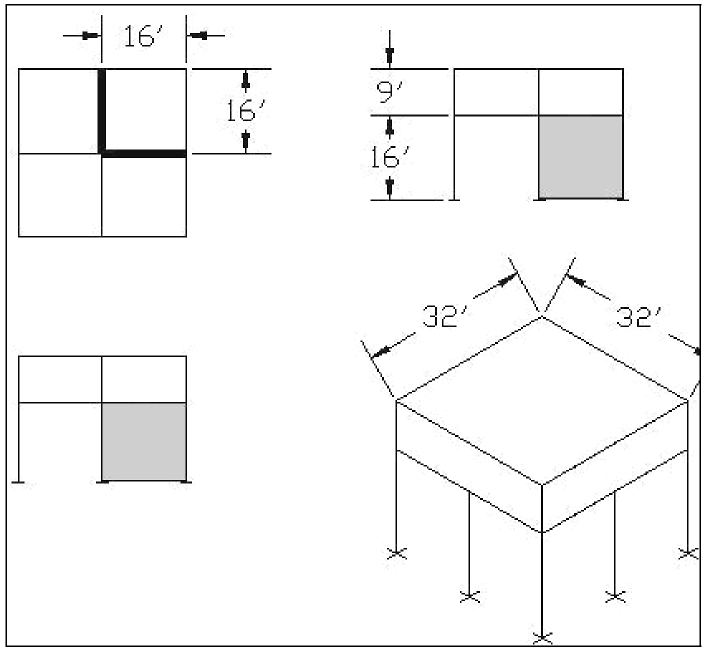


Figure 5.13 – P3-2B prototype with two shear walls

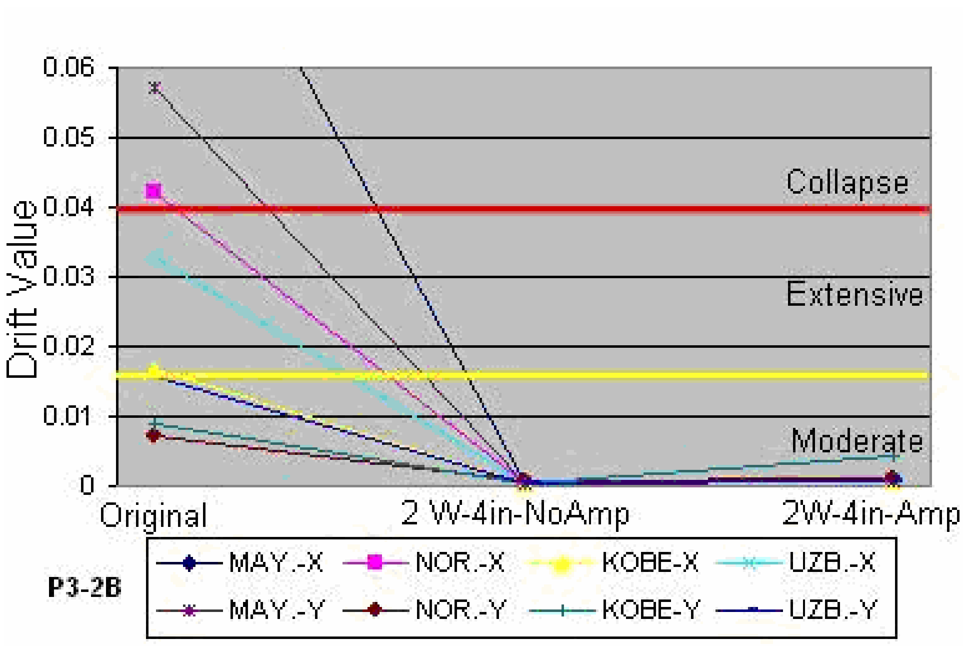


Figure 5.14 – Drift: P3-2B prototype with two shear walls

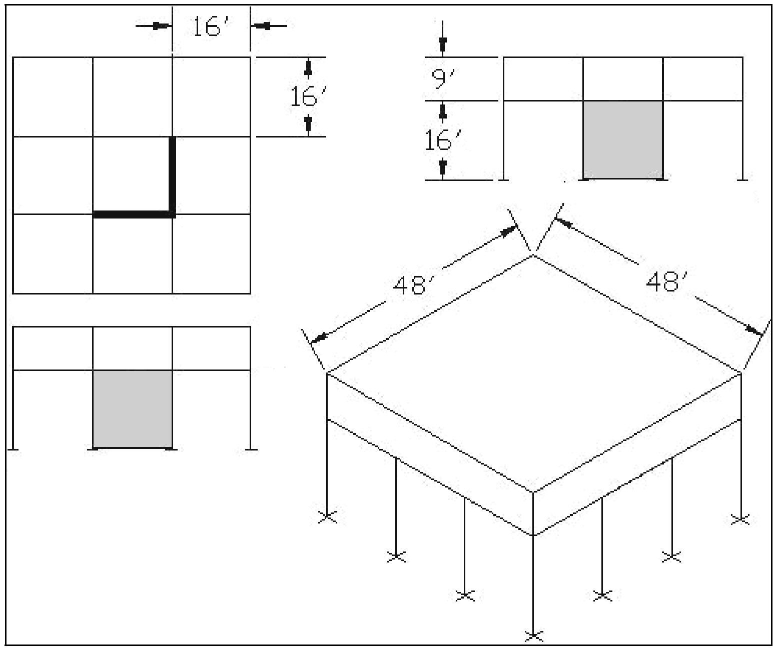


Figure 5.15 - P3-3B prototype with two shear walls

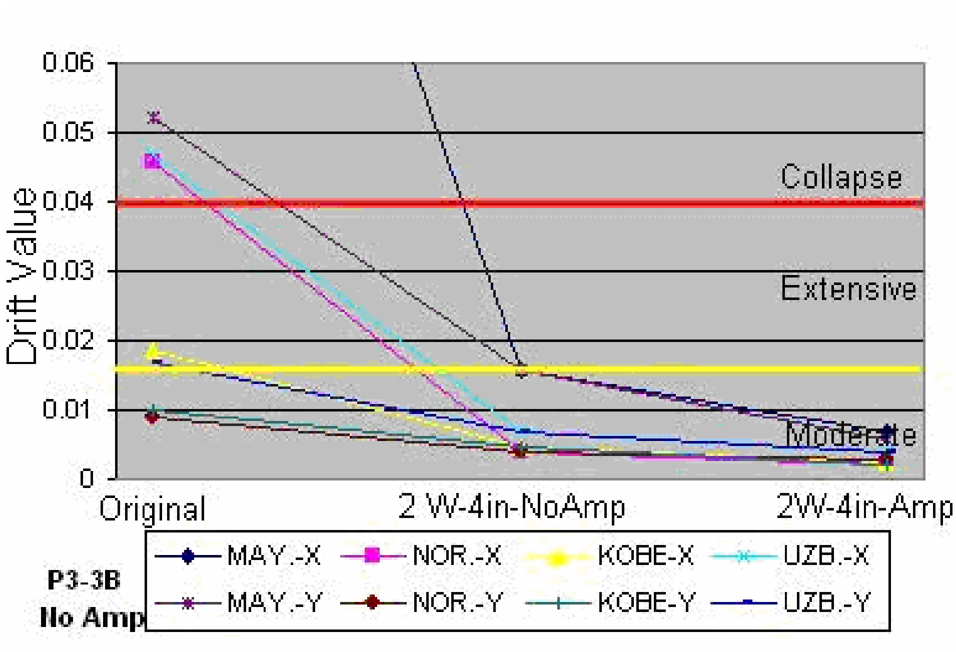


Figure 5.16 - Drift: P3-3B prototype with two shear walls – No amplification

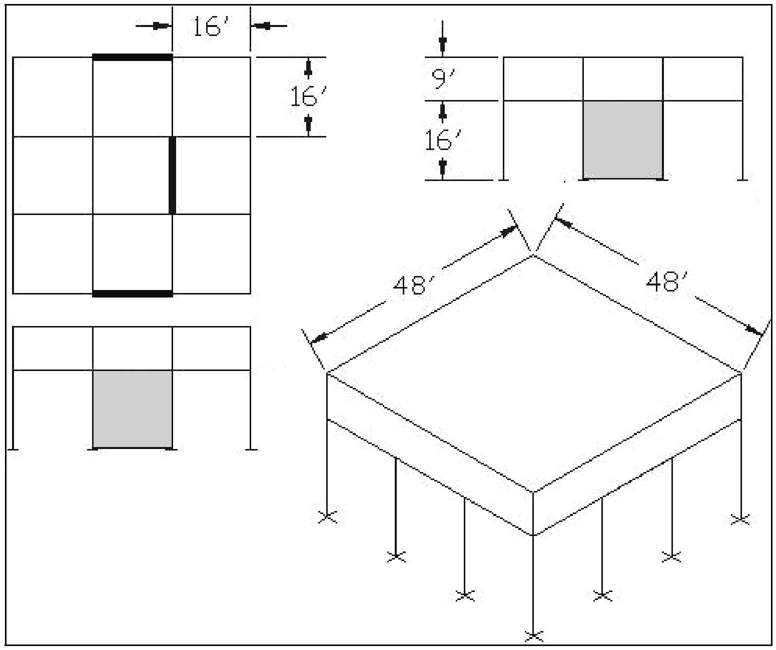


Figure 5.17 - P3-3B prototype with three shear walls

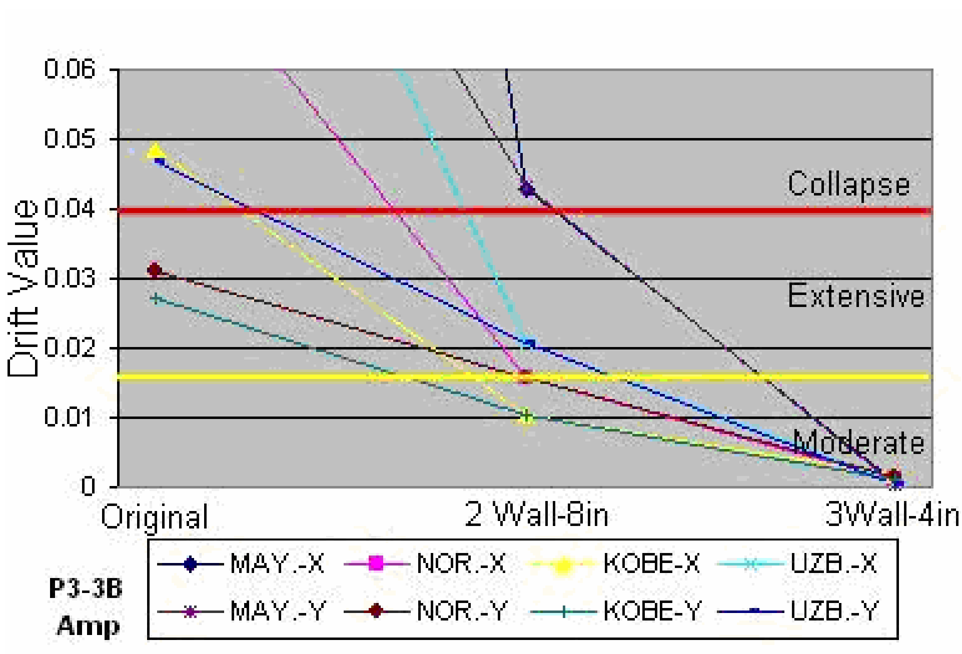


Figure 5.18 - Drift: P3-3B prototype with three shear walls – Amplified Earthquakes

5.5 Model House Calibrations

A summary of retrofitting recommendations for prototype Models are listed in Table 5.1. These recommendations as they apply to Model Houses are as follows:

- House Model 1 (Figure 5.19):
  - Number of bays: 3
  - Floor area: 841 ft^2
  - Recommendation for regular earthquakes: 2 Walls 4 in thick
  - Recommendation for amplified earthquakes: 3 Walls 6 in thick
- House Model 2 (Figure 5.20):
  - Number of bays: 4
  - Floor area: 2184 ft^2
  - Recommendation for regular earthquakes: 2 Walls 4 in thick
  - Recommendation for amplified earthquakes: 2 Walls 4 in thick
- House Model 3 (Figure 5.21):
  - Number of bays: 4
  - Floor area: 1248 ft^2
  - Recommendation for regular earthquakes: 2 Walls 4 in thick
  - Recommendation for amplified earthquakes: 2 Walls 4 in thick

The selected configurations were effective for House Models 1 and 3. The structures suffered only slight and moderate damages as shown in Figures 5.22 and 5.23. House Model 2 is built on a slope and has columns of different lengths. This placed restrictions on where the two shear walls may be located. The configuration was not effective in preventing planar rotation. Without amplification (Figure 5.24) the structure suffered moderate damages. However, the three shear wall configurations in Figure 5.25 were utilized in order to improve the performance when the loading is amplified. The end results are shown in Figure 5.26.

Table 5.1- Retrofitting Scheme Recommendations for Shear Walls

Number of Bays	Area (ft^2)	Scheme No Amplifications	Scheme Amplified Earthquakes
2	up to 1000	2 Walls 4in	2 Walls 4in
3	up to2300	2 Walls 4in	3 Walls 6in
4	up to 2500	2 Walls 4in	2 Walls 4in

