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Performance-Based Seismic Design of Concrete Buildings: State of the Practice

SP-339

Editors:

Jeff Dragovich, Mary Beth Hueste, Brian Kehoe, Insung Kim



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Performance-Based Seismic Design of Concrete Buildings: State of the Practice

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PREFACE

Performance-Based Seismic Design of Concrete Buildings: State of the Practice

Performance-Based Seismic Design (PBSD) of reinforced concrete buildings has rapidly become a widely used alternative to the prescriptive requirements of building code requirements for seismic design. The use of PBSD for new construction is expanding, as evidenced by the design guidelines that are available and the stock of building projects completed using this approach. In support of this, the mission of ACI Committee 374, Performance-Based Seismic Design of Concrete Buildings, is to “Develop and report information on performance-based seismic analysis and design of concrete buildings.”

During the ACI Concrete Convention, October 15-19, 2017, in Anaheim, CA, Committee 374 sponsored three technical sessions titled “Performance-Based Seismic Design of Concrete Buildings: State of the Practice.” The sessions presented the state of practice for the PBSD of reinforced concrete buildings. These presentations brought together the implementation of PBSD through state-of-the-art project examples, analysis observations, design guidelines, and research that supports PBSD.

This special publication reflects the presentations in Anaheim. Consistent with the presentation order at the special sessions in Anaheim, the papers in this special publication are ordered in four broad categories: state-of-the-art project examples (papers 1-5), lateral system demands (papers 6-8), design guidelines (papers 9-10), and research and observed behavior (papers 11-13).

On behalf of Committee 374, we wish to thank each of the authors for sharing their experience and expertise with the session attendees and for their contributions to this special publication.

Editors

Jeff Dragovich
Mary Beth Hueste
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Performance-Based Seismic Design of the Tocumen Airport Terminal 2

Xiaonian Duan, Andrea Soligon, Jeng Neo, and Anindya Dutta

Synopsis: The new Terminal 2 at the Tocumen International Airport in Panama, currently essentially completed, will increase the airport's capacity to 25 million passengers per year. It has a doubly curved steel roof supported on reinforced concrete columns. The gravity force-resisting systems in the superstructure include long span precast and prestressed double tee decks, topped with cast-in-place concrete diaphragms and supported on a combination of unbonded post-tensioned girders and special reinforced concrete moment frame beams. The seismic force-resisting system includes special reinforced concrete moment frames and perimeter columns, special reinforced concrete shear walls and diaphragms, all detailed in accordance with ACI 318. Located in a region of moderately high seismic hazard, the building is classified as an essential facility and requires a non-conventional seismic design approach to maintain operational continuity and to protect life. Adopting the performance-based seismic design methodology and the capacity design principle, the structural engineering team designed an innovative reinforcement detail for developing ductile hinges at the top of the reinforced concrete columns to protect the structural steel roof which is designed to remain essentially elastic under MCE shaking. The structural engineering team's design has been reviewed by internationally recognised experts and three independent peer review teams.

Keywords: nonlinear pushover analysis, nonlinear response history analysis, performance assessment, performance-based seismic design, Tocumen Terminal 2

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INTRODUCTION

Located 24 km (15 miles) east of Panama City, the capital city of the Republic of Panama, Tocumen International Airport is one of the busiest airports in Central America. The new Terminal 2 (T2), currently with construction essentially completed as shown in Fig. 1 and partially operating, will add 20 gates to those of the existing terminal to achieve an estimated total capacity of 25 million passengers per year and will establish the airport as a new hub for the Americas.

Following an international competition and based on the design concept proposed by the winning architectural design firm, a global construction firm was awarded the design-build contract in 2012 to deliver the new terminal. The design firm was subsequently retained to provide full structural engineering services, to be delivered in an integrated manner with those of the in-house architectural and MEP teams.

The new terminal, with a gross area of 116,000 m² (1,247,000 ft²), has a curvilinear shape 660 m (2,174 ft) long by up to 162 m (531 ft) wide on plan and is up to 26 m (85 ft) tall. Arrivals and baggage handling are located on the first (grade) level, departures on the second. A third and fourth level, in the central part of the terminal, provide accommodation for central plant rooms, food courts, airline lounges and offices.

The terminal is divided into five zones along its length, each with its own independent structure from foundations to the roof, via four seismic joints in order to mitigate effects arising from thermal expansion and seismic relative displacements, as shown in Fig. 2.

Among the numerous challenges which are inherent in large scale projects of similar complex occupancies, the major challenges for this project were firstly the fast-track schedule and secondly the complex geometry that led to non-conventional lateral force-resisting systems not listed in Table 12.2-1 of ASCE 7–10¹ and connections not prequalified in accordance with AISC 358–10². The first major challenge was overcome through close collaboration between the integrated multidisciplinary architectural, structural and MEP engineering design team, co-located in the same design office, and the contractor. Structural engineers from the design team were also present on site throughout the two parallel and overlapping processes of design and construction to co-ordinate and assist the contractor with construction administration. This close collaboration enabled construction of the foundations to start only 5 months after project kick-off. The second major challenge was overcome through the adoption of the performance-based seismic design methodology by the structural engineering team.

This paper focuses on the performance-based seismic design and analysis of the Terminal 2 building. The need for a performance-based seismic design methodology as an alternative route to the conventional code-prescriptive approach

is presented first, followed by the seismic performance objectives and the performance-based seismic design and analysis procedure and analysis results. Finally, the peer review process is briefly discussed.



Figure 1— Aerial view of the new Terminal 2 near completion

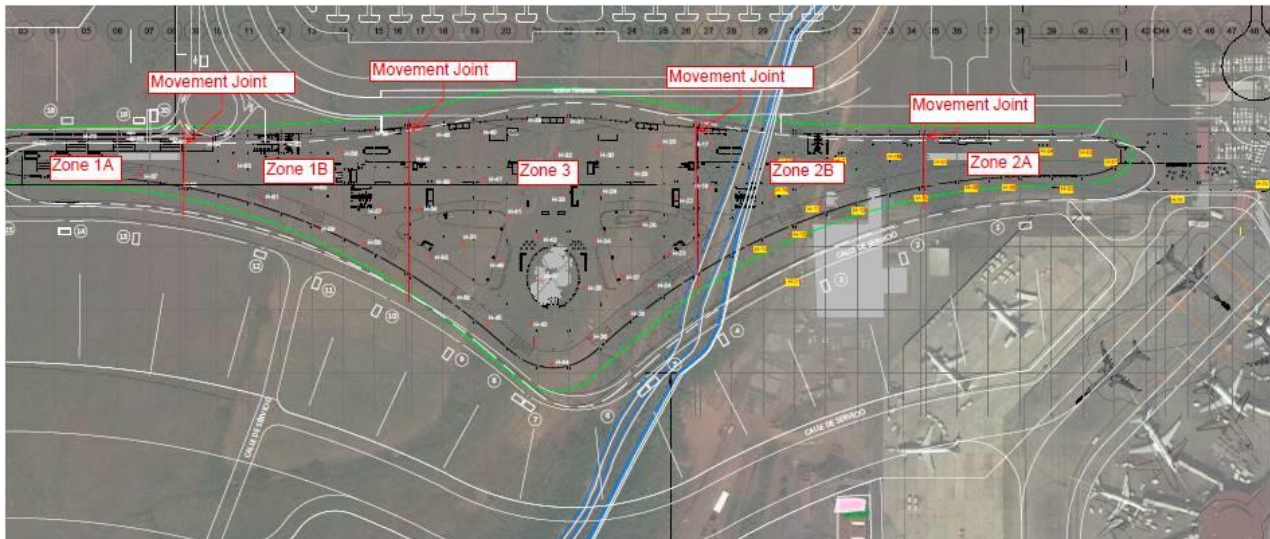


Figure 2— Structural zones and seismic joints of the new Terminal 2

THE STRUCTURAL SYSTEMS

Zones 1A and 2A

Zone 1A is composed of two independent structures - a single-story concrete superstructure 6 m (20 ft) tall and 115 m (377 ft) long and a 16 m (52 ft) tall steel roof structure supported on perimeter concrete columns which span from foundations to roof without any interaction with the concrete superstructure. The lateral force-resisting system for the superstructure is reinforced concrete moment frames in two orthogonal directions. The steel roof structure and the perimeter concrete columns also act as moment frames in two orthogonal directions but in the transverse direction the curved steel beams are not aligned with the concrete columns so as to achieve the architectural design intent shown in Fig. 4. The roof structure as such is a non-conventional lateral force-resisting system not listed in Table 12.2-1 of ASCE 7-10¹ and is not detailed with prequalified steel connections in accordance with AISC 358-10².

The structures of Zone 2A, at the opposite end of the terminal, are similar to those of Zone 1A except that a partial mezzanine extends above the second level.

Fig. 3 illustrates the structural systems of Zone 2A. The perimeter columns and the moment frames in Zones 1A, 2A and all the other zones are detailed to conform to the requirements for special reinforced concrete moment frames in accordance with Chapter 21 of ACI 318–11³.

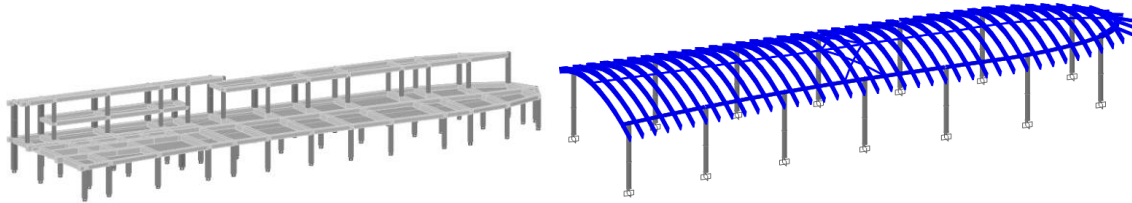


Figure 3— Structural systems in Zone 2A



Figure 4— Architectural rendering of an internal view of the new Terminal 2

Zones 1B and 2B

Zone 1B consists of a five-story 23 m (75 ft) tall and 129 m (423 ft) long structure. While the perimeter columns span between the foundation and the roof without any connections with the interior structural elements similar to those in Zones 1A and 2A, selected interior columns are extended upwards to support the roof in order to reduce the span of the roof secondary steel beams along the transverse direction. Connecting these selected interior columns are steel primary beams running along the longitudinal direction similar to the roof perimeter primary beams framing to the perimeter columns. Therefore, unlike Zones 1A and 2A, Zone 1B consists of a single structure as shown in Fig. 5. Unbonded post-tensioned girders are provided at the departure level in the transverse direction at bays with spans exceeding 18 m (59 ft). Reinforced concrete moment frames, together with the reinforced concrete perimeter columns, form the seismic force-resisting system beneath the roof. To achieve the architectural design intent, roof beams along the transverse direction are not framed directly to the concrete columns, as shown in Figs. 4, 5 and 8. The structure of Zone 2B is similar to that of Zone 1B.

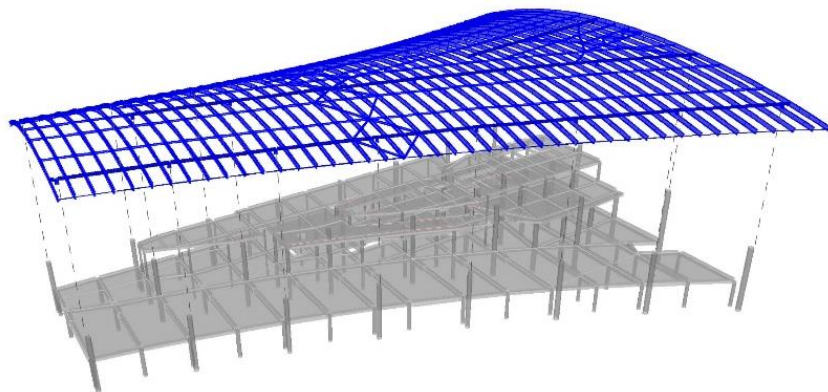


Figure 5— Structural systems in Zones 1B /2B (Exploded view of steel roof and concrete superstructure)

Zone 3

Zone 3, the largest of the five zones, is a single structure of five-stories, 26 m (85 ft) tall and 165 m (541 ft) long by up to 165 m (541 ft) wide. Reinforced concrete shear walls and moment frames, together with the perimeter columns, form the seismic force-resisting system beneath the roof. Shear walls are not extended upwards to support the roof. However, similar to Zones 1B and 2B, selected interior concrete columns are extended upwards to support the roof in order to reduce the span lengths of the roof secondary beams along the transverse direction. Interior roof primary beams are introduced along the longitudinal direction to align with these interior columns. The shear walls are detailed as special reinforced concrete shear walls, while the moment frames and the perimeter columns are detailed to conform to the requirements for special reinforced concrete moment frames in accordance with Chapter 21 of ACI 318–11³.

The original design featured a full height atrium of an elliptical shape on plan, 41 m (134 ft) long by 32 m (105 ft) wide with a tropical garden at the center of the terminal as shown in Fig. 6. This has since been replaced by an independent retail accommodation structure within the atrium void. Shown in Fig. 7 is the structural system of Zone 3. As in all other zones, the roof secondary beams in the transverse direction do not frame directly to the roof columns. Working collaboratively with the design firm of the first three authors of this paper, the California-based engineering design firm of the last author performed the nonlinear response history analyses and implemented the Construction Documents of the steel roof of the Zone 3 structure.



Figure 6— Architectural rendering of an internal view of the tropical garden at the center of the terminal

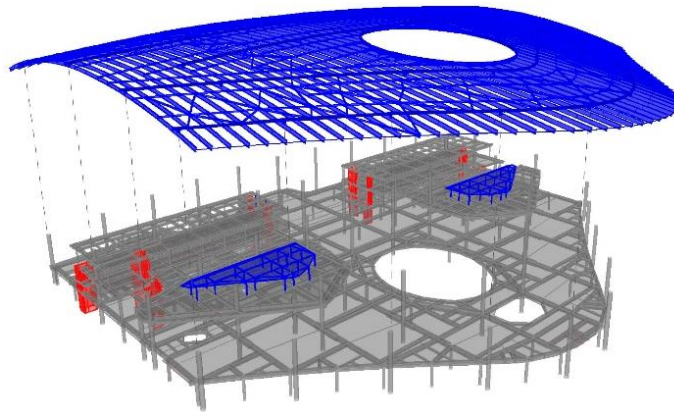


Figure 7— Structural system in Zone 3 (Exploded view of steel roof and concrete superstructure)

The Roof

The roof structural system consists of unfilled metal decking with a profile depth of 75 mm (3 in) spanning between curved built-up wide-flange secondary beams running in the transverse direction at 3 m (10 ft) centers. Supporting the roof secondary beams are the roof primary beams which run along the longitudinal direction and are in turn supported on concrete columns at 18 m (59 ft) centers. Along the perimeter, the secondary beams cantilever out from the perimeter primary beam lines by 4.5 m (15 ft) up to a maximum cantilever span of almost 16 m (53 ft) to form canopies on both the landside and the airside of the terminal as presented in Figs. 8 and 9.

Selection of the cross section shapes and sizes was largely dictated by the needs to achieve the architectural design intent, which requires firstly that the roof primary beams are tubular sections to realise the architectural language as shown in Figs. 4 and 8 and secondly that the bottom flanges of the secondary beams are Architecturally Exposed Structural Steel (AESS) and form part of the roof internal finish as shown in Fig. 4. The spans and sizes of the various roof members in the different zones are summarized in Table 1.

The primary beams are supported on short steel column stubs of the same cross section shape and diameter on top of the concrete columns which support the roof, as shown in Fig. 8. These column stubs provide the transition from the roof structural steel onto the concrete columns and are connected to the concrete columns through a base plate connection detail described later in this paper.

