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UNITED STATES COURTHOUSE: APPLICATION OF PERFORMANCE-BASED DESIGN TO AN ECCENTRICALLY BRACED FRAME STRUCTURE

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ABSTRACT

This paper focuses on the application of performance-based design (PBD) to a tall eccentrically braced frame (EBF) structure. The height of the structure exceeds that permitted by the International Building Code for braced frames and a dual system would typically be required. It was argued that better, more cost-effective seismic performance could be obtained using an all-braced frame scheme. Project-specific performance and analysis criteria were developed limiting overall story drift and link rotation demand. EBF link models utilized recently completed research to calibrate behavior, particularly that associated with estimating link overstrength in thick-flanged link beams. Key findings include more realistic column axial load demands than would have been predicted using current AISC 341 *Seismic Provisions* as well as improved EBF strength distribution throughout the height of the building to delay formation of story mechanisms.

Introduction

A new United States Courthouse, designed by the architectural firm Richard Meier and Partners, is a 17-story, 320-foot tall structural steel building planned to provide state-of-the-art court facilities in a dense urban environment. The building is underlain by a two-story concrete basement. Figure 1 illustrates a typical framing plan indicating the location of the seismic resisting elements as well as an elevation of the building indicating examples of the building's varying floor-to-floor heights, which range from 13.75 ft. at typical office levels to approximately 22 ft. at the courtroom floors to 25 ft. at Level 1 where the jury assembly and public lobbies are located.

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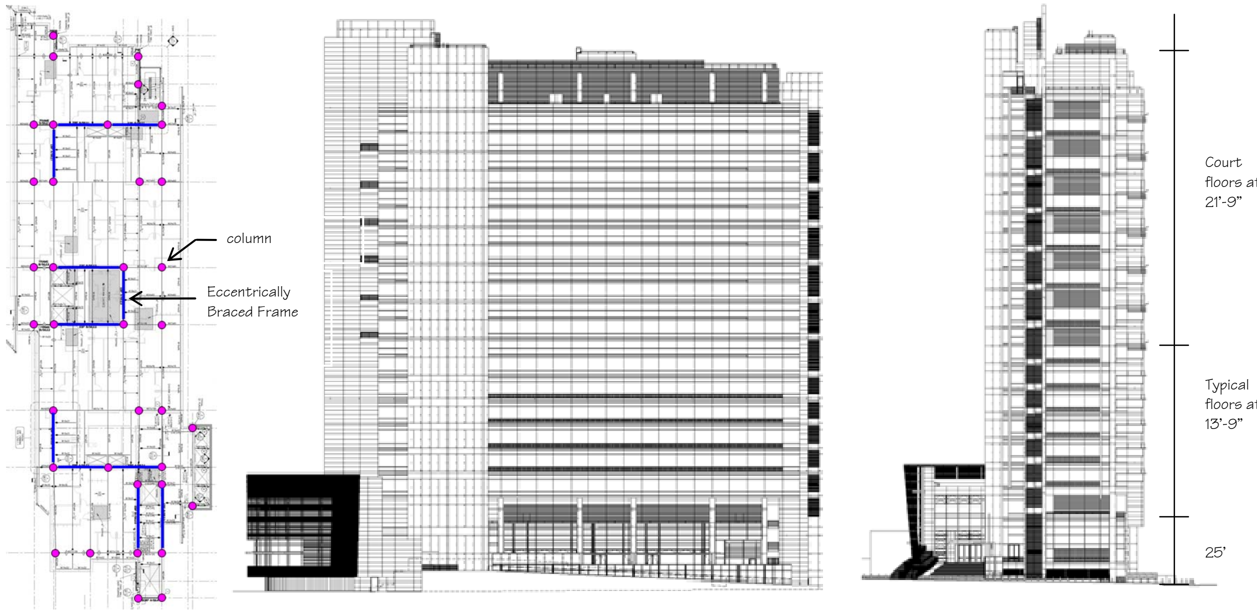

Framing Plan East Elevation North Elevation

Figure 1. Framing plan and elevations

The primary structural design challenge was posed by the high seismicity of the project site and architectural constraints imposed by the narrow building plan and required open spaces at Level 1. Selecting a functional and cost-effective lateral system was difficult. The tall floor-to-floor heights found on many floors and associated drift limits made an all-moment frame structure relatively heavy at a time when structural steel prices were increasing rapidly. The building exceeded the building code-mandated 240 ft. maximum height for all-shear wall and all-braced frame structures satisfying special seismic redundancy requirements. Although many dual systems do not have height limitations, contractors advised that mixing major subcontracting trades required for dual systems utilizing concrete shear walls and steel moment frames would result in cost premiums. The relatively high cost of steel plates compared to wide flange sections made a special steel plate dual system prohibitively expensive. In addition, trial designs of various dual systems suggested that the moment frames required to resist at least 25% of the prescribed seismic forces would not participate effectively in the overall lateral system due to their extreme flexibility relative to the stiffer elements in the dual system.

