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LAX MIDFIELD SATELLITE CONCOURSE EXPANSION: ROLE OF PERFORMANCE-BASED SEISMIC ANALYSIS IN THE DESIGN OF THE NEW SUPERJUMBO AIRCRAFT TERMINAL

Saiful Islam, Ph.D. S.E.(1), M. Skokan, Ph.D. S.E.(2), T. Bouquet, S.E.(3),
D.W. Kim, Ph.D.(4), R.Poomiwatracanont P.E.(5)

(1) *President, Saiful/Bouquet Structural Engineers, Inc., Pasadena, CA, USA, Saiful@saifulbouquet.com*
(2) *Principal, Saiful/Bouquet Structural Engineers, Inc., Pasadena, CA, USA, mskokan@saifulbouquet.com*
(3) *Managing Principal, Saiful/Bouquet Structural Engineers, Inc., Pasadena, CA, USA, tbouquet@saifulbouquet.com*
(4) *Senior Analyst/Engineer, Saiful/Bouquet Structural Engineers, Inc., Pasadena, CA, USA, DKim@saifulbouquet.com*
(5) *Senior Project Engineer, Saiful/Bouquet Structural Engineers, Inc., Pasadena, CA, USA, Royp@saifulbouquet.com*

Abstract

The new LAX Midfield Satellite Concourse (MSC) construction project is ongoing modernization of the Los Angeles Internation Airport. Essential facility like an airport is designed based on the importance factor (I) of 1.5 to achieve better seismic performance. That is, seismic design force is increased by 50% and the expected nonlinear story drift is reduced by 67% as compared to a typical building which has I = 1.0. As a result, it is expected that the building structure designed with I = 1.5 would be stronger and stiffer. However, the performances of buildings in past earthquakes have shown that there is no direct relationship between the prescriptive code approach and expected seismic performance. In addition, higher structural costs is not avoidable if I = 1.5 is used for the design. In these reasons, the procedure used for this project is to design the building based on the prescriptive requirements of the building code using an importance factor of only 1.25 followed by performance-based design using a nonlinear dynamic time history analysis to achieve the enhanced structural performance levels; (1) "O*perational*" after frequent earthquake (43-year return period), (2) "I*mmediate Occupancy*" after rare earthquake (475-year return period), and (3) "L*ife Safety*" after very rare earthquake (2475-year return period).

This paper presents the results of this design methodology and demonstrates that this approach proves to be a much more efficient and reliable method for achieving the desired seismic performance objectives for the project.

Keywords: Performance-Based Design; Seismic Design; Nonlinear Time History Analysis; Reduced Beam Section

1. Introduction

The new $1.2 billion LAX Midfield Satellite Concourse (MSC) Expansion is the next significant construction project in the ongoing modernization of the Los Angeles International Airport. Los Angeles World Airports (LAWA), the governing authority, wanted the airport to be operational following a regional earthquake. Several prior airport projects had been designed as “Essential Facilities” based on a prescriptive approach, with I = 1.5. The primary structural frame of the Midefield Satellite Concourse would be designed using a Performance-Based Engineering approach to more specifically and accurately address the airport’s goals for post-earthquake functionality. Elements and components were then designed using a prescriptive approach which was informed by the results of the Performance-Based analyses. Curtain wall systems were designed to meet performance-based objectives for various deformations and hazard levels.

2. Building Description

LAX Midfield Satellite Concourse (MSC) expansion project is consist of adding new concourses located west of the existing Tom Bradley International Terminal (TBIT). The new structure is a three-level concourse over a single basement, comprising approximately 800,000 square feet. The new concourses are designed to accommodate the new superjumbo intercontinental airplanes such as the Airbus A380 and Boeing 747-8I and will be connected to the existing terminal through a new underground tunnel. Figures 1 to 3 show master plan and both exterior and interior images of the new terminal building.