Table 1— Summary of spans and sizes of roof structural elements

Zone	Max secondary steel beam span	Secondary steel beam depth	Primary steel beam diameter	Roof concrete column diameter
1A, 2A	30 m (98 ft)	850 mm (33.5 in)	650 mm (25.5 in)	1000 mm (39.3 in)
1B, 2B	40 m (131 ft)	904 mm (35.6 in)	700 mm (27.5 in)	1000 mm, 1200 mm (39.3 in, 47.2 in)
3	46 m (151 ft)	972 mm (38.3 in)	750 mm (29.5 in)	1200 mm (47.2 in)



Figure 8— Site photo of the roof structural steel construction in Zone 2B

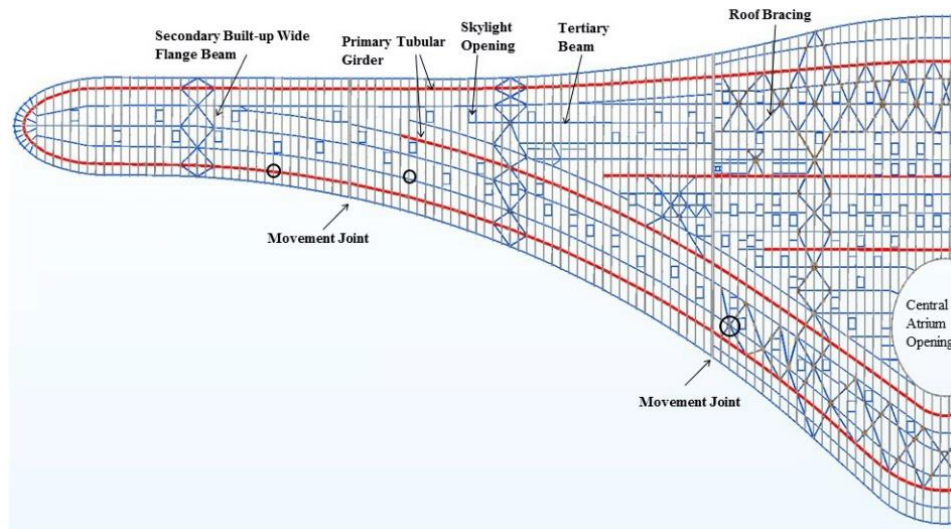


Figure 9— Half roof structural framing plan

THE NEED FOR A PERFORMANCE-BASED SEISMIC DESIGN METHODOLOGY

Site geotechnical conditions and the foundation systems

The Terminal 2 building sits above several layers of fluvial sediments of soft, medium and stiff consistency, overlaid by fill material. An extensive site geotechnical investigation with over 80 boreholes indicated significant variability in the geotechnical conditions across all zones of the 660 m (2,164 ft) long site. Engineering sound rock was found at a depth between 9 m (29 ft) and 23 m (75 ft) below the existing grade level. The foundation system consists of cast-in-place drilled vertical shaft piles socketed into the rock. Pile diameters are 1.37 m (54 in) and 0.9 m (35 in), respectively, with socket length varying between 6 m (20 ft) and 9 m (29 ft). Each column is typically supported by one individual pile with a grillage of grade beams tying the pile caps together to form an integrated foundation system and providing base fixity to the columns to resist the column base bending moments. Static and dynamic pile load testing were performed in the field respectively with bidirectional Osterberg Cell and Pile Driving Analyzer (PDA) methods to validate the design pile geotechnical capacities.

Owner's requirements

Panama is located in a seismically active region of moderately high seismic hazard. The Seismic Design Category (SDC) of the new Terminal 2 would have been D had the code-prescriptive design approach been adopted. In its project brief, the Owner (Tocumen International Airport Authority) classified the Terminal 2 building as an essential facility and specified the following three-level seismic performance objectives:

- Operational capacity of facilities shall be guaranteed in the event of mild and moderate earthquakes. Not only the structure itself shall be free of damage, but also operational continuity shall be ensured. A non-conventional structural seismic design approach is required. This requirement is beyond the scope of the conventional prescriptive seismic design method prescribed in the local and international building codes.
- Structural and non-structural damage caused by severe earthquakes shall be minimised so that facilities can resume their operational capacity in a short period of time following repair work.
- The structural system shall remain stable without partial or total collapse when subjected to Maximum Considered Earthquake shaking.

It is clear in the project brief that the Owner demanded that a performance-based structural seismic design methodology be employed to explicitly verify the fulfilment of the three-level seismic performance objectives and that the conventional code-prescriptive approach did not meet the Owner's requirements.

Seismic force-resisting systems outside code prescriptive requirements

Along the transverse direction, none of the 3 single structures in Zones 1B, 2B and 3 (Figs. 5 and 7) together with the 2 independent structures of the roof plus its supporting concrete perimeter columns in Zones 1A and 2A (the structure shown by the right-hand side diagram in Fig. 3) has a seismic force-resisting system that is listed in Table 12.2-1 of ASCE 7–10¹. This makes the use of the code-prescriptive seismic design approach not feasible, because the critical parameters including the response modification coefficient R , the system overstrength factor Ω_0 and the deflection amplification factor C_d cannot be determined for a non-classified seismic force-resisting system.

The seismic force-resisting systems along the transverse direction do not comply with the code prescriptive requirements because the roof beams do not frame directly to the supporting concrete columns and hence do not form conventional moment-resisting frames. When subjected to lateral loads, the columns supporting the roof still exhibit double curvature bending similar to moment frame columns due to the torsional stiffness of the tubular steel primary beams along the longitudinal direction. However, this mechanism is not compliant with the intent of the prescriptive seismic design approach which requires ductile energy dissipation to occur in flexural plastic hinges at the ends of beams and at the base sections of columns, rather than at torsional plastic hinges.

The two independent roof structures in Zones 1A and 2A, as shown by the right-hand side diagram in Fig. 3, may be classified as a cantilevered column system detailed to conform to the requirements for special reinforced concrete moment frames (seismic force-resisting system G.3 in Table 12.2–1 of ASCE 7–10¹). However, ASCE 7–10¹ imposes a height limit of 10.7 m (35 ft) on such systems. Hence, at 16 m (52 ft) tall, these two independent structures do not comply with the prescriptive requirements of the code either.

Beam-column connections outside code prescriptive requirements

Along the longitudinal direction, the roof primary beams are framed to the supporting concrete columns thereby forming moment-resisting frames in all the five independent structures in the five zones. However, the tubular sections adopted for the roof primary beams and the supporting column stubs do not lead to prequalified beam-column connections for intermediate and special moment frames in accordance with AISC 358–10². Hence, along the longitudinal direction, the moment-resisting frames can only be designed as ordinary moment frames which do not comply with the requirement of special moment frames demanded by the prescriptive approach for SDC D.

As a result of the Owner's requirements and the non-compliance of the seismic force-resisting systems and the beam-column connections with the code prescriptive requirements, the structural team set aside the code-prescriptive approach and instead adopted a state-of-the-art performance-based seismic structural design methodology together with nonlinear seismic analysis methods. The project-specific performance-based seismic design methodology and criteria document, as well as structural analyses, calculations and construction documents were reviewed by internationally recognised experts and three independent peer review teams. These aspects are described in the following sections of this paper.

SEISMIC PERFORMANCE OBJECTIVES

Qualitative seismic performance objectives

After careful consideration of the Owner's requirements as specified in the project brief and recommendations in performance-based seismic design guideline documents and standards (references 4 – 7), the following three-level performance objectives were adopted:

1. Operational – The building is designed to remain operational when subjected to an occasional earthquake shaking having a return period of 72 years (50% probability of exceedance in 50 years).
2. Immediate Occupancy – The building is designed for immediate occupancy when subjected to a rare earthquake shaking having a return period of 475 years (10% probability of exceedance in 50 years).
3. Life Safety – The building is designed for life safety when subjected to Maximum Considered Earthquake (MCE) shaking which has a return period of 2,475 years (2% probability of exceedance in 50 years).

On the basis of the above qualitative performance objectives, quantitative performance objectives in terms of acceptance limits on deformations for deformation-controlled actions/components and those on forces for both the critical force-controlled and non-critical force controlled actions/components were developed following recommendations in current performance-based seismic design guidelines and standards (references 5 – 7) for the purpose of quantitative seismic performance evaluation of the global structures and local structural elements and components.

Quantitative acceptance limits at the global building level

At the global building level, structural damage is best quantified by the total story drift angle. For the Terminal 2, Table 2 summarises the acceptance limits on total story drift angles for each of the three performance levels.

Table 2— Acceptance limits on total story drift angles

Performance level	Acceptance limit on total story drift angle
Operational	No limit imposed. Non-structural components and systems, such as the glass façade system, are to be detailed to accommodate the seismic drift demand without damage
Immediate occupancy	1% for moment frames 0.75% for concrete shear walls
Life safety	2% for moment frames 1.5% for concrete shear walls

Quantitative acceptance limits at the component level

Performance (structural damage) of structural components is quantified by the maximum inelastic deformation sustained by deformation-controlled actions/components. For both the Immediate Occupancy and the Life Safety

performance levels, the acceptance limits on the maximum inelastic deformations of deformation-controlled actions/components were adopted from ASCE 41–06 Including Supplement No. 1⁷.

SITE-SPECIFIC PROBABILISTIC SEISMIC HAZARD ASSESSMENT

The structural engineering team commissioned a seismologist and geotechnical seismic engineering specialist team to perform a site-specific probabilistic seismic hazard assessment to provide:

- 5% damped uniform hazard horizontal and vertical response spectra at ASCE 7–10 type B bedrock level but assuming that the soil above has been removed for 72-, 475- and 2,475-year (MCE) return periods.
- Five sets of tri-axial horizontal and vertical site-specific ground surface response spectra, one set for each zone corresponding to the five zones of the new terminal. The spectra were derived from equivalent linear and fully nonlinear site response analyses, assuming vertically propagating seismic shear wave and P wave and based on dynamic soil properties determined by site downhole shear wave and p-wave velocity measurements.
- A suite of seven sets of tri-axial horizontal and vertical earthquake ground motion time histories spectrally matching the site-specific ground surface response spectra, for each of the five zones of the project site.

Fig. 10 presents the horizontal and the vertical response spectra at the ground surface in Zone 3 corresponding to the three return periods. Response spectra in each of the five zones are markedly different due to the high variability in the ground conditions across the 660 m (2,164 ft) long site.

CAPACITY DESIGN AND CLASSIFICATION OF ACTIONS AND COMPONENTS

In addition to the performance-based seismic design methodology, the capacity design principle was also adopted. Structural elements and components were proportioned, designed and detailed such that yielding and energy dissipation occur in selected deformation-controlled actions and components detailed for ductile behaviour which limit the force demands on non-ductile, force-controlled actions and components. The latter were designed with sufficient strength capacities to remain essentially elastic during responses to a rare earthquake shaking (475 year return period) and to MCE shaking.

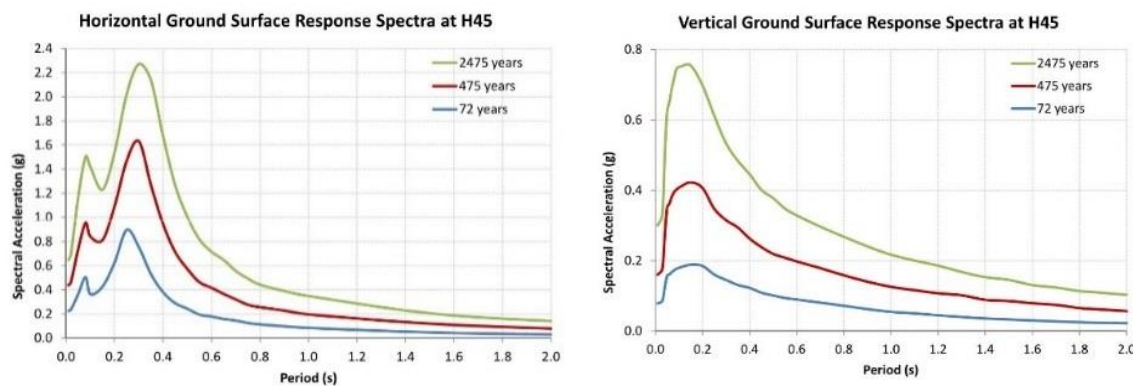


Figure 10— Site-specific horizontal and vertical ground surface response spectra in Zone 3 of the project site for three return periods

Capacity design of the roof steel column stub to concrete column connection

As described earlier, the tubular roof primary steel beams are supported on short tubular column stubs of the same diameter on top of the concrete columns. The steel column stubs provide the transition from the structural steel roof onto the concrete columns and are typically connected to the latter via the connection detail shown in Fig. 11. Also as discussed previously, since the beam-column connections between tubular members are not prequalified for intermediate and special moment frames, the roof structural steel framing has limited ductility capacity. As a result, the roof primary beams and the supporting column stubs were classified as critical force-controlled components and designed to be capacity-protected so as to remain essentially elastic during responses to a rare earthquake shaking and to MCE shaking.

In order to ensure that yielding in the roof structural steel does not occur, an innovative fuse detail at the top of the concrete columns was developed by the structural engineering team. In this detail, the vertical reinforcing bars are locally reduced in size to create a weakened zone between strong segments. The weakened zone becomes the plastic hinge zone where ductile plastic rotation and energy dissipation can occur as shown in Fig. 11. This fuse detail comprises of a ring of smaller diameter reinforcing bars spliced by Type 2 transition mechanical couplers to larger diameter reinforcing bars above and below to create a reinforced concrete equivalent of the steel special moment frame reduced beam section (RBS) connection, commonly referred to as the “dog-bone” connection. Lateral shear is transferred into the concrete column via a short tubular shear lug welded to the underside of the base plate. This fuse limits the moment capacity at the top of the columns and hence limits the magnitude of forces that can possibly be developed in the roof steel column stubs and in the roof primary beams. By performing nonlinear seismic analysis, the moment capacity of the weakened segment (the fuse) can be tuned to capacity-protect the column stubs and the roof primary beams thereby allowing the roof structural steel to remain essentially elastic under MCE shaking. This column top hinging mechanism of energy dissipation is the preferred mechanism for the Terminal 2 within the framework of the performance-based seismic design and is also permitted within the framework of the code-prescriptive design since column top hinging occurs in the perimeter columns, which are single story columns, and in the top story of the selected interior columns extended upwards to support the roof.

The connection between the horizontal and the vertical tubular members was specified as a demand critical CJP weld and the connection strength was checked as a thickened chord can T-joint using the American Petroleum Institute (API) standard “Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms-Working Stress Design (2000)”⁸ with a modified safety factor to derive the nominal capacity and resistance factors added to give the equivalent Load and Resistance Factor Design (LRFD) capacity.