Faced with these constraints, the General Services Administration (GSA), the building's owner, was willing to consider applying performance-based design criteria that would demonstrate that an all-braced frame lateral system exceeding the prescriptive building code height limits would be an effective seismic system. Schemes using special concentrically braced frames were ruled out due to unresolved concerns regarding system performance. Buckling-restrained braced frames, although judged to be potentially good performers, were not pursued due to the GSA's concerns about limited competition in what was, at that time, an overheated construction market. In the end, the eccentrically brace frame (EBF) was selected for further consideration.

One of the primary reasons for using performance-based seismic design is recognition that typical building code requirements may not be well suited for atypical structures, and it is believed that project-specific design criteria will result in a better performing design. In other cases, performance-based design is used to establish special seismic performance targets that are, at best, implicitly defined in building codes. Overall, it is believed that capital invested in a building designed using performance-based criteria will be more effectively deployed than is likely in a conventionally designed building. In the present case, the building code (i.e. the 2003 *International Building Code*), except for the height limit, was used to develop the basic structural design, which was then verified using the project-specific criteria. Since published performance-based design criteria were not available, this paper discusses the criteria used in this project and the results of the design verification effort.

Building System Description

The gravity system of the superstructure is composed of structural steel rolled wide-flange shapes supporting metal deck and concrete fill. Floor-to-floor separation requirements for Type I construction, as well as the need to limit the building's seismic mass, suggested 3-¼" of lightweight concrete over 3" metal deck as the floor system. Repetitive steel infill beams span from 24 ft. to 44 ft. at approximately 8 to 11 ft. on center. Infill beams are carried by structural steel girders spanning up to 45 ft. Spans are relatively large compared to traditional buildings to satisfy the desire for "column-free" spaces within the courtrooms and jury assembly areas. Gravity beams are 24" deep and gravity girders are approximately 27" deep.

Security considerations require that none of the essential parts of the building's resistance to catastrophic blast loads and progressive collapse, including columns, girders, beams, and the main lateral resistance system, be located with a predetermined blast setback zone. This zone is oriented in both the horizontal and vertical planes. Girders are cantilevered by almost 15 ft. in many areas to satisfy this intent. In most instances, the girders are moment connected to the columns to satisfy progressive collapse requirements.

Architectural considerations, balanced by a need to provide some structural symmetry and an appropriate number of frame bays to limit member sizes and foundation demands, dictated the location of the EBF. Figure 1 indicates the locations of EBF bracing elements for the Tower. The EBF beams are moment connected to the EBF columns. While the EBF braces do not extend below the Plaza Level, the EBF columns continue through to the mat foundation. Concrete shear walls are used below the Plaza Level to transfer the lateral loads to the foundations. The link lengths range from 4 to 6 ft., which conforms to a shear link.

Based on an evaluation of alternative link limit states, the links were proportioned to be consistent with a shear link. As a result, link lengths were set between four and six feet depending on the frame span. EBF beams are typically W27x194 to W27x258. The largest EBF column is a W14x730 with some of these columns plated to satisfy load demands. The EBF braces range from W14x159 to W14x211.

Analysis Models

Basic Modeling Parameters

A modified performance-based design was pursued on this project. First, a code-based design was performed for the all EBF lateral system and then the resulting design was evaluated using a nonlinear time history analysis. The design was modified as required to satisfy the demands from the time history analysis based on the requirements of the project-specific performance-based design criteria.

Progressive collapse considerations required the beam-to-column connections to be moment connected. This allowed the use of the response modification coefficient R of 8.0 per Table 1617.6.2 of the 2003 IBC. The complexity of the structure required a linear dynamic analysis procedure incorporating a site-specific response spectrum. The site class was determined to be C and the mapped spectral response parameters, S_s and S_1 were determined to be 1.52g and 0.69g.