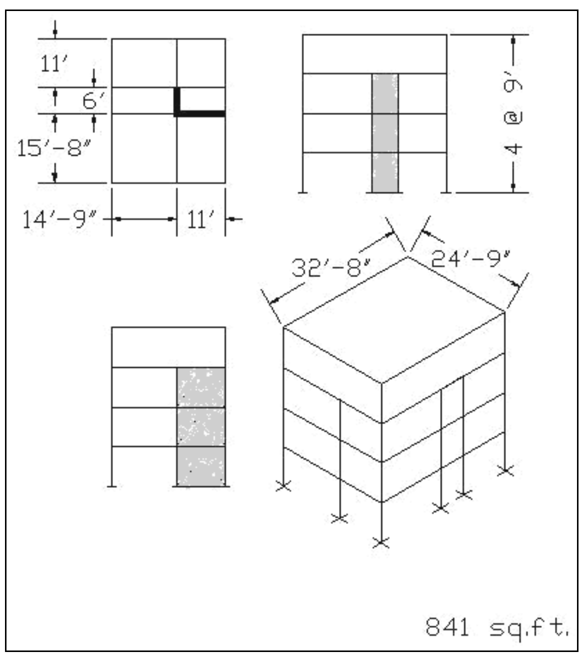


Figure 5.19 – House Model 1 retrofitted with two shear walls

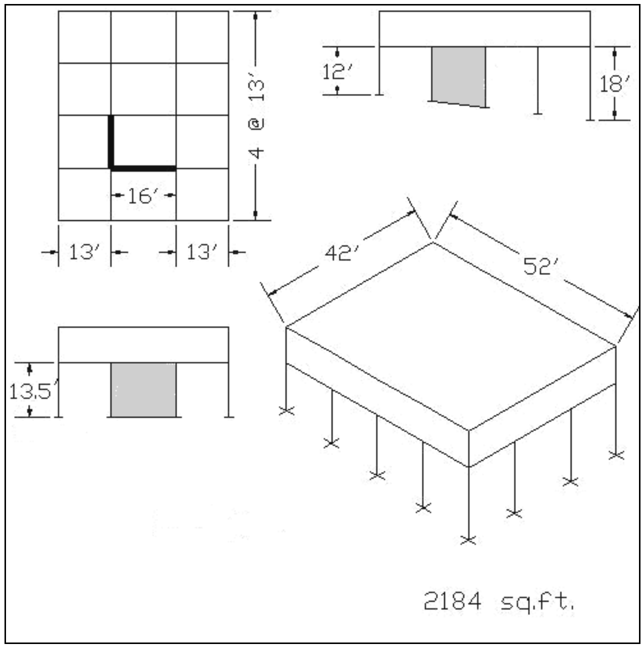


Figure 5.20 - House Model 2 retrofitted with two shear walls

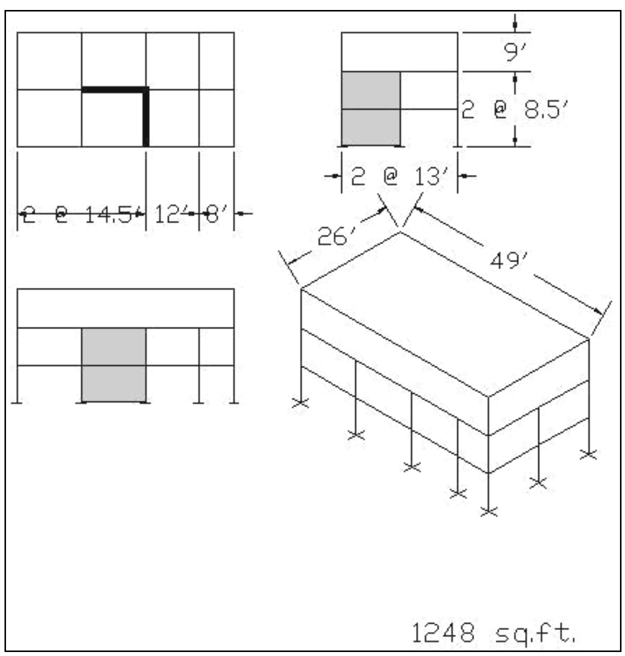


Figure 5.21 - House Model 3 retrofitted with two shear walls

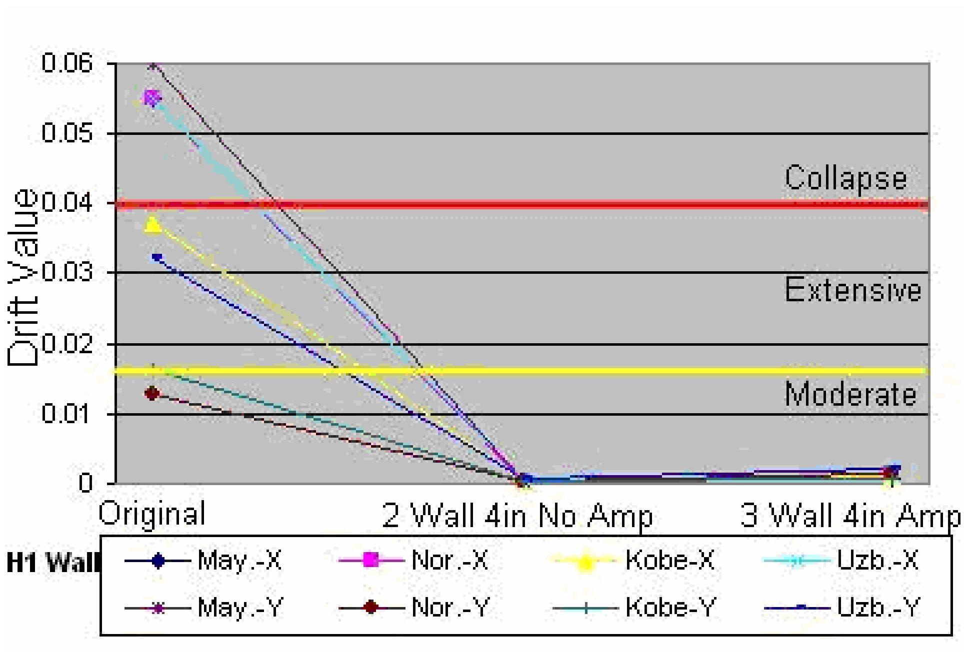


Figure 5.22 – Drift: House Model 1 retrofitted with two shear walls

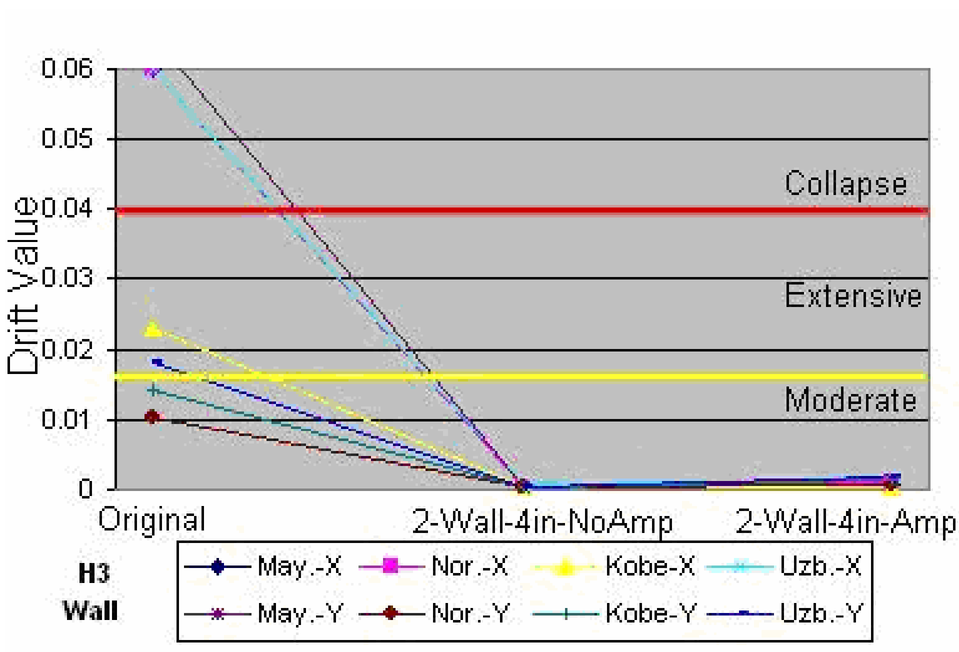


Figure 5.23 - Drift: House Model 3 retrofitted with two shear walls

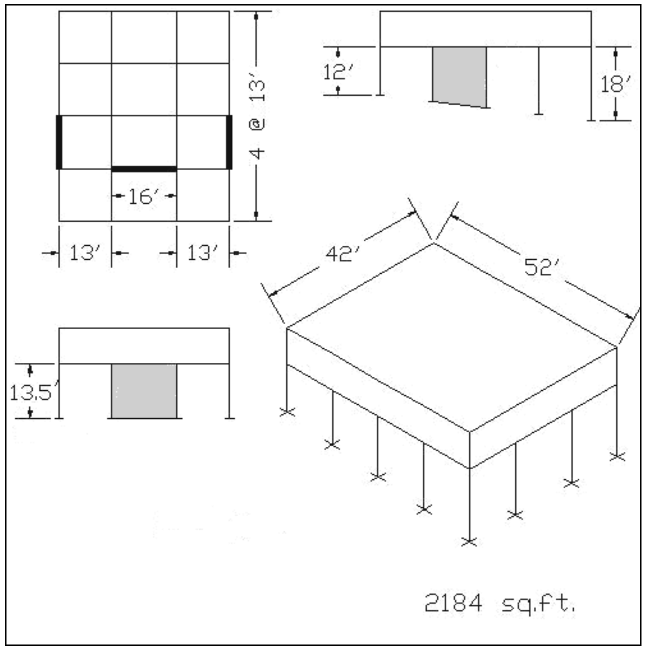


Figure 5.24 - House Model 2 retrofitted with three shear walls

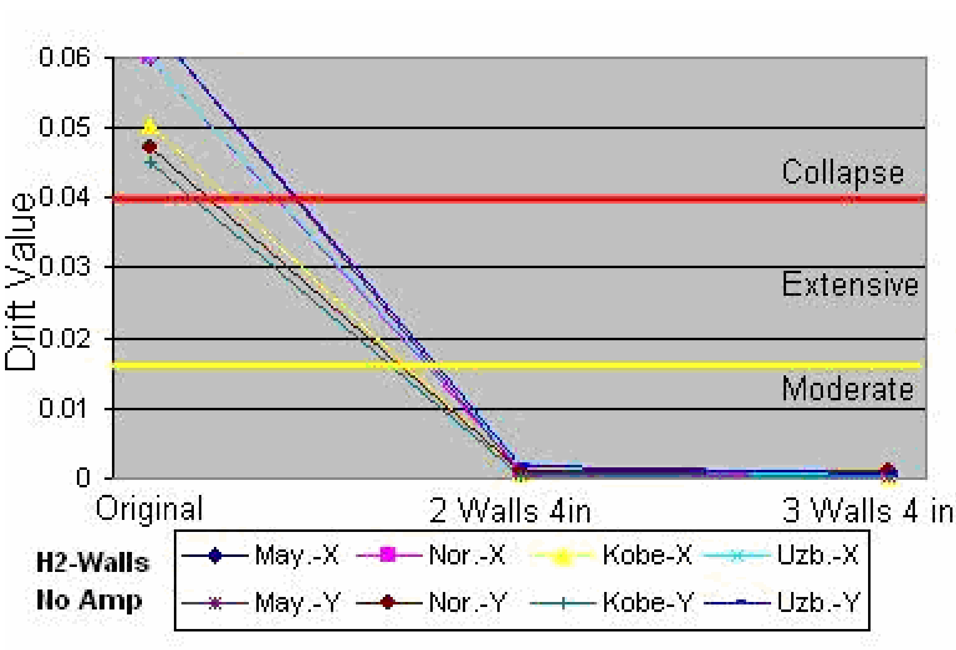


Figure 5.25 - Drift: House Model 2 retrofitted with two and three shear walls – No amplifications

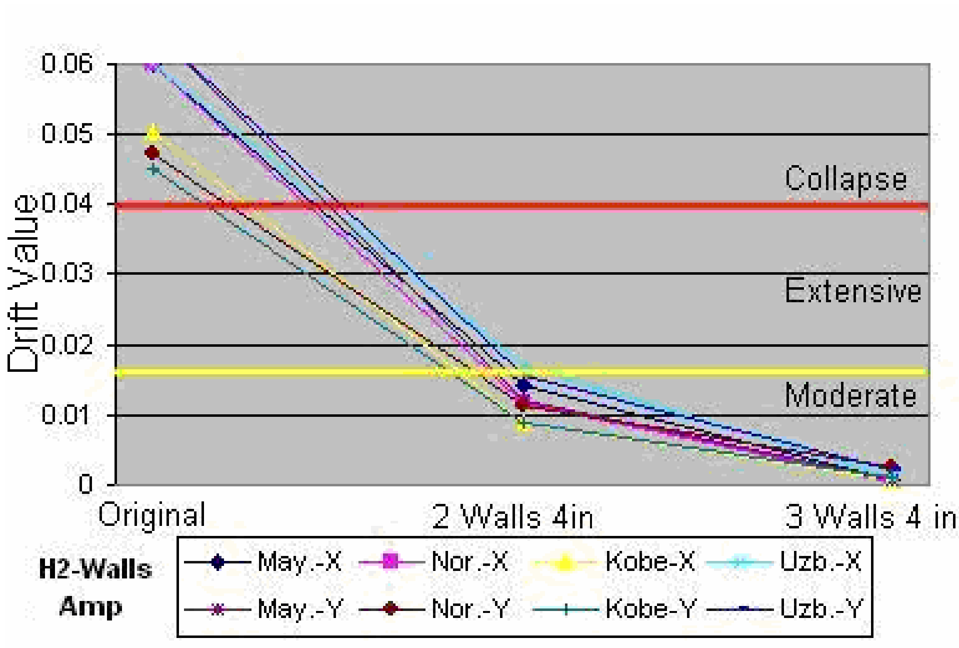


Figure 5.26 - Drift: House Model 2 retrofitted with two and three shear walls –Amplified earthquakes

Overall, the recommended placement schemes for shear walls were proven successful. However, the builder must use his judgment in cases where the site topography or the odd number of bays in one direction may force the placement of a wall in an L-shaped configuration too far from the mass center. In lieu of a direct analysis, a three wall configuration will represent a conservative solution for such cases, especially if the loading is amplified.

6 Economic Analysis

6.1 Introduction

