Study of Preventive Seismic Retrofitting for an Existing Industrial Pharmaceutical Steel Building at Humacao, Puerto Rico

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Abstract — *The project consists on the development of a comprehensive seismic evaluation and possible retrofit for an industrial pharmaceutical steel building. The building was originally constructed during the 1970s decade, before the adoption of modern seismic design codes. The study includes the evaluation of the strength capacity and ductility of existing structural lateral force resisting system members in relation to the actual codes. The structural analysis and evaluation determined if rehabilitation will be required.*
The structural member sections and details were evaluated using the requirements of International Building Code IBC 2009 Structural/Seismic Design Manual, Minimum Design Loads for Buildings and Other Structures ASCE 7-05 and American Institute of Steel Construction Manual, AISC (thirteen edition). Finally, a retrofit proposal, among other solution, was recommended to increase the critical elements capacity to resist lateral loads as required.

Key Terms — *Excessive Buckling in Compression Members, Finite Element Methods, Ordinary Concentric Braces Frames, Slender Column Condition.*

INTRODUCTION

Building structural retrofits has gained more popularity during the last 20 years due to the observed seismic behavior of existing structures on the post earthquake effects and the introduction of more restrictive seismic design parameters on the actual building codes.

Prior to the introduction of modern seismic codes in the late 1960s for developed countries (US, Japan, etc.) and late 1970s for many other parts of the world (Turkey, China, etc.) including Puerto Rico, many structures were designed without adequate detailing and reinforcement for seismic protection. In view of the imminent problem, various research works have been carried out. Furthermore, state-of-the-art technical guidelines for seismic assessment, retrofit and rehabilitation have been published around the world such as the American Society of Civil Engineers (ASCE)-SEI 41 and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines. Whilst current practice of seismic retrofitting is predominantly concerned with structural improvements to reduce the seismic hazard of using the structures, it is similarly essential to reduce the hazards and losses from non-structural elements. It is also important to keep in mind that there is no such thing as an earthquake-proof structure, although seismic performance can be greatly enhanced through proper initial design or subsequent modifications.

For the past 50 years Puerto Rico has experienced a considerable industrial development around the whole Island. Most of those developments were focused on the industrial sectors of the former age for example the tobacco or the swing industry. A considerable amount of those companies went out of business leaving existing facilities available. As the time goes by changes on the technology, industrial needs and tax incentives bring new companies to the island to produce or manufacture different types of products, specifically pharmaceutical products. As there were existing facilities available, those companies brought their operations to the available buildings, performing considerable changes to the facilities in terms of utilities to manufactures and operate the plants. The new utilities included for example, new air handlers units (much of the time above roof structure), cooling towers or chillers to serve the HVAC systems, the plant process, mechanical or electrical equipments among others. Those equipments bring a lot of conduits inside the facility running hanging inside the existing roof area adding more mass to the original structure. Much of the structural improvements associated to the new building use were focused on the gravity loads side, leaving the original lateral resisting structural system (if any) untouched. With time business kept growing and so did utilities inside the building, thus adding additional mass inducing a bigger seismic lateral load. For this reason, it is of interest to analyze the effect of those mass additions and building use change in the lateral load resisting system focusing on the seismic effect using the actual knowledge and latest applicable code.

The Island of Puerto Rico is located in the northeastern edge of the Caribbean Tectonic Plate, which interacts with the North America Tectonic Plate, and therefore it is surrounded by geological faults capable of producing earthquakes. The main objective of this project is to perform a structural evaluation of existing Industrial Steel Building. The building located in the industrial area at the Mariana Ward at Humacao, P.R., is a company dedicated to the manufacturing and packaging of different types of pharmaceutical product. Since the owners of these facilities during the past years have increased concerns in term of security of the overall plant during a seismic event, this company was selected to be used as the building model on the structural evaluation for this project. This facility produces, manufactures and distributes products from the Puerto Rico facility all over the word, being this plant one of the most productive facilities corporative speaking. A partial or a full collapse to the facility will cause a huge loss, not only in terms of human life, but will affect financially all the company business around the word.

The building was constructed on the early 1970's and consists on a structural steel building enclosed by metal siding panels. The lateral load resisting system consists on vertical concentric brace frames located at the building perimeter frames. The roof system is composed by a lightweight cellular concrete and metal deck over steel joist system. The slab in combination with the metal deck acts as a diaphragm that should distribute the seismic load to the lateral resisting system. The interior columns do not contribute to the lateral load resisting system and were designed to support only the dead and live gravity loads. The building column grid system varies from 30'-0" to 34'-0" spans, (see Figure 1).

Figure 1
Photo Plan View of the Industrial Steel Building

Each year buildings and other structures are designed and built with a continually improving understanding of their performance during earthquakes, yet the vast majority of structures were built with substantially less understanding or seismic action that we currently posses. Recently Earthquakes in Chile (2010) and Japan (2011) demonstrate the power of nature and the catastrophic impact of such power upon urban cities. Casualties and damage associated with older buildings, which were designed and constructed using codes that are now known to provide inadequate safety, are far worse than that for newer buildings which has been designed with more stringent codes. The quantity of older buildings build before 1980's is believed to be many times more than the number of newer buildings in most urban and industrial areas.

