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DESIGN AND ANALYSIS OF A MODULAR TEST BUILDING FOR THE 6-DOF LARGE HIGH-PERFORMACE OUTDOOR SHAKE TABLE

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Abstract

The 1-DOF (uni-directional) Large High-Performance Outdoor Shake Table (LHPOST) at UC San Diego has supported the execution of thirty-four (34) landmark projects since its commissioning in 2004. Large and full-scale experimental data generated at this facility has advanced seismic design practice due to its unique ability to generate essential knowledge regarding the performance of a wide variety of structural and geotechnical systems. LHPOST is capable of accurately reproducing even the most severe near- and far-field ground motions and imposing them to a variety of types of structures without height limitations, due to its outdoor nature. This national shared-use facility is currently being upgraded to facilitate shaking in 6-DOF and will subsequently be renamed LHPOST6. The upgraded facility will facilitate research of important aspects of the seismic response of 3-D structural and geotechnical systems. To complement the facilities equipment inventory, a Modular Test Bed Building (referred to as MTB^2) is being designed for direct adaptation to LHPOST6. This 3-story structure is designed to be a reconfigurable, steel-framed building, which can be reused for testing various structural and nonstructural systems by future researchers. In addition, it will be the first structure to be tested on the upgraded LHPOST6 in 2021. To facilitate a variety of dynamic and inelastic characteristics of MTB^2, three inelastic mechanisms are being implemented. Namely, 1) Buckling-Restrained Braced Frames (BRBFs), 2) Special Moment Frames (SMF), the latter facilitated by a novel shear fuse detailing concept, with easily replaceable fuse plates, and 3) column base connections with yielding stretch length anchors. Each of these systems are replaceable, economical, and facilitate the simple and efficient erection of the MTB^2, while also allowing replacement of strategically placed yielding elements. Amongst the simplest of these systems is energy-dissipating column baseplate anchorage systems. Recently observed in both field reconnaissance and component testing to offer appreciable energy dissipation via anchorage elongation (also commonly referred to as "stretch length"), such connections also provide sufficient ductility and the potential for re-centering, without compromising the strength and stiffness, key considerations in the design of column base connections.

In the present paper, the range of configurations of the MTB^2 are investigated using two different numerical analysis platforms, namely ETABs and OpenSees. Eigenvalue analysis, nonlinear pushover, and nonlinear time history analyses using sequentially increasing directions of input earthquake motions (ultimately 3-DOF; longitudinal, transverse, and vertical) are conducted using each of the finite element models. Results from these analyses are cross-compared and used to support the design of the various inelastic elements of the structure, with a particular focus on assuring a clear delineation of the attainment of individual performance limit states of the lateral load resisting systems.

Keywords: shake table testing, finite element modeling, buckling restrained braces, moment frame, base anchorage

1.0 Introduction

Modularity in building structures is becoming a desirable aspect of structural design for engineers to use in lieu of traditional construction and erection methods. This philosophy is particularly favorable in earthquake resistant design, as it allows for a completely reconfigurable structural system with the added benefit of more easily replaceable fuses, reducing retrofit cost and downtime after an earthquake. To assess the viability of this modular design philosophy, a joint project between the University of Utah and UC San Diego involves testing a full-scale, steel-framed Modular Test Bed Building (MTB^2) in Fall 2021 at UC San Diego's Large High-Performance Outdoor Shake Table (LHPOST6). The building is designed to be completely modular so that after each research use, it can be de-erected, safely stored, and then re-erected in other configurations to address other research questions. Currently under upgrade from a 1-DOF (unidirectional) shake table to 6-DOF, the LHPOST6 facility will allow researchers the opportunity to subject the MTB^2 to sequential, and ultimately, simultaneous ground motions in the three translational degrees of freedom. Ultimately, this will facilitate quantification of full-scale building performance and individual member response under coupled vertical-horizonal ground excitation. In preparation for the testing of the MTB^2, a total of seven earthquake input motions are selected and utilized to perform dynamic analyses. These motions were selected based on record sets utilized in the preliminary design of LHPOST6, from actual recordings with a variety of seismological characteristics [1]. In addition to results from these dynamic analyses, in the present paper, the basic design attributes of the MTB^2 are presented. To facilitate modularity in the buildings design, two lateral force resisting systems were selected, namely: 1) Buckling-restrained braced frames (BRBF) utilizing replaceable buckling-restrained braces (BRB) and 2) Special Moment frames (SMF) which make use of replaceable shear fuses at beam-column connections. In addition, yielding base anchorage are incorporated to allow for assessment of the performance of three different configurations of lateral force resisting systems (LFRS). Numerical analysis of the MTB^2 detailed herein include eigenvalue, nonlinear static, and nonlinear time history analyses, as well as numerical validation of nonlinear fuse components used in the design. It should be noted that, although a suite of seven earthquake input motions were used in the analyses, this paper presents results from the analysis of the 1995 M6.9 Kobe Earthquake (KJM000) and 1994 M6.7 Northridge Earthquake (Rinaldi Receiving Station, RRS) only due to length limitations.