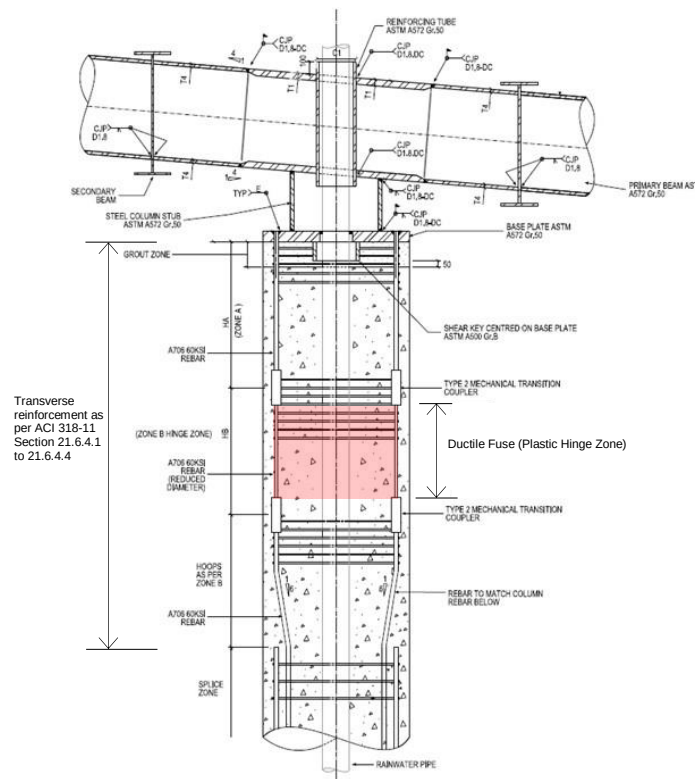


Figure 11— The capacity design fuse connection between the roof structural steel and the reinforced concrete supporting columns

Classification of actions and components

Deformation-controlled actions and components— The following actions and components were selected as deformation-controlled for ductile energy dissipation:

- P-M yielding at the base sections of reinforced concrete shear walls
- Flexural yielding at the ends of reinforced concrete shear wall coupling beams
- Flexural yielding at the ends of reinforced concrete moment frame beams except transfer beams and post-tensioned girders
- P-M-M yielding at the base sections and at the top ends of columns supporting the roof primary beams

Critical force-controlled actions and components— The following actions and components were classified as critical force-controlled and their force demands were increased from those determined by the performance point from a nonlinear static pushover analysis or the mean peak demand from nonlinear response history analysis using the suite of seven sets of MCE ground motion time histories by a factor of 1.2 for actions limited by a well-defined yielding mechanism and otherwise by a factor of 1.5. In all cases, the force demand was limited to the maximum force that can actually be delivered to the component.

- Structural steel roof primary beams and steel column stubs supporting the roof primary beams
- Tubular moment connections/joints between the roof primary beams and the steel column stubs, and other connections in the roof structural steel
- Shear in reinforced concrete shear walls, moment frame beams, columns and beam-column joints
- Shear in transfer beams and in unbonded post-tensioned girders
- Shear in base plates
- Shear force in shear lugs

Non critical force-controlled actions and components— The following list of actions and components were classified as non-critical force-controlled and their force demands were determined by the performance point from a nonlinear static pushover analysis or the mean peak demand from nonlinear response history analysis using the suite of seven sets of MCE ground motion time histories without increase.

- Foundation forces
- Diaphragm shear in reinforced concrete floors and unfilled roof metal decks
- Axial forces in diaphragm chords and collectors (drag struts)
- Roof steel secondary beams and roof horizontal bracings
- Flexure in unbonded post-tensioned girders
- Flexure in base plates
- Tension in anchor rods/reinforcing bars at the top of the roof concrete columns

Strength hierarchy

A desired strength hierarchy was achieved by the use of the strength reduction factor/resistance factor, ϕ , as specified in the applicable material standards ($\phi < 1.0$ for critical force-controlled actions and $\phi = 1.0$ for non-critical force-controlled actions).

PROPORTIONING AND DETAILING OF PLASTIC HINGE REGIONS

Elements and regions selected to develop nonlinear behaviour were proportioned and detailed in accordance with the more stringent requirements of the following ACI code and ASCE standard:

- ACI 318–11³ - Sections 21.5 Flexural members of special moment frames, Section 21.6 Special moment frame members subjected to bending and axial load, Section 21.7 Joints of special moment frames and Section 21.9 Special structural walls and coupling beams
- ASCE 41–06 Including Supplement No. 1⁷ - Tables 6-7, 6-8 and 6-18. As a minimum, reinforced concrete beams, columns, shear walls and shear wall coupling beams are to be proportioned and detailed to meet requirements specified in these tables in order to use the acceptable plastic hinge rotation angles in the performance evaluation of the deformation-controlled actions/components.

SEISMIC ANALYSIS AND NONLINEAR MODELLING METHODS

Selection of applicable seismic analysis methods

Central to the performance-based seismic design approach is the use of nonlinear analysis methods in cases where the expected structural behaviour is nonlinear. In cases where the behaviour of the structure is expected to remain linear elastic, the linear response spectrum analysis method can still be used for performing seismic analysis. For the Terminal 2, the selected seismic analysis methods are summarised in Table 3.

Table 3— Selected seismic analysis methods

Level of earthquake shaking	Return period	Expected structural behaviour	Selected seismic analysis method
Occasional earthquake	72 years	Linear elastic	Linear response spectrum
Rare earthquake	475 years	Slightly nonlinear	Nonlinear static pushover
MCE	2,475 years	Substantially nonlinear	Zone 3 – nonlinear response history Other zones – nonlinear static pushover

All five independent roof structures are low-rise. In principle, the nonlinear static pushover analysis method is applicable for performing nonlinear analysis of these structures subjected to a rare earthquake and to MCE shaking. However, the nonlinear response history analysis method was necessary for nonlinear analysis of the Zone 3 structure subjected to MCE shaking due to its complexity and the lack of a dominant mode in each of the two horizontal principal directions, with the effective masses in each of the two fundamental modes being approximately 1/3 of the total seismic mass.

Analysis software

- Linear response spectrum analysis – SAP 2000 version 15 and ETABS version 9.7.2
- Nonlinear static pushover analysis – SAP 2000 version 15
- Nonlinear response history analysis – Perform 3D version 5.0

Nonlinear modelling methods in Perform 3D

Reinforced concrete columns— Reinforced concrete columns are modelled by the lumped plasticity model with zero-length, rigid-plastic P-M3-M2 plastic hinges located at the ends of a column. The P-M3-M2 interaction yielding surface is calculated by the fiber cross section model, as shown in Fig. 12 for a typical column having a diameter of 1,200 mm (47 ¼ in) and 26 No. 36 (#11) reinforcing bars. The nonlinear plastic moment-plastic hinge rotation backbone curve and strength loss parameters (values of “a”, “b” and “c”) of each column are determined based on Table 6-8 of ASCE 41–06 Supplement 1⁷ as illustrated in Fig. 13. Cyclic stiffness degradation parameters and cyclic hinge moment-rotation hysteresis loops of a typical rigid-plastic P-M3-M2 hinge exhibiting both strength loss and stiffness degradation are illustrated in Fig. 14.

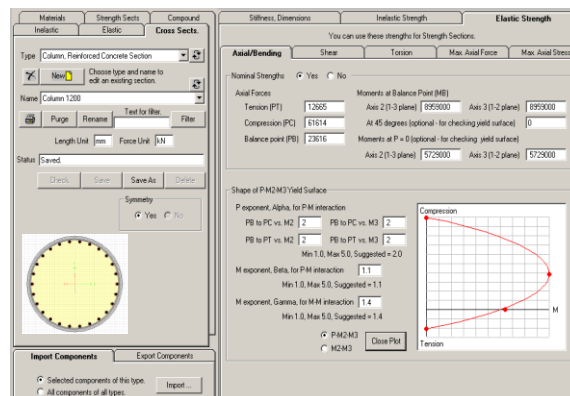


Figure 12— Modelling parameters of yielding surfaces of P-M3-M2 hinges of typical reinforced concrete columns

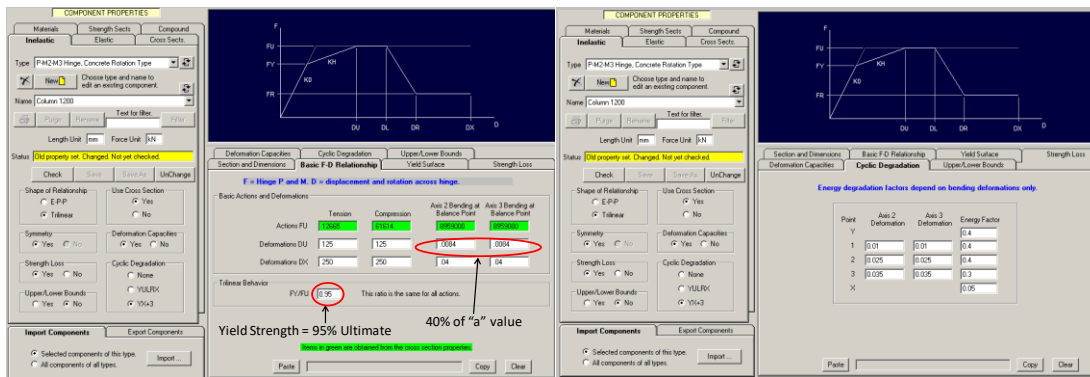


Figure 13— Modelling parameters of P-M3-M2 hinge backbone curves and strength loss parameters of typical reinforced concrete columns

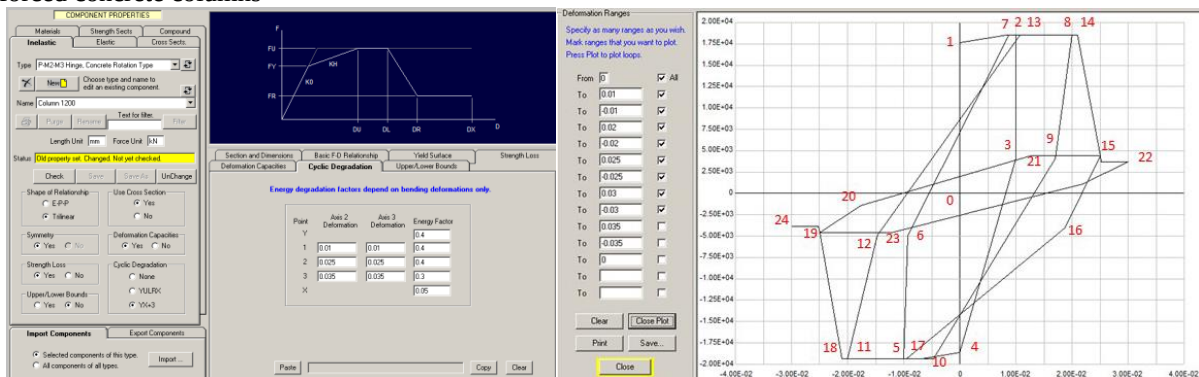


Figure 14— Typical cyclic stiffness degradation parameters and hinge moment-rotation hysteresis loops

Reinforced concrete moment frame beams— Reinforced concrete moment frame beams are modelled similarly to reinforced concrete columns except that the rigid-plastic hinges at beam ends are uniaxial flexural hinges which yield and develop plastic rotations when the bending moments at beam ends reach the plastic moment capacity of the respective beams.

Coupling beams in reinforced concrete shear walls— Each coupling beam is modelled by three frame elements, an inelastic element in the clear span and two elastic element embedded in the reinforced concrete shear wall as illustrated in Fig. 15. Nonlinear modelling of the inelastic frame element is identical to that of reinforced concrete moment frame beams with flexural plastic hinges forming and developing plastic rotations at the wall faces. The two embedded elastic frame elements simulate rigid connections between the coupling beam and the walls. This is necessary because reinforced concrete shear walls are modelled by four-node shell elements which do not have stiffness in the drilling degree of freedom of the shell element nodes. Without the two embedded elastic frame elements, the inelastic frame element would have been connected to the wall by pin connections without bending moments at the ends.

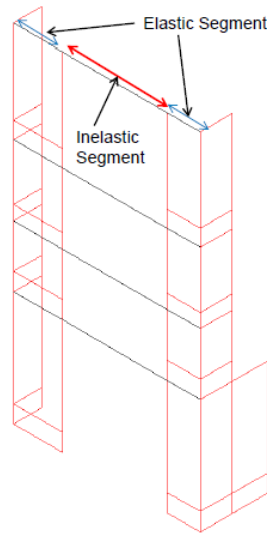


Figure 15— Modelling of coupling beams in Perform 3D

Reinforced concrete shear walls— Each wall pier is modelled by a mesh of four-node shell elements coupled with the fiber cross section model in which the wall cross section is discretized to many concrete fibers and the steel reinforcing bars are lumped together to form steel fibers. Each fiber is assigned its respective uniaxial cyclic stress-strain curve as shown in Fig. 16.

The shell element mesh is sufficiently refined such that at least two shell elements exist over the height of each story. The height of the lowest shell elements was selected to correspond to the plastic hinge length at wall base sections which is typically the shorter of the story height and half of the wall length. Fig. 17 shows a typical shell element mesh near the wall base section together with linear strain gages and rotation gages used to measure wall plastic hinge rotation and linear strains of the various fibers.

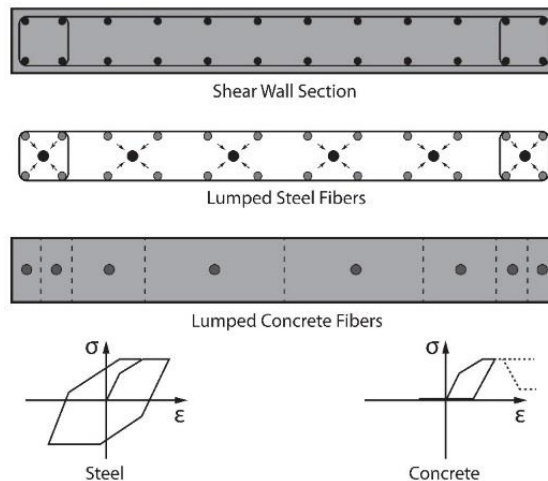


Figure 16— Fiber cross section model of shell elements modelling reinforced concrete shear walls

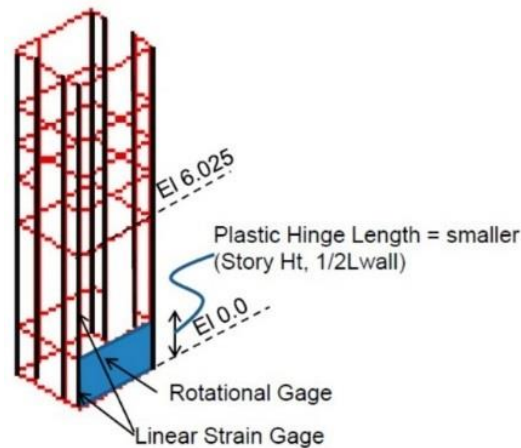


Figure 17— Shell element mesh and deformation measuring elements near wall base in Perform 3D

Damping— The Rayleigh damping matrix $[C] = \alpha[M] + \beta[K]$ for linear dynamic analysis was modified for nonlinear response history analysis. The initial global elastic stiffness matrix $[K]$ was used to form the global damping matrix $[C]$ and for determining values of the mass proportional multiplier α and the stiffness proportional multiplier β , resulting in a constant damping matrix $[C]$ to achieve 5% of critical damping in linear dynamic response in the period range of interests of linear response. This constant Rayleigh damping matrix is modified for nonlinear response history analysis in order to account for changes in mode shapes and elongation in modal periods due to softening of the structure as a result of yielding.

Rather than using a damping matrix that varies with time and is updated during the nonlinear response history due to the high computational cost arising from recalculation of the mode shapes and modal periods at every nonlinear event such as yielding, unloading and re-loading, a simplified method as recommended in the Perform 3D User Guide⁹ was adopted as described below in order to maintain a constant damping matrix $[C]$ but modified for nonlinear analysis:

- Modification to account for period elongation due to yielding – the period range of interests is increased by assuming that the fundamental period (the upper bound of the period range of interests) is elongated by a factor equal to the square root of the expected ductility ratio.
- Modification to account for stiffness reduction due to cracking of reinforced concrete components – when assembling the stiffness proportional damping term, $\beta[K]$, of the global damping matrix, $[C]$, the stiffness proportional multiplier, β , associated with contributions from reinforced concrete elements is reduced by a factor of 0.1.