The typical floor masses varied from 5500 kips at Level 2 to 2859 kips at the typical court levels. The total superstructure mass was estimated to be 48,000 kips.

The static analysis suggested the potential for torsional irregularity which required the amplification of the accidental eccentricity for the static analysis. The static analysis was primarily used to determine the mass distribution and the building code-based design base shear. The "code-based" base shear, V, was estimated to be 2200 kips. The story force distribution and the frame force distribution were determined based on the response spectrum analysis method. The frame forces were then modified based on the provisions of Section 9.5.7.3 of the ASCE 7-02. These forces were used to design the EBF link, although in most instances, the sizes of the EBF beams were dictated by the progressive collapse considerations. The expected capacity of the EBF link was then used to design the EBF beam outside the link, the EBF braces, and the EBF columns.

An interstory drift check according to the provisions of Section 9.5.2.8 of the ASCE 7-02 was then performed. The code-based limit of 2% of the story height was used to verify conformance.

The structure at this stage was intended to be fully compliant with the provisions of the 2003 IBC except for the height limit restriction. A nonlinear dynamic procedure was used to confirm the performance of the structure. Detailed analysis criteria, discussed below, were prepared and submitted to the GSA and a structural peer reviewer for review and approval. A nonlinear time history analysis was suggested with pairs of scaled time histories. Based on project scheduling constraints, the design confirmation was based on a nonlinear time history analysis using the maximum demand resulting from three time histories rather than the more common average of seven (or more) time histories.

Ground Motion Selection

The geotechnical engineer, after taking due consideration of the site seismicity, selected three different earthquake time records. The first was based on the Landers Earthquake as measured at the Joshua Tree Station. The second was based on the Chi-Chi Earthquake as measured at the TCU087 station. The third was based on the Northridge Earthquake as measured at the Sylmar-Olive View Station. The original records were scaled for the site seismicity and the building fundamental period by the geotechnical engineer and were provided for the structural analysis. Each earthquake time history was selected to reflect ground motion characteristics that might have a significant influence on the dynamic response of the building. The Landers Earthquake represented a sustained duration earthquake. The Northridge Earthquake represented an earthquake with a large pulse in the beginning of the record. The Chi-Chi Earthquake represented an intermediate earthquake with sustained acceleration during the first part of the time history.

The records corresponded to an earthquake with an average return period of 2475 years or 2% probability of exceedance in 50 years. This level of earthquake is identified as the maximum considered earthquake (MCE) in the 2003 IBC. The records were also scaled to simulate a Design Basis Earthquake (DBE), which was approximately equal to an earthquake with an average return period of 475 years or 10% probability of exceedance in 50 years. Each record was divided into a pair of time histories representing the "fault normal" direction and the "fault parallel" components compared to the major axes of the building. The "fault normal" direction was determined to be 110 degrees from the longitudinal direction of the building.

Project-Specific Performance-Based Criteria

The project-specific performance-base criteria specified limits on overall building response and EBF element demands as well as analytical modeling considerations. Some of the criteria are summarized below:

Maximum Link Beam Rotation: The maximum link beam rotation at the DBE and MCE level earthquakes was limited to no more than 0.08 and 0.12 radian, respectively. For the DBE limit, the intent was to use the 2/3 scaling factor as noted in the 2003 IBC to convert parameters from the MCE to a DBE level, although this was not verified explicitly. The MCE limit was based on test data and is further explained in subsequent sections.

Maximum Interstory Drift: The interstory drift at the DBE level was limited to no more than 2% of the story height. The interstory drifts were checked at extreme points as required by ASCE 7 because the building was categorized as "torsionally irregular". This drift limit was also translated to 3% of the story height for the nonlinear time history analysis, the basis for which is the 2/3 scaling factor of the 2003 IBC. For the linear analysis, the drifts obtained were compared to the 2% limit. For the nonlinear time history analysis, the time histories provided were at the MCE level, and the corresponding interstory drifts were compared to the 3% limit.

Beam-to-Colum Rotation: The rotation demands at the beam-to-column connection and column base was limited to no more than 1% at the MCE level.

Column Performance: The column was to be essentially elastic at the DBE level.

EBF Brace Performance: The brace axial strength was to be greater than the maximum demand generated by the nonlinear analysis. Expected steel yield strength and a resistance factor of 1.0 was used when evaluating the beam capacity.