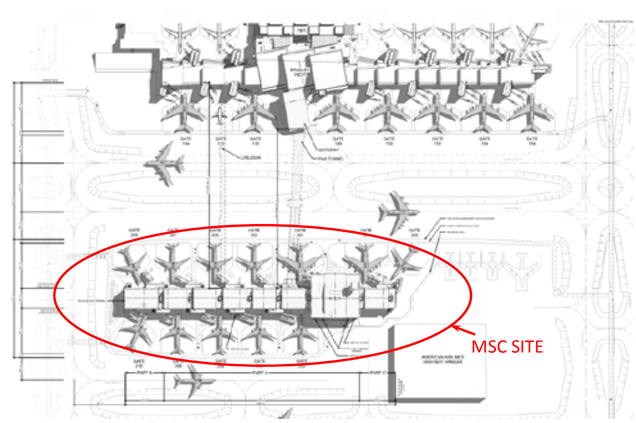


Fig. 1 – Site Plan

Fig. 2 – Overall View of New Terminal Building

Fig. 3 – Interior Image of Concourse

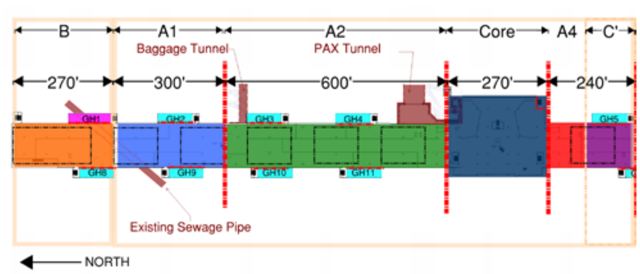


Fig. 4 – Overall Plan and Component Definition

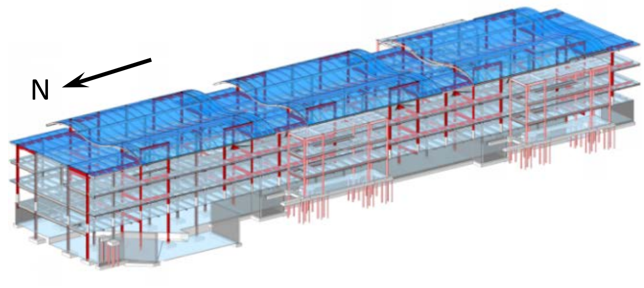


Fig. 5 – Typical Concourse Structure (A2)

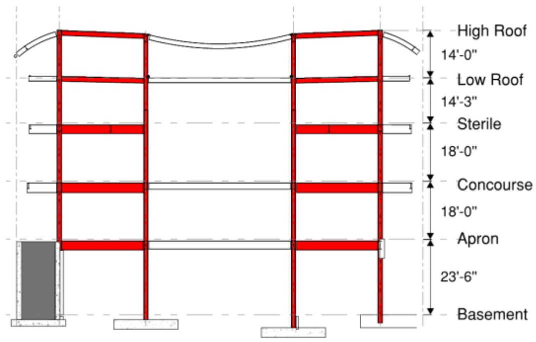


Fig. 6 – Section View of Typical Concourse (A2)

The MSC is organized into several structurally discrete buildings separated by expansion joints (see Figure 4). The Concourse section of the MSC is a three story steel structure with a partial basement. The Core section is an enlarged main entry hall of four stories over a basement, with a Ramp Control Tower separated above the basement level by a seismic expansion joint. Structural systems for the superstructure consist of composite steel floor framing and non-composite roof framing supported by steel columns. Figures 5 and 6 show a typical concourse structural framing and a cross section. The building is founded on a relatively flat site and has a 100 ft. wide basement under all portions except area A1 and B. The basement is bisected by concrete shear walls. Floors and roofs for typical concourses are designated as Apron, Concourse, Sterile, Low Roof, and High Roof. The Core has one more floor specified as Club which is located between Sterile and Low Roof.

ANSI/AISC 358 [2] provides various types of prequalified moment connections. Among them, non-proprietary reduced beam sections (RBS) and the proprietary SidePlate connections were considered for the lateral seismic force resistance system. RBSconnections are very popular and commonly used by fabricators. However, RBS connections require significant field Complete Joint Penetration (CJP) groove Weld at the beam-to-column connections and may need thicker doubler plate to avoid significant panel zone damage. Compared to RBS momento connections, SidePlate connections provides bolted or fillet weld for the connection assembly which result in fast erection time. Also, SidePlate typically reduce the steel tonnage due to increased frame stiffness and minimize damage to the panel zone, thus doubler plate is not needed. However, license fee is required to use SidePlate. Both connections were modeled and analyzed in the beginning of the analysis and the results were disscussed with architects and contractors to pick one lateral force resistance system. From the discussion, RBS connection was selected for LAX MSC project because the estimated cost was not too much different for both connections but fabricator prefer to use RBS connection for their familiarity.

3. Performance-Based Seismic Design

3.1 General

Performance-Based Seismic Design (PBSD) refers to the structural engineering design procedures to achieve predictable building performance in response to specified levels of earthquake ground shaking. Table 1 compares performance-based design to code-based or "prescriptive" design. As shown in the comparison, performance-based design provides a more realistic estimate of the building behavior so that there is a greater assurance of the desired building performance for the different intensity of earthquakes.