SUMMARY OF RESULTS OF NONLINEAR ANALYSIS AND PERFORMANCE EVALUATION

Zone 2A

The nonlinear analysis of the independent roof structure shown by the diagram at the right hand side of Fig. 3 was carried out employing the nonlinear static pushover analysis method using SAP 2000 first and then validated by the nonlinear response history analysis method using Perform 3D. Fig. 18 illustrates the SAP 2000 model. Figs. 19 and 20 show the deformed shapes and plastic hinge locations and performance evaluation at the performance point corresponding to MCE shaking from the nonlinear pushover analysis along both the positive and the negative directions of the two principal axes. The colour coding in the legends indicates the damage states of the plastic regions, with pink implying Immediate Occupancy performance (IO, between yielding and the threshold value for reaching the Life Safety damage state indicated by blue). It can be observed that the damage states of all column plastic hinge regions are within acceptable limits for Immediate Occupancy which is a lighter degree of structural damage than the performance objective of Life Safety, implying satisfactory performance at the component level.

The capacity curves and the performance points corresponding to the three-levels of earthquake shaking intensity are presented in Fig. 21. For brevity, results from pushover along the positive directions only of the longitudinal and the transverse axes are presented herein. Those from pushover along the negative directions are nearly identical. The horizontal axis is the drift ratio at the center of mass of the roof and the vertical axis is the ratio between the base shear

force and the seismic weight. The performance points corresponding to the 72-year return period earthquake shaking are well within the linear elastic range of response, suggesting no structural damage. Those associated with the 475-year return period earthquake shaking are just beyond the linear elastic limits at a drift ratio of $0.7\% <$ the acceptable limit of 1% for the Immediate Occupancy performance objective, suggesting light damage and permitting immediate occupancy of the structure after a rare earthquake shaking. Finally, the performance points associated with MCE shaking are well into the nonlinear range of response at a drift ratio of $1.1\% <$ the acceptable limit of 2% for the Life Safety performance objective, suggesting significant inelastic behaviour and structural damage, which is indeed the structural design intent. It can also be seen that the MCE performance points occur before the structure has reached its ultimate strength capacity, with the pushover curves still rising slowly beyond the MCE performance points, implying that the structure still possesses a small amount of reserve strength capacity and a significant amount of stable deformation capacity to protect life safety after MCE shaking. The above observations indicate that the performance of the Zone 2A roof structure is satisfactory at the global structural level.

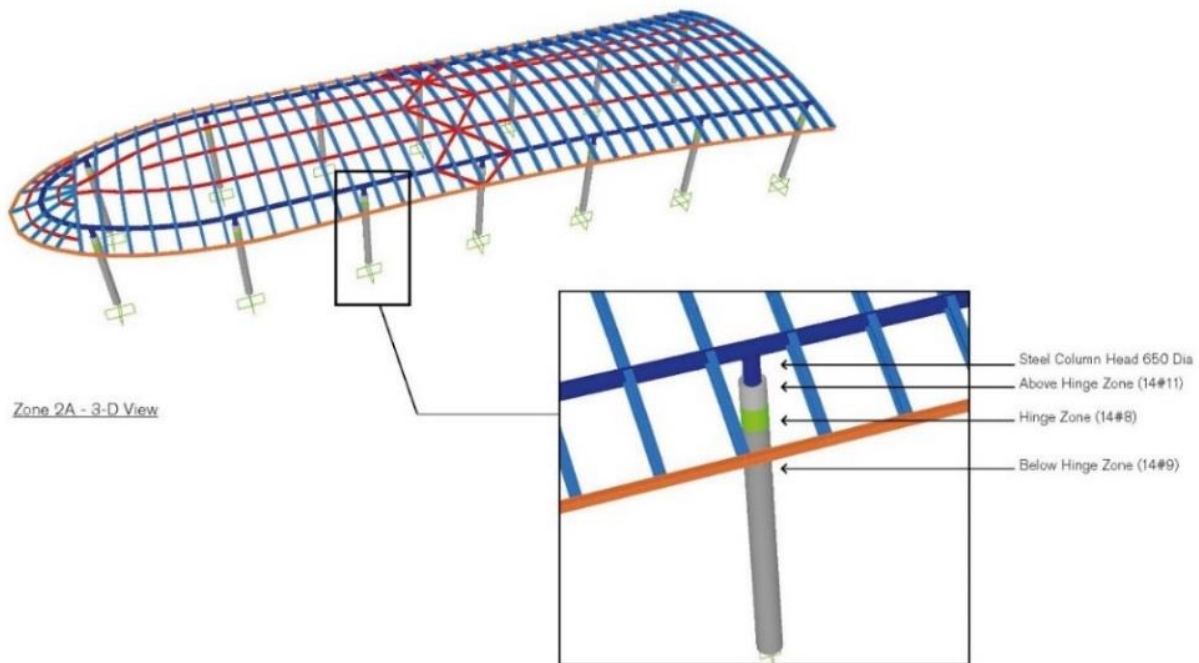


Figure 18— The SAP 2000 nonlinear pushover analysis model of the Zone 2A roof structure

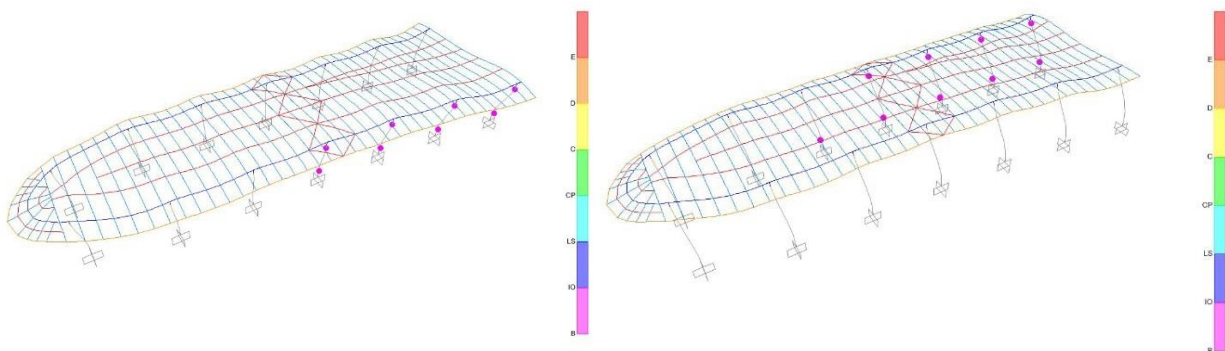


Figure 19— Hinge locations and performance evaluation of the Zone 2A roof columns – pushover along the transverse axis (positive and negative directions, respectively)

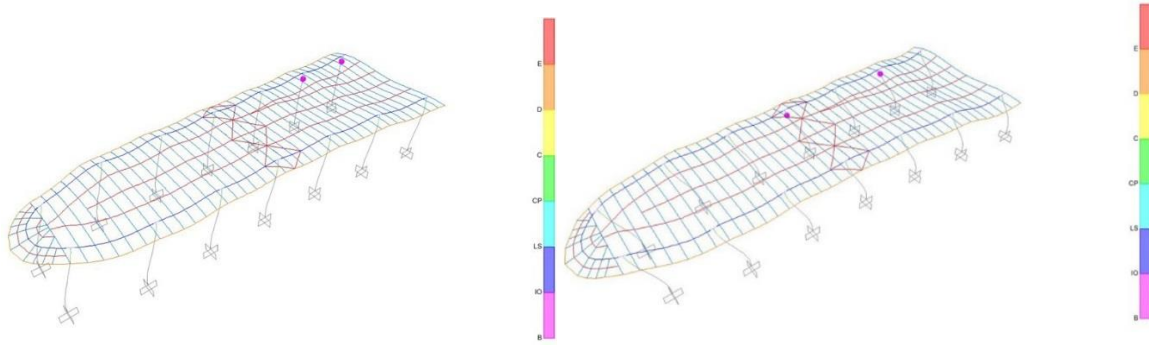


Figure 20— Hinge locations and performance evaluation of the Zone 2A roof columns – pushover along the longitudinal axis (positive and negative directions, respectively)

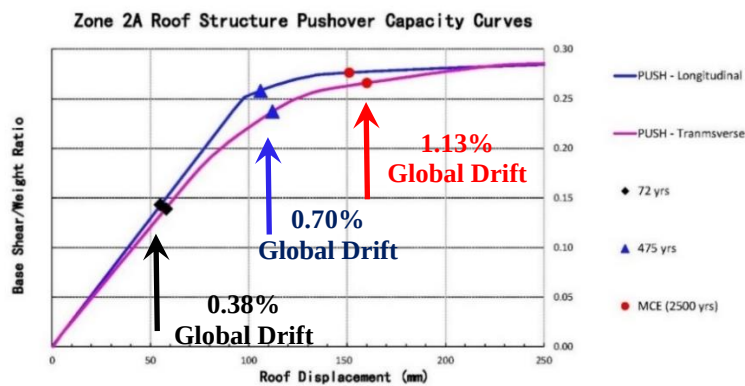


Figure 21— Capacity curves and performance points of the Zone 2A roof structure – pushover along the longitudinal and the transverse axis, respectively

In order to validate results by the nonlinear static pushover analysis, the Zone 2A roof structure was further analysed by the nonlinear response history analysis method using the suite of seven pairs of MCE ground motion time histories as input. Table 4 and Figs. 22 to 23 present drift ratios and column plastic hinge rotations. The plastic hinge rotations listed in Fig. 23 are expressed in terms of usage ratio which is equal to the Demand/Capacity ratios calculated as the peak plastic rotation normalised to the limiting plastic rotation acceptable for the performance objective – Life Safety in this case. It can be observed that the highest usage ratio from the suite of seven sets of MCE ground motion time histories is 0.15, implying much lower plastic rotations than the threshold rotation reaching the Life Safety damage state and achievement of the Immediate Occupancy performance. The average drift ratio from the suite of seven sets of MCE ground motion time histories is approximately 1%. Compared with results by the nonlinear pushover analysis method, the agreement is satisfactory.

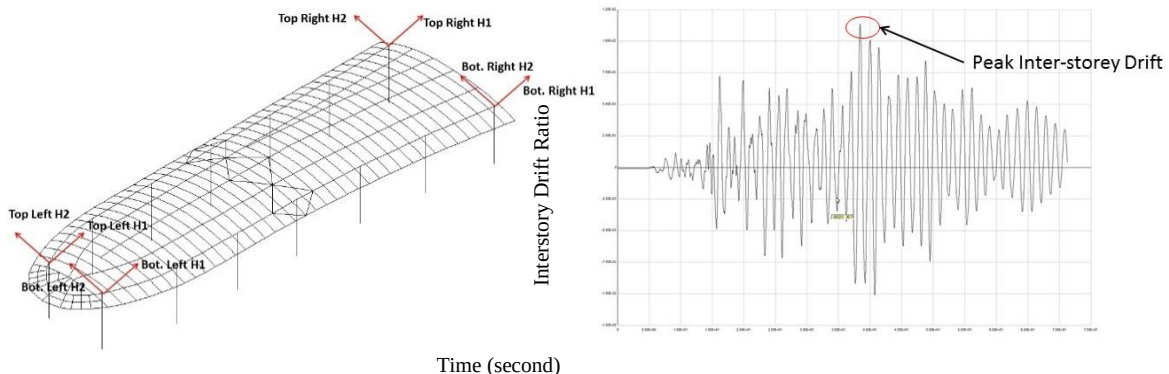
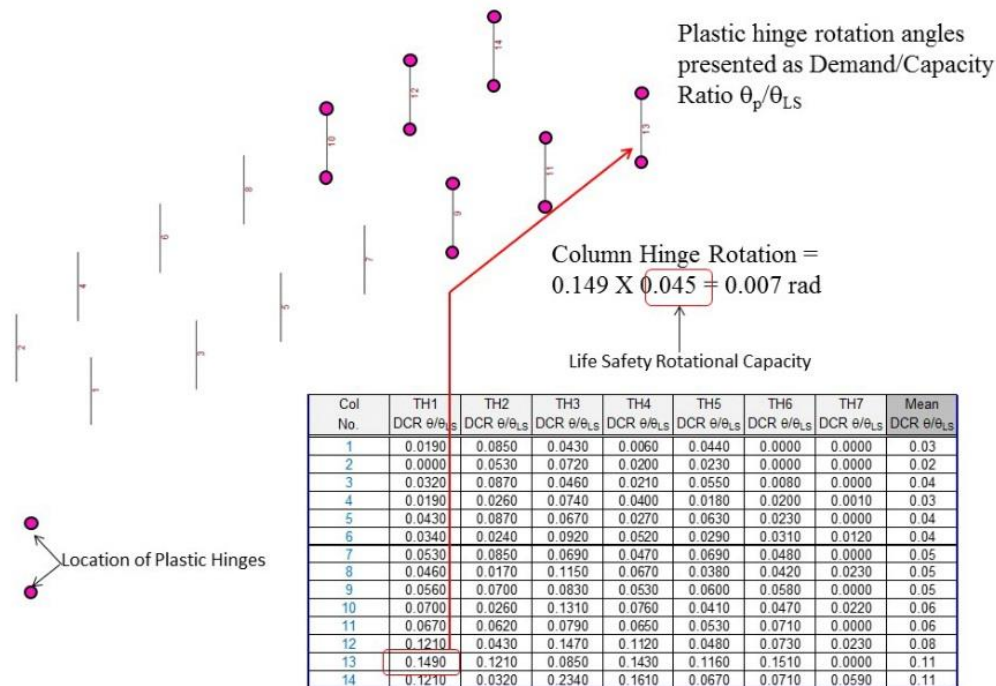


Figure 22— Locations of drift output and time history of drift

Table 4— MCE drift ratios from nonlinear response history analysis*

Drift No.	Drift Name	TH1 Drift%	TH2 Drift%	TH3 Drift%	TH4 Drift%	TH5 Drift%	TH6 Drift%	TH7 Drift%	Mean Drift%
1	Top Left H1	1.137	0.964	0.956	0.975	0.985	0.954	0.953	0.989
2	Top Left H2	0.841	1.028	0.895	0.933	0.968	1.041	0.825	0.933
3	Top Right H1	1.098	0.926	0.921	0.929	0.956	0.962	0.937	0.961
4	Top Right H2	1.328	1.093	1.417	1.191	0.953	1.204	1.039	1.175
5	Bottom Left H1	1.163	1.017	1.005	1.035	0.985	0.880	0.957	1.006
6	Bottom Left H2	0.846	1.042	0.877	0.919	0.982	1.026	0.810	0.929
7	Bottom Right H1	1.135	1.007	1.005	1.037	0.948	0.871	0.929	0.990
8	Bottom Right H2	1.237	1.246	1.206	1.258	1.059	1.141	0.864	1.144

* Note: TH1 through to TH7 refer to the 7 sets of strong motion time histories

**Figure 23**— Plastic hinge rotations of columns in Zone 2A subjected to MCE ground motion time histories

Zone 3

The nonlinear seismic analysis of the independent structure in Zone 3 subjected to MCE shaking was carried out by the nonlinear response history analysis method using the seven sets of tri-axial MCE ground motion time histories as prescribed boundary motion condition at the nodes fixed on the ground. The nonlinear equations of motion were solved directly in the time domain and the response histories together with the peaks of the various response parameters of interests were extracted and post-processed. The mean peak values of the various response parameters of interest from the suite of seven sets of MCE ground motion time histories were then calculated for performance evaluation.

Drift ratios— Fig. 24 summarizes the drift ratios of the Zone 3 structure subjected to MCE shaking. The mean peak drift ratios of all the tall perimeter columns supporting the roof are approximately 1% and the maximum mean peak drift ratio of all perimeter and interior columns is approximately 1.5%, occurring in the multi-story mezzanine reinforced concrete special moment frame – special shear wall structure between floor levels 4 and 5. The drift ratios satisfy the acceptance criterion of not exceeding 1.5% for Life Safety.