Beam-Outside-the-Link: The strength of the beam-outside-the-link was to be greater than the maximum demands generated by the nonlinear time history analysis. Expected steel yield strength and a resistance factor of 1.0 was used when evaluating the beam capacity.

Shear Link Modeling: The link beams were modeled as a shear hinge although the modeling parameters for this key structural element required careful selection. After a review of available research and considering the suggested loading protocol for link-to-column connections published in AISC 341-05, Section S6.3, the recommended link model was based on the results in Richards (2006) and Taichiro (2006). This model was judged to be a reasonable representation of the link behavior because the plastic moment capacity to plastic shear capacity ratio was similar to the beams being used in this project. Based on the cited research and the use of links with relatively thick flanges, the link overstrength was modeled using an approximate upper limit of 1.5 rather than the value of 1.25 recommended in AISC 341-05. This resulted in increased demands on all other members in the EBF.

CSI PERFORM 3D, version 4.0.3 was used to model the structure. The sizes determined from the design based on the linear analysis were used in the initial nonlinear analysis model. A 3-D model of the gravity columns and all EBF elements was developed. Masses were lumped at each floor at the center of mass. The masses of the mechanical penthouse were lumped to the roof. These masses were directly derived from the linear elastic structural model. The center of mass obtained from this model was used to locate the masses in the PERFORM 3D program

The EBF brace, EBF beam-outside-the-link and the column elements are required to be elastic under mechanism loads. The columns and the beam-outside-the-link were modeled as elastic elements to conform to the code intent of confining nearly all of the inelastic behavior in the EBF link at the DBE level. The beam-outside-the-link was modeled as a beam column with strengths based on its unbraced length and section size and with fixity at the beam column joint. The brace was modeled as an axial-only element with moment release at each end. The column was modeled as a beam column element with strengths based on its unbraced length and section size.

P-delta effects were considered in the model. Gravity and lateral columns were modeled in PERFORM. Gravity columns were set to be bar elements with tributary gravity loads being applied at each level. The damping was set to be 5% for all modes, as is customary for steel buildings.

Nonlinear Dynamic Results

After the design based on the linear elastic code-based forces was completed, the nonlinear analysis results were then considered. The link rotation and the interstory drifts were the major design verification items in the analysis. Based on the results reported by Richards (2006) and the project-specific performance-based design criteria, the link rotation was limited to

12% for the MCE level earthquake, and the interstory drift was to be limited to 3% at the MCE level earthquake.

Except as noted below, the EBF link rotations for all three earthquakes at all time instances and at all levels were found to be less than 12%. There were two EBF link beams where the link rotation for a short instant of time reached 12.25%, which is within 2.1% of the 12% limit. The peer reviewer concluded that this slight deviation from the acceptance criteria was not significant because it was isolated in nature. A figure presenting the EBF link rotations for the Sylmar time history, which generally represented the most punishing earthquake, is presented in Figure 2.

