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SEISMIC ANALYSIS ISSUES FOR EARLY 20TH CENTURY STEEL BUILDINGS

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SUMMARY

Based on experience analyzing multiple San Francisco mid- to high-rise steel framed buildings, constructed in the early part of the 20th century, multiple issues specific to this building type are required to be addressed to appropriately model, analyze and seismically retrofit these buildings. These issues are to include, but are not limited to: incomplete documentation, plan geometry irregularities and asymmetries, composite frame behavior, riveted connection behavior, and participation of exterior masonry cladding. The authors present practical implemented methodologies for approaching these issues. Non-linear static and linear dynamic analysis results are also presented. FEMA guideline documents as well as other documents are utilized and referenced.

INTRODUCTION

Many high-rise structures in the United States were constructed in the early 20th century utilizing a construction technology of a structural steel frame encased with concrete and with an exterior unreinforced masonry infill. Included were many prominent buildings in downtown San Francisco, California. Two buildings utilizing this construction technology have been thoroughly analyzed by the authors. Both are located in downtown San Francisco, a location with significant seismic hazard, located within 10 miles of the San Andreas Fault and 15 miles of the Hayward Fault. In the past, conventional frame analyses have been performed on these building types with little success in predicting earthquake behavior. Conventional frame analysis procedures typically assume that the steel moment resisting frame and the masonry walls will act independently. The masonry infill walls are typically checked as "shear wall" elements and are expected to crack and degrade under relatively low seismic loads. The steel moment resisting frames will then provide a more flexible and yet typically stronger lateral resisting system. Such analysis will typically indicate long structural periods, deficient connections, and soft story yielding mechanisms. However, the actual earthquake performance of these buildings has been significantly better than conventional engineering analysis would predict. Building of this construction type survived the 1906 San Francisco, 1971 San Fernando, and the 1989 Loma Prieta earthquakes without collapse.

DOCUMENTATION

Due to the composite nature of the construction of this building type, documentation is particularly important. Yet, structural or architectural drawings for these older structures may no longer exist or may be incomplete. Thus tackling this problem is a daunting task. Information must be gathered from past

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structural engineering experience, review of incomplete factual documentation, site visits, non-destructive testing and interviewing of key people who have worked on or been around the building over the years. In regards to early 20th century steel buildings, a particular issue related to missing or incomplete structural knowledge is that the structure's steel frame is encased in concrete for fire protection.

Columns are generally built-up riveted sections and beams and girders are either built-up sections or rolled shapes. Usually the building has a lightly reinforced concrete slab, which is composite with the concrete encasing of the steel beams and columns. Unfortunately some of these structural items will also be hidden and inaccessible due to the architectural features of the structure. For example, concrete encased beams and girders might be hidden by a stucco ceiling. Or, piping that runs next to a column will have architectural wall pop-outs that cover both the column and piping, hiding the true dimensions of the column. Hence demolition is needed to be able to assess the sizes of these structural members.

Non-destructive probing can be utilized to expose the encased steel to confirm steel member sizes and how the built-up sections are constructed and to learn how the beam-column connections are constructed.

Building engineers and contractors who have worked on the building over the years are another source of information. For instance, one can learn how much of the clay tile partitions have been removed, what kind of remodeling and retrofit has been done on the building over the years and other pertinent information related to the building's history. Also, early 20th century steel structures are often historical structures and are often of some importance; one might be lucky and find photographs of the structure under construction.

Building Descriptions

Building A is an eleven-story structure, triangular in plan, located in downtown San Francisco. A typical floor plan is shown in Figure 1. The hollow triangular core extends from the third floor to the roof. Story height is typically sixteen feet; however, the ground floor is 20.5 feet high for a total height of 180.5 feet. Construction of the building began in 1908; it was one of the first buildings built after the 1906 San Francisco earthquake.

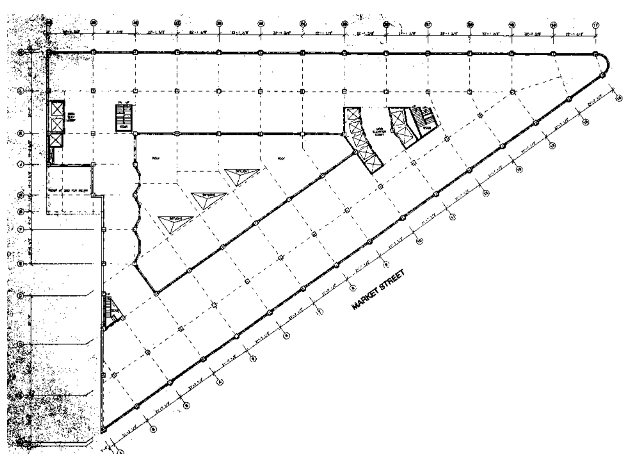

Figure 1. Building A Typical floor plan. The triangular core is open from the third floor to the roof.

