(W)RAPPER TOWER: ENGINEERING AN OCCUPIED SCULPTURE

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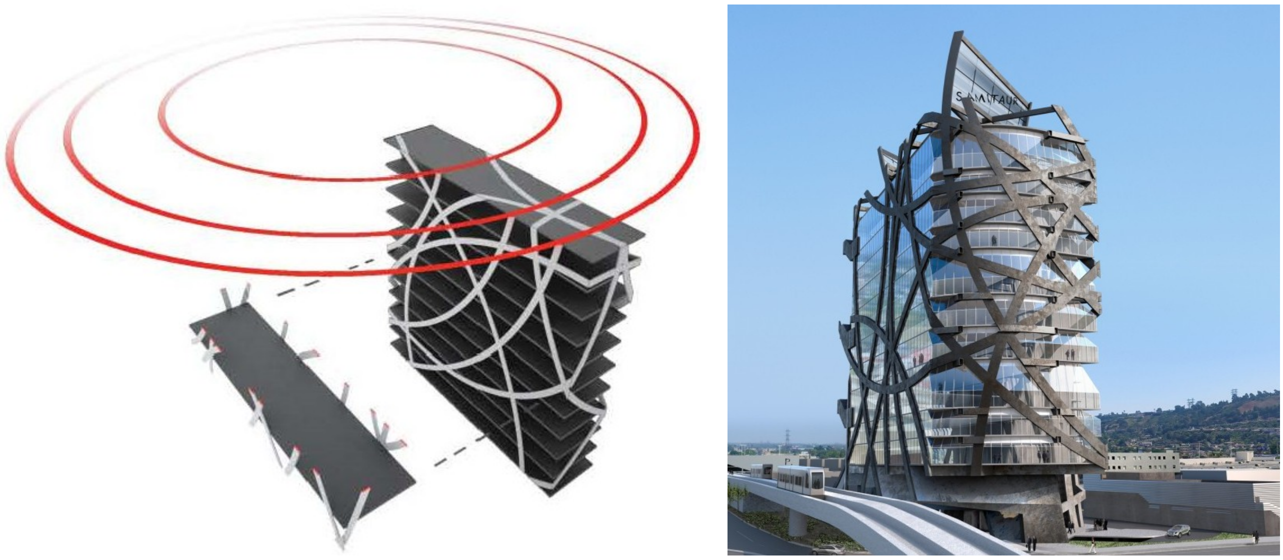
Abstract

(W)rapper Tower is a 240 ft tall building located in Los Angeles, California, which features a "continuous system of curvilinear ribbons" (Steel Bands) wrapping around its exterior. The steel exoskeleton allows a column free interior, with floor plates typically 14,000 square feet in area, and double-height ceilings of 24 ft on certain levels. The lateral force-resisting system of the Tower consists of base isolators, a Steel Plate Shear Wall (SPSW) core, Steel Bands and trussed steel framing diaphragms at each level. The Tower is seismically isolated on Triple Friction Pendulum (TFP) bearings located under the Bands and SPSW core columns at the ground level. The Steel Bands are an unclassified framing system per ASCE 7 and require an alternative means of compliance to the code, using performance-based design. Due to the unclassified nature of the Steel Bands, and in recognition of their contribution to the lateral frame system, an elastic superstructure is proposed for the Design Basis Earthquake (DBE) level evaluation. In further recognition of the unique combination of lateral framing (Isolators, SPSW and Bands) for this structure, it is proposed to evaluate the performance of the Steel Bands, Web plates and Horizontal Boundary Elements (HBEs) of SPSW at Maximum Credible Earthquake (MCE) level and design them to stay essentially elastic by employing a response modification factor of R_I=1.5. Vertical Boundary Elements (VBEs) and all components of the substructure supporting the isolation plane are designed to remain elastic under an MCE level of intensity. Due to the unique structural features and eliminating the need for physical testing, Arup proposed and agreed additional analysis and performance acceptance criteria with the peer review panel and the Los Angeles Department of Building and Safety. These additional criteria are; (1) Limiting inter-story drifts to be less than 1% at MCE, (2) Performing Finite Element Analysis (FEA) of Steel Bands and Nodes to validate the performance and demonstrating any local, limited yielding of Bands did not result in structural instability, (3) Performing Fracture Mechanics analysis of Bands and welds by a metallurgist to demonstrate that the Bands resistance to localized fracture against fabrication, geometry and material related imperfections. The (W)rapper Tower is currently under construction and is scheduled to open in 2022. This paper aims to describe the approach to the design and performance evaluation of this unique occupied sculpture.

Keywords: Unclassified Structural System; Steel Plate Shear Walls; Seismic Isolation; Steel Bands; Performance Based Design; Nonlinear Time History Analysis

1. Introduction

The (W)rapper Tower was first conceived by Eric Owen Moss Architects nearly 20 years ago. The project represents a new architectural paradigm for an office building where architecture seamlessly integrates with the structure, provides large open office space unobstructed by interior columns and creates a new landmark. (W)rapper Tower is a 16 story, 240 ft tall building located in Los Angeles, California, which features a "continuous system of curvilinear ribbons" (Steel Bands) wrapping around its exterior. The steel exoskeleton allows a column free interior, with floor plates typically 14,000 square feet in area, and double-height ceilings of 24 ft on certain levels. All Bands run in a plane parallel to the building face and are geometrically defined by large radii. The Bands act as the primary gravity load carrying system and also add lateral strength and stiffness to the building. Fig. 1 illustrates the overall form of the Tower. The long floor spans, irregular location and shape of the supports, absence of interior columns, eccentric core and location in a high seismic region represented major challenges for the structural engineers for this project.



(a) (b)

Fig. 1 (W)rapper Tower steel exoskeleton concept (a) and rendering from Northwest (b) (Eric Owen Moss)

The gravity framing of the regular office floors is composed of typical steel-concrete composite floor system supported by external, curved, structural Steel Bands with no internal columns. Selected 8 external bands run continuously to ground floor level. At ground level, the bands are supported by seismic isolators anchored to reinforced concrete columns to the foundation. Horizontal tie members (beams) at ground level are used to resolve the axial forces of the curved tower bands to vertical and horizontal forces. Bands that are not used to support primary floor members are connected to the floor diaphragm for lateral bracing. An eccentric core on the south side of the building houses vertical transportation, MEP rooms and restrooms. An additional escape route is provided by means of an external staircase located on the east face of the building. 3D Isometric, elevation and plan view of the (W)rapper Tower analysis model is shown at Fig. 2.