The cost of a retrofitting scheme is often the deciding factor in its selection. Although, occasionally, the need to maintain open spaces or remain true to architectural features, especially in historical buildings, may be overriding. In this Chapter, the retrofitting cost of shear walls are compared to that of inverted-Y braces. The design configurations for which these costs are estimated are in compliance with the recommendations outlined in Chapters 4 and 5. The calculations are based on unit prices for materials and processes. It is hoped that the results from this analysis will be helpful to those contemplating such undertakings.

6.2 Shear Wall Cost Estimates

Table 6.1 lists the unit costs for various components in a shear wall structure and adds the total, assuming a coverage area of 12 ft x 14 ft and a wall thickness of 4 inches. The coverage area represents the average opening areas between columns for the Houses surveyed by Vázquez (2002). The first item listed is concrete, including the delivery but not the pouring cost which is noted separately. The cost for formwork includes materials and installation.

The tabulated two dollars per unit cost is based on the current rental rates in Puerto Rico. The price to own was estimated at six dollars per unit.

For the shear wall to work as a system with the existing structure, anchor bolts must be installed in some of the reinforced concrete beams. The drilling cost listed in Table 6.1 is the installation cost for these anchor bolts. Footing is the cost of extending or modifying the existing foundation to support the shear walls. It assumes pouring concrete to cover a volume of 14 ft x 30 in x 18 in. The cost of the footing includes the concrete, formwork and man-hours. Using the total cost from Table 6.1, the average cost for various retrofitting schemes are calculated and listed in Table 6.2.