The building's framing consists of built-up and rolled wide-flange steel beams and girders and built-up steel columns encased in concrete. Exterior columns are encased in concrete and masonry. Exterior spandrels run around the perimeter of the building and interior framing is oriented parallel (beams) and perpendicular (girders) to the perimeter lines. The floor diaphragm at each story consists of a five inch thick concrete slab. Exterior walls are partially in-filled with unreinforced masonry brick. The west side of the building is a concrete wall.

No structural drawings were available for review, so one interior beam-column connection and one exterior beam-column connection were exposed to determine member sizes and connection details that could be extrapolated to other connections in the building. These connections are shown in Figure 2. Sketches of these connections are shown in Figure 3.

Interior Connection - Basement

Exterior Connection – 5th Floor

Figure 2. Building A connections exposed for detailed evaluation.

First floor girders and exterior spandrels consist of built up steel I-shapes. Representative first floor girder webs are ⅜ inch thick by 36 inch deep plates. Girder flanges are comprised of four 6x4x⅝ inch angles, two angles per flange, riveted to the web plate to create the built-up section. Similar to the first floor girders, exterior spandrels are 40 inches deep and have flange angles that are 5x3-½x½ inch.

Columns generally consist of plates and angles riveted together to form an I-shape cross-section. A typical column web plate has dimensions of 14x¾ inches; however, thicknesses as small as ½ inch are used. Four angles, typically 6x6x⅝ inch, are riveted to the web plate. Variability between column sections arises from the number of plates used to reinforce the column flanges. Plate sizes range from 16x⅞ inch to 16x½ inch, and columns with zero, one, two, and three plates per flange are present.

Upper story girders are typically 24 inch deep rolled wide-flanges. Floor beams are rolled wide-flanges ranging from 15 to 20 inches deep and have flanges that are typically 5-½ to 6 inches wide.

Girders are connected to column flanges and beams and spandrels are connected to column webs. For interior columns, this means that the girders are in the strong axis plane of the column and the beams are in the weak axis plane.