The lateral force-resisting system of the Tower consists of base isolators, a Special Perforated Steel Plate Shear Wall (SPSW) core, Steel Bands and trussed steel framing diaphragms at each level. Seismically Base isolating the Tower reduced the lateral demand by factor of 4. In the east-west direction (longitudinal), the SPSW core and the Bands resist approximately 40% and 60% of the total seismic shear respectively. In the north-south direction (transverse), the SPSW core and the Bands resist approximately 90% and 10% of the total seismic shear respectively. The contribution of the SPSW core and the Bands varies along the height of the Tower (above level 3 contribution of bands increases more in the north-south direction). The Tower is seismically isolated on 60 in diameter Triple Friction Pendulum (TFP) bearings located under the Bands and SPSW core columns at the ground level. There are total of 18 isolators under the bands and SPSW columns.

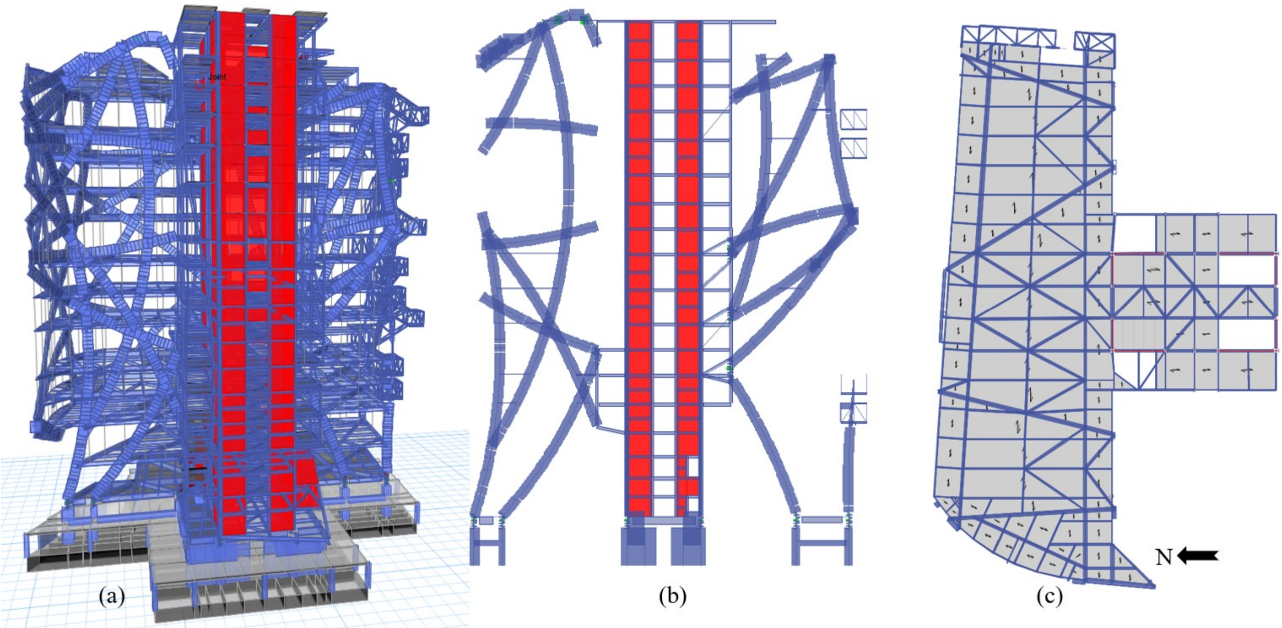


Fig. 2 3D Isometric (a), elevation (b) and typical floor plan view (c) of the Tower analysis model in ETABS

The bearings are supported on concrete columns and supported on a 6 ft to 8 ft thick mat foundation. There is a moat around the perimeter of the Tower at ground level to accommodate movement of the isolated Tower structure. 3D isometric view of the seismic isolation plane and framing is shown in Fig. 3 below. An all-steel framed, base isolated structure resulted in a significant reduction of lateral loads that together with the use of one primary material (steel) resulted in significant construction savings.

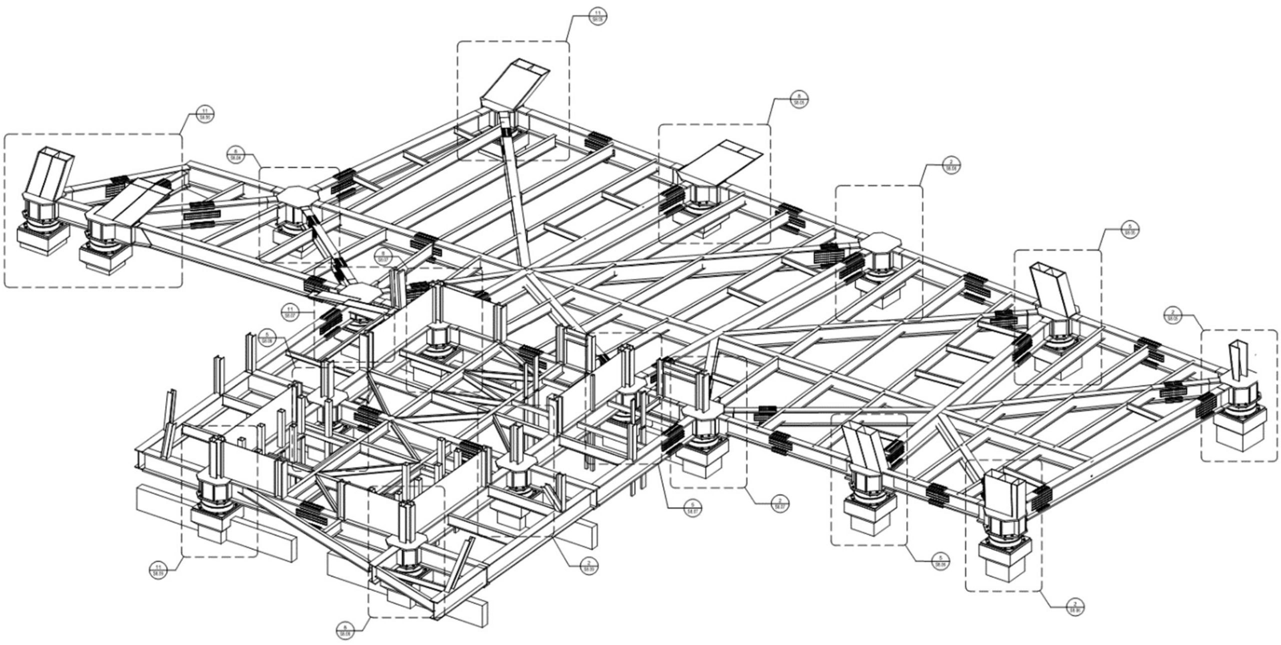


Fig. 3 3D Isometric view of the seismic isolation plane

The (W)rapper Tower is currently under construction and is scheduled to open in 2022. This paper aims to describe the approach to the design and performance evaluation of this unique occupied sculpture.