Table 1: Comparison of Code and Performance-Based Designs

Item	Code-Based Design	Performance-Based Design
Loading	Response sepctrum analysis of design-basis (475-year mean return period) earthquake with forces fictitioly reduced	A suite of 7 time history analyses for three different earthquake hazards using unreduced force levels
Model	Linear elastic 3D model of the structure	Nonlinear 3D model of the structure based on physical connection tests
Evaluation	Simplified stress and drift checks	Acceptance criteria varies based on performance objectives so that reparability and other objectives can be specifically considered

The Performance-Based Seismic Design approach for the project will be based primarily on the National Standard of Seismic Rehabilitation of Existing Buildings (ASCE 41-13) [4]. Although it is originally intended for existing buildings, the ASCE 41-13 document is currently the most comprehensive and practical performance-based analysis and design guideline available and represents the state-of-the-art knowledge related to performance-based seismic design analysis methods, modeling assumptions and acceptance criteria that are applicable to both new and existing buildings alike. The ASCE 41-13 guidelines will be supplemented as needed for this project based on other reference standards, guidelines, and research data.

3.2 Performance-Based Seismic Design Objective

Prescriptive code-based seismic design for "essential facilities" require using an importance factor (I) of 1.5. That is, 50% higher seismic design forces compared to typical building should be used to make building structures stronger and stiffer. However, the performance of buildings in past earthquakes have shown that there is little direct link between the intended seismic performance and the prescriptive requirements of the building code and that the use of performance-based seismic design principles provide a greater assurance that the design will yield the desired seismic performance. As such, Los Angeles World Airports (LAWA) decided that the buildings be designed based on the prescriptive building code using an importance factor (I) of only 1.25. But the building should be verified by a nonlinear dynamic time history analysis using a three-dimensional nonlinear computer model of the building to determine whether the building performance is within the acceptable limits.

For the new LAX MSC project, a three-fold enhanced seismic perforamance objective has been identified (see Figure 7 for the comparison of the building performance level for LAX MSC and typical buildings):

(1) "*Operational*" structural performance for a Frequent (43-year mean return period) earthquake (denoted EQ1).

(2) "*Immediate Occupancy*" structural performance for a Rare (475-year mean return period) earthquake (denoted EQ2).

(3) "*Life Safety*" structural performance for a Very Rare (2,475-year mean return period) earthquake (denoted EQ3).

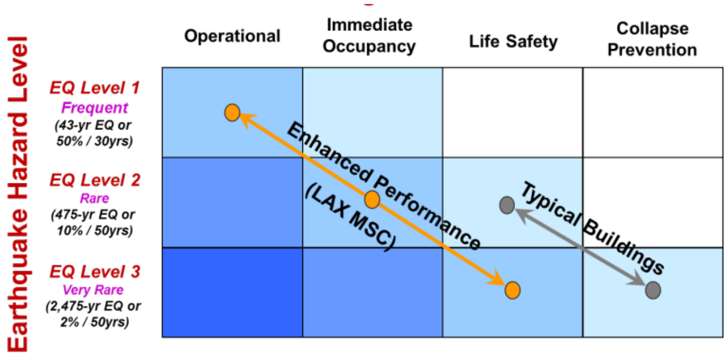


Fig. 7 – Building Performance Level

3.3 Design Procedures

Since the structural frame must also meet all prescriptive Building Code requirements for the LAX MSC project, the first step in the design process is to investigate the nonlinear behavior of a first iteration, "Code-based" structural system of the buildings, by conducting nonlinear dynamic time history analysis with selected ground motions. The results are evaluated and interpreted, and used to suggest modifications to the Code-based design. The Code-based design is then modified based on these results, and checked/ modified for Code compliance. This typically involves the redistribution of steel within the steel frames, and modifications of the joint designs. The process iterates until the design meets both Performance- and Code-based requirements with satisfactory material cost parameters (see Figure 8 for the design flow chart).