Table 6.1 - Cost estimates for a shear wall retrofitting

Shear Wall 14 ft Wide x 12 ft High x 4 in Thick				
Item	**Unit**	**Dollar per Unit**	**Number of Units**	**Dollar Cost**
Concrete	Cubic Yard	91	2.07	189
Rebar	Ton	740	0.105	78
Formwork	Square feet of the contact area	2	336	672
Pouring Concrete	Cubic Yard	45	2.07	93
Drilling	Each	6.68	104	695
Anchor bolt	Each	3.37	104	351
Footing	Cubic Yard	238.6	1.94	463
Total				2,541

Table 6.2 - Cost estimates for recommended shear wall retrofitting

Number of Bays	Area ft^2	Earthquakes not amplified		Earthquakes amplified	
		Scheme	Dollar Cost	Scheme	Dollar Cost
2	Up to 1000	2 Walls 4in	5,082	2 Walls 4in	5,082
3	up to1400	2 Walls 4in	5,082	3 Walls 6in	8,615
4	up to1700	2 Walls 4in	5,082	2 Walls 4in	5,082

6.3 Cost Estimates for Inverted-Y Bracing Systems

The cost of the inverted-Y steel braces as a retrofitting measure will be calculated using the approach previously outlined for shear walls. Table 6.3 lists the unit costs for various components in an inverted-Y retrofitting scheme and adds the total, assuming a braced area of 12 ft x 14 ft. Once again, the braced area represents the average opening areas between columns for the Houses surveyed by Vázquez (2002).

Many of the same items are listed in Tables 6.1 and 6.3. Obviously, the use of formwork is minimal; it is only used for the mortar that fills the gap between the steel and concrete beams. The cost for shear studs includes the welding. The unit cost for steel members is based on the average weight of the C sections used as beams and diagonal braces. A cost for constructing a single brace in a frame that is twelve feet high and fourteen feet wide will be estimated. The determined cost will be applied to the retrofitting recommendations to estimate the cost of each one of the schemes. The C channel is the steel section utilized for the steel frame and the diagonal bracing members. The shear links utilized range from a W8x10 to a W16x26 section, since they are only one foot in length, the cost of them only constitutes a small percentage of the total cost. The cost of the shear link utilized in the estimate will be that of a W16x26. This system also utilizes anchor bolts to transfer the shear load form the reinforced concrete frame to the steel frame.

Table 6.3- Cost estimates for an inverted-Y bracing system

Braced Area 14 ft Wide x 12 ft High				
Item	**Unit**	**Dollar per Unit**	**Number of Units**	**Dollar Cost**
Concrete	Cubic Yard	91	0.13	12
Formwork	Square feet of the contact area	2	14	28
Pouring Concrete	Cubic Yard	45	0.13	6
Drilling	Each	6.68	14	94
Anchor bolt	Each	3.37	14	47
Shear Studs	Cubic Yard	1.56	14	22
Steel Members	Linear foot	13.56	69	936
Shear Link Assembly	Each	130	1	130
Total				1,275

Table 6.4 - Cost estimates for recommended inverted-Y retrofitting scheme

Number of Bays	Area ft^2	Earthquakes not amplified		Earthquakes amplified	
		Scheme	Dollar Cost	Scheme	Dollar Cost
2	Up to 1000	2Y (W8X10)	2,550	2Y (W16X26)	2,550
3	0-1400	2Y (W10X12)	2,550	3Y (W12X14)	3,825
	1400-2300	3Y (W10X12)	3,825	4Y(W16X26)	5,100
4	0-1700	2Y (W12X14)	2,550	3Y(W16X26)	3,825
	1700-2500	3Y (W10X12)	3,825	4Y(W16X26)	5,100

6.4 Cost Estimates for the House Models

The estimated costs for the inverted-Y bracing retrofits in House Model 1 are based on two different configurations shown in Figures 4.30 and 4.31. The case to choose will depend on whether or not the loading is amplified. Table 6.5 lists the relevant data. For the shear wall retrofits, the configuration in Figure 5.19 is used. The total expenses are calculated by counting unit numbers for each wall three times, excluding the footing cost, to account for the multilevel configuration. A summary of results is given in Table 6.6.

The calculations are similar for House Models 2 and 3. Bracing configurations in Figures 4.32 and 4.34 are used for earthquakes without amplifications. Shear wall configurations are those shown in Figures 5.20 and 5.21. For amplified earthquakes, bracing configurations in Figures 4.33 and 4.35 and shear wall configurations in Figures 5.21 and 5.24 are used. As noted, the shear wall retrofits for House Model 3 do not change when the loading is amplified. The results are summarized in Tables 6.7 through 6.10.

Table 6.5 - Cost estimates for House Model 1 with inverted-Y retrofits

Item	Unit	Dollar Cost per Unit	6 ft x 9 ft openings		11 ft x 9 ft openings	
			Number of units	Dollar Cost	Number of units	Dollar Cost
Concrete	Cubic Yard	91	0.06	5.46	0.1	9.10
Formwork	Square feet of contact area	2	6	12	11	22
Pouring Concrete	Cubic Yard	45	0.05	2.25	0.1	4.5
Drilling	Each	6.68	6	40.08	11	73.48
Anchor bolt	Each	3.37	6	20.22	11	37.07
Shear connectors	Each	1.56	6	9.36	11	17.16
Steel members	Linear foot	13.56	38	515.28	48.2	653.59
Shear link assembly	Each	130	1	130	1	130
Total				735		947
Total cost for system in Figure 4.30 = 3 x 735 + 3 x 947 = $5,073						
Total cost for system in Figure 4.31 = 3 x 735 + 6 x 947 = $7,914						

Table 6.6 - Cost estimates for House Model 1 with shear wall retrofit

Item	Unit	Dollar Cost per Unit	6 ft x 9 ft wall		11 ft x 9 ft wall	
			Number of units	Dollar Cost	Number of units	Dollar Cost
Concrete	Cubic Yard	91	0.5	45.5	1.23	111.90
Rebar	Ton	740	0.03	22	.06	44.5
Formwork	Square feet of contact area	2	108	216	198	396
Pouring Concrete	Cubic Yard	45	0.5	22.50	1.23	55.35
Drilling	Each	6.68	30	200.40	80	534.40
Anchor bolt	Each	3.37	30	101.10	80	269.60
Total				607.5		1411.5
Footing	Cubic Yard	238.6	0.83	198	1.53	365
Total cost for system in Figure 5.19 = 3 x 607.5 + 3 x 1411.5 + 198 + 365 = $6,620						

Table 6.7 - Cost estimates for House Model 2 with inverted-Y retrofits

Item	Unit	Dollar Cost per Unit	16 ft x 13.5 ft openings		13 ft x 14.25 ft openings*	
			Number of units	Dollar Cost	Number of units	Dollar Cost
Concrete	Cubic Yard	91	0.148	13.48	0.12	10.92
Formwork	Square feet of contact area	2	16	32	13	26
Pouring Concrete	Cubic Yard	45	0.148	6.67	0.12	5.40
Drilling	Each	6.68	16	106.88	13	86.84
Anchor bolt	Each	3.37	16	53.92	13	43.81
Shear connectors	Each	1.56	16	24.96	13	20.28
Steel members	Lb/ft	13.56	73.7	999.37	69.4	941.06
Shear link assembly	Each	130	1	130	1	130
Total				1,367		1,264
Total cost for system in Figure 4.32 = 1,367 + 2 x 1,264 = $3,895						
Total cost for system in Figure 4.33 = 2 x 1,367 + 2 x 1,264 = $5,262						

* Using the average height of 14.25 ft.