2. Lateral Design Criteria

2.1 Basic Design Criteria

The Steel Bands are an unclassified framing system per ASCE 7 [1]. Due to the unclassified structural system nature of the building (Steel Bands) and base isolation, this building falls outside of a traditional building code. Moreover, SPSW core building has a height limit of 240 ft per Table 12.2-1 and Section 12.2.5.4 of ASCE 7. So, an alternative means of compliance using a Performance Based Design (PBD) approach following the LATBSDC [2] guidelines had to be used in the design. Due to the unclassified nature of the Steel Bands, and in recognition of their contribution to the lateral frame system, R_I=1.0, an elastic superstructure is proposed for the Design Basis Earthquake (DBE) level evaluation. In further recognition of the unique combination of lateral framing (Isolators, SPSW and Bands) for this structure, it is proposed to evaluate the performance of the Steel Bands, Web plates and Horizontal Boundary Elements (HBEs) of SPSW at Maximum Credible Earthquake (MCE) level and design them to stay essentially elastic by employing a response modification factor of R_I=1.5. Vertical Boundary Elements (VBEs) and all components of the substructure supporting the isolation plane are designed to remain elastic under an MCE level of intensity. A seismic peer review panel (PRP) of industry and academic experts was formed to help permitting of the building. The performance objectives were set together with the PRP. The proposed basic design approach is shown in Table 1 below.

Table 1 – Basic design approach for (W)rapper Tower

SCENARIO HAZARD	**BASIC DESIGN CRITERIA**		
	SUPER-STRUCTURE	**ISOLATOR**	**SUB-STRUCTURE**
DBE	R_I=1 (ELASTIC)	R_I=1 (ELASTIC)	R_I=1 (ELASTIC)
MCE	R_I=1 VBEs (SPSW) (ELASTIC)	R_I=1 (ELASTIC) SIZED & STABLE	R_I=1 (ELASTIC)
	R_I=1.5 BANDs, HBEs, CBs and Web Plates (SPSW) (ESSENTIALLY ELASTIC)		

This performance objective limits significant hinging at MCE level earthquakes and minimizes the risks of uncertainty with inelastic behavior of the Bands and the Band intersections (nodes). The Bands act partially as a braced frame and partially as a moment frame – the elements resist both significant axial forces and bending moments. These effects are amplified by the fact that the Bands are curved and loaded by floor girders between nodes. Like more traditional braced frame structures, the Bands are essentially force controlled and therefore ductility and energy dissipation during earthquakes are limited. Ductility and energy dissipation are primarily provided by the base isolation and the web plates of the SPSW core which are allowed to yield. Band geometry (radii, location) and sizes (5' x 1') were defined by the architect. Band sections were established by varying the plate thicknesses depending on required strength and stiffness. Bands are stiffened with a continuous middle stiffener plate which ensures the sections remain non-slender and/or compact per AISC 360 B4.1b.

2.2 Additional Acceptance Criteria

Band sections were established based on DBE demand and AISC 360 combined axial, flexure, shear and torsion. Due to the unique structural features and eliminating the need for physical testing, Arup proposed and agreed additional analysis and performance acceptance criteria with the peer review panel and the Los Angeles Department of Building and Safety. These additional criteria are; (1) Limiting inter-story drifts at MCE to be less than 1% in the north-south direction (transverse) and 0.7% in the east-west direction (longitudinal), (2) Performing Finite Element Analysis (FEA) of Steel Bands and Nodes to validate the performance and demonstrating any local, limited yielding of Bands did not result in structural instability, (3) Performing

Fracture Mechanics analysis of Bands and welds by a metallurgist to demonstrate that the Bands resistance to localized fracture against fabrication, geometry and material related imperfections.

3. Analysis Approach

The following procedures are applied in different stages of the analysis and design of (W)rapper Tower. (1) Equivalent Lateral Force (ELF) and Response Spectrum (RS) Procedure: Utilized for preliminary analysis and design, defining isolator properties and for basic comparison purposes. A linear ETABS model is used for this purpose. (2) Nonlinear Response History Analysis (NLRHA) Procedure: Final verification analysis and performance checks for all components of the structure. A Nonlinear LS-DYNA model is employed for NLRHA (see Fig. 4). LS-DYNA is also used for local FE models. The Bands, SPSW web plates and the TFP isolators are all modeled with nonlinear properties in LS-DYNA [3]. The nonlinear model includes a complete representation of the lateral-force resisting system as well as the gravity frame, including 2D elements for composite floor diaphragms with offset gravity beams sufficient to capture both the horizontal and vertical dynamic behavior. The LS-DYNA model has more than 26,000 beam elements and more than 68,000 shell elements with total of 104,000 nodes and 600,000 degrees of freedom.