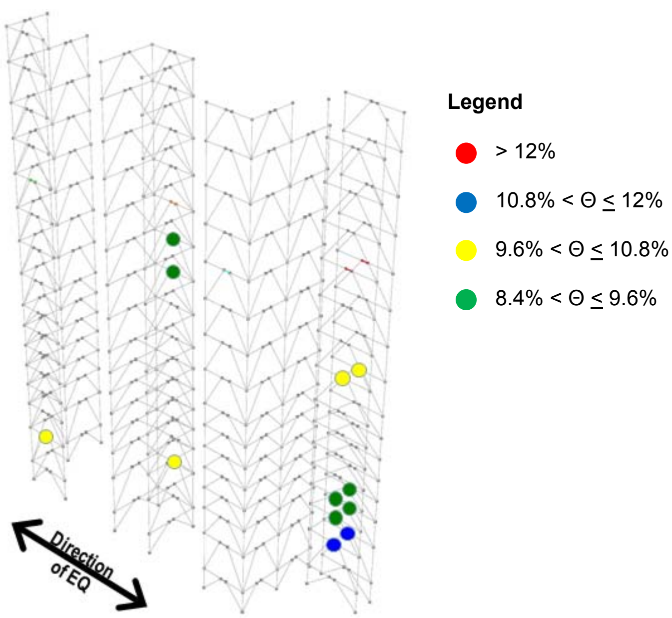

Figure 2. Link beam limit state for Sylmar Earthquake

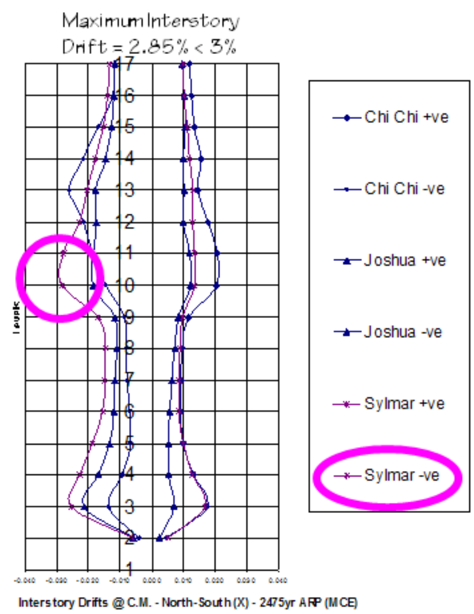

Figure 3. Interstory drift in north-south direction

The interstory drifts determined from the MCE were well within the limit of 3% of the story height. Figure 3 presents a representative plot of interstory drifts for the three sets of time histories. The Sylmar Earthquake produced the largest interstory drifts.

The beam-outside-the-link was designed for the effects of combined flexure and axial loads generated due to the mechanism loads. The mechanism load in AISC 341-05 is defined as $1.1R_yV_n$ for the beam-outside-the-link, which is considered to be a reasonable estimate demand generated by the link overstrength. Since the beam sizes were governed by non-seismic considerations (i.e. progressive collapse), the "amplified" maximum possible mechanism loads are assumed to be the one generated from the time history analysis, which corresponds to a beam link strength calculated assuming the link overstrength to be based on $1.5F_y$. This is consistent with the assumptions from the link beam model derived from the tests reported in Richards (2006), as discussed earlier. When calculating the capacity of the beam-outside-the-link, the AISC 341-05 allows the use of the realistic strength of the beam section. Relating $1.1R_yV_n$ to $1.5F_y$, a revised R_y of 1.32 was obtained.

This more realistic overstrength was assumed when calculating the capacity of the beam-outside-the-link. In addition, the ϕ -factor in such instances was assumed to be 1.0 to compensate for the extreme magnitude of the assumed loads. The load factor used for the design of the beam outside the link was 1.0D + 0.25LL + E_{mech}, where E_{mech} corresponded to the forces generated assuming the yield strength was 50% over the specified yield strength (consistent with the link beam model assumed). The forces generated from the nonlinear time history analysis cannot exceed the mechanism strength inasmuch as the maximum force generated in each link cannot be greater than that equal to $1.5V_n$ (assuming that the maximum link overstrength has been accurately estimated). The sizes of the beam-outside-the-link as determined from the linear analysis were adequate to satisfy the criteria set forth in the performance criteria based on the nonlinear analysis loads.

The EBF column was designed considering two different axial loads. The maximum axial load represents the instantaneous peak load. It is a unique event. There also exists a band of axial loads with roughly similar magnitudes that occur multiple times during the time history secondary peaks. Figure 4 illustrates the maximum peak and the set of secondary peaks, with the maximum value in that set of second peaks identified.

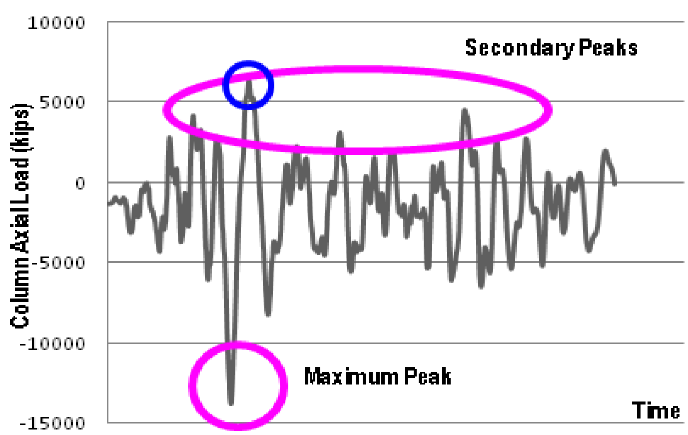

Figure 4. Representative column axial load time history

The axial load associated with the first peak is assumed to be transient in nature and, therefore, a value of k = 0.8, the effective length factor, and a resistance factor ϕ = 1.0, was assumed in the design of the column. The magnitude of the axial load, generated from the second set of peaks, is used for the conventional design of the column elements with k = 1.0 and ϕ = 0.85. The final column size was based on the maximum column size generated considering the two loads and design criteria described above. This approach is reasonable since the magnitude of the column load depends on the number of links that yield simultaneously up the height of the building and considers the maximum peak load as well as the magnitude of column loads that occur repeatedly.