Table 6.8 - Cost estimates for House Model 2 with shear wall retrofit

Item	Unit	$/Unit	16 ft x 13.5 ft wall		13 ft x 14.25 ft wall*	
			Number of units	Cost	Number of units	Cost
Concrete	Cubic Yard	91	2.67	242.61	2.28	207.48
Rebar	Ton	740	0.133	98.42	0.114	84.36
Formwork	Square feet of contact area	2	432	864	370.5	741
Pouring Concrete	Cubic Yard	45	2.67	120.6	2.28	102.60
Drilling	Each	6.68	59	394.12	54.5	387.44
Anchor bolt	Each	3.37	59	198.83	54.5	183.67
Total				1918		1683
Footing	Cubic Yard	238.6	2.22	530	1.81	432
Total cost for system in Figure 5.20 = 1,918 + 1,683 + 530 + 432 = $4,563						
Total cost for system in Figure 5.24 = 1,918 + 2 x 1,683 + 530 + 2 x 432 = $6,678						

* Using the average height of 14.25 ft.

Table 6.9 - Cost estimates for House Model 3 with inverted-Y retrofits

Item	Unit	Dollar Cost per Unit	14.5 ft x 8.5 ft openings		13 ft x 8.5 ft openings	
			Number of units	Dollar Cost	Number of units	Dollar Cost
Concrete	Cubic Yard	91	0.134	12.20	0.12	10.92
Formwork	Square feet of contact area	2	14.5	29	13	26
Pouring Concrete	Cubic Yard	45	0.134	6.03	0.12	5.40
Drilling	Each	6.68	14.5	96.86	13	86.84
Anchor bolt	Each	3.37	14.5	48.87	13	43.81
Shear connectors	Each	1.56	14.5	22.62	13	20.28
Steel members	Lb/ft	13.56	54.5	739.02	51	691.56
Shear link assembly	Each	130	1	130	1	130
Total				1087		1015
Total cost for system in Figure 4.34 = 2 x 1087 + 2 x 1015 = $4,204						
Total cost for system in Figure 4.35 = 2 x 1087 + 4 x 1015 = $6,234						

Table 6.10- Cost estimates for House Model 3 with shear wall retrofit

Item	Unit	$/Unit	14.5 ft x 8.5 ft wall		13 ft x 8.5 ft wall	
			Number of units	Cost	Number of units	Cost
Concrete	Cubic Yard	91	1.52	138.32	1.36	123.76
Rebar	Ton	740	0.079	58.46	.070	51.80
Formwork	Square feet of contact area	2	246.5	493	229.5	459
Pouring Concrete	Cubic Yard	45	1.52	68.4	1.36	61.2
Drilling	Each	6.68	46	307.28	43	287.24
Anchor bolt	Each	3.37	46	155.02	43	144.91
Total				1,220		1,128
Footing	Cubic Yard	238.6	2.01	480	1.8	430
Total cost for system in Figure 5.21 = 2 x 1,220 + 2 x 1,128 + 480 + 430 = $5,606						

6.5 Overall Results

In all cases where earthquake records were not amplified, the cost of retrofitting with the inverted-Y braces was less for the House Models analyzed. Figure 6.1 charts the data. The results were mixed when the amplification factor was applied (Figure 6.2). In such cases, the design configurations generally use a higher number of braces than shear walls to limit damages to an acceptable level. However, the results are still competitive enough to recommend inverted-Y bracing throughout. A supporting argument will be the overly generous estimates for shear walls that ignore the added labor costs and time constrains due to complexities inherent when used as a retrofit.

From the cost estimates conducted for both systems we can see the trend that the Inverted-Y Steel bracing system is a much more economic retrofitting option than the shear wall even when a higher number of braces are utilized over the number of shear walls.

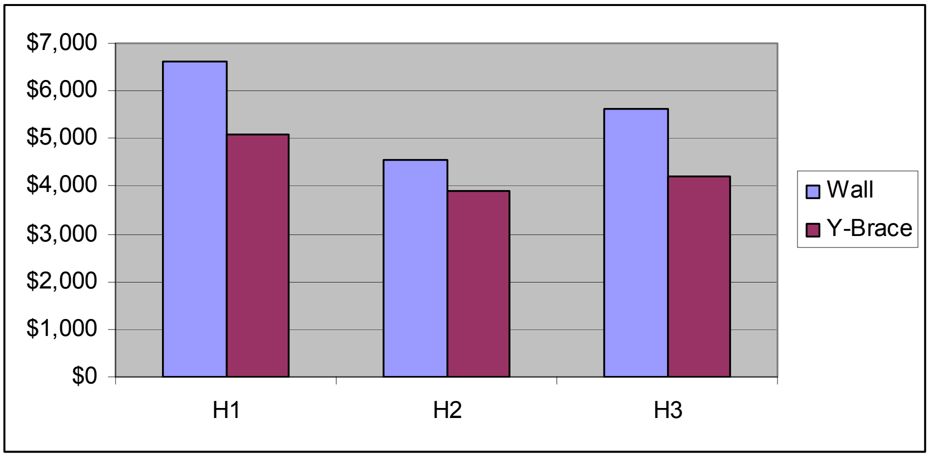


Figure 6.1- Summary of retrofitting costs for the House Models – No amplifications

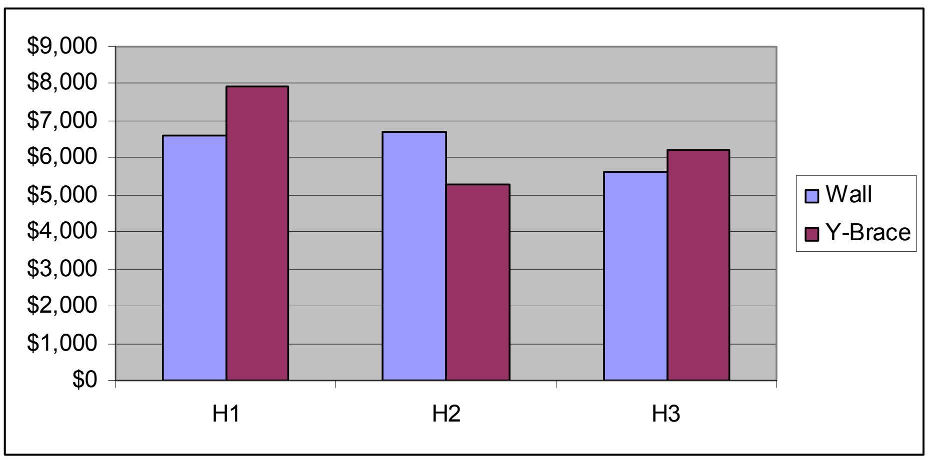


Figure 6.2 - Summary of retrofitting costs for the House Models – Amplified earthquakes

7 Conclusions and Recommendations

7.1 Conclusions

The inverted-Y steel bracing system was proven to be a cost effective retrofitting measure for Houses that are supported on gravity columns. This conclusion was reached after numerically analyzing a database of retrofitted prototype Models and applying the knowledge gained to real House Models. The more conventional approach of adding shear walls was also examined. It was noted that in most cases, the same number of walls and inverted-Y braces were required to reduce the damage states. When earthquakes were amplified, something that is expected at hillsides or escarpments, the average number of inverted-Y steel braces was increased but the material cost remained competitive.