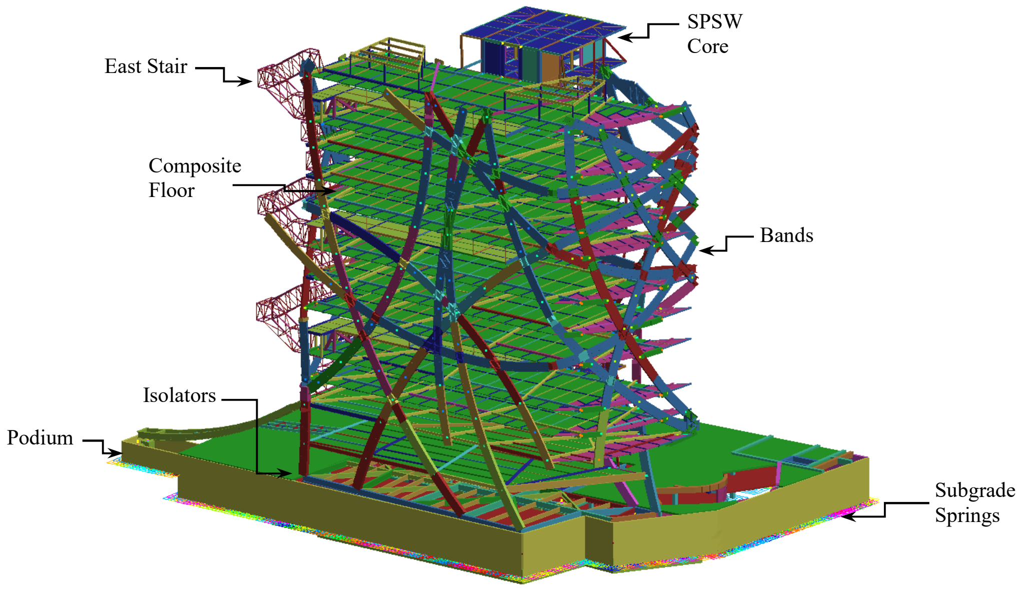


Fig. 4 3D Isometric view of the LS-DYNA model of the (W)rapper Tower

Nonlinear response history analyses are performed using the suite of seven tri-directional time histories developed to sufficiently represent the site-specific DBE and MCE seismic hazard by the Geotechnical engineer. The structure is analyzed for the effects of these horizontal and vertical motions simultaneously with the effects gravity acting on the seismic mass of the building including not less than 25% of the required live loads (per TBI PBD-2010 and LATBSDC-2014). To capture varying earthquake hazard levels and isolator, subgrade and material properties, nine suites of seven-time history records were performed at each design iteration (a total of 63 individual NLTHA analyses) - two suites at DBE, five suites at MCE for member checks and two suites at MCE for global stability (see Table 2). For each ground motion, the maximum of each individual response parameter over the time history (e.g., individual member forces, floor accelerations, isolator displacements, and story drifts) are determined. The maximum displacement of the isolation is calculated as the resultant of the two orthogonal displacements at each time step. The design value of each individual response parameter (element forces, displacements, etc.) is based on the average value of maximums from each of the 7 ground motions in one “suite”. If multiple suites are considered, then the maximum average value is used in design. In Table 2, UB and LB stands for isolator upper bound and lower bound friction properties. See previously published paper of the authors for the details of the various parameters used in the analysis and design [4].

Table 2 – List of all nine “Suits” with various parameters used in the design and performance checks

Name	GMs and EQ Level	Isolators	Subgrade Modulus	Band	SPSW	Purpose / Check
1- DBE_UB_EL_25	(7) DBE	UB	25 pci	Elastic	Elastic	DCRs for Bands and SPSW
2- DBE_UB_EL_76	(7) DBE	UB	76 pci	Elastic	Elastic	DCRs for Bands and SPSW
3- MCE_LB_NL_25	(7) MCE	LB	25 pci	Non-linear	Non-linear	Isolator displacements and uplift, Substructure, Foundations, DCRs for Bands and SPSW
4- MCE_LB_NL_51	(7) MCE	LB	51 pci	Non-linear	Non-linear	Isolator displacements and uplift, Substructure, Foundations, Drifts, DCRs for Bands and SPSW
5- MCE_LB_NL_76	(7) MCE	LB	76 pci	Non-linear	Non-linear	Isolator displacements and uplift, Substructure, Foundations, DCRs for Bands and SPSW
6- MCE_UB_EL_25	(7) MCE	UB	25 pci	Elastic	Elastic	DCRs for Bands and SPSW, Substructure, Foundations
7- MCE_UB_EL_76	(7) MCE	UB	76 pci	Elastic	Elastic	DCRs for Bands and SPSW, Substructure, Foundations, Diaphragm truss
8- MCE_UB_NL_25	(7) MCE	UB	25 pci	Non-linear	Non-linear	Global stability
9- MCE_UB_NL_76	(7) MCE	UB	76 pci	Non-linear	Non-linear	Global stability

4. Structural Nonlinear Modeling

4.1 Bands

Bands are modeled are using beam elements with a combination of Elastoplastic material and elastic material. Elastic material is used for panel zones where two or more bands intersect (see Fig. 5). The major axis bending stiffness of beam elements within nodes (band intersection/panel zones) is increased with a stiffness modifier 2.0. This value is based on a local shell analysis of three node geometries that were subjected to a bending moment and compared to an equivalent stick model.

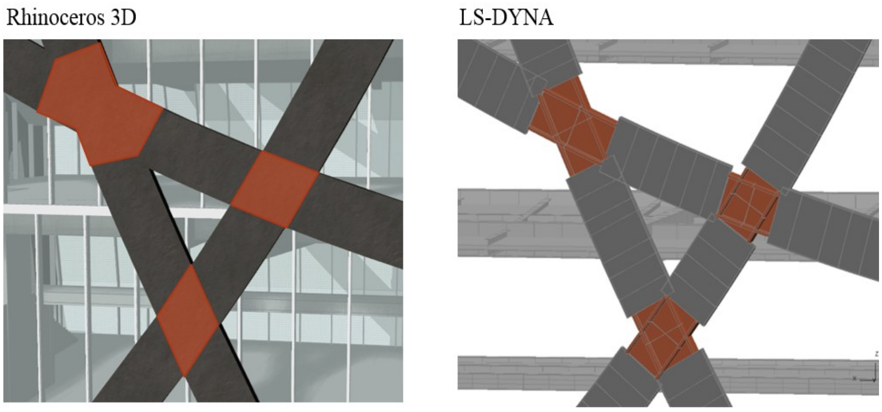


Fig. 5 – Intersection of band elements in 3D geometry model and stiffened panel zone extent in LS-DYNA

The band elements are approximately 24” long with distributed plasticity and fiber section containing 12 integration points (see Fig. 6). Each integration point follows a bi-linear plastic stress-strain relationship with hardening defined by *MAT_PLASTIC_KINEMATIC.