The original column design was based on the demands generated by using a linear analysis and the requirements of AISC 341-05, which assumes that all links above column under consideration have reached their maximum strength. This represents a significant demand, particularly at the lower levels of the building. As a result of the inelastic time history analysis, most of the column sizes did not need to be increased suggesting that the AISC EBF column design requirements are relatively conservative. Some of the column sizes were increased compared to the sizes determined from the linear analysis. This was due to the higher overstrength assumed for the EBF link beams in the nonlinear analysis (i.e. an overstrength factor of 1.5 in the nonlinear analysis versus 1.1 based on the requirements of AISC 341-05), which generated higher demands on a few columns at the lower levels of the structure.

The EBF brace was also designed for mechanism loads. Since the EBF brace is driven by the link strength at each level, the two step approach was not taken. The EBF brace is designed as a column section conforming to the requirements of Section E of the AISC 360-05. The sizes of the EBF brace as determined from the linear analysis were adequate to satisfy the criteria set forth in the performance criteria based on the nonlinear analysis loads.

During the peer review process, a question was raised about the incidental impact of the short moment connected beams adjacent to several of the EBF columns on the behavior of the lateral system. The beams are moment connected to satisfy progressive collapse requirements and were not explicitly intended to participate in the lateral system but would certainly be exposed to the same building deformation as the rest of the lateral system. Would the short span beams be forced to resist unexpectedly high loads due to deformation compatibility? A 3-D model was created by adding the extra bay of moment connected beams and columns at the end of the three EBF.

An analysis of the condition suggested that there was a small redistribution of axial loads in the EBF column to the short-bay moment frame column. The effect on all other columns was insignificant. The EBF column design was not changed to account for the reduced axial loads since the moment connected beams and columns are not part of the lateral force resisting system. The column that is not part of the EBF was designed for the full axial loads resulting from gravity, progressive collapse, and incidental seismic demands. The impact of this short frame on the EBF link was also evaluated. The EBF link rotations were also within the specified limit except at the links on Level 13 for two frames for the Chi Chi Earthquake. The demand was approximately 6% over the 12% limit (compared to 2.1% over the 12% limit for the base model) for link rotation. The short bay beams and columns were designed and detailed based on the requirements of the AISC *Seismic Provisions*.

A study was also performed to determine the effects of providing fixity for the brace connection. A 3-D model which incorporated this change was created and the nonlinear time history analysis was rerun. It was determined that the link beam rotations remained essentially the same despite the changed brace boundary conditions. The moment demand of the beam outside the link was reduced slightly. Since demand on the beam-outside-the-link is highest for the pinned brace condition, the design was not modified.

The EBF shear force at the Plaza Level was transferred to the concrete diaphragm above the subterranean levels using reinforcing welded couplers to the column flange. The design force for each connection was $\Omega_0 V_{\text{base shear}}$, which was on the order of 500 kips for a single bay EBF and about twice this amount for a double bay EBF.

As noted earlier, the steel EBF column was extended to the top of the 8-foot thick mat foundation to simplify the force transfer details that would have been required had the steel column been stopped at the top of a heavily reinforced concrete column. The steel column is also encased in a 42” x 42” concrete column to accommodate the reinforcement from the beams and girders framing into the columns at the Plaza Level and the subterranean levels. The compression load is resisted by bearing on the concrete mat which determined the plan dimensions of the base plate. The tension loads are resisted by providing anchor rods that extend into the footing. An additional mat of reinforcing is provided in the mat foundation to help transfer the uplift force.

Conclusions and Summary

The performance of EBF systems for a building taller than 240 ft. was found to be satisfactory based on an analysis of the seismic response due to three sets of time histories. Progressive collapse considerations governed the sizes of many beams and columns. The importance of not over designing EBF link beams was clearly illustrated by the nonlinear time history analysis because the size of braces and columns dominated by the size of the EBF link beams required to satisfy progressive collapse requirements. The drift limits were easily satisfied in spite of the large story heights. The link rotations were found to be within the limits exhibited by experimental studies.

Acknowledgements

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