Evolution of Building Codes Requirements

Building Codes have significantly changed as far as to seismic requirements in the past fifteen years because of new investigations, new technologies and the experience gained by Engineers through past earthquakes. Therefore, a significant part of the old Industrial Buildings were designed following existing building Code regulations that contained little or no seismic design requirements that will enable reinforced steel structures to sustain significant inelastic deformation without collapsing. This study will take into account the design provisions of current design codes which include several considerations concerning the effects of earthquake-induced lateral forces upon buildings and other structures. The study described above will include the evaluation of the strength capacity and ductility of existing structural members under different load conditions. At each load stage several demand/capacity checks will be performed to determine the adequacy of the existing members based upon the IBC 2009[1] and the AISC manual American Institute of Steel Structural [2] /Seismic Design Manual AISC 341 05 [3]. The rehabilitation method to be selected will be shown in new construction drawings if retrofitting is determined to be necessary from the above mentioned evaluation.

Impact of Evolution of Building Code Requirements

The combined effect of the location and geologic settings of the Island of Puerto Rico, and the fact that a significant part of the Island buildings were designed following building code regulations that contained little or no seismic design requirements should be of great concern to the Company Management, due to the fact that the occurrence of an earthquake with a seven and half magnitude may be imminent and that the occurrence of such an earthquake could cause serious damage to the building, with adverse effect on the company economy and, most importantly, could also result in significant loss of human life.

Even thought industrial buildings do not have very high occupancy densities, one person per hundred square foot, this type of facilities, most of the times, manufactures, pack and storage a considerable amount of products that represent billions of dollars to the company.

For this reason many Companies management are taking immediate action to analyze existing structures and determine compliance with seismic requirements stated in current Building Codes, and recommend, if necessary, rehabilitation measures. Most of the references of the Structural Rehabilitation Methods and/or Procedures group reviewed are part of the publications by the ASCE/SEI 41-06 [4] regarding Seismic Rehabilitation of Existing Buildings.

Literature Review

Concentrically braced frames are frequently used to provide lateral strength and stiffness to low and mid-rise buildings, to resist wind and earthquake forces (study case). This type of structures needs to be designed for appropriate strength and ductility. In a manner consistent with the earthquake-resistant design philosophy, modern concentrically braced frames are expected to undergo inelastic response during infrequent, yet large earthquakes. Specially designed diagonal braces in these frames can sustain plastic deformations and dissipate hysteretic energy in a stable manner trough successive cycles of buckling in compression and yielding in tension. The preferred design strategy is, therefore, to ensure that plastic deformations only occur in the braces, leaving the columns, beams, and connections undamaged, thus allowing the structure to survive strong earthquakes without losing gravity-load resistance.

Past earthquakes have demonstrated that this idealized behavior may not be realized if the braced frame and its connections are not properly designed. Numerous examples of poor seismic performance have been reported and collapses have been occurred of such uncontrolled inelastic behavior, (see Figure 2).

Two types of systems are permitted by the AISC-341-05[3] Seismic provisions: Special Concentrically Braced Frames (SCBFs) and Ordinary Concentrically Braced Frames (OCBFs). SCBFs are designed for stable inelastic performance using response modifications factor in the order of R=6, OCBFs are subjected to smaller deformation demands due to the use of a smaller response modification factor R=3. However, if an earthquake greater than that considered for design occurs, SCBFs are expected to perform better than OCBFs because of their substantially improved deformation capacity.

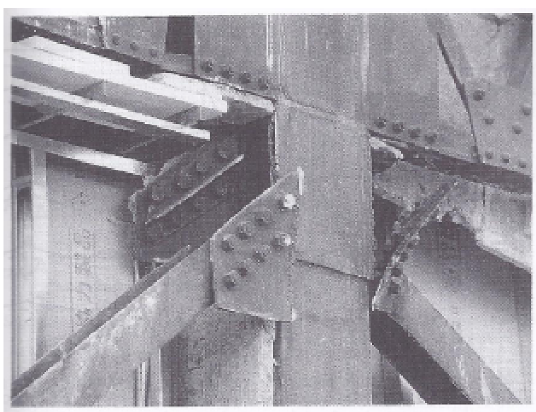


Figure 2
Fracture of Welded Connection and Web Tear-out in Brace

Given that diagonal braces are the structural members chosen to plastically dissipate seismic energy, the physical behavior of a single brace subjected to axial load reversal is very important. As a result, progressively larger axial forces can be applied; the bracing member cannot be brought back to a perfectly straight position before the member yields in tension. Consequently, when unloaded and reloaded in compression, the brace behaves a member with an initial deformation and its buckling capacity, P'_{cr} is typically lower than the corresponding buckling capacity upon first loading, P_{cr}. The Hysteretic curve repeats itself in each subsequent cycle of axial loading and inelastic deformation, (refer to Figure 3).

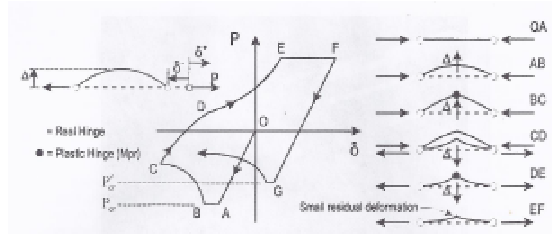


Figure 3
Hysteretic of a Brace Under Cyclic Axial Loading

Concentrically braced frames exhibit their best performance when both yielding in tension and inelastic buckling in compression of their diagonal members contribute significantly to the total hysteretic energy dissipation. The energy absorption capability of the brace in compression depends on its slenderness ration (KL/r) and its resistance to local buckling during repeated cycles of inelastic deformation.