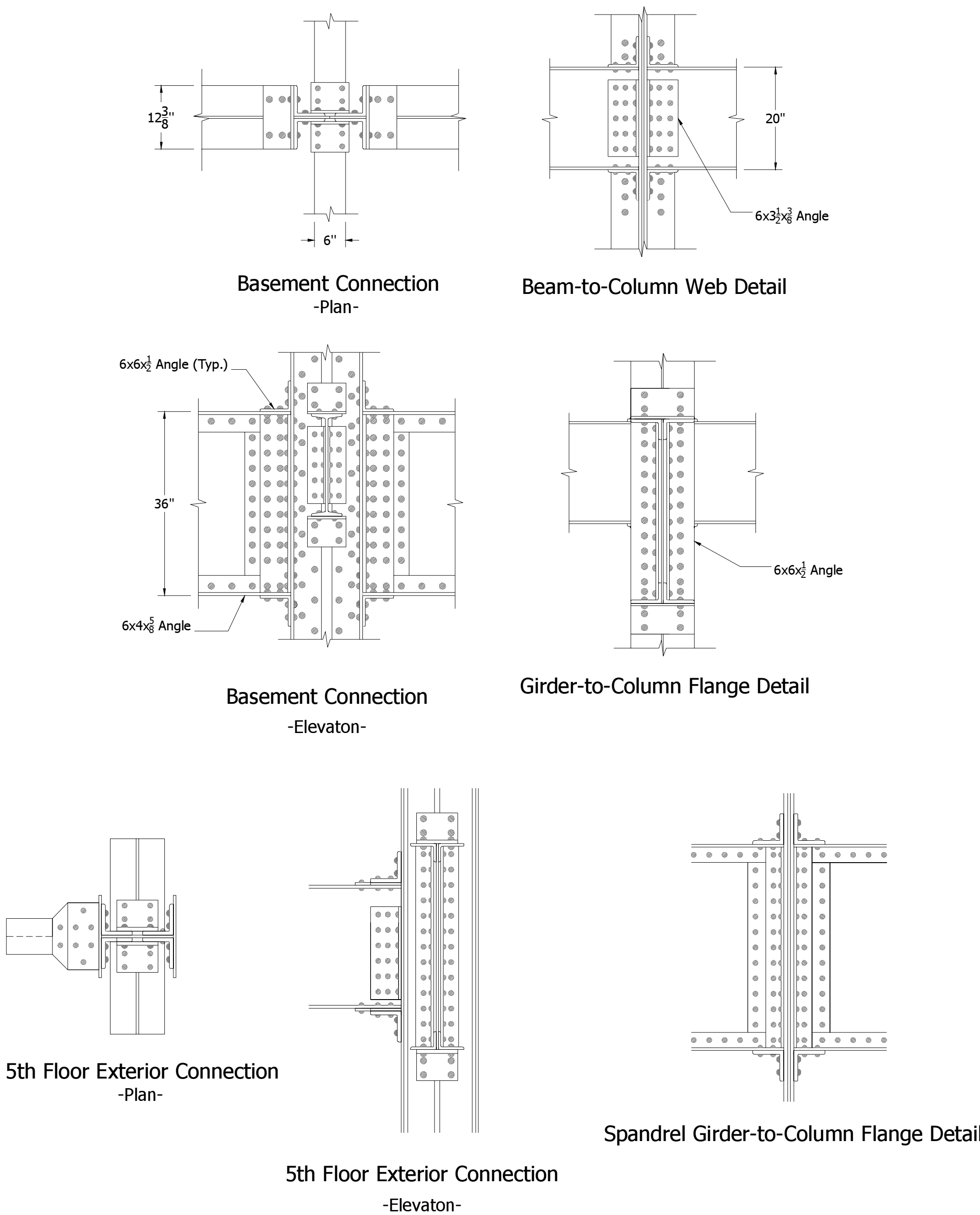

Figure 3. Building A Typical Connection Details.

Beam and columns are connected by riveted clip angle connection, which FEMA 356 (Section 5.5.3) classifies as a partially restrained moment connection. Girder-to-column and beam-to-column connections are comprised of four angles: two angles on each side of the girder (beam) web and one angle on each girder (beam) flange. The rivets used to attach the beams and girders to the columns are ¾ to ⅞ inches in diameter.

Building B

Building B is a 320,000 square foot, 24-story structure located in downtown San Francisco, California at the edge of the original shoreline, with foundations bearing on dense dune sand. The building was initially designed in 1926 and reportedly constructed during the following years. The typical floor has a “C”-shaped building footprint with overall dimensions of 160 ft x 100ft. The standard story height is 12’-0” and the total height is approximately 300 ft. See Figure 4 for the typical floor plan. The building has two subterranean levels with 1’-6” to 2’-0” thick concrete basement retaining walls which extend approximately 23’ below grade. The foundation system consists of a 3’-0” square concrete mat supported by a grid of 5’-0” wide by 8’-0” thick concrete grade beams.

The building’s existing structural system consists of interior and exterior composite steel moment resisting frames supporting 4” to 6” reinforced concrete roof and floor slabs. These frames were originally designed to carry both gravity and wind loads; however, the bulk of the building’s initial lateral stiffness and strength is provided by the exterior masonry cladding and clay tile partitions. With the subsequent or future removal of the clay tile partitions, the building’s existing lateral system will primarily consist of the exterior and interior composite steel moment resisting frames. The exterior moment frames typically consist of steel beams or trusses incased in concrete and faced with brick or terra cotta masonry, which forms part of the exterior cladding. These exterior beams are riveted to the column flanges or webs. (See Figure 5 for typical interior and exterior beam and columnconfigurations.) The exterior columns are built-up steel I-shapes and are clad similarly to the beams. The interior frames typically consist of steel beams and built-up I-shaped columns encased in concrete fireproofing. The interior steel beams are typically connected to the columns semi-rigidly using angles riveted to both the top and bottom beam flange. The large number of these partially rigid connections creates a highly redundant lateral system.

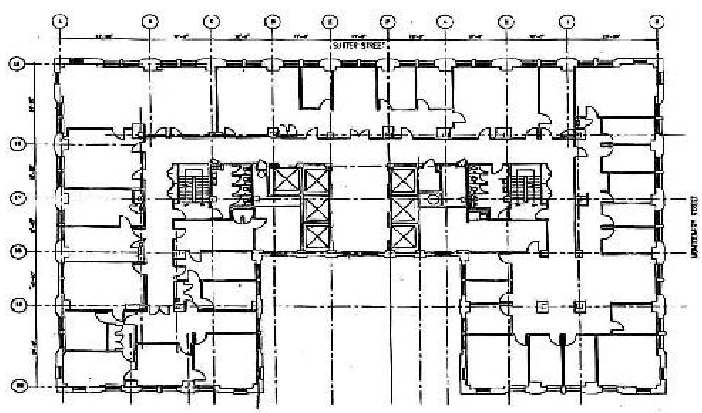

Figure 4. Typical Floor Plan

Analysis Procedures (Conventional vs. Composite Frame Analysis)

In order to better predict seismic response for this building type, a composite frame analysis procedure can be utilized. (Hamburger and Chakradeo, 1993) This procedure is based on the evidence that the masonry infill will act integrally with the steel to resist flexural demands. Cracking in the masonry is anticipated at a relatively low seismic load, resulting in masonry compression struts or braces developing within the frame as indicated in figure 6. Research data (Schneider et al,) suggests masonry strength deterioration at approximately 1% drift. At a drift greater than 1.5%, significant spalling of the masonry may occur and the bare steel frame will become the structure’s primary lateral force resisting system.

Figure 5. Typical beam and column sections

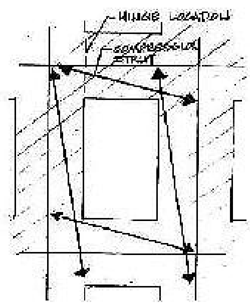

Figure 6. Masonry strut pattern for an infill masonry frame.