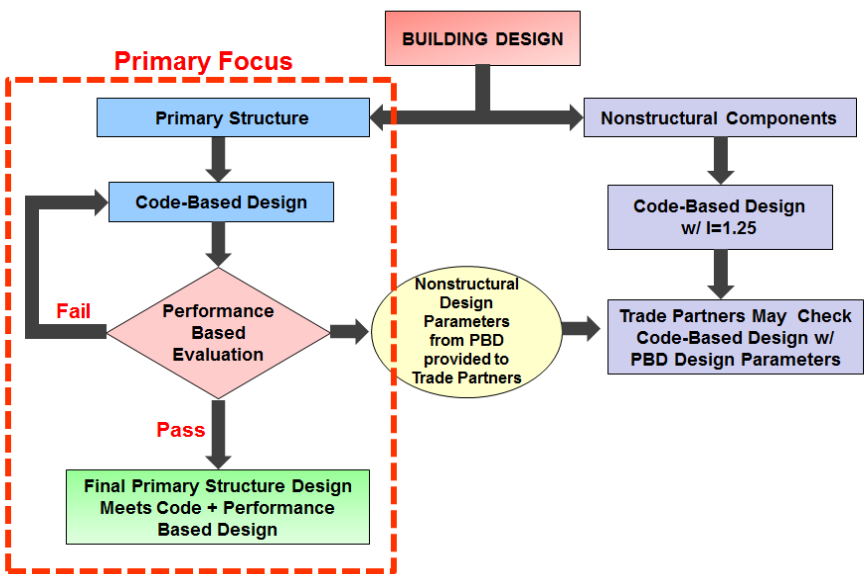


Fig. 8 – Design Flowchart

4. Nonlinear Dynamic Time History Analysis

4.1 Modeling Technique

A three-dimensional comercial nonlinear analysis software package, Perform 3D [6], was used to investigate the nonlinear seismic behaviors of LAX MSC concourses. Since the primary objective is to investigate and design the special moment frame (SMF), all basement shear walls and the building floor slabs (assumed to be semi-rigid diaphragms) were modeled with elastic membrane and shell elements, respectively. Figure 9 summarizes the general modeling approach for the beams, columns, and panel zones.

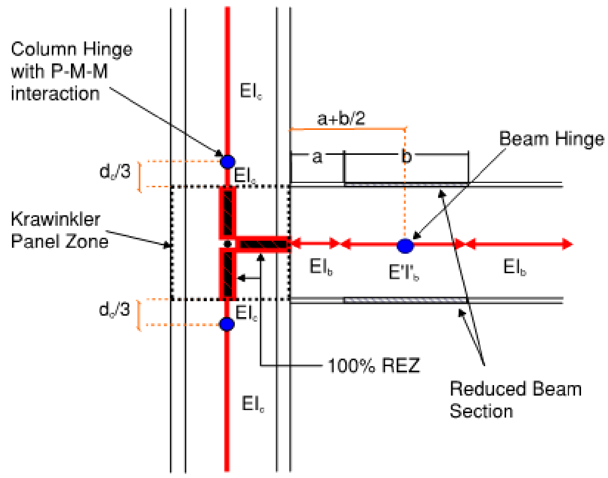


Fig. 9 – Perform 3D Model of Beam-Column Joint with RBS Moment Connections

Elastic beam element with moment plastic hinges and rigid end zones at each end are used. Beam plastic hinges are modeled at the center of the reduced beam section and moment-rotation backbone curve for the flexural plastic hinge characteristics is defined based on Table 9-6 of ASCE 41-13. Elastic column element with P-M2-M3 hinges and rigid end zones at each end are used. P-M2-M3 hinges were located at one third of column depth (d_c/3) away from the face of the beam. Inelastic panel zone elements are used to model the panel zones. Doubler plate thickness is also considered in the strength and stiffness calculation of the panel zone element. The relationship between the moment and shear deformation is developed based on Krawinkler's [8] work up to four time yield strain ($4\gamma_y$). Since ASCE 41 specifies the shear deformation of $12\gamma_y$ as the limit state for the performance level of life safety, strain hardening which is 3% of elastic stiffness is considered beyond $4\gamma_y$.

Expected material properties recommended by ASCE 41 were used to determine element strength and stiffness. Expected yield stress is calculated based on the minimum specified yield stress multiplied by appropriate facotrs (R_y) recommended in AISC 341 [1] to translate from nominal to expected values. Resistance factors for the element strength calculation was not considered, as allowed by ASCE 41 (i.e., $\phi = 1.0$).

Damping was considered by using combined constant modal damping and Rayleigh damping as recommended by Perform 3D Manual [6].

4.2 Seismic Ground Shaking Hazard

Ground motion evaluation, including recommended response spectra and acceleration time histories, for the LAX Midfield Satellite Concourse project was conducted. A total of 42 pairs of matched acceleration time histories, 14 pairs for the 43-yr (EQ Level 1, frequent earthquake) hazard level, 14 pairs for the 475-yr (EQ Level 2, rare earthquake) hazard level, and 14 pairs for the 2475-yr (EQ Level 3, maximum considered earthquake) hazard level, are provided from the evaluation. For each hazard level, seven pairs are developed to matche the short period (SP) conditional mean scenario and seven pairs matched to the long period (LP) scenario, respectively. Each pair consists of pairs of appropriately scaled orthogonal acceleration time histories (H1 and H2 which are acceleration time histories for longitudinal and transverse directions, respectively). Table 2 summarizes 7 representative ground motion records for each seismic hazard level of long period. Figure 10 shows 5% damped site-specific response spectra for frequent, rare, and very rare (maximum considered) earthquake events.