Very slender brace members (such as bars and plates) can result from a practice called tension-only design, often used prior to the promulgation of modern seismic provisions for steel buildings, and still used in non seismic regions. In that design approach, the tension brace is size to resist all the lateral loads, and the contribution of the buckled compression is ignored. While tension-only design may be acceptable for wind resistance, it is not permissible for earthquake resistance.

Is our intention to evaluate the lateral resisting system of the building to identify the actual seismic behavior under the above mentioned conditions.

INFORMATION AS PER STRUCTURAL ORIGINAL DRAWINGS

Gathering of the available construction drawings and geotechnical report was performed. Existing conditions of the building were corroborated on several site visits to validate the actual drawings information. Other related document, such as major rehabilitation work performed in other buildings was obtained to incorporate them into this project. The building was constructed on the early 1970's and consist on a structural steel building enclosed by metal siding panels. The lateral load resisting system consists on vertical cross type brace frames located at the building perimeter frames. The roof system is composed by a lightweight cellular concrete and metal deck over steel joist system. The slab in combination with the metal deck acts as a diaphragm that distributes the seismic load to the lateral resisting system. The interior column does not contribute to the lateral load resisting system and were designed to support only the dead and live gravity loads. The building column grid system varies from 30'-0" to 36'-0" spans, (see Figure 4).

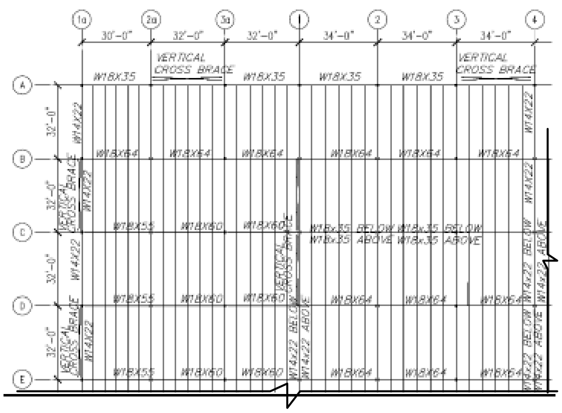


Figure 4
Existing Typical Steel Building Partial Roof Framing Plan

The Building is a one stories braced frame structure as shown in Figure 5. The total height of the structure is mostly in the order of 17.00 feet with an area that reach a height of 21 feet.

A visual inspection was done to determine the present condition of the building structure. Most of the braces systems elements are actually covered by gypsum wall walls because are located inside manufacturing areas. At the existing warehouse area (east side) all the lateral force resisting system are visible and seems to be in good conditions. It was noticed that at axis 1, as part of a previous expansion, the vertical braces at axis 1,C & D were removed. This obviously will have an adverse effect on the seismic performance of building structure during an earthquake event. The structure appears to have been constructed as per original drawings and the rest of the vertical and horizontal steel elements appears to be in good condition.

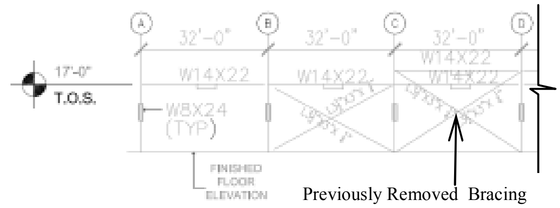


Figure 5
Existing Partial Typical Steel Brace Frame Elevation

Foundation of this building are formed by individual spread foundations that varies from 5'-0" x 5'-0" to 7'-6" x 7'-6" square shallow footings. Soil properties as per record drawings use a soil type D for seismic design purposes. Allowable soil bearing capacity was taken as 3,500 pounds per square foot at 2 feet below existing grade elevation, (refer to Figure 6).

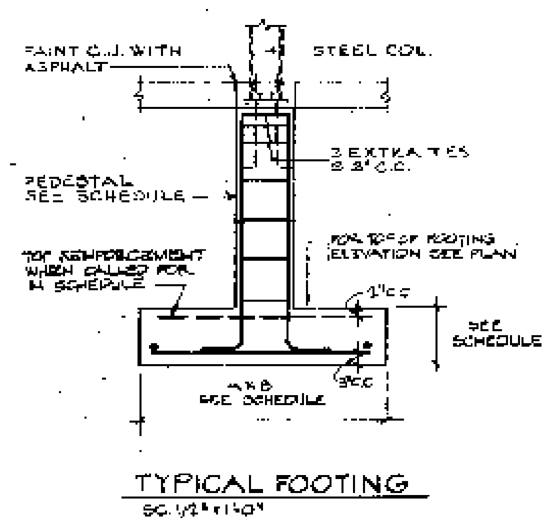


Figure 6
Existing Typical Foundation Detail

Original design of the building was based on the Puerto Rico Building Code 1968, American Concrete Institute (ACI) Code of 1973 and the American Institute of Steel Construction (AISC) of 1970. All estructural setal seccionas ere base en te ASTM specifications A-36.