Summary of Structural Modeling Schemes

Both building's lateral force resisting systems were analyzed globally with an ETABS model. A three-dimensional model of Building A was developed in ETABS Nonlinear v. 7.24. A wire-frame view of the model is shown for Building B. This version of ETABS performs pushover analysis using nonlinear rigid-plastic hinges that are very similar to the backbone curves of FEMA 356. Input parameters for the ETABS model included site specific design spectra with 5% damping, the building mass, moment resisting frame locations, and composite beam and column section properties obtained from element models.

Analysis Approach

Initial evaluations of each building were performed based on FEMA 310 methodology. FEMA 310 provides for three tiers of analysis to evaluate the susceptibility of a structure to damage and determine the most effective retrofit strategies. A Tier 1 analysis consists of checklists that help identify common deficiencies that can lead to extensive damage in an earthquake. This analysis is useful for determining compliance with modern detailing standards and gives a general picture of the strengths and weaknesses of the building, but it does not explicitly address the performance of the structure under anticipated seismic loading. The Tier 2 analysis incorporates anticipated seismic loading; however, it is a linear static procedure and its scope is typically restricted to the deficiencies identified by the Tier 1 analysis. A Tier 3 analysis is a complete evaluation of the lateral system using the provisions of FEMA 356. The FEMA 356 guidelines cover calculation of seismic hazard, linear and nonlinear static and dynamic analysis procedures, and force/deformation limits for many structural components.

Connection Capacity

Capacities were calculated for the connections according to FEMA 356 Section 5.5.3.3 and the methodology of Hamburger and Chakradeo (1993). Several types of partially restrained moment connections are presented in this section of FEMA, each with there own different types of failure modes. Of these several types, Buildings A and B had the clip angle connection type, which comprises of steel angles riveted to the beam and column as discussed earlier in this paper. FEMA 356 gives the connection capacity as the minimum capacity associated with four possible failure modes. The first of the four failure modes is shear failure in the rivets connecting the beam flange and beam flange angle (Limit State I). Limit State II is the tension failure of the horizontal leg of the flange angle. Limit State III is the tension failure of the rivets attaching the vertical leg of the clip angle to the column flange. Last is Limit State IV, flexural yielding of the flange angles.

With both buildings, limit state IV was the controlling failure mode for the connection. This type of failure mode governed due to the size and thickness of the angles and the number and size of rivets used in the connections. Of these four failure modes, this failure mode is the most ductile since there is the creation of a plastic hinges in the beam flange angles. Of the four limit states, this is certainly the most desirable failure mode to have in a connection. The building is thus able to deform in a ductile manor without brittle failure mechanisms forming. Yet, P-D effects will be accentuated due to this type of ductile mode of failure. Table 1 presents a summary of the concrete thickness and dimensions used in the calculations of the connection (and member) capacities for Building A.

Clip angle connections are among the more flexible and weaker partially restrained moment connections. Since this type of connection will usually develop only a small portion of the capacity of the beam, composite action due to the concrete encasement will provide significant increase to the strength and rotational stiffness of the connection. Thus, a fuller, more accurate assessment of the connection capacity is achieved. Predicting and understanding the type of failure mode in these types of connections is vital since the type of limit state can produce significant ductility and inelastic deformation or it can produce a meager amount of ductility capacity and often a brittle failure mode. Accurate calculation of the connection capacity is difficult because of the interaction between flexure in the column flanges/flange angles and tension in the rivets through prying action in the connection. Even though these prying forces introduce additional tension in the tensile connectors, the prying force may be relieved by tensile yielding of the rivets.

The moment capacity of the connection not only is increased by the contribution due to the encasing concrete but there is also an increase in the moment capacity of the connection due to the beam web connection. The web connection develops forces, forming an additional force couple and thus adds to the capacity of the connection. However, this additional capacity typically can only be developed when there is significant rotation in the connection. Thus, limit states with limited rotational capacity will not be able to develop this additional contribution to the connection capacity. Therefore, in general, clip angle connection Limit State IV is able to develop this extra capacity, where the other limit states cannot. As past research has shown, without considering the additional contribution to the connection capacity due to the web connection, a significant underestimation of the connection capacity occurs with clip angle connections.