In terms of logistics, the inverted-Y steel frames are a practical choice for the use in hillside or escarped terrain. The use of heavy machinery is reduced to a minimum because there is no extensive digging, the steel components are (for the most part) of manageable size, and only a small amount of mortar is required. By contrast, building a shear wall will often require expanding the existing foundation which in turn may require excavation.

Most of the Houses in hilly or escarped terrain have narrow site access, making excavation process considerably more difficult. The manual labor and the construction time are also markedly increased. Placing the reinforcement bars and making sure that the required development lengths inside the existing reinforced concrete beams are achieved is another labor intensive task. Forms must be rented or bought and assembled and the concrete must arrive by mixing trucks and pumped in place, making sure that the wall-to-beam juncture is adequately bounded.

In the aftermath of an earthquake, it is most likely that the shear link will be the only component of the inverted-Y steel bracing system need replacing. The shear wall retrofits, on the other hand, may be cracked or damaged beyond repair. This is an added cost benefit of the inverted-Y bracing that was not accounted for in the cost analysis of Chapter 6.

7.2 Recommendations

The number and placement of inverted-Y braces as retrofits is based on geometric properties like the square footage, column heights, and number of bays as well as whether or not the earthquake is amplified. Table 4.1 provides details on the acceptable configurations. The recommended shear links in this table are in four sizes. They are built-up sections using a W shape at its core. The basic design as presented in Chapter 4 is easy to execute. It is assumed that the link is shop built and is welded on site to the supporting steel beam before the whole assembly is raised and attached to the existing concrete beam. The C sections used as diagonal braces are then bolted to the link assembly. Non-shrinking grout will be used to fill the gap between the steel and concrete beams.

Although the overall inverted-Y retrofitting scheme was designed to be implemented without any additional computations, engineering supervision is recommended. On the production side, it would have helped if rolled W shapes with thin webs and a flange to web thickness ratio of at least 3 to 1 was available. Such a section would have eliminated the need of flange cover plates for the shear link sections. Recognizing the wide spread use of shear links in the modern design, serious considerations should be given to expanding the existing inventory of W shapes. As is, the detailing guidelines for the shear link assembly must be closely followed to insure successful transfer of story shear to the links.

Recommendations on the use of shear wall retrofits were presented as part of this research. Although fairly popular in Puerto Rico, the cost analysis for this retrofitting system indicates that any potential advantage is limited to hillside and escarpments and then only because the cost overruns associated with the access to the site was ignored. However, there are occasions when the underside area is used to make additional housing. In those occasions, the recommended configurations in Table 5.1 can be made part of the new addition.

For a future work, it is recommended to analyze the interactions between the retrofitted structures and the soil in more details. The existing prototypes assume that the building foundations can be easily modified to support the proposed retrofitted schemes. As a result, potential problems like sliding, tilting, uplifting or bearing failure are not considered. The conditions under which the building foundations must undergo major modifications to minimize the risk of failure at that level need to be clearly established.

8 References

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Appendix A

SAP2000 Model

The following are the recommended guidelines for creating the SAP2000 model of a reinforced concrete frame structure retrofitted with the inverted-Y steel bracing system. The guidelines are intended for users that are already familiar with the program.

Use a three dimensional frame template to define the geometry of the structure. Provide the size, material properties, and reinforcement data for beams and columns in the usual manner. The floor slab in the selected structures is supported by gravity columns since they have no concrete slab at ground level. The weight of the floor slab, the roof slab and the columns and beams located above the floor slab can be assigned as a lumped weight located at the height of the floor slab. A suggested procedure would be to define a shell element at the height of the floor slab with a thickness that corresponds to the lumped weight.

Define a multi-linear plastic link element at every node connecting beams to the columns. The equivalent of the behavioral equations for the concrete links is the moment-curvature relationship of the concrete element cross section, selecting Takeda as the hysteresis type. To draw the concrete links, remove a one inch section from the end of the concrete element at a distance equal to five to ten percent of the total length. Replace the created void with the defined concrete link element.

To model the shear links for the inverted-Y bracing, the behavioral equations presented in Tables 2.1 and 2.2 will be utilized. First, the user must define a multi-linear plastic link element in SAP2000. At this time, select a name for the link and input the mass and weight of the element as shown in Figure A.1. Next, define the force-displacement and moment-rotation relationships in the directional properties sections and select kinematics as the hysteresis type (Figure A.2). The links utilized have a longitude of 12 inches but the length can be varied if desired, provided that the change in longitude is taken into consideration in the calculation of the behavioral equations. Draw a 2 joint link from the center of the beam down to a vertical distance equal to the link length. From the lower part of the link attach the two diagonal bracing and connect them to the base of the columns.

When creating the time history function for the analysis, one must provide the file containing the acceleration record (Figure A.3). Once the file is uploaded to the program, usually as a text file, specify number of header lines to skip, prefix characters to skip, points per line and the values of the time intervals. In the example shown in Figure A.4, the user can select u1 as the load name and insert the previously defined time history function. If the earthquake record utilized had two horizontal components, add a new acceleration load with u2 as the load name and apply the function of the additional component. The scale factor varies depending on the units of the earthquake record and the units being utilized in the analysis. It is common to define scales as a percentage of the acceleration of gravity. Modal damping is selected as constant at 0.05 for all modes.