Investigation of Capacity/Demand Ratios under Seismic Loading

A Structural analysis was performed to determine the weakest part of the system in the seismic event. The evaluation will consist of the building behavior assuming the resistances provided by the brace frames for each direction assuming a tension-only approach ignoring the compression resistance of the brace frame system. This evaluation will investigate if the actual braces section has enough capacity to support all the tension forces imposed by the seismic loads.

The second study will consider both effects tension and compression acting together to evaluate the actual forces and the required improvements or retrofit to the lateral resisting system if this was the case.

Analysis of the existing structure was performed using the computer programs called ETABS[5], Extended 3D Analysis of Building Systems, developed by Computers and Structures, Inc. The said program is based upon Finite Element Methods of structural analysis, **(**see Figure 7).

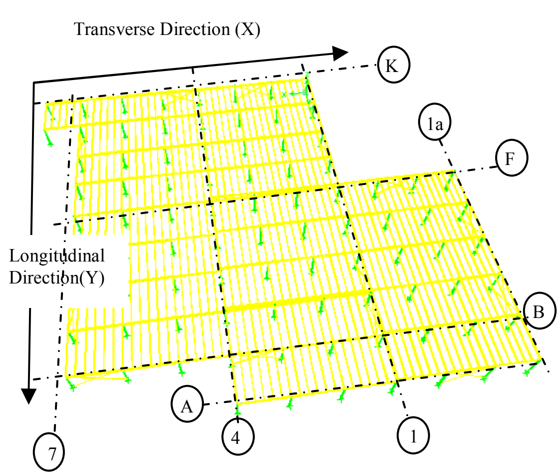


Figure 7
Schematic Three Dimensional Frame of Steel Building

At each load stage examination of the demand/capacity ratios will be performed to compare the strength capacity (moment, shear and axial) of structural members with the forces obtained from the analysis. Study of the linear static analysis will take into consideration the load combinations specified in the IBC 2009[1] & ASCE 7-05 Codes [6].

Determination of capacity will be based upon the following assumptions and conditions:

- Considering that all the seismic lateral loads will be resisted by the brace frames systems.
- Considering that the actual concrete filled metal deck system will act as a flexible diaphragm to distribute the lateral loads.

Evaluation Of Seismic Loads As Per IBC 2009[1] As Amended For P.R. Building Code And ASCE Standard 7-05[6]

Earthquake loads are those lateral loads produced by the ground motion of the earth and will be modeled following a simplified static lateral force as described by the IBC 2009 [1] and ASCE Standard 7-05 [6], and also other requirements such as load combinations.

The accelerations in Figure 1613.5(13) of IBC 2009 [1] shall be particularized by the following accelerations corresponding to the municipalities of Puerto Rico. The following maps present the spectral response acceleration for buildings with 0.2 sec period of vibration (as seen in Figure 8) and for period of 1.0 sec (stiff structures), (as shown on Figure 9). Accelerations indicated on these Figures are measured in percentage of gravity, and correspond to occurrences with 2% probability of exceedance in a 50 years interval.

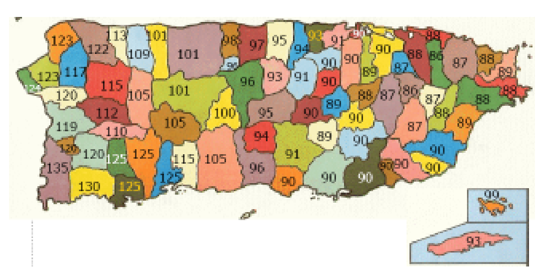


Figure 8
Spectral Response Acceleration at Period of 0.2 Seconds

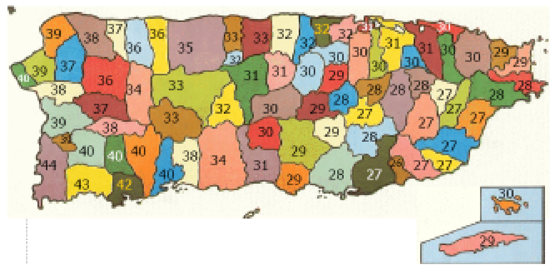


Figure 9
Spectral Response Acceleration at Period of 1.0 Seconds

Figure 10 shows the response spectra in IBC 2009 [1] for different areas in Puerto Rico using a Soil Class D, importance factor I=1.25 and response modification factor R=3.25 related to Ordinary Concentric Braced Frames(OCBF). For overall average dead load of 48 psf and period of 0.275 seconds , the total base shear for a total gravity load of W=3,471 kips is equivalent to a total base shear of approximately V=908 kips.

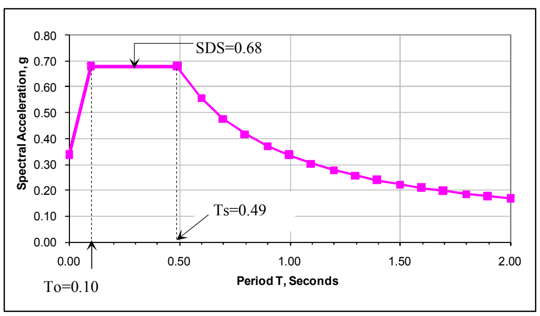


Figure 10
Elastic Response Spectrum, R=3.25, Soil D, I=1.25

The vertical distribution of the 908 kips base shear, to identify the lateral forces F_x at different levels of the building is shown in Table 1.