The Hamburger-Chakradeo method considers an alternate load path. A vertical couple (see Figure 7) is formed by the beam-column shear connection and bearing of the beam on concrete or masonry infill. Because this load path is stiffer than the horizontal couple assumed in the FEMA procedure, the connection is rigid and the horizontal couple does not resist load until loading exceeds the capacity of the vertical couple. Significant infills are present in the exterior frames and interior beam-column connections where the beam frames into the column web. In these cases, the vertical couple provides a higher capacity than the horizontal couple

Concrete Structural Element	Dimension	Source	Comments
Slab thickness at floors/roof	5″	Measured from non-destructive probing	Measured at 5th floor, assumed for other floors and roof
Concrete encasement thickness– Beams[1]	1-½″ – 3″	Measured from non-destructive probing and site visits	Measured at several locations, assumed for analysis
Concrete encasement thickness– Columns[1]	2″ – 3″	Measured from non-destructive probing	Measured at several locations, assumed for analysis
Width of slab used in composite analysis (Interior girders)[2]	Typical total width[4] ~ 93″	FEMA 356 §6.4.1.3 – Flange Construction	Actual number used in analysis averaged from site visit observations
Width of slab used in composite analysis (Interior beam – 1st floor)[1]	Typical total width[4] ~ 109″	FEMA 356 §6.4.1.3 – Flange Construction	Actual number used in analysis averaged from site visit observations
Width of slab used in composite analysis (Interior beams)[2]	Typical total width[4] ~ 93″	FEMA 356 §6.4.1.3 – Flange Construction	Actual number used in analysis averaged from site visit observations
Width of slab used in composite analysis (Exterior spandrels)[3]	Typical total width[4] ~ 60″	FEMA 356 §6.4.1.3 – Flange Construction	Actual number used in analysis averaged from site visit observations

[1] – Governing case - ½ the distance to the next web or 1/5 the span of the beam

[2] – Governing case - $8 \cdot t_{slab} + w_{beam\ web}$

[3] – Governing case - ½ the distance to the distance to the next web

[4] – Typical total width = beam (web) width + 2·the effective flange on each side of the beam (web) – as defined by FEMA 356 §6.4.1.3

Table 1 Concrete Structural Element Summary for Building A

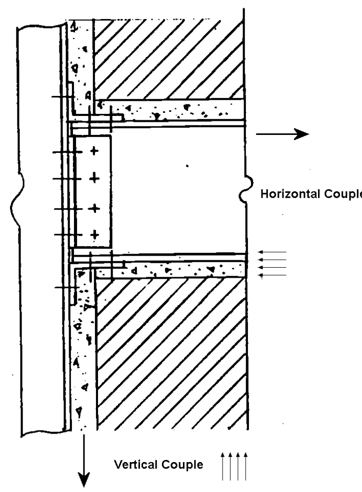

Figure 7. Typical exterior column and masonry infill; schematic depiction of horizontal and vertical resisting couples for beam-column connection.

Beams/Columns:

Beam capacity is based on the lesser of beam flexural strength and beam-column connection strength calculated. Connection strength controls in all cases, so connection failure is the only failure mechanism modeled. The connection is modeled as a nonlinear moment hinge located at the face of the column. Hinge properties are based on the recommendations of FEMA 356. Moment curvature relationships, called backbone curves in FEMA 356, for connection hinges are shown in Figure 8; allowable plastic rotations are listed in Table 2.

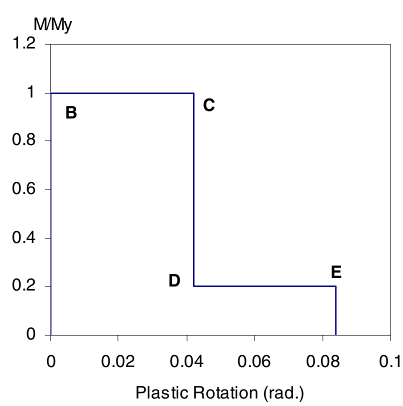

Figure 8. Beam-column connection backbone curve (FEMA 356).

Point B on the backbone curve indicates the yield point of the connection. Post yielding, the connection has no stiffness, but it is able to maintain the same level of loading as it deforms to point C. At point C, the strength of the connection is reduced to 20% of its yield capacity and the "extra" load must be redistributed to other structural elements. In its degraded state, the connection is still able to deform to point E where it loses all of its load carrying capacity and functions as a pinned connection. The other values – IO, LS, and CP – are the Immediate Occupancy, Life Safety, and Collapse Prevention thresholds. They indicate the amount of damage the connection and adjoining members suffer as the connection progresses from point B to point C.

	Performance Level				
	IO	**LS**	**CP**	**C**	**E**
Beam-Column Connection	0.010	0.025	0.035	0.042	0.084

Table 2. Allowable inelastic rotations (rad.) for hinge elements (FEMA 356).

In building B the beam elements were divided into three groups for the purpose of determining their moment capacities: beams or girders framing into a column flange, beams or girders framing into a column web and flange, and trusses framing into a column flange or web. The capacity of each beam, girder, or truss was calculated at both ends, accounting for the connection properties, the surrounding concrete and masonry (if any), and the size and orientation of the columns into which they frame.

The moment capacity of a beam element has two components: the traditional couple associated with reinforced concrete in flexure, tension in the steel and a block of concrete or masonry in compression shown in Figure 9a and a shear-bearing couple consisting of shear at the face of the column and bearing against the adjacent concrete or masonry shown in Figure 9b. These may be referred to as the horizontal and vertical moments respectively. Based on element parametric studies, (Hohbach and Lee, 1998) the vertical resisting couple, having sufficient concrete or masonry bearing length, was calculated to be significantly stiffer than the horizontal resisting couple. The total moment resisting couple was assumed to be the sum of the horizontal and vertical resisting moment if the connection is expected to yield in a ductile manner. It also was determined that a 50% rigid end off set was most appropriate.