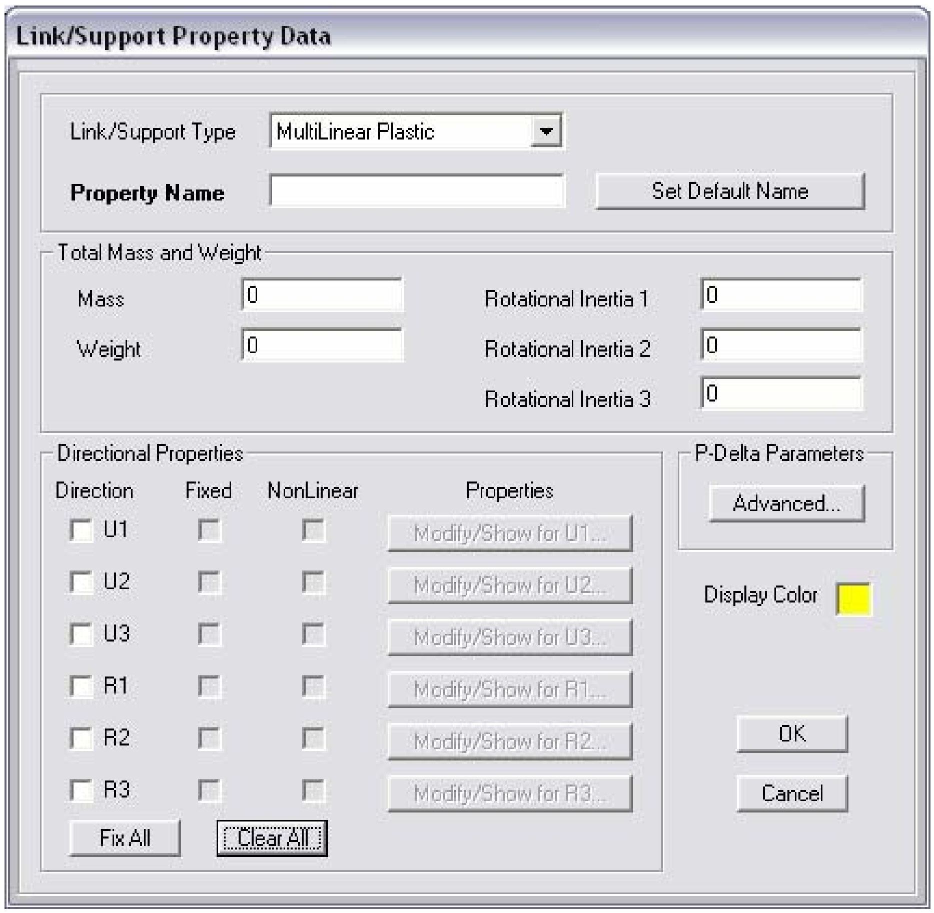

Figure A.1 - Link property data

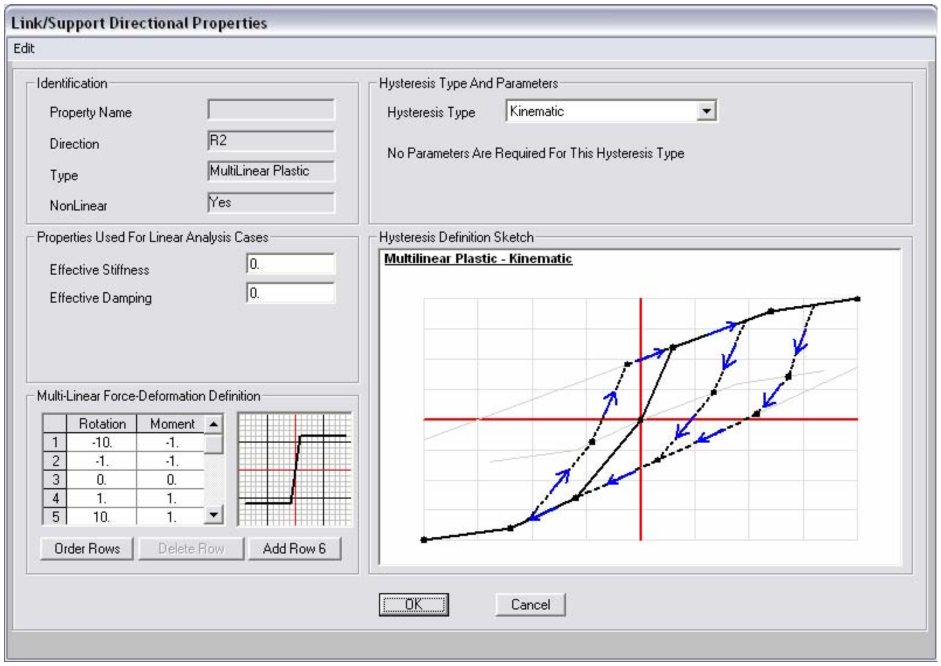

Figure A.2 - Link directional properties

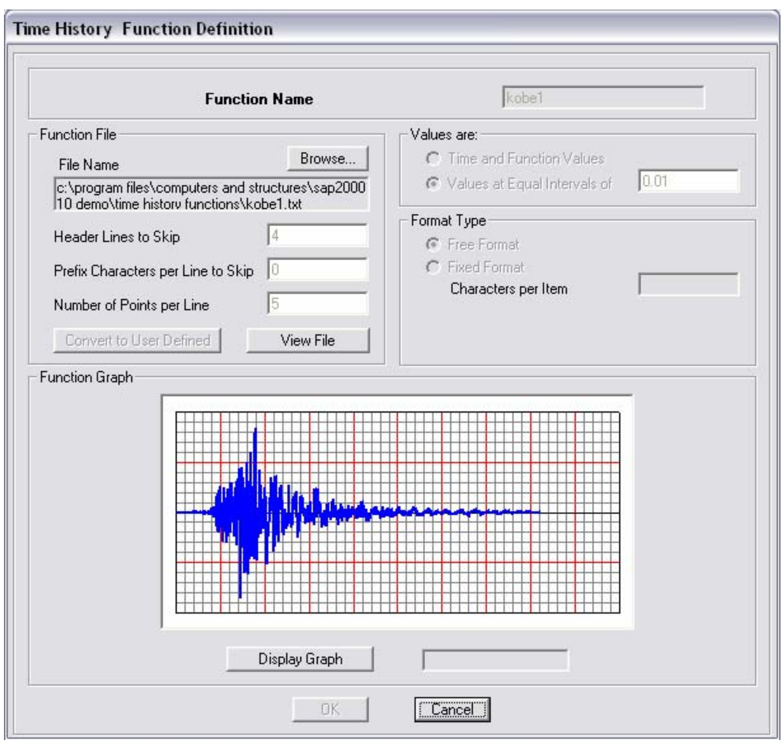

Figure A.3 - Time history function definition

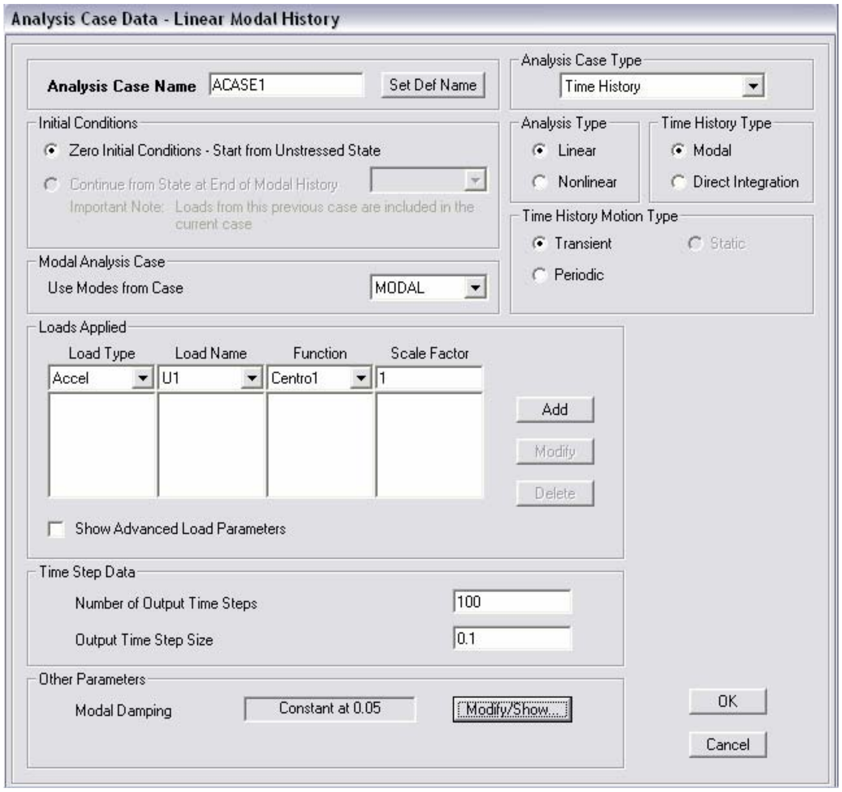

Figure A.4 - Analysis case data