Table 1
Lateral Forces at each level (Kips)

Level	$W_{x(K)}$	$h_{x(ft)}$	$W_x h_x$	$W_x h_x / \sum W_x h_x$	$F_{x(k)}$
High Roof	1,182	21.50	25,413	0.39	354.12
Low Roof	2,297	17.00	39,049	0.61	553.88
Ground	3,471	0	0	0.00	0.00
					V = 908 kips

Diaphragm Strength Evaluation

As per structural drawings the metal deck is connected to the collector elements with weld washers using a 36/4 pattern at all supports, (refer to Figure 11).

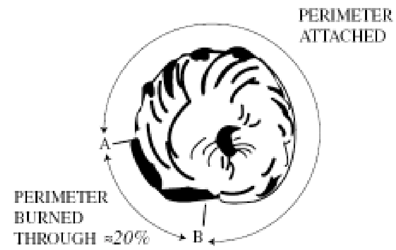


Figure 11
Typical Weld Washer Connection

Results from the structural analysis of the building using the computer software ETABS[5] determine that the higher lateral load is resisted by the brace frame at axis 7, (refer to Figure 15). As per Steel Deck Institute Diaphragm design Manual [7] the individual weld strength can be calculated by the following equation:

$$Q_f = 99t\left(0.5 + 0.3F_{xx} * t\right), kips \qquad (1)$$

where,

t = base metal thickness, inches

F_{xx} = electrode strength, ksi

In our case the maximum distributed load along the perimeter collector elements was 2.85 k/ft. From the previous formula the available weld strength was calculated to be Q_f = 3.10 k/ft. We can conclude that the existing deck diaphragm can transfer the expected loads to the lateral resisting system.

Structure Response of Brace Frames

Results of the Frame Analysis combining Gravity and Seismic Loads, based on Ordinary Concentric Brace Frames(OCBF), using R=3.25, is presented in Figure 12 & 13 below, including structural demands at the braces frame members.

The axis line 7 as per structural analysis was the brace line that receives the biggest lateral load.

Using the basic load combinations as per IBC 2009 [1], maximum earthquake load calculated as follows:

$$E = \rho * Q + / - 0.2 * SDS, kips \quad (2)$$

Were,

SDS = 0.68 *(Spectral Acceleration)*

Q= Horizontal seismic load

ρ=Redundancy factor=1.3

Maximum load combination for the tension case refers to equation (3).

$$Pu = 0.90 * P_D + \rho * Q, kips \quad (3)$$

were,

P_D = *Dead load*

Q= Horizontal seismic load

ρ=Redundancy factor

Pu=277 kips (Tension).

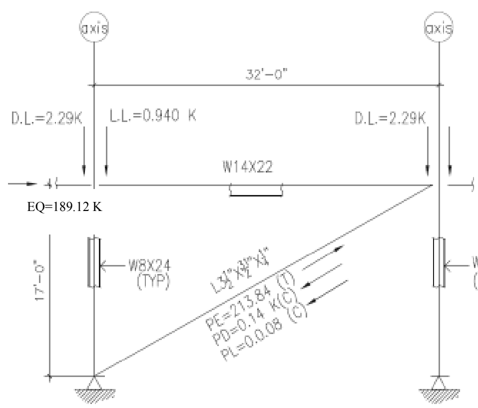


Figure 12
Brace Frame Axis 7 Seismic Load Tension Only Case

Using the basic load combinations as per IBC 2009 [1], maximum earthquake load calculated as follows:

$$E = \rho * Q + / - 0.2 * SDS, kips \quad (2)$$

Were,

SDS = 0.68 *(Spectral Acceleration)*

Q= Horizontal seismic load

ρ=Redundancy factor=1.3

For the Maximum load combination of the compression case refers to equation (4).

$$Pu = 1.2 * P_D + \rho * Q + P_L, kips \quad (4)$$

were,

P_D = *Dead load*

PL = Live load

Q= Horizontal seismic load

ρ=Redundancy factor=1.3

Pu=-142 kips (compression).

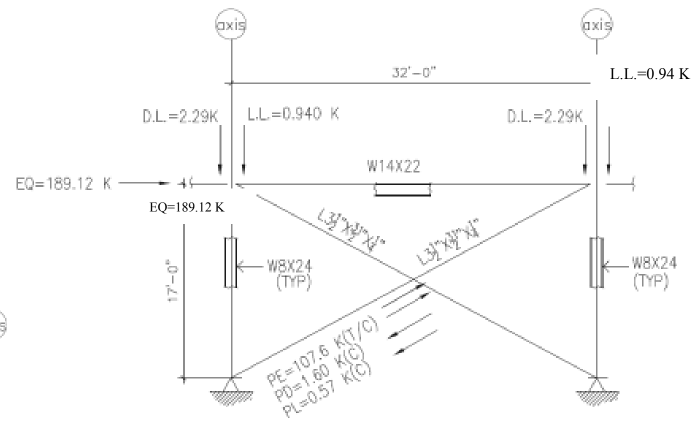


Figure 13
Brace Frame Axis 7 Seismic Load Tension and Compression Case

Results of the study of Demand/Capacity imposed by the seismic loads

Existing braces and columns associated with the lateral resisting system do not have the capacity resist the required demands related to the seismic forces developed using conservatively the low response modification factor R=3.25, compatible with Ordinary Braced Frames. For the tension only loading case, the maximum tension capacity can be calculated using equation (5).

$$\phi Pn = F_y * A_g, kips \quad (5)$$

were,

F_y = *Steel yield strength=36 ksi*

A_g = *Steel section gross area.*

φ= *Reduction factor=0.75*

ϕPn = 66 kips (Tension).