Column plastic hinge rotations— All columns are classified as either Condition i or ii in accordance with ASCE 41–06 Supplement No. 1⁷. Where the full backbone curve is explicitly modelled including strength degradation and

residual strength in accordance with Section 3.4.3.2 of this standard as implemented in the Perform 3D nonlinear modelling of the concrete columns of the Zone 3 structure, the acceptance limits on plastic hinge rotations of both primary and secondary components are taken as those for the secondary components listed in Table 6-8 of this reference in accordance with note 4 of this table. The acceptance limits of all columns for Life Safety range from 0.017 radians for internal special moment frame columns (Condition ii) to 0.045 radians for the perimeter columns (Condition i). The column plastic rotation usage ratio, expressed as the average peak plastic rotation from the suite of seven sets of MCE ground motion time histories of a column normalised to its acceptance limit on plastic rotation for Life Safety, ranges from 0.0 (remaining elastic) to 0.55, implying satisfactory performance of the columns for achieving the Life Safety performance objective when subjected to MCE shaking. Out of the suite of seven sets of MCE ground motion time histories, the maximum peak column plastic hinge rotation is 0.0325 radians occurring in a Condition i column as shown by the column plastic hinge moment-rotation hysteresis loops presented in Fig. 25. The acceptance limit for Life Safety of this Condition i column is 0.045 radians and its plastic rotation usage ratio based on the mean peak plastic rotation is 0.49 only.

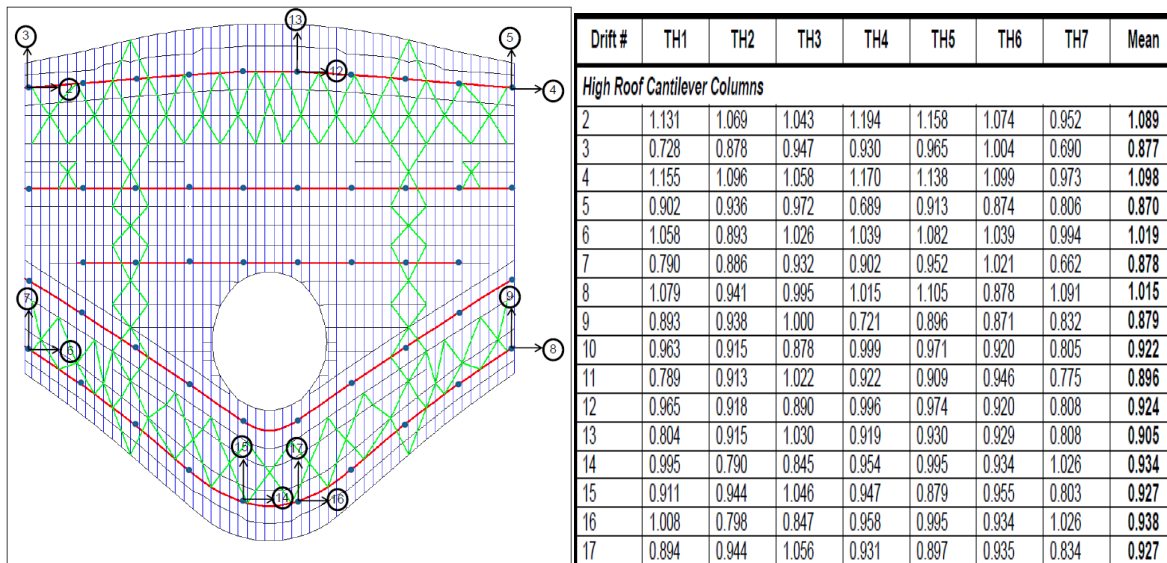


Figure 24— Summary of drift ratios of the Zone 3 structure subjected to MCE shaking

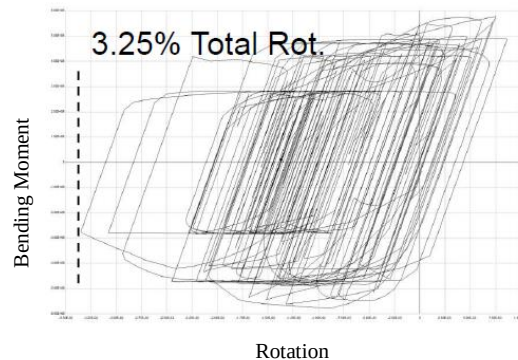


Figure 25— Plastic hinge moment-rotation hysteresis loops of a Condition i column subjected to MCE Shaking

Plastic hinge rotations of moment frame beams and wall coupling beams— All special moment frame beams and coupling beams in the special reinforced concrete shear walls are controlled by flexure and have an acceptance limit on plastic rotations of 0.02 radians for Life Safety. Table 5 summarizes the mean plastic rotations of beams at each level. It can be seen that the maximum mean plastic rotation of 0.022 radians occurs in a Level 5 beam at elevation +19.725 m (64.7 ft) and is approximately equal to the acceptance limit of 0.02 radians, implying acceptable performance.

Plastic hinge rotations near the base sections of special reinforced concrete shear walls— As shown in Fig. 17, rotation gages were used to measure flexural rotation within the plastic hinge zone near the wall base sections. Among all the reinforced concrete special shear walls, the highest mean wall plastic rotation near the wall base sections from the suite of seven sets of MCE ground motion time histories is 0.01 radians, lower than its acceptance limit of 0.15 radians for Life Safety, again implying acceptable performance.

Table 5— Plastic rotations of special moment frame beams and special shear wall coupling beams

Level / Elevation	Maximum Beam Mean Plastic Rotation (rad.)	Acceptance Limit
Level 2 / +6.025 m (19.7 ft)	0.006	0.02
Level 3 / +10.025 m (32.9 ft)	0.010	0.02
Level 4 / +14.025 m (46 ft)	0.010	0.02
Level 5 / +19.725 m (64.7 ft)	0.022	0.02

Energy balance and distribution of dissipated energy— The time history plot in Fig. 26 presents an energy balance plot of the Zone 3 structure responding to one of the suite of seven sets of MCE ground motion time histories. Those associated with the other sets of MCE time histories are very similar. The horizontal axis is time and the vertical axis is the percentage of the total external work imparted to the structure by MCE shaking, ranging from 0% to 100%. It can be seen that energy balance has been achieved with the sum of all internal energies equal to 100% of the total external work. It can also be noted that an optimum distribution of the dissipated energies by different energy dissipating mechanisms has also been achieved, with energy dissipated by inelastic actions/components reaching 45% of the total internal energy.

Distribution of inelastic dissipated energy— The pie chart presented in Fig. 26 shows a break-down of the inelastic dissipated energy, namely the distribution of the total inelastic energy dissipated in the various parts of the Zone 3 structure. It can be observed that most of the inelastic energy is dissipated in the reinforced concrete columns supporting the roof and the special reinforced concrete shear walls, implying that the design intent has been realised.

Force-controlled actions/components— Nonlinear response history analysis outputs force demands on force-controlled actions/components. Sufficient strength capacities are provided for these actions/components for them to remain essentially elastic subjected to MCE shaking for both critical and non-critical force-controlled actions/components respectively as described previously.

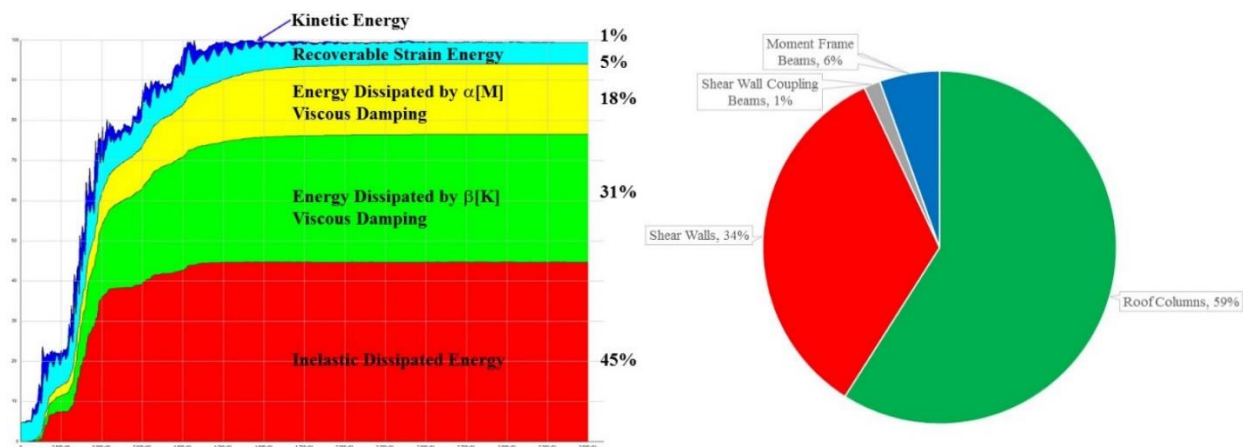


Figure 26— Energy balance and distribution of inelastic dissipated energy from one set of MCE time histories

INDEPENDENT PEER REVIEW

An important part of the performance-based seismic design methodology is the review process by independent peers possessing expertise and experience in the fields related to this newly emerging design approach. Recognizing this, the structural engineering team initiated a review process of the project-specific performance-based seismic design

methodology and criteria report and the site-specific probabilistic seismic hazard assessment report as well as the subsequent design and analysis work by engaging two internationally recognised experts at the earliest opportunity.

As the structural design of the Terminal 2 progressed, two US structural engineering firms were appointed as independent peer review teams to review the structural design basis and criteria reports, structural analysis models and calculations, as well as the contract documents. After completion of the structural design, the Owner commissioned a consortium of four European-based leading firms experienced in airport planning and design to conduct the Owner's own review. The structural engineering team worked closely with the peer review teams and closed out all review comments to the satisfaction of all parties.

CONCLUDING REMARKS

The performance-based seismic design methodology is increasingly employed by structural engineers to meet the demands of owners and users and to achieve the design intent of modern architecture of tall as well as low to intermediate-rise buildings. This paper has presented a successful application of the performance-based seismic design approach to the new Terminal 2 at the Tocumen International Airport in Panama City to meet these design goals and to achieve a fast-track project delivery. The performance-based design framework not only provides solutions to complex, non-code-prescribed structural configurations and systems and non-prequalified connections but also encourages and provides a framework for innovations such as the bespoke reinforced concrete equivalent of the “dog bone” beam-column connection fuse detail developed by the structural engineering team for this project. Hence, the performance-based methodology is increasingly employed by structural engineers to solve complex structural engineering problems that cannot be addressed within the conventional prescriptive framework.

ACKNOWLEDGEMENTS

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Revitalizing a Community Space Using Performance-Based Seismic Design

Saeed Fathali, Bret Lizundia, and Francisco Parisi

Synopsis: This paper summarizes the benefits and challenges of implementing performance-based seismic design (PBSD) for two concrete buildings of the Lower Sproul Plaza Redevelopment Project in one of the busiest areas of the UC Berkeley campus. The project included new construction of Eshleman Hall and the additions to Martin Luther King (MLK) Hall, and the seismic retrofit of the existing MLK Student Union as a result of the expansion. The peer-reviewed PBSD implemented three-dimensional nonlinear response history analyses at two levels of seismic hazard. The analytical simulations using pairs of near-fault ground motions, scaled to match the site-specific spectrum, were intended to establish the expected seismic behavior of the buildings under rare and frequent earthquakes. The choice of PBSD over code-prescriptive procedures was prompted by multiple layers of complexity of the project. Several challenges including those related to the horizontal and vertical irregularities, or connecting new and existing concrete buildings with different lateral force-resisting systems would have made a code-prescriptive design a cumbersome analytical endeavor without providing reliable insight about the expected seismic behavior of the buildings. The PBSD, however, proved a powerful framework to design for a reliably predictable seismic behavior with sufficient ductility, and a designated ductile hinge zones with sufficient confinement and shear capacity. The PBSD methodology also enabled the designers to avoid unnecessary conservatism to deal with the complexities, when designing drift- and acceleration-sensitive elements including the cladding system. Finally, the PBSD methodology allowed the design to consider all potential modes of failure of concrete elements retrofitted by FRP material including the debonding failure between FRP material and substrate.

Keywords: performance-based seismic design, acceptance criteria, fiber-reinforced polymer (FRP), hazard level, nonlinear response history analyses (NLRHA), reinforced concrete, seismic design, seismic evaluation

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INTRODUCTION

The Lower Sproul Redevelopment Project was a student-based initiative to reinvigorate an outdated existing central campus space in UC Berkeley. Encompassing a site area of approximately 184,000 SF (17000 m²), the project included strengthening of three existing buildings, namely Martin Luther King Jr. Student Union (MLK Student Union), Plaza/parking Garage, and Anthony Hall, and replacement of Eshleman Hall. This paper summarizes the motivations, methodologies, and benefits of implementing performance-based seismic design throughout this project focusing only on MLK Student Union (strengthening/retrofit) and Eshleman Hall (new design). The project was completed in 2015.

PROJECT SCOPE AND COMPLEXITIES

The Lower Sproul Plaza project redeveloped one of the busiest areas in UC Berkeley campus, and included work in multiple adjacent structures that were separated by seismic joints from each other. The following factors contributed to complexity to the project: new construction, retrofit of existing structures, use of different construction materials, desire for a higher seismic structural performance for the buildings, reduction of impact on the existing structures, and the close proximity among these different buildings in a limited construction site (including the adjacent Zellerbach Hall and Cesar Chavez Student Center, which remained open during construction). Given the large number of different goals for the use of concrete in various elements and buildings, including elaborate board-formed patterns in the exposed concrete walls, the project had nearly 20 different concrete mix designs.

Existing MLK Student Union

The existing 117,000 SF (10850 m²), five-story MLK Student Union, constructed in 1958, was renovated in this project by two major additions (on the west and south sides), adding 29,333 SF (2725 m²) of student mixed-use space to the existing building. Both south and west additions consist of a concrete basement and two-story steel framed structure above the plaza level with reinforced concrete shear walls, connected to the existing structure at all levels.



Figure 1— Two new two-story buildings structurally connected to west side (top image) and south side (bottom image) of existing MLK Student Union (images by project architect, Moore Ruble Yudell)

New Eshleman Hall

The existing Eshleman Hall (constructed in 1965) was demolished and replaced with a new building that consists of a 64,800 SF (6020 m²), five-story (plus basement) concrete structure with special reinforced concrete shear walls as the lateral force-resisting system. The gravity load-carrying system uses concrete slabs supported on concrete beams and columns. Some of the gravity beams were post-tensioned to control long-term creep deflections at the cantilever floors, which overhang the plaza at the upper levels on the north façade of the building.



Figure 3— Old Eshleman Hall (left) was demolished and replaced (Photos by Moore Ruble Yudell)



Figure 4— New Eshleman Hall (image by project architect, Moore Ruble Yudell)

New and existing structures connections

The connection between existing and new structures of MLK Student Union required very complicated detailing that dealt with various existing conditions, which in several instances differed significantly from the as-built information available. The structures also must accommodate their relative displacement through their perimeter seismic joints. Moreover, the bridge connecting MLK Student Union with Eshleman Hall was designed with an articulated end to accommodate the movement between the two buildings under a seismic event.

WHY PERFORMANCE-BASED SEISMIC DESIGN (PBSD)

The reasons and motivations for choosing PBSD over code-prescriptive design for this project can be summarized in three categories:

- 1) *Reliable prediction of building response despite the project complexities:* Several challenges such as the geometry of the buildings, connecting new and existing concrete buildings with different lateral force-resisting systems, and connecting two buildings with a pedestrian bridge would reduce reliability of the code-prescriptive design for this project. PBSD, on the other hand, enabled the design team to find insight into complex aspects of the response at the global and element level (e.g. the extent of hinge zone of concrete walls and distribution of story shear between the interconnected new and existing buildings).
- 2) *Limitations of code-prescriptive design:* Some of the current limitations of the code-prescriptive design would impact the understanding of the building seismic behavior and would limit the retrofit options. The two main limitations were lack of guidelines and provisions for using fiber elements to model flexural behavior of concrete walls (the current ASCE 41 standard is based on lumped plasticity modeling), and modeling of existing concrete elements retrofitted by FRP materials.
- 3) *Design optimization (avoiding unnecessary conservatism):* The code-prescriptive design of structures with complexities such as the buildings in this project with plan and vertical irregularities is often a cumbersome analytical endeavor with accumulative layers of conservatism. Implementing PBSD provided the design team with justifications to avoid those unnecessary conservative assumptions that would impact the design of drift-sensitive components such as the cladding systems of the two buildings.