2.0 Archetype Design Methodology

2.1 Geometric Design Philosophy

The MTB^2 is a 3-story steel-framed structure with two bays in the East-West (E-W) direction and one bay in the North-South (N-S) direction. In plan, the building footprint and floor area span 32 ft by 20 ft (9.8 m by 6.1 m), and the floor-to-floor height is 12 ft (3.7 m) on all levels. As shown in Fig. 1, the floor plan is completely open to allow researchers the potential to change the configuration, add walls, test nonstructural components, etc. Moreover, the footprint has been selected to nearly encompass the footprint of the LHPOST6. Large, open spaces between the transverse girders have been provided to allow for the inclusion of testing stairs, or other vertically spanning egress systems, such as elevators, in future configurations. These floor plans were designed with these considerations in mind to embrace the philosophy of a completely reconfigurable and reusable building. All the beams used in the building are A992 W14x34 and all columns are W12x96. The beams are detailed to have staggered holes along the top flange to facilitate mass attachment. The primary mass, which also serves as a rigid diaphragm are large steel plates, four total per floor. The staggered holes on top of the beams may also accommodate future researchers needs for varying the mass, adding equipment, and providing a walking platform to name a few. In the open areas between the girders of the transverse beams (red region of Fig. 1), modular, drop-in decking systems constructed of steel decking overlaid with either poured-in-place concrete or sheathing are provided at each floor level. These modular floors fill the areas between the masses and contribute to overall building weight but foremost complete the structural function of a floor diaphragm, as depicted in Fig. 1. The total weight applied to the building provided by the steel plates and modular decking on the floor and roof levels is approximately 135 kips. This is in addition to the weight of the structure itself,

which will vary based on its configuration as each BRB adds an additional ~1.3 kips. Thus, the total final weights for the BRB-1, BRB-2 and SMF configurations are approximately 208, 202, and 191 kips, respectively, which translates to a maximum of approximately 5% of the vertical payload capacity of LHPOST6.

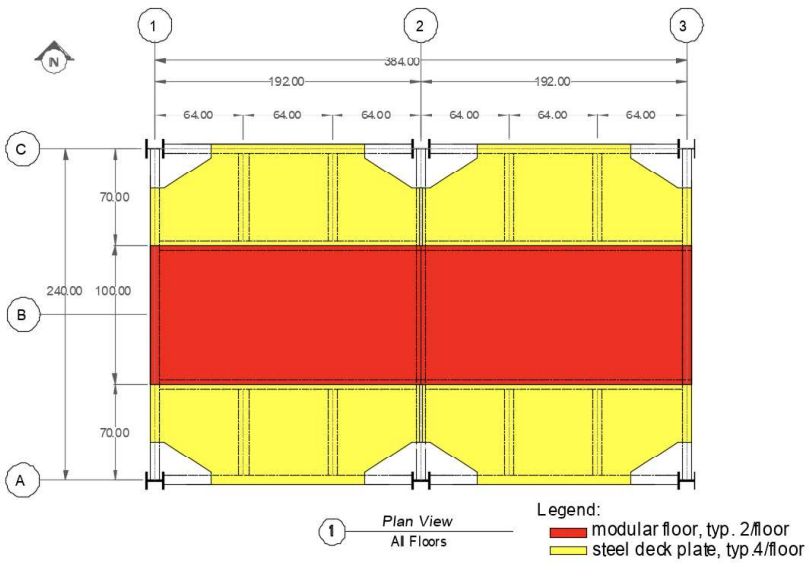


Fig. 1 – Typical floor plan of the MTB[2]. All units in inches (1 in. = 2.54 cm).