Table 2: Long Period Event Seed Time History Records

No.	EQ Name	Recording Station	Year	M	V_{s30} (m/s)	Distance from the recording station to ruptured area (km)
GM1	Imperial Valley-06	EC Country Center FF	1979	6.53	192	7.3
GM2	Chi-Chi, Taiwan	TCU087	1999	7.62	474	7.0
GM3	Duzce, Turkey	Duzce	1999	7.14	276	6.58
GM4	Kocaeli, Turkey	Yarimca	1999	7.51	297	4.83
GM5	Northridge-01	Burbank-Howard Rd.	1994	6.69	822	16.88
GM6	Loma Prieta	Saratoga-Aloha Ave	1989	6.93	371	8.5
GM7	Landers	Coolwater	1992	7.28	271	19.74

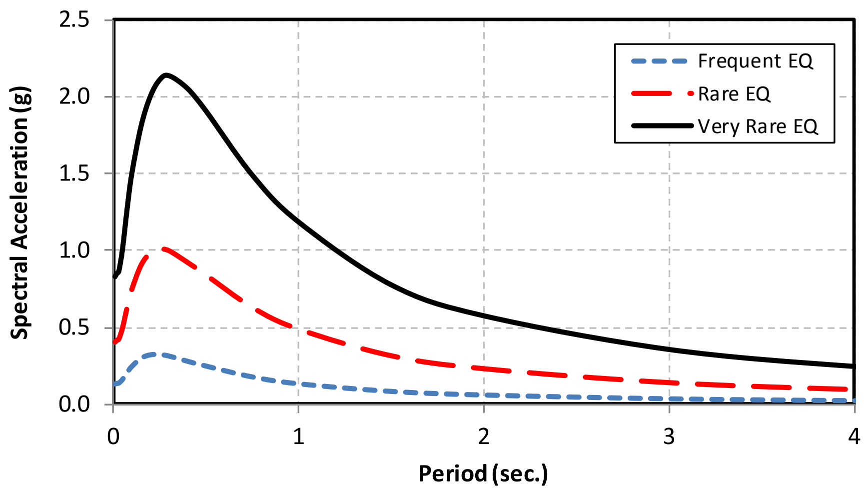


Fig. 10 – Site-Specific Response Spectra with 5% Damping

4.3 Acceptance Criteria

Primary component demands shall be within the acceptance criteria for nonlinear components at the selected structural performance level. Expected deformation capacities were calculated based on ASCE 41-13.

Also story drift limits were specified for this project; 0.75% of story drift ratio for operational structural performance level, 2% of story drift ratio for immediate occupancy structural performance level, and about 3% of story drift ratio for life safety structural performance level. These story drift ratios were based on fragility and repairing cost of the structures based on FEMA P58 [5] and FEMA 352 [9]. That is, for the operational performance level, no damage occurs. However, structrual beam elements in SMF systems would experience local buckling at the plastic hinge location for immediate occupance performance level. This damage state may be repaired by heat straightening of the buckled flanges and web per FEMA P58 or providing stiffeners at the buckled beam flanges per FEMA 352. For the life safety performance level, structural beams may experience both local buckling and lateral torsional buckling. The repair of this damage state may necessitate removal and replacement of distorted portion of the beam.

4.4 Results of Primary Structural Frame

An iterative code-based and performance-based process was used to develop a solution that meets Building Code standards with an importance factor of I = 1.25 while satisfying the performance based objectives at each level. The PBSD design generally validated the overall layout of steel moment frames developed for the Code-based seismic design, but resulted in a significant redistribution and redesign of those frames.

In broad terms, the performance-based design generally required a significant strengthening of the column panel zones, and suggested a redistribution of structural steel in the moment frames. This feedback, combined with numerous iterations of the Code-based design, resulted in a net savings of close to 2 psf of steel on the overall project compared to initial Code-based designs.

Based on the 7 ground motions for each earthquake hazard, the average interstory drift ratios and component deformation was calculated and compared with the limits (see Table 3). The average interstory drift ratios were calculated at the frame location near the center of mass of each floor. The maximum average interstory drift ratios are 0.54% at high roof for EQ Level 1, 1.65% at high roof for EQ Level 2, and 3.0% at sterile for EQ Level 3 (see Figures 11 through 13 for the story drift ratio profiles).