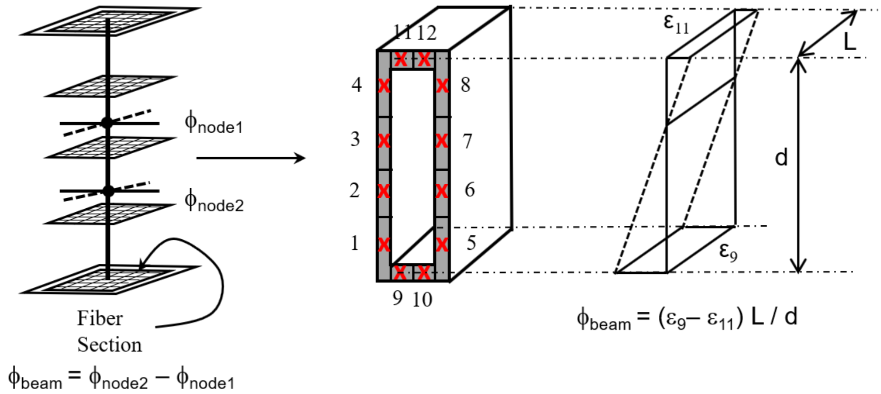


Fig. 6 – Modeling of Bands using fiber section with 12 integration points

4.2 SPSW Core Modeling

The SPSW consists of plates (web plate), boundary elements, gravity framing and concrete floors [5]. The two main approaches for modelling of the web plate of a SPSW are strip elements and shell elements. Strips (non-linear tension only diagonal beam elements) have relatively low computational demand however their representation of web plate yielding is limited. Shell elements can capture the full non-linear behavior of the web plate including buckling and tension yielding. Moreover, they are geometrically simpler to handle. For (W)rapper the core web plates are modelled with shell elements of mesh size 12” × 12” with bi-linear steel material (*MAT_PIECEWISE_LINEAR_PLASTICITY) with hardening (See Fig. 7). A sensitivity study was performed to select this mesh size as sufficient to capture the buckling of the web plates. Only web plates are modeled with non-linear properties. The horizontal and vertical boundary elements (HBEs and VBEs), gravity framing, and concrete floors are modeled elastic.

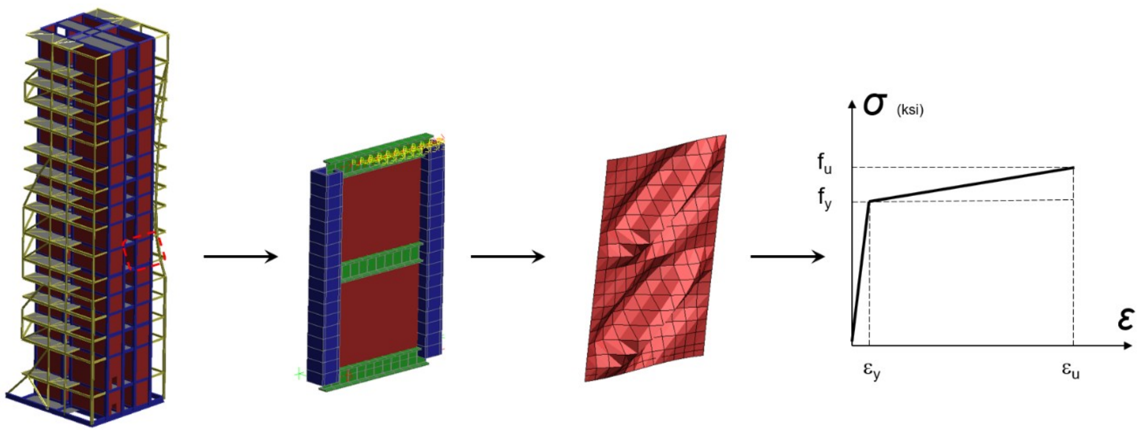


Fig. 7 – Modelling of SPSW Web Plate – mesh and material model

4.3 Seismic Isolator Modeling

Stiffness and damping varies in the TFP isolator as a result of various combinations of sliding that occur on the multiple concave surfaces. To conduct nonlinear time history analysis, the variability in stiffness and damping must be captured properly. MAT_197: SEISMIC_ISOLATOR material model is employed to capture the behavior of TFP bearings in LS-DYNA. In total four elements were used at each isolator, three single concave friction pendulum elements with properties according to model developed by Fenz and Constantinou (2008) [6] (see Fig. 8) and one element representing the isolator rim failure (perfectly plastic element failing at proportion of axial demand according to supplier’s specification). None of the elements can carry tension which is consistent with the designed isolator type. A nonlinear hysteresis model used for isolators is matched to the manufacturer’s prototype test data per §17.8 and §18.9 of ASCE 7.

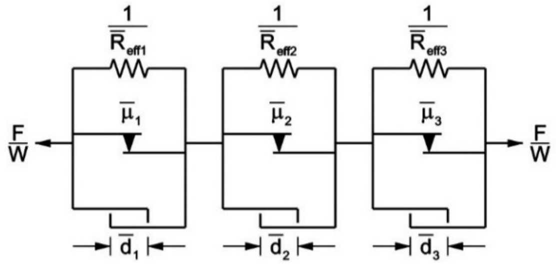


Fig. 8 – Three single concave FP elements in series used to model the behaviour of the TFP bearing

The notations and the properties of the TFP isolator are summarized in the Fig. 9 and Table 3 below.

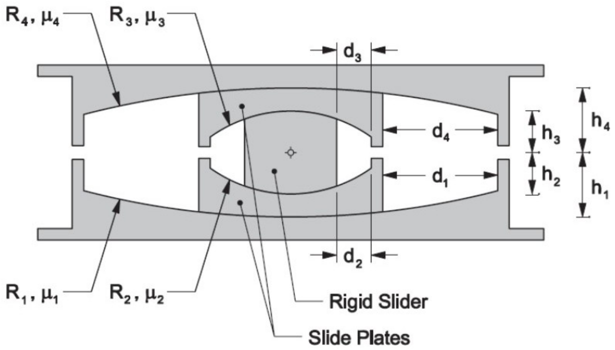


Fig. 9 – Notations used for the implementation of the TFP properties

Table 3 – Proposed TFP isolator properties

Nominal friction coefficient	μ_1	μ_2	μ_3	μ_4
	0.080	0.050	0.050	0.100
Radius (in)	**R_1**	**R_2**	**R_3**	**R_4**
	156	22	22	156
Height (in)	**h_1**	**h_2**	**h_3**	**h_4**
	11.25	6.5	6.5	11.25
Effective radius (in)	**R_{eff1}**	**R_{eff2}**	**R_{eff3}**	**R_{eff4}**
	144.75	15.50	15.50	144.75
Displacement capacity (in)	**d_1**	**d_2**	**d_3**	**d_4**
	13.58	2.18	2.18	13.58

One bearing type (total of 18 isolators) is used under the Tower. Considering manufacturing tolerances, prototype test issues such as first-cycle effects, and long-term environmental effects (aging, contamination, etc.), upper and lower bound friction properties are accounted for in the analysis and design. For FTP15656/30-22S/20-13 bearing, suggested upper and lower bound friction values are summarized in Table 4 and later used in nonlinear response history analysis. The friction determined by the isolator prototype tests fell within the range used for design, therefore no redesign or additional analysis was required.