The building has also been designed allow for reconfiguration of the LFRS in the E-W direction. In the N-S direction in all configurations, BRBs are oriented in a chevron BRBF in the outer two bays of the MTB[2], see Fig. 2a, while the center frame (line 2) is designed as a gravity frame. Three different LFRS were selected for this study, as follows: 1) BRBF-1 Configuration which uses BRBs in all three stories in both E-W bays in the configuration shown in Fig. 2b, 2) BRBF-2 which has replaceable shear fuse connections at the beam-column connections at the top of the first story, and BRBs in the remaining two stories, shown in Fig. 2c and 3) SMF Configuration which has replaceable shear fuse connections on all three stories in the E-W direction at beam-column connections, Fig. 2d. The objective of designing for three different configurations is to assess global dynamic building response for each configuration and characterize BRB, shear fuse and stretch length anchor [see e.g. 2, 3] behavior although all three configurations may not be physically tested. These fuses are specifically designed and detailed to be easily replaceable after they have achieved their targeted performance limits, while the rest of the structure remains virtually undamaged.

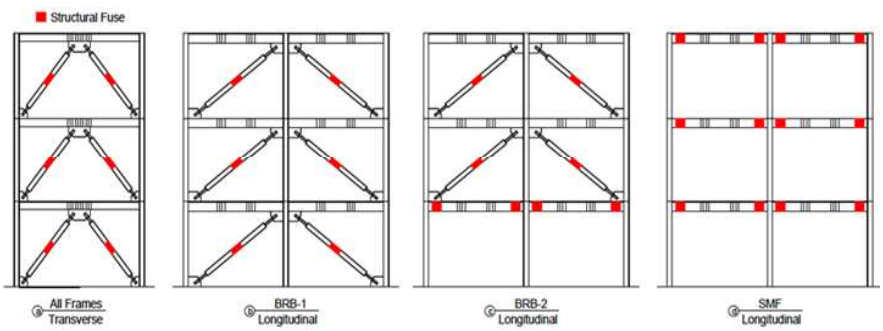
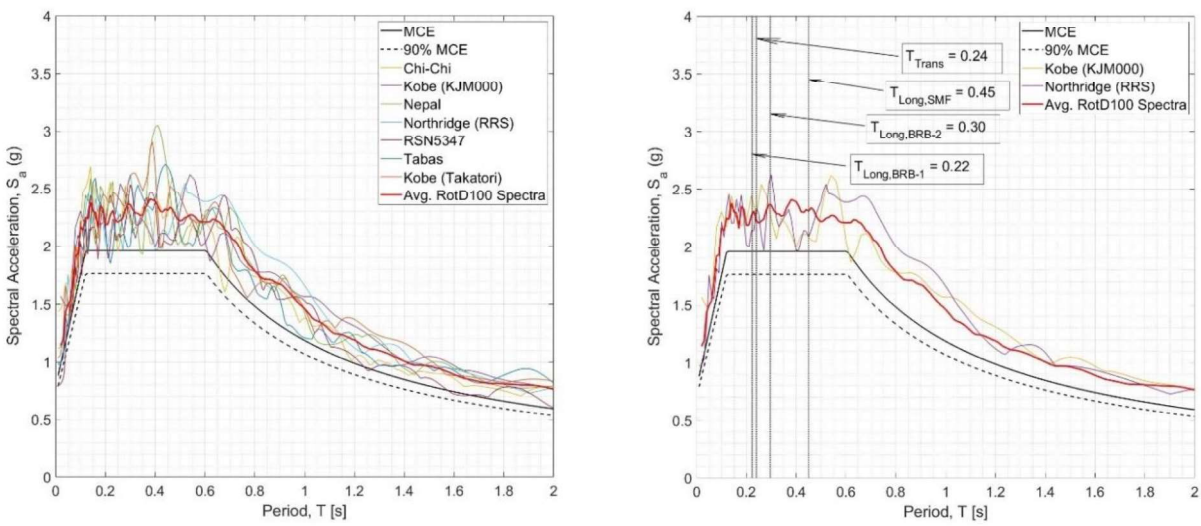


Fig. 2 – Various configurations of the LFRS bays of the MTB[2]: (a) Transverse LFRS Frame, (b) BRB-1 Frame, (c) BRB-2 Frame and (d) SMF Frame.