PBSD METHODOLOGY

The PBSD methodology for this project was developed by the design team at Rutherford+Chekene as an “acceptance criteria document”. The document was peer-reviewed by Tipping Mar Associates and was approved by the UC Berkeley Seismic Review Committee (SRC) before implementation. The “acceptance criteria document” for each building provided answers to three main questions:

- 1) What levels of earthquake hazard concern the seismic evaluation and/or design of the building?
- 2) How to establish the building responses at each hazard level (modeling assumptions) reliably?
- 3) What are the accepted global building responses and component-level responses at each hazard level?

The following sections provides a summary of the acceptance criteria document for the two buildings that are the focus of this paper.

Earthquake hazard levels

For design of new Eshleman Hall, two different levels of seismic hazard were considered: the hazard level corresponding to a return period of 475 years (Design Earthquake or DE), and the hazard level corresponding to a return period of 975 years (BSE-C). For the renovation of MLK, on the other hand, the two hazard levels correspond to a return period of 225 years (BSE-R) and 975 years (BSE-C) in accordance with the Chapter 34 requirements of California Building Code (CBC) 2013. Note that in 2013 CBC Section 3417.5, if the floor area of an addition is greater than 50% of the floor area of the existing building, the BSE-R and BSE-C levels are replaced with the somewhat larger BSE-1 and BSE-2 levels, respectively. Since the size of the two additions to MLK Student Union did not exceed the 50% threshold, the hazard levels were BSE-R and BSE-C. For the purpose of brevity and clarity, from this point on, this paper refers to DE (BSE-R in case of MLK Student Union) as “LS” (Life Safety), and to MCE (BSE-C in case of MLK) as “CP” (Collapse Prevention).

Seismic response simulations

Seismic responses of each building were established by nonlinear response history analyses (NLRHA) on three-dimensional (3D) analytical models developed using Perform3D Version 5. In the following section, selected key assumptions and procedures used to develop the analytical models for NLRHA are described.

Reinforced concrete walls— The in-plane bending and axial behavior of the reinforced concrete walls were modeled using fiber elements with the corresponding expected material properties. The shear stress-strain relationship of the wall, on the other hand, assumed nominal material properties for the LS-level analyses, and expected material properties for the CP-level analyses. The stress-strain relationship of the steel fibers of the reinforced concrete walls used a simplified tri-linear model. During the peer review process, it was verified that, for the range of acceptable seismic response of the building, consideration of strength loss or cyclic degradation of steel fibers was unnecessary. The stress-strain relationship of concrete fibers of the reinforced concrete walls used a simplified tri-linear model as well; however, the model considered strength loss under large compressive strains, but did not include cyclic degradation. The shear behavior of the reinforced concrete walls was modeled using nonlinear shear stress-strain behavior (with elastic-perfectly-plastic backbone curve and degradation). The ultimate strength of the shear stress-strain curve of shear materials was defined using the ACI 318-08 formula for shear capacity of reinforced concrete walls. The elastic shear modulus was assumed to be 20% of that of uncracked reinforced concrete (based on recommendations of ATC 72-1, Chapter 5), and the yield shear strength was assumed equal to 60% of the ultimate shear strength which was established without a reduction factor. The idealized stress-strain relationship considered the strength loss (beyond 1% shear strain) but would ignore the effects of cyclic degradation. Although the model included strain limit states associated with the nonlinear shear materials of the walls, shear design of the walls was not based on such limit states. Rather shear design of the walls was based on stress limit states that would confirm shear stress demand would not exceed shear stress capacity calculated per ACI 318-08 (see Table 2 for reduction factors and material strength assumptions).

Reinforced concrete beams and columns— To model the concrete beams and columns, lumped plasticity models were used. To establish simplified backbone curves of each concrete beam or column cross section, expected material properties were used as input to a fiber-based section analysis program. Both LS- and CP-level analyses assumed expected material properties to model flexural behavior of beams and columns. For the range of acceptable seismic response of the building, consideration of strength loss or cyclic degradation for frame elements was verified unnecessary.

Diaphragms— Except for Level 2 of Eshleman Hall, all analytical model nodes at each level were constrained to each other by a rigid diaphragm assumption. At Level 2 of Eshleman Hall, the nodes on the east and west side of the bridge were assigned to separate rigid diaphragms. The total translational mass in the horizontal direction and mass moment of inertia around the vertical axes were lumped at the location of center of mass of each diaphragm.

Global P-Delta effect— The gravity loads contributing to the global P-Delta effect were assigned to the end points of columns in the model.

Soil-structure interaction— The soil compression forces under the foundation and behind retaining walls were simulated in the analytical models of the buildings using compression-only elastic springs. The stiffness values of the springs representing the compression forces under the foundations were calculated based on the tributary area of the spring. During the peer review process, it was verified that assuming elastic behavior under compression forces was justified for the soil springs as they were not subjected to the forces corresponding to the soil yielding.

Bounding analyses— Based on the results of a bounding analyses conducted for stiffness values of the springs representing the soil-structure interaction (provided by the geotechnical report for the project), it was concluded that most building response quantities are not affected by variation of spring stiffness values between lower and upper bounds. However, for some response quantities including the total drift and soil pressure, the analyses using lower-bound values of soil spring stiffness resulted in larger responses. Thus, it was decided to perform the entire NLRHA using only lower-bound values of soil spring stiffness.

Ground motions— For each seismic hazard level, site-specific response spectra were developed which considered all aspects of the site including the base-slab averaging and embedment effects per FEMA 440. Then, seven pairs of horizontal ground motions with fault-normal and fault-parallel components were selected and scaled to match the

corresponding spectrum. The angle to apply the fault-normal component of the ground motions were defined based on the location of the buildings relative to the Hayward fault.

Acceptance Criteria

The structural design of the buildings in this project aimed to achieve a seismic behavior that is governed by ductile nonlinear actions at targeted elements and essentially elastic response of remaining structural elements. The targeted nonlinear actions that will help dissipate earthquake energy include limited flexural yielding of the concrete walls at designated hinge zones, and limited flexural yielding of frame elements. On the other hand, the design ensured that brittle nonlinear actions such as shear failure of concrete wall piers or frame elements are avoided.

A set of acceptance criteria was developed for the building global response and individual structural element response under each of the LS and CP levels. The demands at LS level were calculated using a numerical model that assumed expected material for flexural behavior, and nominal material properties for shear behavior of structural walls. The demands at CP level, on the other hand, were calculated using a numerical model that used expected material properties for both shear and flexural stiffness. Although nonlinear shear behavior of the walls were modeled and accepted under individual ground motions, per Table 2, concrete walls were checked and retrofitted if needed to have a capacity larger than the mean peak shear response.

At both hazard levels for each building, per Section 16.2.4 of ASCE/SEI 7-05, the mean value of peak response under seven pairs of ground motions was used to show the corresponding design criterion is met. Tables 1 through 4 summarize the key parts of the project acceptance criteria.

Table 1— Acceptance criteria for global response

Design Check Name (Type)	Acceptance Criteria
Peak transient interstory drift (deformation controlled)	For all story levels, mean value of interstory drift under seven pairs of ground motions shall not exceed 2.5% ^a at LS and 3.0% ^b at CP level.
Residual interstory drift (deformation controlled)	Mean value of three largest residual drifts under seven pairs of ground motions shall not exceed 1.0%, and peak residual interstory drift shall not exceed 1.5% ^c at both LS and CP levels.
Sliding at foundation-soil interface (force controlled)	Maximum base shear under seven pairs of ground motions normalized by seismic weight shall not exceed ultimate coefficient of friction at the foundation-soil interface multiplied by a reduction factor of 0.67 ^d at LS level and without a reduction factor at CP level.

^a Per Section 16.2.4.3 of ASCE/SEI 7-05, the allowable drift per Table 12.12-1 can be increased by a factor of 1.25 (total interstory drift includes contribution of rotational displacements (due to building rocking over the foundation), as well as deformation of structural elements).

^b Per professional judgment and a similar set of design criteria in Section 8.7.1.1 of PEER/TBI Guidelines for Performance-based Seismic Design of Tall Buildings (2010)

^c Per professional judgment and similar set of design criteria in Section 8.7.12 of PEER/TBI Guidelines for Performance-based Seismic Design of Tall Buildings (2010)

^d $\phi = 0.67$ corresponds to the safety factor of 1.5.

Table 2—Acceptance criteria for reinforced concrete walls

Design Check Name (Type)	Acceptance Criteria
Elastic shear behavior of walls (force controlled)	For all wall piers and spandrels, mean value of shear force demand under seven pairs of ground motions shall not exceed their shear capacity ^a .
Tensile strain of flexural steel reinforcement (deformation controlled)	For all wall piers and spandrels, mean value of tensile strain of flexural steel reinforcement under seven pairs of ground motions shall not exceed 6% ^b at both LS and CP levels.
Concrete crushing (deformation controlled)	For all wall piers and spandrels, mean value of concrete compressive strain under seven pairs of ground motions shall not exceed 0.15% ^c at both LS and CP levels.

^a Shear capacity of the walls (both existing and new) is calculated per Section 21.9.4, ACI 318-08 using nominal material properties and $\phi = 0.75$ at LS level and $\phi = 1.0$ at CP level.

^b Flexural steel rebars are directly modeled using the expected strength value and a nonlinear model considering strain hardening. The 6% strain considered for the acceptance criterion is the fracture strain recommended by Priestly et al. (Displacement-Based Seismic Design of Structures, 2007), which considers concentration of strain due to concrete cracking.

^c The compressive strain of concrete is directly monitored in the nonlinear analytical model using fiber elements with expected properties of confined concrete. The 0.15% compressive strain considered for the acceptance criterion is half of the specified concrete crushing strain defined by ACI 318-08 based on a study by Orakcal and Wallace (2006) that showed nonlinear models such as the one used for this building underestimate the actual demand of concrete compressive strain by a factor of 2.

Table 3—Acceptance criteria for reinforced concrete beams and columns

Design Check Name (Type)	Acceptance Criteria
Elastic shear behavior of frame elements (force controlled)	For all concrete beams and columns, mean value of shear force demand under seven pairs of ground motions shall not exceed their shear capacity ^a .
Concrete beam plastic rotation (deformation controlled)	For all concrete beams, mean value of plastic rotation shall not exceed 0.02 and 0.04 radian ^b at LS level and CP levels, respectively.
Concrete column rotation (deformation controlled)	For all concrete columns, mean value of plastic rotation shall not exceed 0.018 and 0.025 radian ^c at LS level and CP levels, respectively.

^a Shear capacity of beams and columns is calculated per Section 11 of ACI 318-08 using nominal material properties and corresponding reduction factor at LS level and without a reduction factor at CP level

^b Per Table 6-7, ASCE/SEI 41-06 (minimum acceptance criteria for secondary concrete beams with conforming transverse reinforcement at Life-Safety (LS) and Collapse-Prevention (CP) performance level).

^c Per Table 6-8, ASCE/SEI 41-06 (minimum acceptance criteria for secondary concrete columns with conforming transverse reinforcement at Life-Safety (LS) and Collapse-Prevention (CP) performance level).

Table 4— Acceptance criteria for foundations

Design Check Name (Type)	Acceptance Criteria
Elastic shear behavior of foundation (force controlled)	Mean value of shear demand under seven pairs of ground motions shall not exceed shear capacity of the foundation calculated using nominal material properties and reduction factors at LS level and expected material properties and without reduction factors at CP level.
Soil pressure (force controlled)	Mean soil pressure under seven pairs of ground motions shall not exceed allowable soil pressure at both LS and CP levels.

KEY BENEFITS OF IMPLEMENTING PBSD

Optimized design of ductile lateral force-resisting systems

PBSD enabled the design team to detail the lateral force-resisting systems of the two buildings to achieve reliably predictable seismic behavior with sufficient ductility. The detailed insight about the global and local behavior of the building gained by implementing PBSD facilitated the optimization of the concrete walls layout of the additions to the MLK Student Union (to minimize the impact on the existing structural elements with different material and behavior). Moreover, modeling the flexural behavior of the walls by fiber elements provided the key to determine the hinge zones and the horizontal and vertical extent of boundary zones. Fig. 5 showcases how the PBSD relied on the results of analyses to detail the boundary elements of Eshleman Hall concrete core walls.

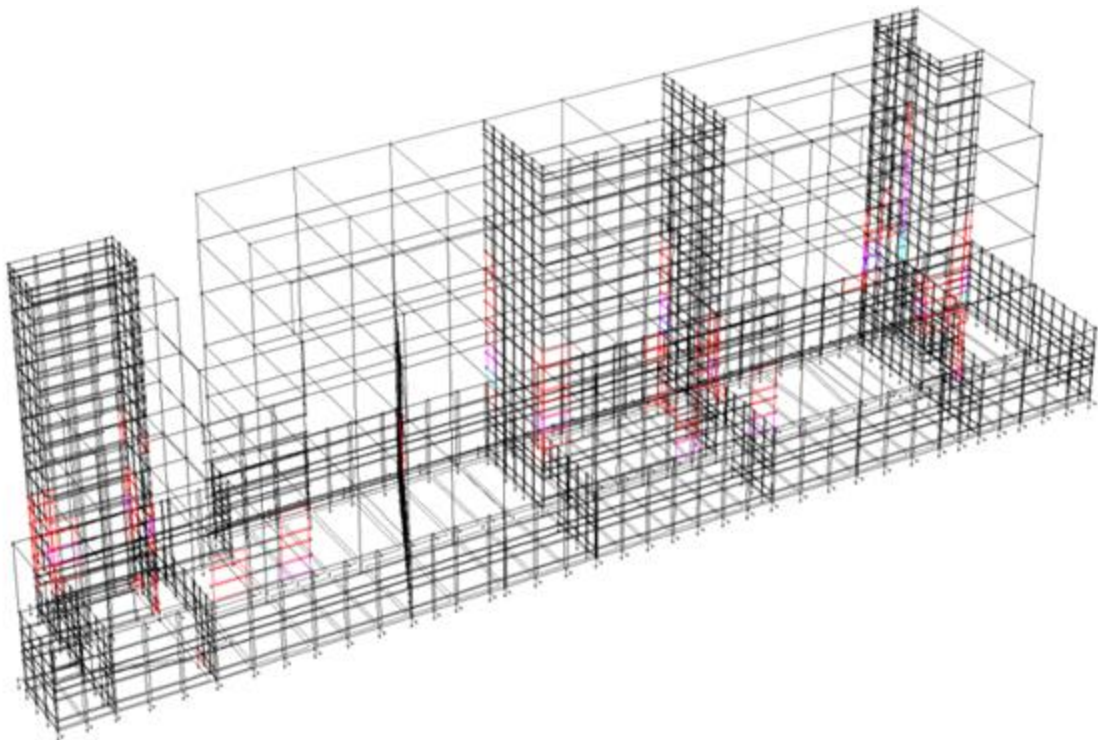


Figure 5— Using colored elements to check concrete compressive strain does not exceed its corresponding acceptance criteria and to establish the horizontal and vertical length of boundary zone; red, yellow, green and blue elements correspond to compressive strain of 0.15, 0.11, 0.08, and 0.04%, respectively

Optimized design of drift- or acceleration-sensitive components

PBSD provided the design team with drift values (see Figs. 6 and 7) that were used to design the cladding system, and the bridge connecting MLK with Eshleman Hall, with an articulated end to accommodate the movement between the two buildings (translational and rotational movement) under an earthquake (see Fig. 8). The corresponding design values per code provisions would be conservatively larger than those based on PBSD.