Table 3: Summary of Analysis Results

EQ Level.	Target Performance	Interstory Drift (%)		Beam Plastic Hinge Rotation (rad.)		Panel Zone Shear Deformation (rad.)	
		Project Limit	Analysis Result	Acceptance Limit	Analysis Result	Acceptance Limit	Analysis Result
Level. 1 (Frequent)	Operational	0.75%	≤ 0.54%	0	0	≤ 0.0025	≤ 0.0011
Level. 2 (Rare)	Immediate Occupancy	2.0%	≤ 1.65%	≈ 0.02	≤ 0.01	0.0025	≤ 0.0023
Level. 3 (Very Rare)	Life Safety	3.0%	≤ 3.0%	≈ 0.045	≤ 0.042	0.030	≤ 0.0047

Designed based on the strong-column/weak-beam provisions of AISC 341, all columns satisfied the acceptance limit state for each hazard level with little modification. See Table 3 for the summary of the beam and panel zone deformation summary from the nonlinear analysis. As shown in the table, all beams and panel zones were within the acceptance limit for each target strucrual performance level. Figures 14 and 15 present the range of the beam plastic rotation shown in the beam backbone curve and expected beam damage state. In general, under rare earthquakes, most of beams experienced 0.6% of plastic rotation and the maximum plastic rotation achieved was about 1.0%. The expected beam damage is yielding and local buckling in the plastic hinge location as mentioned in Section 5.3. Under very rare earthquake, most of beams experienced the plastic rotation between 2.3% ~ 4.2%. With that amount of plastic rotation, the beam may experience both local and lateral-torsional buckling as shown in Figure 15 (b).

For the panel zone deformation, shear yield deformation (γ_y) is specified as a limit for both operational and immediate occupancy performance level and $12\gamma_y$ is for life safety performance level per ASCE 41. Due to the enhanced performane level required in LAX MSC project, panel zone shear deformation has to be less than γ_y for EQ level 2 (design basis earthquake). Note that AISC Specifications allow $4\gamma_y$ for design basis earthquake. Therefore, the design of the panel zone required much thicker doubler plates per ASCE 41-13 than required by AISC Specifications. That means, it might cause very conservative design for the panel zone to meet immediate occupancy performance level per ASCE 41-13. After immedeiate occupancy level for panel zone is achieved with thicker doubler plate, very small increase in the panel zone shear deformation at the EQ level 3 is observed (see Figures 16 and 17). Tha maximum panel zone shear deformation achieved is 0.47% which is still less than $2\gamma_y$. This is due to the fact that most of nonlinearities occur at the beam at EQ level 3.

Thus, it is recommended to revisit the ASCE 41-13 code requirement for the panel zone deformation limit state to avoid unnecessary strengthening.

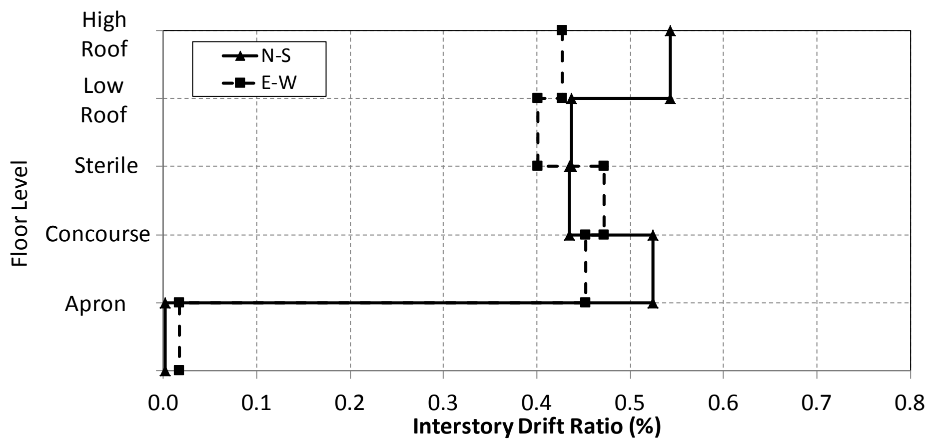


Fig. 11 – Interstory Drift Ratio Profiles under Frequent Earthquake (Average of 7, RBS Connection)

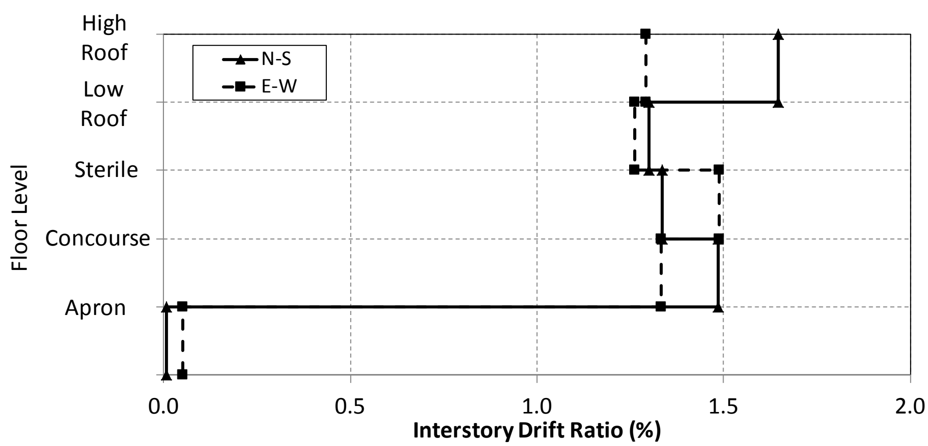


Fig. 12 – Interstory Drift Ratio Profiles under Rare Earthquake (Average of 7, RBS Connection)