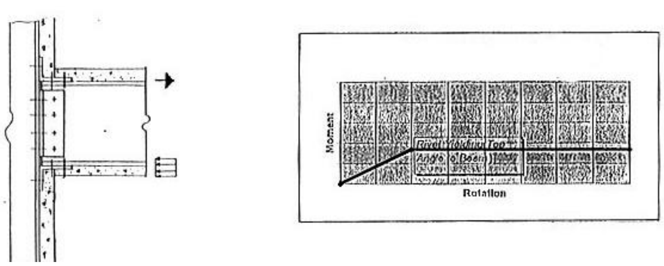

Figure 9. a.) Traditional connection couple.

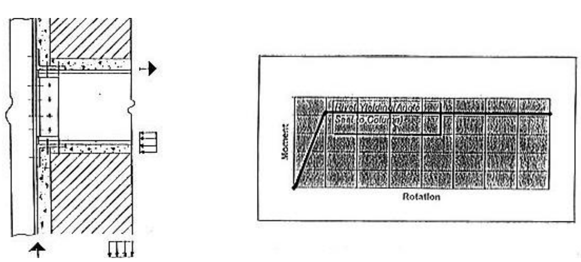

Figure 9. b.) Shear-bearing couple

Nonlinear Static Analyses

A pushover analysis of building A was performed in accordance with FEMA 356 Section 3.3.3. Loading was applied in four directions as shown in Figure 10. Corresponding inter-story drift ratios for each load case are shown in Figure 11. The resulting pushover curves are shown in Figure 12.

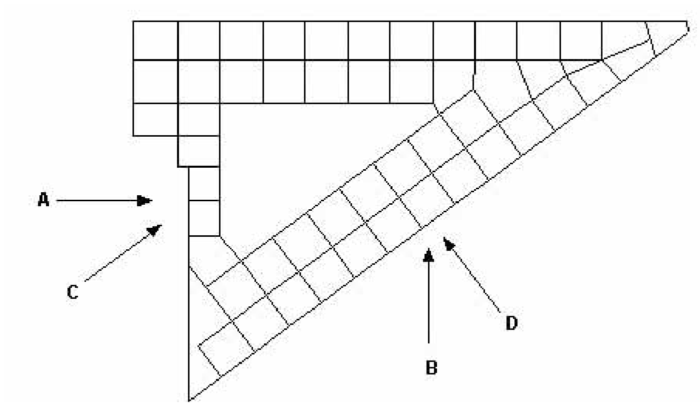

Figure 10. Loading orientation for pushover analyses.

Load cases A and C reached the target displacement of 20 inches, albeit at low levels of base shear. The building exhibited a high degree of system ductility in these directions with yielding distributed throughout the frame. Inter-story drift exceeds 1.5% in the third through sixth stories indicating that these areas sustain extensive damage. The target displacement was not reached for load orientations B and D because loading in these directions produced severe deformations at the point of the building causing the analytical model to become unstable. Note that the displacement at loss of stability is nearly four times the center-of-mass displacement and inter-story drifts exceed 3% in the middle stories. This indicates that severe damage is likely to occur in this portion of the structure as a result of the building's torsional response.

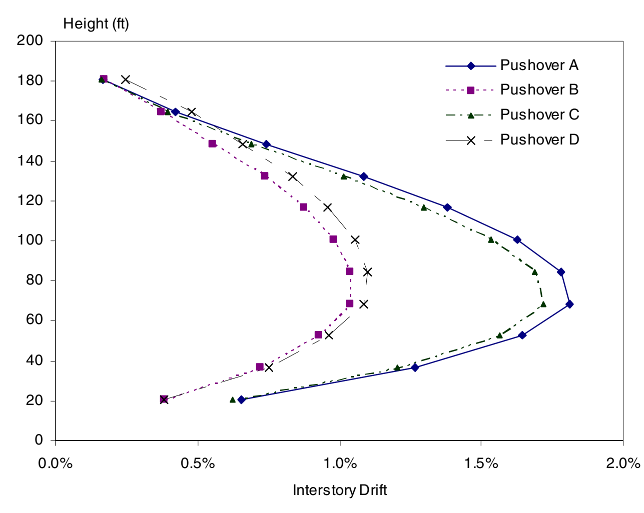

Figure 11. Inter-story drift ratios at the target displacement (orientations A and C) and instability point (orientations B and D).