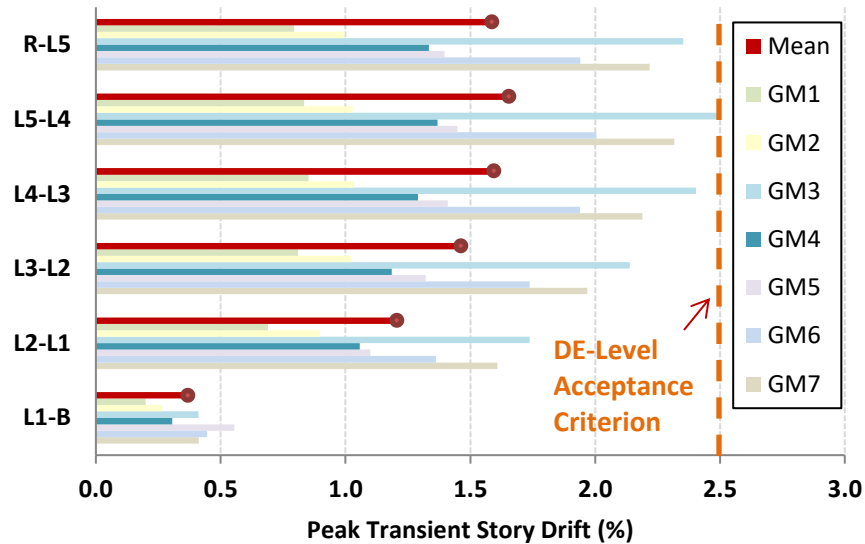


Figure 6— Peak transient interstory drift values of Eshleman Hall under seven DE-Level ground motions

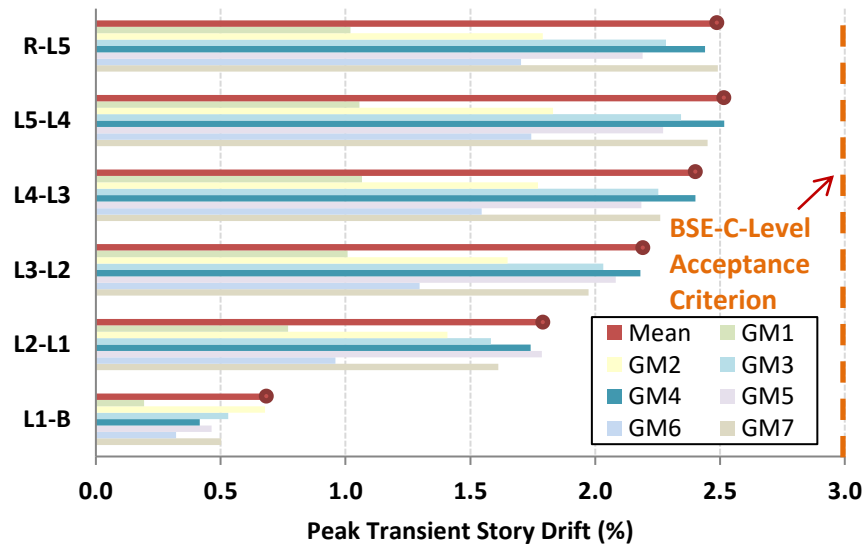


Figure 7— Peak transient interstory drift values of Eshleman Hall under seven BSE-C-Level ground motions



Figure 8— Bridge connecting MLK with Eshleman Hall, with an articulated end to accommodate the movement between the two buildings under an earthquake



Figure 9— Connections between existing MLK diaphragms and new diaphragms

Modeling and design of existing elements retrofitted with FRP material

The preliminary pushover analyses of MLK Student Union indicated that connecting the two new buildings on the west and south sides of the MLK Student Union would stiffen the first two levels of MLK Student Union and would drive the seismic forces to the upper stories of the building. The analyses also showed that while in the EW direction, the behavior will be governed by the flexural yielding of the walls at Level 3 of the existing building (see Fig. 10), in the NS direction the walls would suffer shear failures that would lead to a story mechanism (see Fig. 11).

To prevent such brittle behavior the design team decided to use carbon fiber-reinforced polymer (CFRP) strips to increase the shear capacity of the NS Walls of Level 3. The analytical modeling of the walls retrofitted by CFRP material would be a challenge if the project had pursued a code-prescriptive design (the current ASCE 41 code does not have provisions for modeling and acceptance criteria of retrofitted elements). However, thanks to the peer-reviewed PBSD methodology implemented for the project, the design team was able to develop suitable modeling assumptions and corresponding acceptance criteria for the inelastic shear behavior of the walls retrofitted by CFRP material.

The modeling assumed that the elastic shear stiffness and the ratio between yield and ultimate shear strength ($F_y/F_u = 0.6$) remains unchanged after adding the CFRP to the existing wall. However, the external application of CFRP would increase the ultimate strength of the wall. In the absence of sufficient experimental or numerical studies to capture the shear behavior of concrete walls retrofitted with FRP, it was assumed that under shear strains larger than 0.004 the CFRP no longer contributes to the shear strength due to the bond failure.

Fig. 12 is an example to demonstrate how the nonlinear shear stress-strain relationship of the walls retrofitted with CFRP was modeled. The blue line in this figure corresponds to the shear stress-strain relationship of an existing 12-inch-thick wall (305 mm-thick wall). The orange line in this figure would be a less conservative model that inherently assumes the CFRP overlay and its bond to the existing wall can undergo the same shear strains as the wall without an overlay. The purple line would be a more conservative model which inherently assumes that shortly after 0.004 shear strain, the contribution of the CFRP overlay to the wall shear strength rapidly diminishes and the behavior of the wall would then become identical to that of the existing wall. Commercially available analysis programs such as the one used for this project do not have the option of modeling the post-degradation behavior of concrete material with multiple segments. Therefore, the black line was selected as the simplified version of the purple line such that the area under the black curve is close but smaller than the area under the purple curve.

External application of FRP materials on the walls of MLK Student Union to change their behavior from shear governed to flexure governed was an elegant and cost-effective retrofit solution that was possible since PBSD approach allowed the design team to work with the peer reviewer and develop appropriate modeling and acceptance criteria.

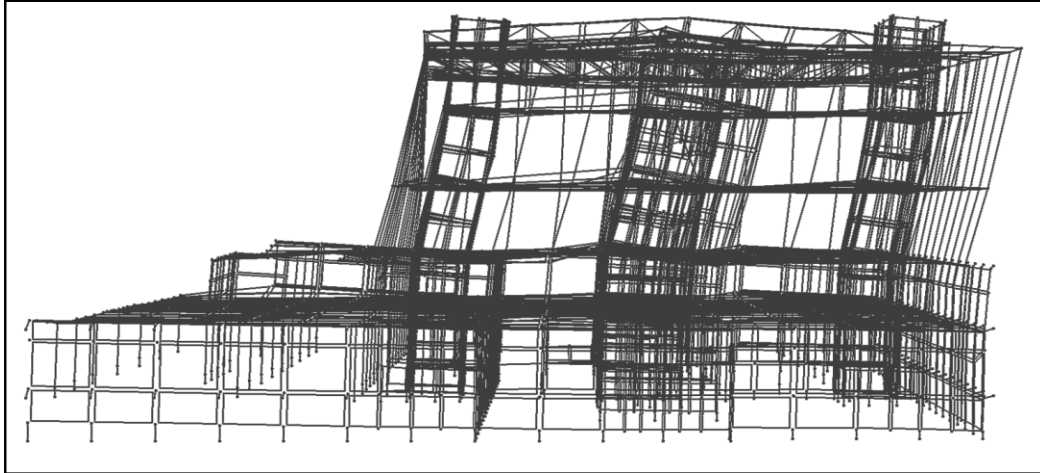


Figure 10— Pushover analyses of MLK Student Union (push to East) showed that addition of new buildings on west and south side would result in flexural hinging of EW walls above Level 3

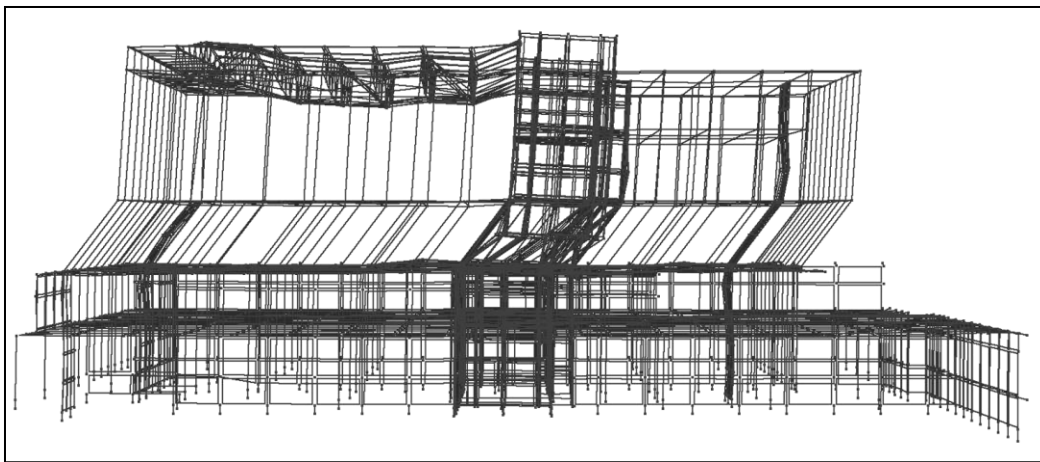


Figure 11— Pushover analyses of MLK Student Union (push to South) showed that addition of new buildings on west and south side would result in a shear failure of existing NS walls above Level 3

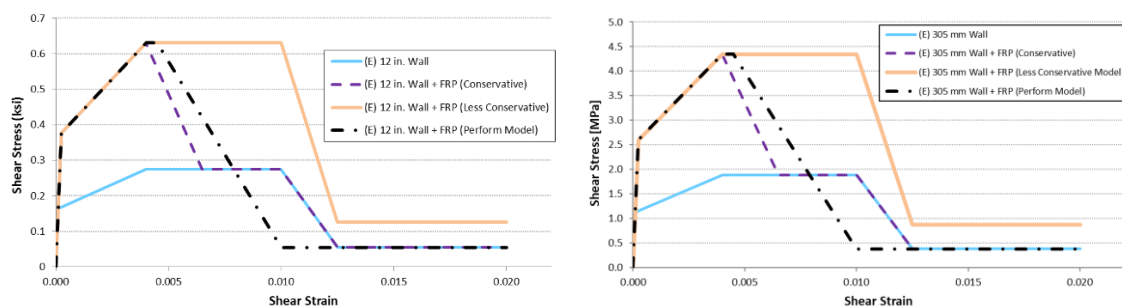


Figure 12— Comparison of inelastic shear behavior modeling of existing and retrofitted wall (right: SI units, Left, Imperial units)

CONCLUSIONS

Seismic design of for the concrete buildings of the Lower Sproul Plaza Redevelopment Project in one of the busiest areas of the UC Berkeley campus is an example of the state-of-practice of performance-based design methodologies for concrete buildings. The peer-reviewed PBSB implemented in this project proved useful and even essential to optimize the seismic design, to avoid unnecessary conservative assumptions that are conventionally used to address the complexity of a project, and enabled the project to take advantage of a retrofit technology (externally applied FRP material) that is currently not addressed by the code-prescriptive methodologies. In many cases, such as design of drift-sensitive elements, and connections between new and existing elements, PBSB provided realistic design values that resulted in cost saving compared to alternative code-prescriptive methods. The readers are advised to note that although the overall framework presented here is applicable to any other project, the specifics of modeling assumptions or acceptance criteria may or may be suitable for another project. The details of the PBSB implemented in this project were developed by the design team and approved by the peer reviewer based on all the characteristics and circumstances specific to the project.

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First Performance-Based Seismic Design Tower in Oakland, California

Devin Daniel and Ian McFarlane

Synopsis: The use of a Performance-Based Seismic Design (PBSD) approach to design buildings that exceed 240-feet (73.2 m) tall has been common among many west coast cities. More recently, Oakland, California has been an epicenter of development that has created a market for taller buildings. The residential tower at 1640 Broadway, which is currently under construction, is the first tower designed using PBSD exceeding 240-feet (73.2 m) tall in Oakland. This is notable in terms of establishing the implementation of PBSD in a new jurisdiction. This is also notable because of the near fault location of Oakland, given that the Hayward fault is less than 3.1 miles (5 km) from the downtown region, which raises new issues such as fault normal/fault parallel ground motion scaling issues and designing for extremely high demand levels. Due to these extreme demand levels, the project consisted of high reinforcement ratios within the walls and embedded steel coupling beams. Finally, the foundation conditions were challenged by the proximity to BART tunnels and therefore consist of a hybrid mat foundation supported on deep soil mixed panels and cased steel piles. A summary of the unique aspects of the building are presented and compared with typical code compliant and PBSD towers.

Keywords: performance based seismic design, near-source, soil-structure interaction

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INTRODUCTION

The use of a Performance-Based Seismic Design (PBSD) approach to design buildings that exceed code-prescriptive building height limits has grown rapidly among many west coast cities within the last decade. Oakland has been an epicenter of new development, creating a demand in the market for high-rise towers. Traditional code-compliant design approaches impose significant impediments to the fiscal sense of a real estate development project. For high-rise towers located in high-seismic zones (Seismic Design Categories D, E or F) and exceeding 240-feet (73.2 meters) in height; a code-compliant design would prescriptively require an additional special moment frame system or "dual-system". This classical approach to building design imposes restriction on unit layouts, utility routing, perimeter windows and balconies that otherwise contribute to the overall value of a given project.

At 330-feet (100.6-meters) tall the 34-story high-rise tower at 1640 Broadway is the first PBSD project to exceed 240-feet (73.2-meters) in Oakland, California. The primary use of the building is residential; at 416,000 gross square feet (38,648 square meters) of developed area, the tower will accommodate 254 rentable units with above grade parking, amenity spaces and ground level retail.



Figure 1— Residential tower at 1640 Broadway (rendering by Solomon Cordwell Buenz)

The lateral force resisting system consists of special reinforced concrete shear walls surrounding a central circulation core. An additional shear wall was provided at the easternmost face to provide torsional stability. Concrete coupling beams are either embedded steel reinforced, diagonally reinforced or conventionally reinforced, depending on the length/depth (L/d) ratio and the shear stress ratio. Above the ground level, 8-inch post-tensioned concrete floor slab diaphragms deliver distributed inertial loads to the lateral force resisting system. Additionally, at the tower-podium interface, a 12-inch post-tensioned slab is used to distribute transfer forces between the structure's lateral force resisting elements. At ground level, a concrete structural slab on grade is used as a structural diaphragm to engage all foundation elements. The structure has several columns that transfer at the tower-podium interface to allow for drive-isle clearances in the lower parking levels. These transfers are possible through the use of six-foot deep by seven-foot wide non-prestressed transfer beams.

The primary tower footprint is supported on a mat foundation of variable thickness, ranging from 10-feet (3-meters) thick at the center to 6-feet (1.8-meters) thick at the perimeter. The podium structure is supported by a combination of spread-footings and pile foundations. An additional below grade consideration is the location of the Bay Area Rapid Transit (BART) tunnels on the east side of Broadway. BART has developed strict guidelines for construction near their below grade structures.