The maximum brace capacity was calculated to be 66 kips. The demand capacity ratio for this section is in the order of 4.20.

For the combined loading case the calculated KL/r was 400, bigger than the allowed 200 for compression members. Maximum compression capacity for the brace section using the following equation (6):

$$\phi Pn = F_{cr} * A_g, kips \quad (6)$$

were,

F_{cr} = *Flexural buckling stress, ksi*

A_g = *Steel section gross area.*

φ= *Reduction factor=0.75*

φPn=2.34 kips (compression).

This value validates that the structure falls below as a tension-only design. The remaining bracings of the building were analyzed behaving quite similar to the presented before. It's clear that the existing lateral resistance system does not have the required capacity to withstand the expected seismic loads under an imminent mayor earthquake. Figure 14 below contains the Column Strength Diagram for the existing brace section.

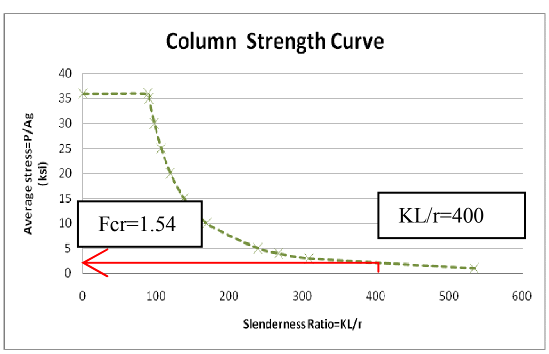


Figure 14
EULER Colum Strength Curve

Analysis of Retrofitted Frame using Additional Brace Frames

Figure 15 illustrates the retrofitting procedure corresponding to the proposed solution of the possible brace failure by the installation of additional bracings at some axis and the replacement of the existing remaining brace elements by bigger sections. The existing building conditions does not allows, at some frame areas, the installation of additional bracings due to the existence of manufacturing equipment, manufacturing rooms and hallways that can't be interrupted by structural elements.

It can be noticed that four new OCBF bracings were added at west and south west areas and brace replacement were implemented in all the remaining existing bracing lines.

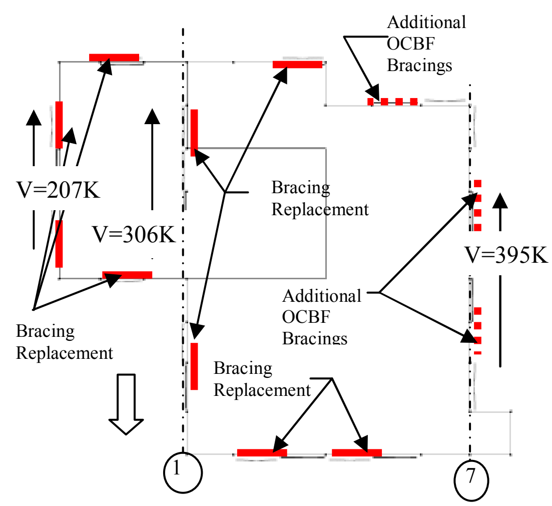


Figure 15
Proposed New Bracing Locations to Resist the Expected Seismic Lateral Loads

Figure 16 presents the analytical model of the typical retrofitted frame lines showing the elements, including the seismic load corresponding to the base shear calculated previously.

Verification of Capacities in the Retrofitted Structure against Demands imposed by the Seismic Loads

After several trials it was established that the optimum bracing size for the retrofit conditions of the brace system were hollow structural sections HSS8x8x5/8". This element complies with the minimum width to thickness and slenderness ratios as required by the ICB 2009[1] and AISC 341-05[3].

As part of the retrofit evaluation the existing beams and columns were verified. It was found that the existing W14 beams complies with the code requirements in terms of geometry and demand/capacity ratios, but that was not the case for the existing W8 columns.

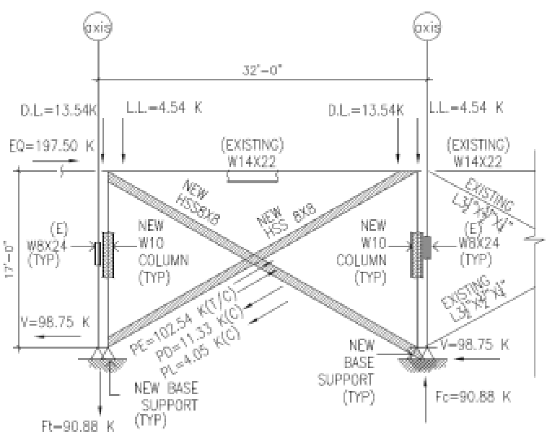


Figure 16
Axis no. 7 Frame Elevation with additional Ordinary Concentric Brace Frame System (R=3.25)

New W 10 columns were added to comply with the new imposed loads and width to thickness ratios requirements, also for constructability purposes was determined that adding new columns was cheaper than reinforced existing columns.

