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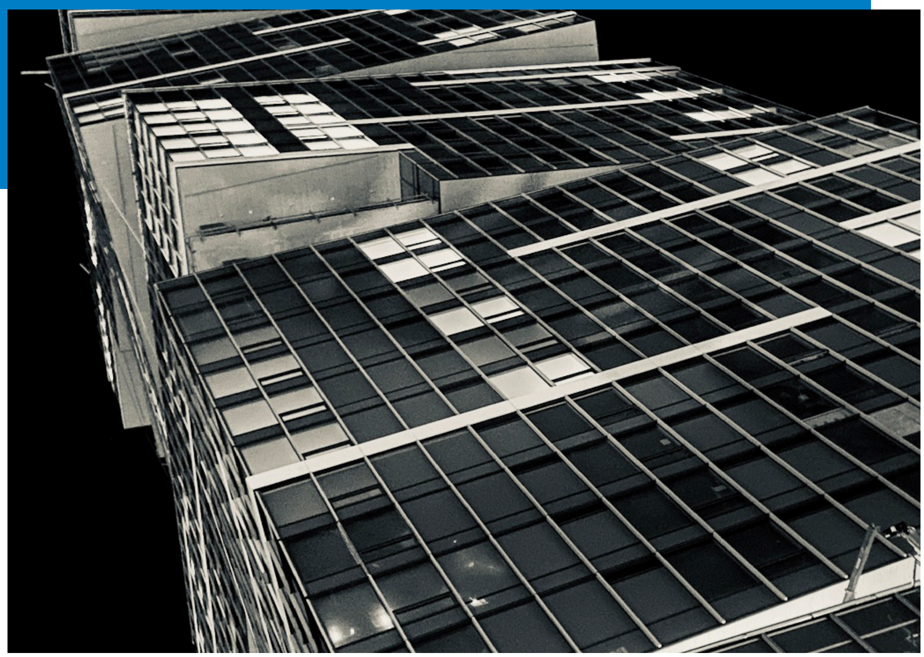

Performance-Based Seismic Design of Concrete Buildings: State of the Practice

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
Brian Kehoe
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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

Xiaonian Duan is a Partner at Foster + Partners and a Chartered Engineer in the UK. He has a BEng, MSc and PhD in structural engineering and has over 20 years of experience in seismic analysis and design of a wide spectrum of structures worldwide. His particular expertise is in performance-based seismic design and nonlinear response history analysis of tall buildings and has served in CTBUH Seismic Working Group and Review Panel on this subject.

ACI member **Andrea Soligon** is a Partner at Foster + Partners and a registered Professional Engineer in California and Italy. He graduated in structural engineering at University of Padova, Italy and received his MS in structural engineering at the University of California, Berkeley, CA. He has 19 years of experience in structural and earthquake engineering and has led the structural design of a wide range of buildings worldwide, including high-rise, museums, transportation and mixed-use projects.

Jeng Neo is an Associate Partner at Foster + Partners and a Chartered Engineer and a member of the Institution of Structural Engineers in the UK. He has a MEng from Imperial College London and over sixteen years of experience in the analysis and design of a wide variety of complex building structures.

Anindya Dutta is a Senior Project Manager at Simpson Gumpertz & Heger Inc., and a registered Structural Engineer in California. He received his BEng from Jadavpur University in India and his PhD from the State University of New York at Buffalo. He has over eighteen years of experience in structural and earthquake engineering including analysis and design and seismic evaluation and strengthening of a variety of structures in high seismic zones.

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. 1and 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 m^2 (1,247,000 ft^2), 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 7–10[1] and connections not prequalified in accordance with AISC 358–10[2]. 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

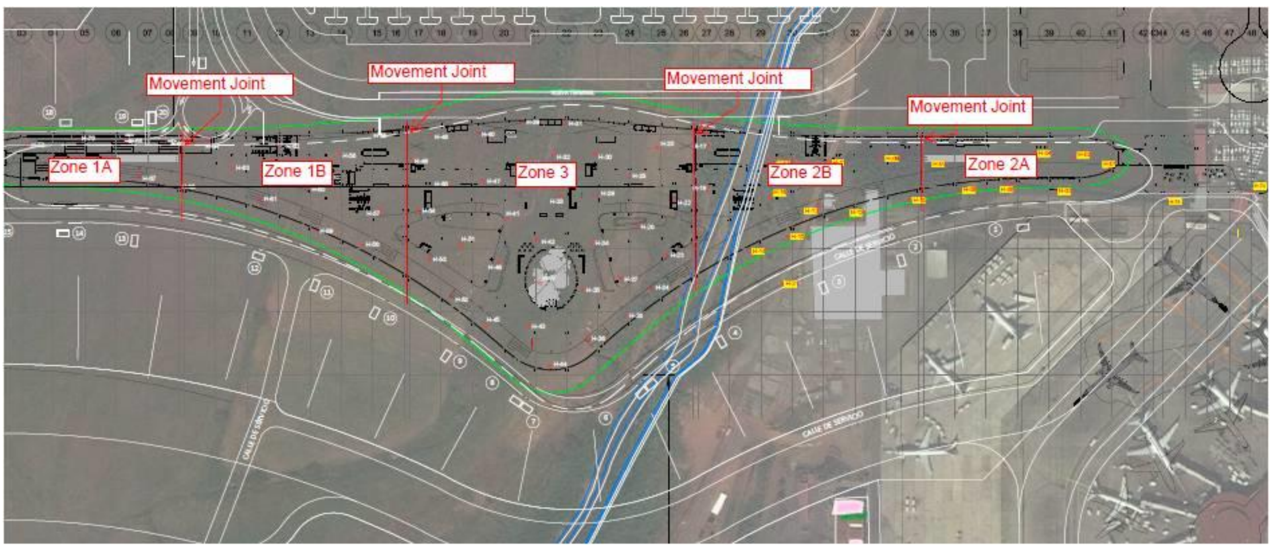

Figure 2— Structural zones and seismic joints of the new Terminal 2

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 7–10[1] and is not detailed with prequalified steel connections in accordance with AISC 358–10[2].