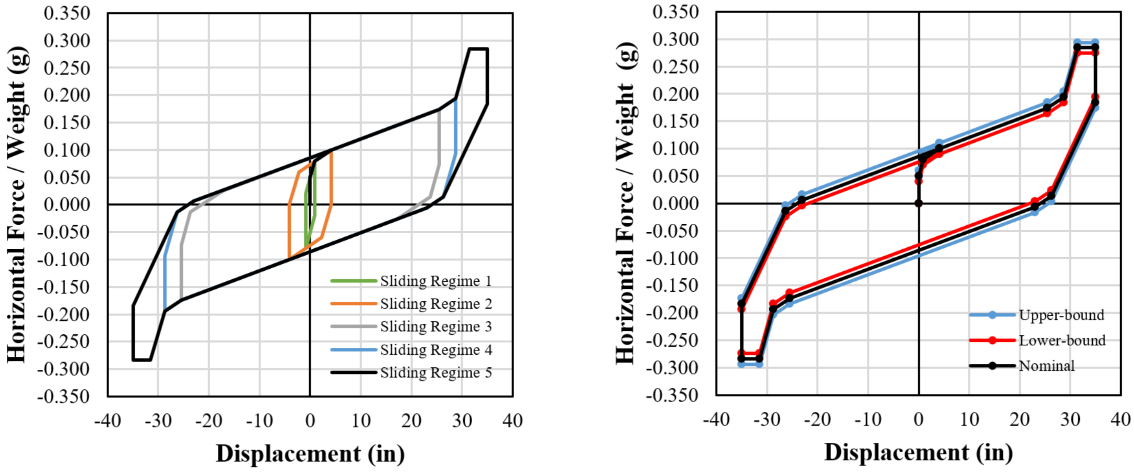
Table 4 – TFP Upper and lower bound friction coefficient

Friction Coefficient	Nominal	Upper-bound	Lower-bound
μ_1	0.080	0.090	0.070
μ_2	0.050	0.060	0.040
μ_3	0.050	0.060	0.040
μ_4	0.100	0.110	0.090

Table 5 summarizes the different stage of the sliding regime used in the nonlinear modelling of the isolators. The theoretical backbone curves are illustrated in Fig. 10 below.

Table 5 – Parameters defining sliding regimes and backbone curve (Nominal)

Regime	Displacement [in]	Horizontal Force [g]	Slope
Sliding Regime 1	0.00” ~ 0.93”	0.050g ~ 0.080g	$1/(R_{eff2} + R_{eff3})$
Sliding Regime 2	0.93” ~ 4.14”	0.080g ~ 0.100g	$1/(R_{eff1} + R_{eff3})$
Sliding Regime 3	4.14” ~ 25.50”	0.100g ~ 0.174g	$1/(R_{eff1} + R_{eff4})$
Sliding Regime 4	25.50” ~ 28.70”	0.174g ~ 0.194g	$1/(R_{eff2} + R_{eff4})$
Sliding Regime 5	28.70” ~ 31.50”	0.194g ~ 0.284g	$1/(R_{eff2} + R_{eff3})$



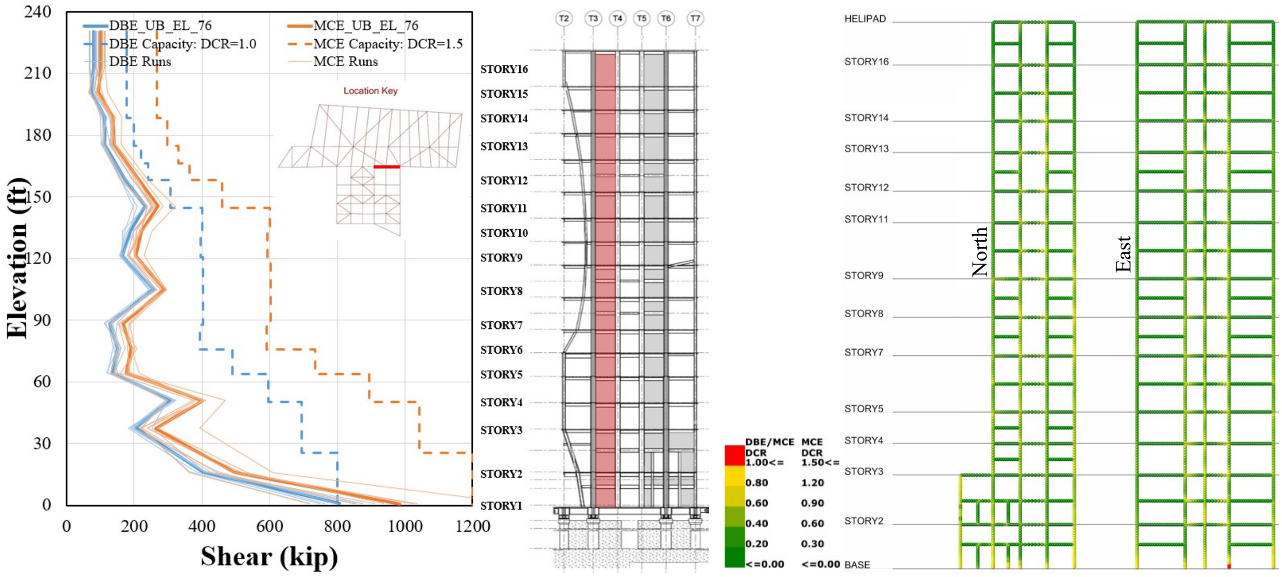
(a) (b)

Fig. 10 – Nonlinear hysteresis backbone curves for isolator FTP15656/30-22S/20-13 used in the analysis

5. Tower Performance

5.1 SPSW Plate Shears and Boundary Frame Demand Capacity Ratios (DCRs)

Web plate thicknesses are established with NLTHA shear demand. The Fig. 11-(a) shows the distribution of demand between the seven-ground motion runs as well as the average demand across all seven runs. The fully yielded capacity of the SPSW plates is shown to demonstrate that the average shear demands do not exceed this level.