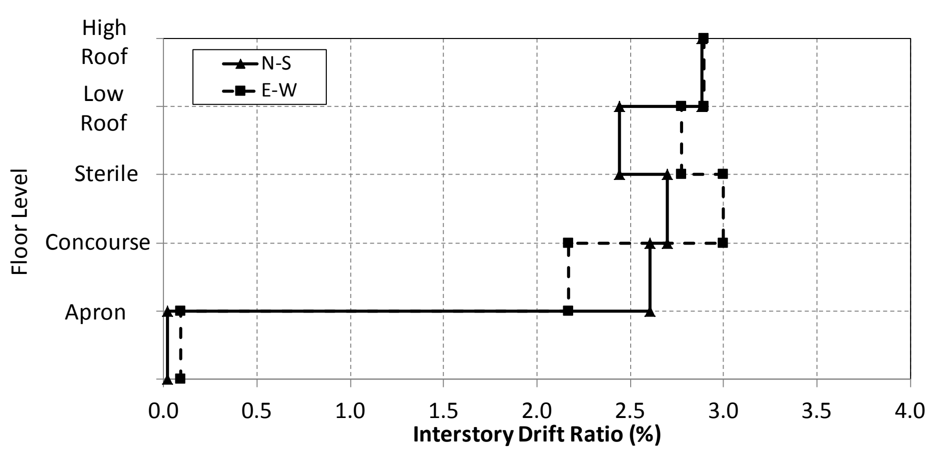
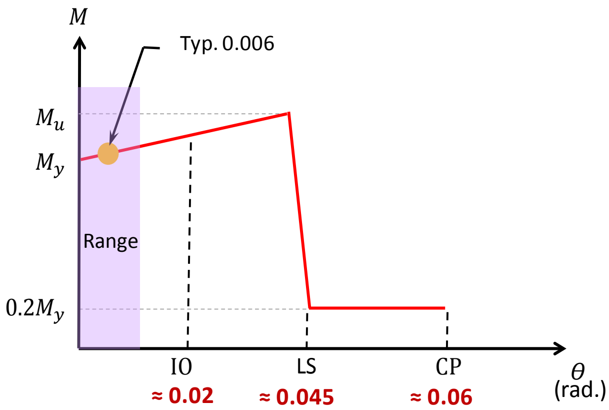
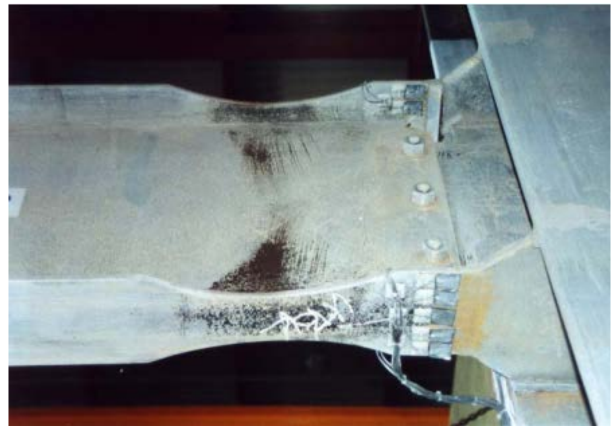


Fig. 13 – Interstory Drift Ratio Profiles under Very Rare Earthquake (Average of 7, RBS Connection)

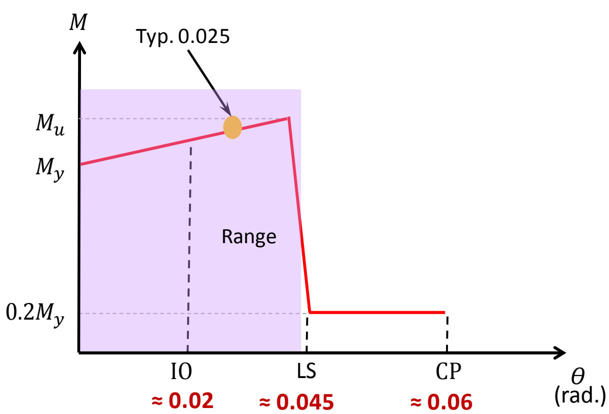


(a) Beam Plastic Rotation



(b) Beam Damage at Plastic Rotation of about 0.01 Radian (Pictures from Gilton et al. [7])