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 318–11[3].

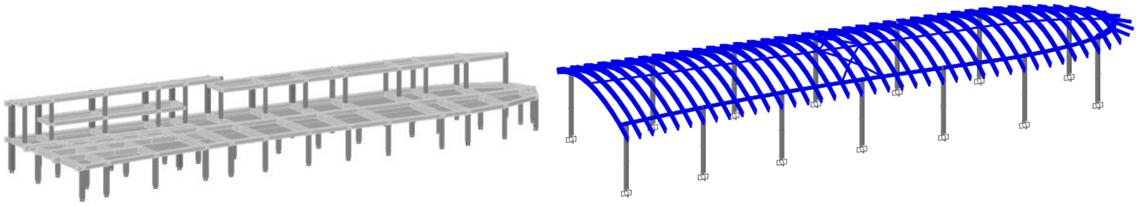

Figure 3— Structural systems in Zone 2A

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.

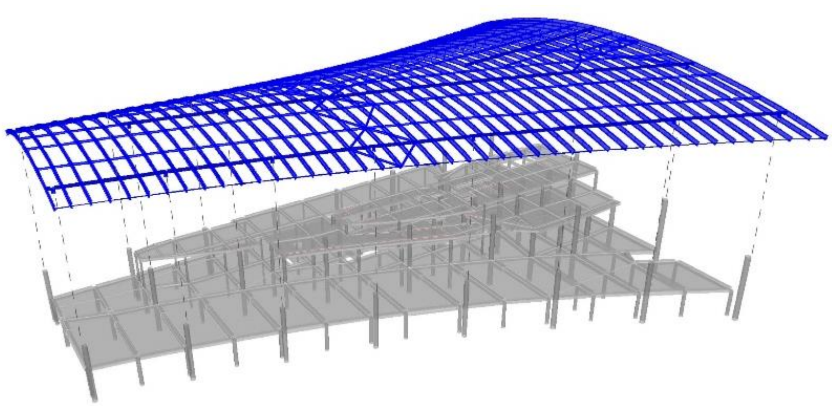

Figure 5— Structural systems in Zones 1B /2B (Exploded view of steel roof and concrete superstructure)

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 318–11[3].

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

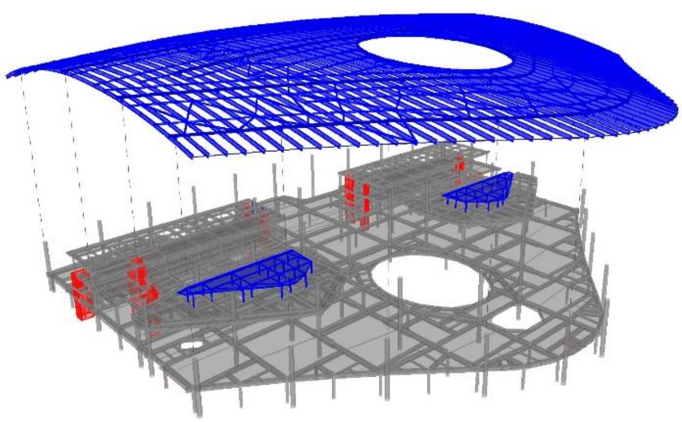

Figure 7— Structural system in Zone 3 (Exploded view of steel roof and concrete superstructure)

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

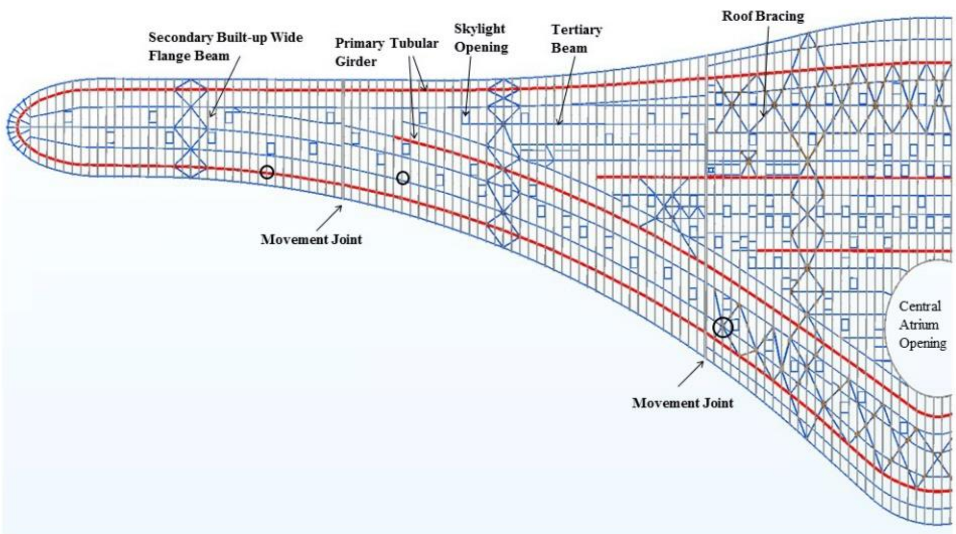

Figure 9— Half roof structural framing plan