(a) (b)

Fig. 11 – Web Plate Shear (a) and HBE-VBE (b) DCRs from NLTHA

Capacity design approached is used for boundary elements per AISC 341. VBE's are designed to have sufficient capacity to sustain simultaneous web plate yielding, hinging of HBE's and Coupling Beams and therefore remain elastic at MCE. Whereas, HBE's and Coupling Beams can be essentially elastic (DCR < 1.5) at MCE. The Fig. 11-(b) show the VBE, HBE and Coupling Beam DCR's for all seven runs of the MCE_UB_EL_76 suite. The DCR's are calculated based on the NLTHA demands for each member of the core. Each member has been sub-divided into approximately 12-inch segments. Each coloured sphere on these plots shows a location of the segment where the combined DCR's were calculated according to AISC360.

5.2 Band Performance

Band sizes were established based on DBE demand and AISC 360 combined axial, flexure, shear and torsion then checked for compliance with MCE demands. In Fig. 12, the DCR equations are mapped back onto an "unwrapped" elevation of the Bands. Fig. 12 shows the Band DCR's as an average, the maximum and separately for all seven runs of the MCE_UB_EL_76 suite. Of the more than 2100 Band sections, only two Band sections in one run of the seven-run suite exceed DCR of 1.0 at MCE, which complies with the performance criteria of essentially elastic (DCR<1.5). These results agree with the conclusion seen in the Tower performance that the Bands do not yield, they are elastic at MCE.

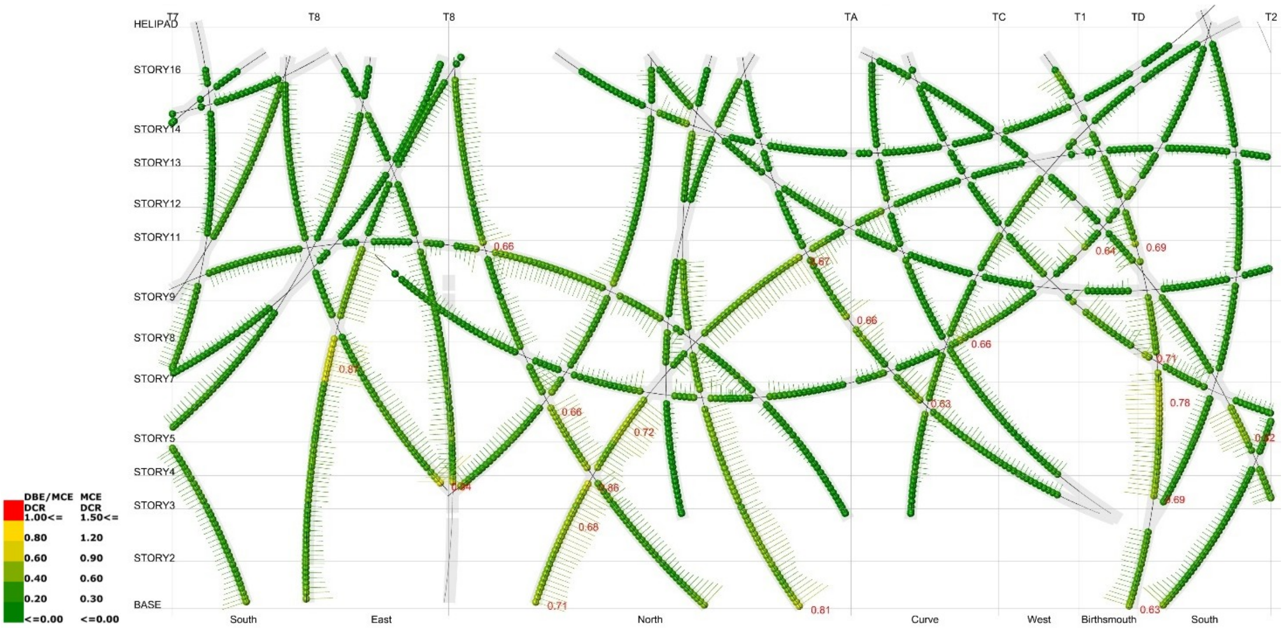
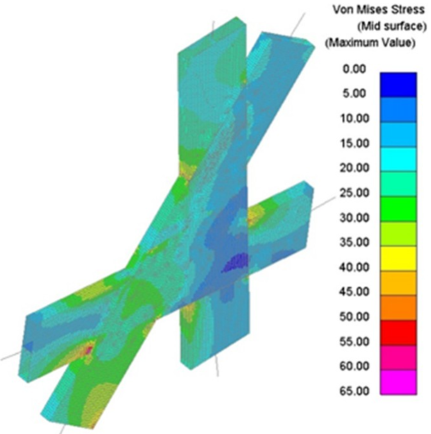
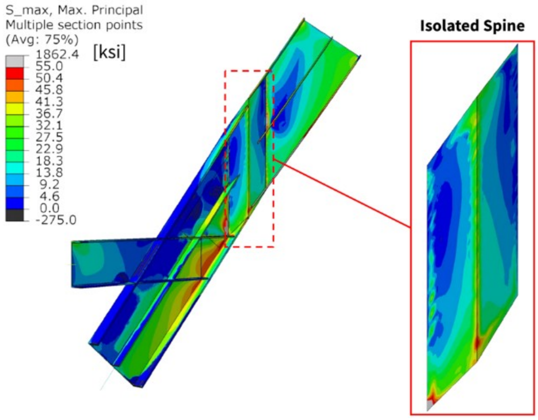


Fig. 12 – Band DCRs from NLTHA in unwrapped elevation view

5.3 FEA for Local Yielding and Fracture

One of the criteria for acceptance of the unclassified Band structure is to ensure that local effects of the Band geometry and cross section design would not result in global instabilities as captured by FEA shell element modelling. Four locations in the Band structure were selected by the design team and PRP for shell element modelling using LS-DYNA. Those are (1) Two Band Node, (2) Multi-Band Node, (3) Corner Node, and a (4) Touchdown column. The models were further examined for ductility and weld fracture by a metallurgist. In addition to typical analysis parameters (von-mises stresses, plastic strains, etc.) an exploration of low-cycle fatigue of the steel material was undertaken in LS-DYNA utilizing a material model that calculates accumulated damage related to ductile fracture with consideration to stress triaxiality. 3-D solid ductile fracture analysis showed low levels of PEEQ vs. TRIAX at MCE conditions when compared to documented ductile fracture curves of similar steel materials. For the geometry and load cases selected at MCE conditions, the linear elastic fracture mechanics analysis of Node welds showed toughness demands well below the estimated critical fracture toughness (See Fig. 13).