Fig. 14 – Summary of Beam Bahavior (EQ2, RBS Connection)



(a) Beam Plastic Rotation

(b) Beam Damage at Plastic Rotation of about 0.04 Radian (Pictures from Gilton et al. [7])

Fig. 15 – Summary of Beam Bahavior (EQ3, RBS Connection)

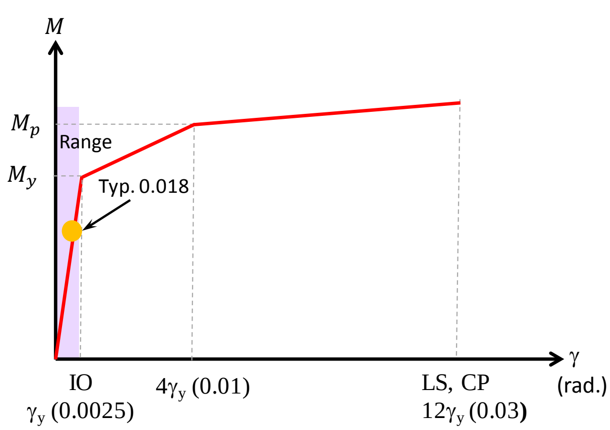


Fig. 16 – Panel Zone Shear Deformation (EQ2, RBS Connection)

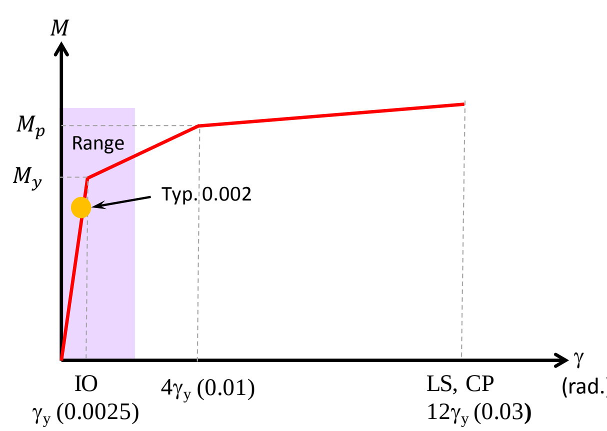


Fig. 17 – Panel Zone Shear Deformation (EQ3, RBS Connection)

5. Nonstructural Components

While the primary focus of a performance-based evaluation and design is to ensure that the structural frame meets the seismic performance objectives for the facility, the goal is for the entire facility to meet those same objectives. In this case, the LAX MSC is intended to be suitable for "*Immediate Occupancy*" after a "*Rare*" earthquake, and "*Operational*" after a "*Frequent*" earthquake. This necessarily depends on the functionality of numerous non-structural components and systems.

The PBSD analysis provides a wealth of information that may be used to determine whether non-structural components can meet the performance objectives. Some non-structural elements are sensitive to the intensity of the floor accelerations, while others are sensitive to building floor-to-floor drifts. Expected floor accelerations and interstory drifts can be estimated for each earthquake hazard level from the nonlinear time history analysis and can be used to evaluate non-structural systems. As shown in Figure 8, the PBSD results for interstory drift and floor acceleraton are provided to the trade partners for the use in designing the nonstructural systems.

6. Summary and Conclusions

Both code-based and performance based seismic designs were conducted for the LAX Midfield Satellite Concourse expansion project. Prescriptive code-based design requires using importance factor of 1.5 for "Essential Facilities" which results in higher structural costs. However, the performance of the building with importance factor of 1.5 does not necessarily gurantee the enhanced seismic performance. Based on the advanced performance-based seismic evaluation and design by using nonlinear dynamic time history analyses, the following conclusions were made.

1. Using importance facotor of 1.25 (I = 1.25) for code-based design was justified by conducting the nonlinear dynamic evalation. All buildings were able to meet the enhanced performance levels: operational for frequent earthquake, immediate occupancy for rare earthquake, and life safety for very rare earthquake.
2. Several iterations between code- and performance-based design resulted in additional net saving of the steel (about 2 psf of steel on the overall project).
3. Very rare earthquake (maximum considered earthquake) governed the design of the beams and columns to meet the life safety structural performance level but panel zone design was governed by rare earthquake (design basis earthquake) to meet the immediate occupancy structural performance level.
4. Since panel zone deformation acceptance criteria does not provide reasonable limit state in this project, it is recommended to revisit the ASCE 41 code requirement to be consistent with AISC 341. Therefore, balanced yielding in both beams and panel zones can make the building structures work better and stable, avoiding conservative panel zone design.
5. Interstory drift and peak floor acceleration determined from the nonlinear dynmaic analyses can be used to design nonstructural components.

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