As per code requirements the maximum compressive strength load combination was obtained using the following equation:

$$Pu = (1.2 + 0.2SDS) * P_D + \rho * Q + P_L, kips \qquad (7)$$

$Pu = 168.50\ Kips$

Maximum compression capacity for the brace section using the following equation (8):

$$\phi Pn = F_{cr} * A_g, kips \qquad (8)$$

$\phi Pn = 88.77\ Kips$

The demand/capacity ratio will be 1.89. Table 2 below, indicates the capacity/demand ratio resulting from the original condition and Table 3 indicates the strengthening of the building in accordance with the study developed previously incorporating the new bracing sections and column as described above.

Table 2
Summary Demand/Capacity (*original condition*)

Member Section	Demand	Capacity	Ratio
W8x24 columns	168.40	88.77	1.89>1
W14x22 Beam	117.00	168.09	0.70<1
L3.5x3.5Brace	-141.81C	2.34	60.60 >1
L3.5x3.5Brace	138.40T	77.74	1.80 >1

Table 3
Summary Demand/Capacity (*retrofitted condition)*

Member Section	Demand	Capacity	Ratio
W10x49 columns	168.40	381.68	0.44<1
W14x22 Beams	117.00	168.09	0.70<1
HSS8x8x5/8Brace	-152.40C	170.98	0.89 <1
HSS8x8x5/8Brace	134.20 T	754.40	0.18 <1

Base Column Connection Evaluation

Verification of the base connection to existing pedestal and foundation was performed using the 318-08ACI [8] and AISC Steel Design Guide Line no.1, Base Plate and Anchor Rod Design [9]. The existing base plate connection was evaluated for compression, tensile, pull-out and base shear forces. It was found that the existing anchor connection to the concrete pier does not comply with the Tension/Shear combined forces. For this reason new 1 ½" F-1554 bolt were included on the new base connection (Refer to Figures no. 17 trough 21) to resist the combined forces and comply with the interaction equation (refer to figure no. 17), and equation (9).

$$Nu / Nn + V / \phi Vn, kips < 1.2 \qquad (9)$$

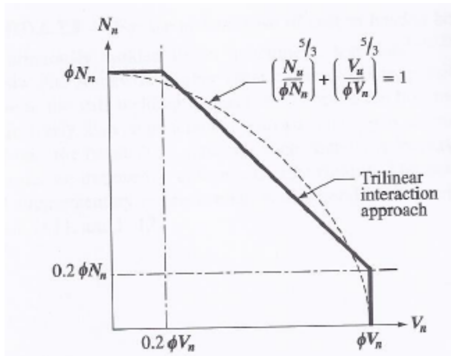


Figure 17
Shear and Tensile Loads Interaction Equation Graph

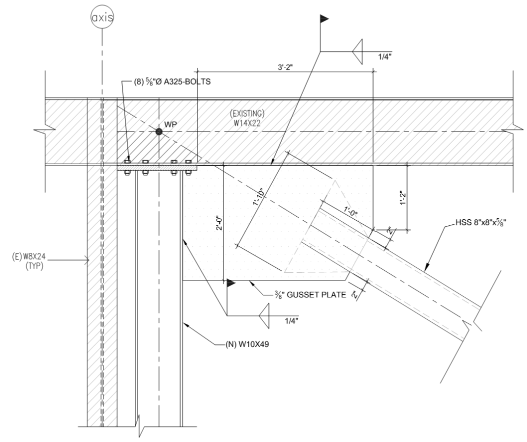
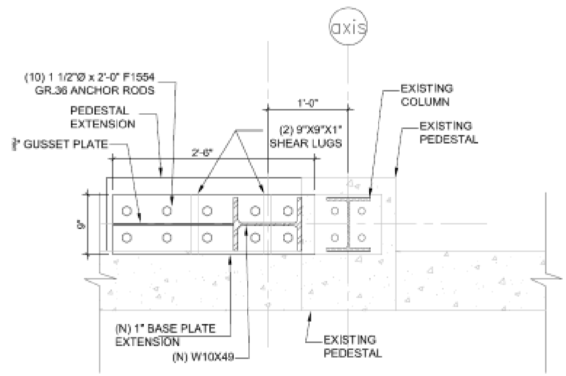


Figure 18
Typical Gusset Beam to Column Connection

Existing anchor bolts were not take inconsideration because of the limited pullout capacity and lack of concrete pier reinforcement. Table 4 indicates the capacity/demand ratio of the existing base plate condition using the code load combinations.