PERFORMANCE BASED SEISMIC DESIGN IN THE CITY OF OAKLAND

The California Building Code (CBC) is the governing standard for the design of buildings in the state of California. Additionally, for the majority of the loads imposed, such as snow, wind and seismic, the CBC references to ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures*, for design requirements. The requirements under these standards are prescriptively-based, requiring the loading and subsequent design to adhere to the provisions specified in the standard. These standards impose limitations to restrict the type, location and height of the seismic force resisting systems used to meet compliance. Due to these limitations, PBSD alternatives have been introduced throughout the United States to provide engineered solutions to meet owner and architectural design directives. The use of a PBSD approach is accepted under the “alternative means and methods” clause of the IBC, Section 104.11: “The provisions of this code are not intended to prevent the installation of any material or prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.”

While the PBSD approach has been in use for some time, the actual implementation varies from jurisdiction to jurisdiction. The tower at 1640 Broadway is significant in terms of establishing the framework for PBSD practices in a jurisdiction where it has not previously been applied. The PBSD approach focuses on the behavior and performance under design objectives rather than prescriptive demands under design requirements. It is the intent for a building designed under the PBSD approach to meet or exceed the requirements presented within the code for promoting the health, safety, and welfare of its occupants.

Because of the complexity of the analysis used to demonstrate building behavior and performance, most building departments have implemented a requirement for an independent peer review panel when designs are submitted for permit under the alternative means and methods clause of the IBC. Prior to initiating the design, Magnusson Klemencic Associates (MKA) briefed the City of Oakland on the current state of practice implementing the PBSD approach among major west coast jurisdictions. This brief included minimum acceptable scope of work for the peer review panel, composition of the structural peer review panel, review timeline, and interaction between the city and structural peer review panel. Based on this discussion, the City of Oakland agreed upon a standard for handling future PBSD projects. Within a few months, MKA approached the City of Oakland with a detailed proposal to proceed with a PBSD approach for the tower at 1640 Broadway, after which, the PBSD process was initiated.

Due to the complexity of the seismic issues in the vicinity to the City of Oakland, it was deemed necessary for the structural review panel to consist of a licensed professional structural engineer and a licensed professional geotechnical engineer. The structural engineer’s objective is to review the behavior and performance of the super structure in relation to the performance objectives. This requires the review structural engineer to be well versed in performance-based seismic design (PBSD). The geotechnical engineer’s objective on the other hand is to review the seismic hazard used in the design of the superstructure. This requires the review geotechnical engineer to have ample knowledge of near-source seismic hazard considerations. Additionally, the geotechnical engineer reviews the geotechnical design as related to the foundation support.

Table 1 — Performance objectives for risk category II

Level of Earthquake	Structural Performance Objectives
Serviceability: 50% probability of exceedance in 30 years (43-year return).	Serviceability: Structure to remain essentially elastic with minor yielding of structural elements, minor cracking of concrete and minor damage to non-structural elements.
Design Basis Earthquake (DBE): As defined by ASCE 7-10, two-thirds of the magnitude of the MCE_R ground motions.	Code Level: Moderate structural damage; extensive repairs may be required, without generation of significant hazards to individual lives through design measures intended to assure that non-structural components and systems remain anchored and secured to the structure and that building drifts are maintained at levels that will not create undue hazards.
Risk-Targeted Maximum Considered Earthquake (MCE_R): As defined by ASCE 7-10.	Collapse Prevention: Structure has a low probability of collapse (on the order of 10 percent) which will be demonstrated implicitly through analyses that indicate the structure has stable, predictable response to MCE_R shaking at response levels which do not result in loss of gravity load carrying capacity or substantial degradation of lateral resistance. Extensive structural damage may occur; repairs to structural and non-structural systems are required and may not be economically feasible.

REGIONAL SEISMICITY AND DESIGN CONSIDERATIONS

Several major fault lines exist within close proximity to the City of Oakland. The transform boundary often defined by the San Andreas Fault is not a straight line; rather the movement between the Pacific and North American plates is diffused over a conglomeration of fault lines with many capable of producing high-seismic demands. The Hayward-Rogers Creek Fault (sub-fault), the San Andreas Fault (primary-fault) and the Calaveras (sub-fault) are located at 3.4-miles (5.5 kilometers), 14.3-miles (23 kilometers) and 14.9-miles (24 kilometers) respectively from the City of Oakland.

It is the intent of a performance-based approach to provide a building that meets or exceeds code-equivalency. However, the detailed nature of the performance-based approach often leads to informing upon the code-based design process and the standards that comprise it. The process takes into consideration many of the uncertainties that remain inherent in the frequency and magnitude of potential events and provides a better overall estimate of the actual response of the structure during a seismic event. In context with the site, the seismic hazard for the design earthquake from a code-based approach was determined to be approximately two-thirds the seismic hazard under a site-specific, performance-based approach. This ratio was determined at the period range of interest for the tower, where the primary modes of translation are approximately 3.7 seconds and 3.1 seconds in the east-west and north-south directions respectively. The code-based design under ASCE/SEI 7-10 utilizes NGA West 1 hazard analysis data whereas the site-specific information is based on the more current dataset, NGA West 2. NGA West 2 provides a better prediction for buildings within the $t > 1$ second period range built upon soft soil structures. ASCE/SEI 7-16 requires site-specific analyses for site classes D and E structures and utilizes NGA West 2 hazard data.

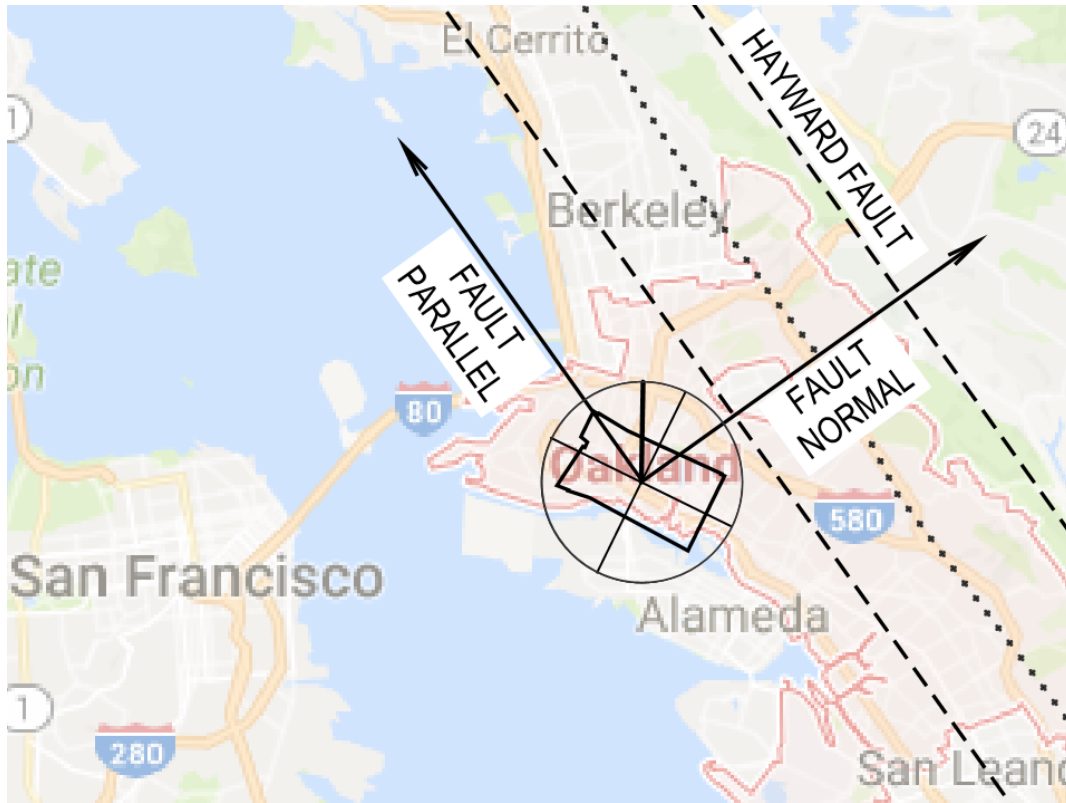


Figure 2— Site orientation and proximity to the Hayward fault

Table 2— Mean probability of occurrence

Fault	Approximate Distance from Fault	Mean Characteristic Moment Magnitude	Probability of Occurrence
Hayward-Rodgers Creek	3.4 miles (5.5 kilometers)	7.33	31 %
Mount Diablo	13.7 miles (22 kilometers)	6.70	1 %
Calaveras	14.3 miles (23 kilometers)	7.03	7 %
N. San Andreas	14.9 miles (24 kilometers)	7.23	21 %
Concord-Green Valley	16.8 miles (27 kilometers)	6.80	3 %
San Gregorio	18.6 miles (30 kilometers)	7.50	6 %

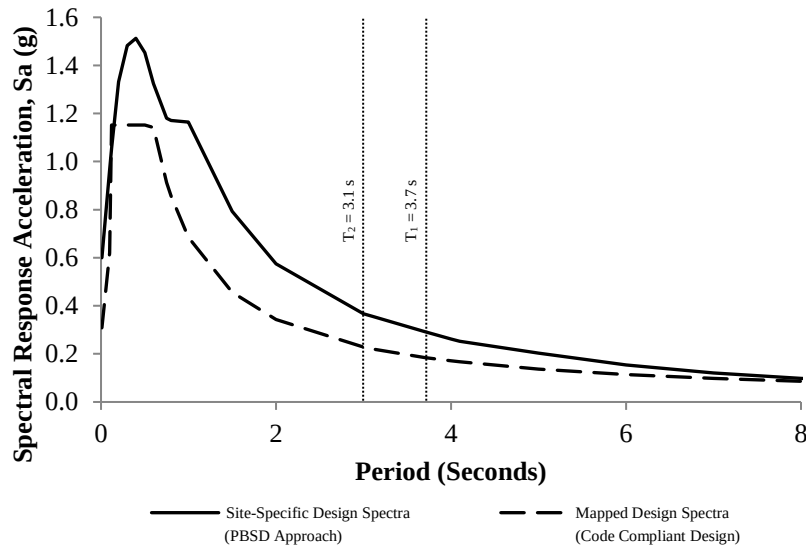


Figure 3— Site-specific design earthquake response spectra

For a code-based design approach, a dual lateral force resisting system is required for structures exceeding 240-ft in height (ASCE 7-10, Table 12.2-1). The lateral system consists of special reinforced concrete core/ blade walls with an additional back-up moment frame capable of resisting twenty-five percent of the system's total base shear. In order to accommodate this design requirement, back-up moment frames can become an extensive portion of the primary structure. The core wall stance for the code-based design is 29'-6" (9.0 meters) in the north-south direction and 36'-6" (11.1 meters) in the east-west direction. The blade wall stance is 30'-0" (9.1 meters). The typical floor-to-floor height for most residential towers ranges from 9'-6" to 10'-0" (2.9 to 3.0 meters). The incorporation of a back-up moment frame system can restrict unit-layouts, mechanical and electrical routing, internal circulation and increase the total amount of formwork required at all floor levels, all with added costs to the project. The routing of mechanical systems at moment frame beams traditionally is represented by reduced rentable area due to an increase of vertical distribution shafts or lower ceiling heights for horizontal distribution under beams. In context with the tower at 1640 Broadway, this consisted of approximately six 2'-6" (0.76 meters) wide by 2'-6" (0.76 meters) deep moment frame beams located near and around the perimeter of the tower floor plate (see Fig. 4).

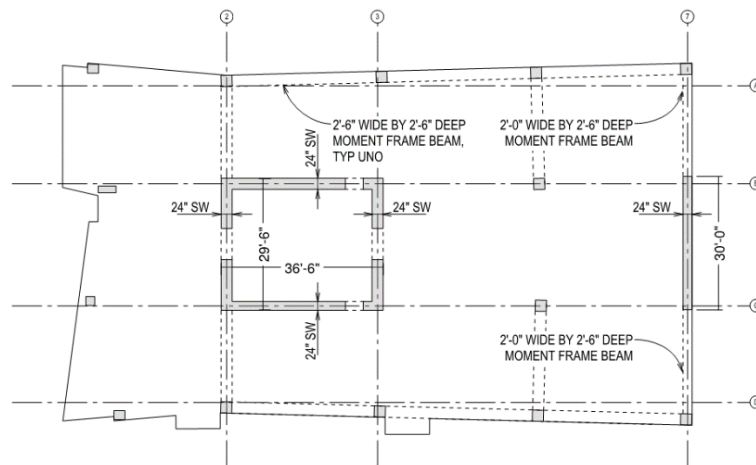


Figure 4— Code-complaint design approach

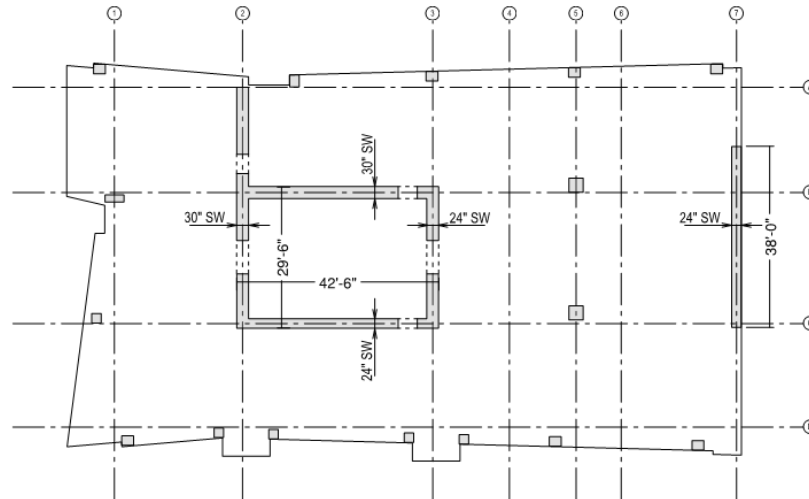


Figure 5— Performance based design approach

The performance-based approach requires an increase in the overall capacity of the system to match the increase in seismic demands. However, the lateral system maintains a reduced overall footprint in comparison the code-based design approach. The performance-based design approach requires a larger core-stance in the east-west direction. The primary core-stance is 29'-6" (9.0 meters) in the north-south direction and 42'-6" (12.9 meters) in the east-west direction. Additionally, the design required an added wing-wall in the north-south direction matched with increased length of the easternmost face blade wall to maintain torsional balance. The western most core wall has a total stance of 50'-6" (15.4 meters) while the eastern most blade wall has a stance of 38'-0" (11.6 meters). The performance-based approach required a slightly higher concentration of core wall longitudinal and horizontal reinforcing. In order to target an acceptable building drift, additional reinforcing was added to zones of high strain to reduce the towers overall floor-to-floor drifts. Additionally, the post yield behavior (secant stiffness) of traditional concrete coupling beams represents a substantial reduction in stiffness. This post yield behavior inhibits the cores ability to act as a coupled system. Steel coupling beams were added along the western wall to promote a more stable post yield behavior.

NEAR-SOURCE GROUND MOTION SELECTION

Recommended site-specific MCE_R ground motions were developed in accordance with ASCE 7-16 (2016) and PEER TBI Version 1.0 (2010). Two suites of eleven ground motions were used for the nonlinear response history analysis (NRHA). These suites represent a Conditional Mean Spectra (CMS, "Method 2" per ASCE 7-16, Section 21.2.1) with a conditioning range of primary translation periods for the long-period spectral suite, and secondary translational periods for the short-period spectral suite. The intent of ASCE 7-16 is for the average of the maximum direction spectra (ROT_{D100}) determined from the suite of eleven ground motions to meet or exceed the target response spectrum throughout the period range of interest. The period range of interest for each of the spectral suites is defined as $0.2T$ to $2.0T$, where T is the target period. Figs. 6 and 7 present the Site-Specific Uniform Hazard Spectrum (denoted as MCE_R) as well as the Conditional Mean Spectrum (denoted as Avg. (μ)) for both short-period spectral suites and long-period spectral suites respectively.