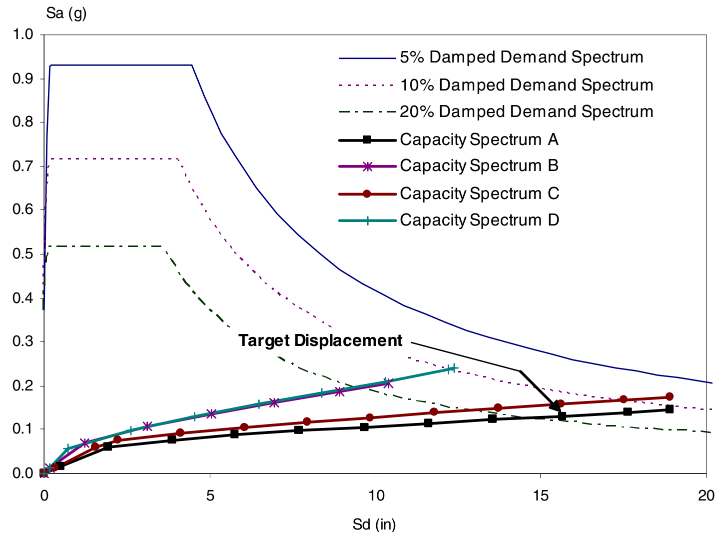

Figure 12. Demand and capacity spectra for Building A

To determine which point on the pushover curve corresponds to the BSE-1 seismic demand, the authors converted the pushover curve and the BSE-1 response spectra to the ADRS format. The results are shown as Figure 12. The pushover curve becomes the capacity spectrum and the response spectra become the demand spectra. Note that FEMA 310 suggests reducing the demand by 25% for existing structures so that retrofits are not required without ample cause. The damping level of the demand spectrum that intersects the capacity spectrum at the target displacement indicates the expected performance of the structure. Higher damping implies that the structure dissipates more energy through inelastic deformation and sustains more damage.

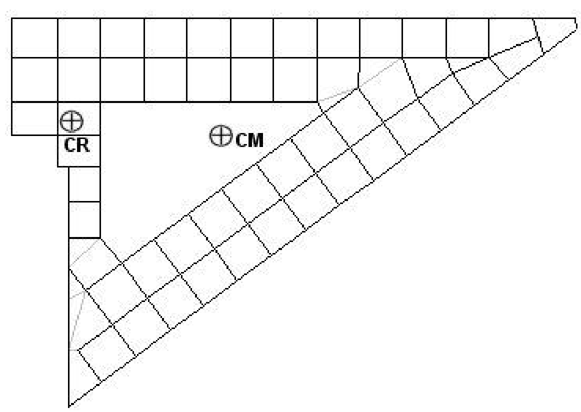

Figure 13. Plan view of the Building A showing the center-of-mass (CM) and center-of-rigidity (CR).

Load cases A and C reach the target displacement at about 20% damping. This implies a moderate to high level of damage is associated with the BSE-1 earthquake in these orientations. Cases B and D do not reach the target displacement, and loading in these orientations leads to severe damage, especially at the point of the building.

Based on the pushover results, we investigated the following additional sources of damage.

- Torsional Response: Due to its triangular shape and mostly solid west-side wall, the Building A undergoes a significant torsional response. Modal analysis indicates that the first torsional mode contains 40% of the building mass. Figure 13 shows the location of the building's center-of-mass (CM) and center-of-rigidity (CR). When loaded at the center of mass, as is the case in an earthquake, the building will rotate about its center of rigidity. This causes the "point" of the building to undergo the largest deflections and suffer extensive damage under the BSE-1 earthquake.

- Diaphragm Strength: The ability of the concrete slab diaphragm to span across the hollow core in the upper stories was investigated. The diaphragm is heavily loaded in this region because the building's hollow core reduces the area of the diaphragm available to resist shear forces. The diaphragm is a five inch thick concrete slab. Assuming that the lower bound f'c is approximately 2,500 psi, the slab has an expected shear strength of 122 psi, based on a 50% increase in f'c to convert from lower bound to expected strength. The highest stress in the slab is found at the roof level and is roughly 42 psi, so the slab is expected to perform adequately as a diaphragm.

- Overturning Forces/Column Tension Splices: Lateral loads applied in directions B and D induce tension in the columns at the "point" of the building. At the final point on the pushover curve, the tensile force in these columns is shown in excess of 1,000 kips. Tension splices are largely non-existent because the columns are designed to resist gravity and wind loads, not seismic loads. Column failure is neglected in the analytical model, so the 1,000 kip tension force is unrealistic – the columns will yield in tension well before reaching this load. As the columns yield, load will distribute to the remaining columns. Therefore, we expect additional damage will occur at the point of the building in the BSE-1 earthquake due to the inability of the columns to withstand uplift forces. The west-side wall provides enough counter-balance to prevent overturning from becoming an issue at the other side of the building.