Table 4
Summary Demand/Capacity (*Retrofitted Base Plate) in Kips*

Member	Demand	Capacity	Ratio
Plate	178.12 C	230.42	0.77>1
1 1/2"dia. Anchor Bolts F-1554	8.94/bolt (Up Lift)	9.10/bolt	0.98<1
Shear(bolts)	21.38	78.51	0.27 <1
Shear(Shear Lug)	140	142	0.99 <1



SECTION-1

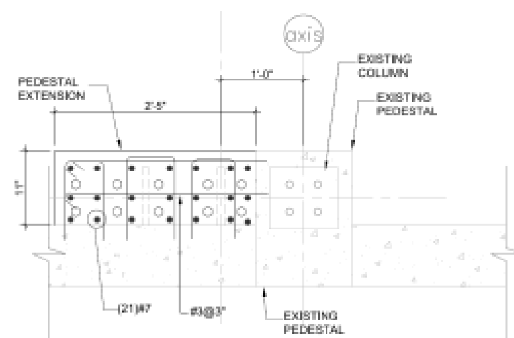
Figure 19
New Base Plate Detail

Additionally, for shear, a total of 2 shear lugs plates were added to provide additional shear resistance and increment the shear/tension loads combination resistance, refer to Figures 19 & 21.

Existing foundations were also evaluated to verify the actual uplift and compression capacity. It was found that the existing foundation can resist the superimposed loads with no modifications requirements.

RETROFITTING CONSTRUCTION DETAILS

Figures 18, 19 and 20 & 21 presents the pertinent construction details to achieve the retrofitting design previously discussed.



SECTION-2

Figure 20

New Pedestal Detail

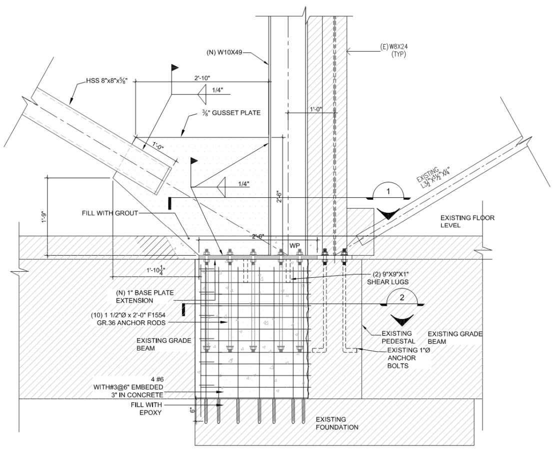


Figure 21
Typical Gusset Column to Base Connection

Recommendations for Future Work

Several other retrofit options were evaluated during the design process of this report, but the presented option was the selected one due to the short implementation time and cost savings.

One of the system were the Steel Plate Shear Walls, the Steel Plate Shear Walls (SPSW) has been used as the primary lateral load resisting system in the high-rise buildings in the recent three decades. This structural system that has spread increasingly in the word has been utilized in constructing of new buildings and also in retrofitting the existing buildings, especially in countries with seismic vulnerability such as USA and Japan. In general, steel plate shear wall system consists of steel plate wall, two boundary columns and horizontal floor beams. The steel plate wall and two boundary columns act as a vertical plate girder. The columns act as flanges of the vertical plate girder and the steel plate wall act as its web.

The other system explored was the installation of Fluid Viscous Dampers. Damping provides a large reduction in stress and deflection by dissipating energy from the structure. Spring forces are supplied by the building columns or base isolators which both support the building and deflect under load. It requires only a small amount of viscous damping force to reduce building deflection by a factor of two or three while simultaneously reducing overall column stresses. Both systems present an alternate to be used as a retrofit option in the future. The previously mentioned systems are not very common used in Puerto Rico as a retrofit option. Such evaluation is suggested as an expansion of the present work.

Conclusions and Recommendations

- Structural building members have been examined to check whether they have enough capacity to resist the loads applied to them, even when codes at time of design did not include requirements for present seismic lateral loads.
- Some seismic Code requirements did exist at the time of design of the building. Such requirements were intended to help a structure or its components to maintain resistance in the inelastic domain of response. Code provisions should include the ability to sustain large deformations to absorb energy without significant loss of capacity.
- As a proposed method of rehabilitation, we suggest the installation of stronger brace frame sections and columns to control the actual failure due to excessive buckling condition and lack of adequate strength on existing columns and piers. A total of 26 new W columns and HSS bracings were added on a total building approximate area of 73,100 square foot.
- Future studies should examine the possibility of the use of a state of the art retrofit system, such as viscous damping system or as mentioned before Steel Plate Shear Walls (SPSW).

References

[1] 2009 IBC Structural/Seismic Design Manual, Volume 3

[2] AISC. Steel Construction Manual, Thirteenth Edition. American Institute of Steel Construction: USA 2005.

[3] AISC-341-05 Seismic Design Manual, Thirteenth Edition. American Institute of Steel Construction: USA 2005.

[4] American Society of Civil Engineers, “Seismic Rehabilitation of Existing Buildings”, *ASCE/SEI Standard 41-06*, Vol. No. 1, May 2007, pp 1-100.

[5] ETABS Version 9.6 CSI.

[6] Minimum Design Loads for Buildings and Other Structures, ASCE 7-05, American Society of Civil Engineers, New York, 1995.

[7] Steel Deck Institute Design Manual, Third Edition

[8] ACI, committee 318, “Building Code Requirements for Reinforced Concrete (ACI 318-08)”. American Concrete Institute, Detroit, 2008.

[9] AISC Steel Design Guide no. 1, Base Plate and Anchor Rod Design, Second Edition, James M. fisher PhD., PE/Lawrence A. Kloiber, PE.