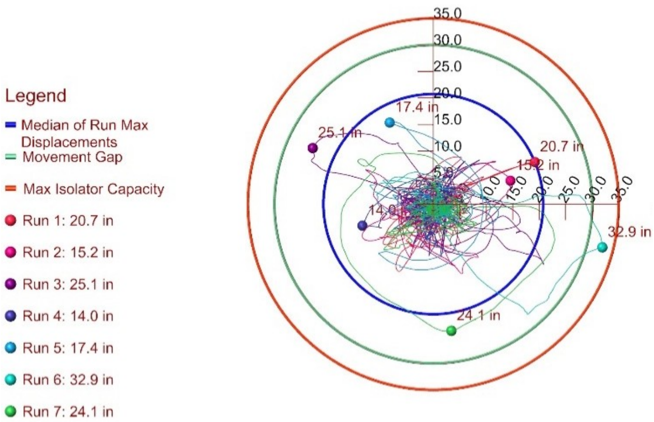


(a) (b)

Fig. 13 – FEA model of a multi-band node (a) and weld fracture analysis models for corner spine (b)

5.4 Isolator Performance

There are nine suites of seven sets of tri-directional input motions in the NLTHA. For each set of motions, the maximum isolator displacement is computed as the peak resultant of the orthogonal isolator displacements over all time steps for the record, for all isolators in the building. The MCE Displacement, D_M, is taken as the mean of these seven maxima, taking the maximum D_M computed for either the lower bound or upper bound isolator property analyses. Fig.14 shows the orbital plots for a selected corner isolator under MCE_LB_EL_76 suite togather with corresponding total base shear. The results show that maximum isolator displacement (averaged over seven-records) is 21-inches which is less than 30-inches of isolator rim impact displacement and less than 32-inches of isolation joint gap. The proposed isolator is stable under the demand of 4.2-inches of maxium uplift.



Earthquake Records	Base Shear (kips) X, u2 (EW)	Base Shear (kips) Y, u3 (NS)
1. Imperial Valley	5,152	4,136
2. Superstition Hills	5,497	3,526
3. Landers	6,369	4,668
4. Kobe	4,302	3,580
5. Duzce	4,807	4,260
6. Darfield	10,957	4,689
7. ChiChi	5,784	5,734
Median NLTHA	5,497 (0.18 W)	4,260 (0.14 W)
ELF	5,209 (0.17 W)	5,209 (0.17 W)

Fig. 14 – Orbital isolator displacements plots from NLTHA at the building corner and total base shears

5.5 Building Drifts

The ASCE 7 limit for the building drift is based on the Life Safety performance objective which states that the drift of the isolated building at the Design Earthquake level shall not exceed 1.5% when using NLTHA. The Band behavior lies somewhere between a brace frame and a moment frame. A typical braced frame has yield drifts on the order of 0.375%, while typical moment frames are closer to 1%. As such, the Bands are likely to yield between these two values (0.7%). Based on this, Arup proposed and agreed with the PRP to limit inter-story drifts at MCE to be less than 1% in the north-south direction (transverse – SPSWs are more effective like a moment frame) and 0.7% in the east-west direction (longitudinal – Bands are more effective). As a result, the mean of the maximum drifts from the seven NLTHA runs were confirmed to be smaller than these values. Fig. 15 shows the inter-story drifts for all runs in the suite MCE_UB_EL_76 at the northeast corner of the Tower floor plate.

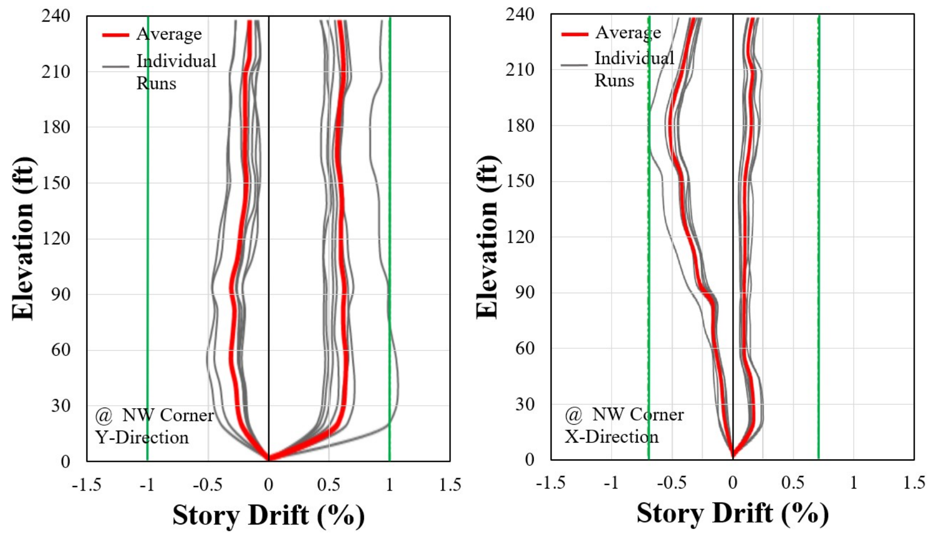


Fig. 15 – Inter-story drifts in MCE_UB_EL_76 suite at the northwest corner of the Tower floor plate.

6. Concluding Remarks

This paper described the approach to the design and performance evaluation of a unique occupied sculpture. It has been shown how an unclassified framing system with an eccentric steel plate shear wall core and seismic base isolation is analyzed, designed and permitted using an alternative means of compliance to the code, and performance-based design. This paper also demonstrates the use of analytical approaches to alleviate the need for physical testing of an unclassified system. SPSW is shown to be an efficient pairing with the Steel Bands and base isolation due to its relatively high stiffness and light weight. An all-steel framed, base isolated structure resulted in a significant reduction of mass and lateral loads, that together with the use of one primary material (steel), resulted in significant construction savings. The project will be one of the cutting edge structural designs in Los Angeles. The project is currently under construction and when completed in 2022, it will become the first base isolated building with a SPSW core in the world.

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