- Column Yielding: Flexural yielding of the columns is neglected in the analytical model because the weak beam-column connections limit the moment applied to the column ends. However, large moments can be induced at the column bases where we have assumed a fixed-end condition. Evaluating the forces at the column bases at the final step of the pushover analyses (i.e., at the target displacement) showed that yielding is likely to occur in most of the columns. Based on the interaction equation for force-controlled steel columns in FEMA 356 (equation 5-12 with m = 1.0), 73 of the 92 columns do not meet the acceptance criterion. Again, the model neglects yielding in the columns, so the expected behavior of the structure will differ somewhat from the model. Once columns begin to yield, the structure will become more flexible leading to larger displacements and greater damage.

For building B the non-linear static analysis indicated that the building's global lateral system, in general, is expected to yield in a ductile manner. The existing moment resisting frames virtually in all cases meet strong column weak beam (beams & connections) requirements. The connections and the majority of the structural elements are expected to be displacement controlled. See Figures 14 and 15 for load deformation (push-over) curves for the two principal axes of loading. The pushover curves are graphed relative to a site specific earthquake with a 20% probability of exceedance in 50 years. The building's drift at the performance point is anticipated to be approximately 1%. Spalling of the exterior masonry cladding is not expected at this drift. However, the following deficiencies were noted which were deemed to require mitigation:

- Diaphragm Strength: The existing concrete diaphragm at the 5th floor does not have sufficient shear capacity to resist the in plane shears in the longitudinal direction.

- Beam Yielding: Selected exterior beams connecting large masonry piers are anticipated to yield in shear.

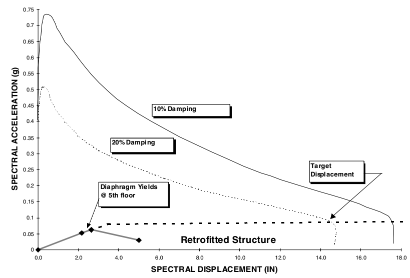

Figure 14. Demand and Capacity Spectrum for Building B – X direction

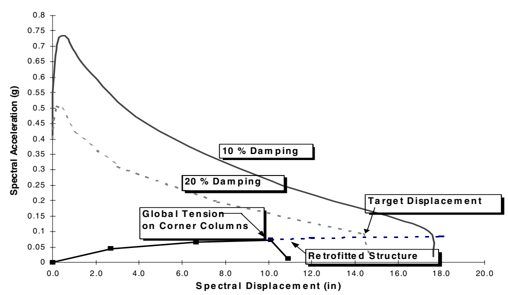

Figure 15. Demand and Capacity Spectrum for Building B – Y direction

Linear Dynamic Analyses

A linear dynamic analysis in accordance with FEMA 356 Section 3.3.2 using the BSE-1 response spectrum was performed for Building A to validate the nonlinear pushover results. The first twelve building modes were used in each direction giving a mass participation of roughly 90%.

The FEMA 356 linear dynamic analysis methodology accounts for the post-yield capacity of structural elements through m-factors that represent the allowable demand-capacity-ratio. The m-factor performs a similar role to R in the UBC; however, whereas one value of R is applied to an entire building, m-factors vary by material, element type (e.g., beam), and loading conditions.

Beam-column connections were evaluated as deformation-controlled actions with an m-factor of five from FEMA 356 Table 5-5. The beam-column joints exceeded the allowable DCR in 912 of 1,422 beams. Demand-capacity-ratios in excess of ten were observed in several locations.

As in the pushover analysis, column yielding at the beam-column joints was neglected because the connections will yield before enough force is delivered to the columns to result in column yielding. The columns were evaluated at their bases using FEMA 356 equation 5-12. This equation treats axial load as a force-controlled action – i.e., the m-factor is 1.0 – and bending about both axes as deformation-controlled actions with m-factors derived from Table 5-5. The column is acceptable if equation 5-12 evaluates to less than unity. Thirty-eight of the 92 columns were found to be unacceptable. The worst cases were at the corners of the building where equation 5-12 typically gave values larger than two. Additionally, 61 columns experienced uplift forces ranging from 10 kips to 4,700 kips.

The linear dynamic analysis procedure does not account for the yielding of structural elements or the redistribution of forces that occurs after elements yield. Forces in some members will likely be over-predicted, especially if elements are loaded beyond their yield strength. The base shear coefficients for this analysis were 40% to 50% depending on load orientation; however, the pushover results show that the building cannot exceed 20% base shear. Therefore, the load levels used in the linear dynamic analysis will never be reached and the structure will be sufficiently damaged in the BSE-1 earthquake such that some of the problems identified above may not occur.

The linear dynamic analysis for Building B demonstrated that the building has a torsional response in the east-west direction and that the existing corner columns have significant tension demands even at low lateral force levels. These high-tension demands combined with relatively weak column splices indicated a need for column splice strengthening in addition to the deficiencies identified by non-linear static analysis.

CONCLUSION

Displacement based analyses per FEMA Guidelines, incorporating the composite frame properties of these steel framed buildings with masonry infill, provide valuable information regarding the expected behavior of the building in a seismic event and can be effectively utilized to design a minimally invasive seismic retrofit, particularly in comparison to that which would be produced by a conventional code retrofit design.

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