2.2 Modal Characteristics

For the three different LFRS systems of the MTB^2 used during the test program, the modal characteristics are of great importance in design. As explained in Section 2.1, supplemental mass is provided via steel plates and concrete over metal decking on stories 1 and 2 and metal roof decking at the top level of the structure. The choice of decking was dictated by the desire to remain consistent with decking used in a traditional office building while providing a realistic vertical distribution of mass. The target supplemental mass of the MTB^2 was determined by determining a target period range of interest for buildings of this height in practice, given any of the three LFRS configurations and iterating. This iteration was completed through eigenvalue analyses, discussed in more detail in Section 4. The final dispersion of modal properties can be seen in Fig. 3, which plots the first modes of each MTB^2 configuration on the suite of ground motion spectra to illustrate how the desire to have modes which span a range of high spectral demand was ultimately achieved. Note that in these figures, there is only one representation for the N-S or transverse period, although there are three configurations; that is because the eigenvalue analysis showed that the change in N-S period is negligible from configuration to configuration as the change in mass is relatively small.



(a) (b)

Fig. 3 – a) Spectral Acceleration plot of the conditioned ground motion suite and b) dispersion of modes compared to 1994 Northridge(RRS) and 1995 Kobe(KJM000).

2.3 Performance Objectives

To facilitate robust seismic performance of a structure, it must be designed to dissipate energy efficiently at well-defined fuses throughout the structure, which have an intended performance objective. When these fuses attain their performance objective (PO), they can easily be replaced before damage occurs to elements intended to remain elastic. The desire to facilitate this in the MTB^2 as a building system is coupled with the need to target the PO across both member and global levels. Motion scaling in turn must consider the span of these objectives, as this will influence the ground motion selection and scaling discussed in Section 3.0.

The LFRS of the MTB^2 configurations used in this program utilize either BRBs, shear fuses, yielding anchors or a combination of both. These yielding elements have individual performance objectives, which have been defined based on component testing performed prior to these full-scale tests [4-6]. The BRBs used

have a displacement-based PO of approximately 2.5% axial strain, for each of the BRBs in both the transverse and longitudinal BRBFs. The shear fuses used in SMF frames also have a displacement-based performance objective, which is approximately 0.045 rad at the face of the column. The PO of the yielding anchors is approximately 15-20% axial strain which create a base plate detail that allows for rigid rotation of column bases to accommodate the predicted large roof drifts. The benefit of these elements is that they are all quite ductile, while also incorporating the ability of the structure to dissipate significant amounts of hysteretic energy while being simultaneously able to withstand large displacements. Numerical modeling of the yielding elements is discussed in Section 4.0.

3.0 Ground Motion Selection and Scaling

The suite of earthquake input motions used in the design and analysis of the MTB^2, are those selected for the preliminary design of the upgrade of LHPOST6 [1]. These motions encompass a range of near-fault and far-field motions with varying frequency content and intensity as to facilitate the range of seismological settings envisioned in practice. Following *ASCE 7-16* [7] procedures for ground motion scaling, the seed (original recorded) motions were conditioned to create Maximum Considered Earthquake (MCE) compatible records to calculate the maximum direction spectrum to be used in non-linear time history analyses. The elastic acceleration response spectra were created with an assumption of 5% damping and calculated via Newmark's constant average acceleration method. Directionality of earthquake ground motions are an important design consideration to account for; thus, the maximum-direction spectrum (RotD100) is computed by taking the maximum envelope of the pseudo-accelerations of 0° to 180° in increments of 2°. Although these conditioned and scaled spectra coupled with the average RotD100 spectra would subsequently be computed and scaled to 90% MCE, according to present ASCE 7 procedures, herein a slightly different approach was adopted. Namely, rather than applying a uniform scale factor across all spectra within the ground motion seeds in the suite, it is recognized that this may not lead to attainment of the targeted performance objective. In this regard, individual seed motions were scaled, the scale factor estimated by attainment of the targeted PO for a given building configuration.

4.0 Numerical Modeling

The modeling for this structure was performed using two different programs, namely, *OpenSees* [8] and *ETABS* [9]. *OpenSees* is an open-source, research grade software often used for its advantageous ability to accurately model nonlinear behavior and targeted earthquake analysis capabilities. *ETABS* is a general-purpose civil engineering software for the design and analysis of structural systems, it is widely used by practicing engineers. The advantage of using two different software packages to model the MTB^2 is two-fold. One is that there are constant checks and balances as modeling progresses from linear static analyses up to nonlinear time-history analyses. The next advantage is a comparison of research-grade software to a general-purpose software for complex nonlinear modeling can be made and used to evaluate the adequacy of the nonlinear modeling of these specific elements in *ETABS*. In what follows, the characteristics of each model are presented, and a cross-comparison amongst the results using the two different FE software platforms discussed.

4.1 OpenSees Numerical Model

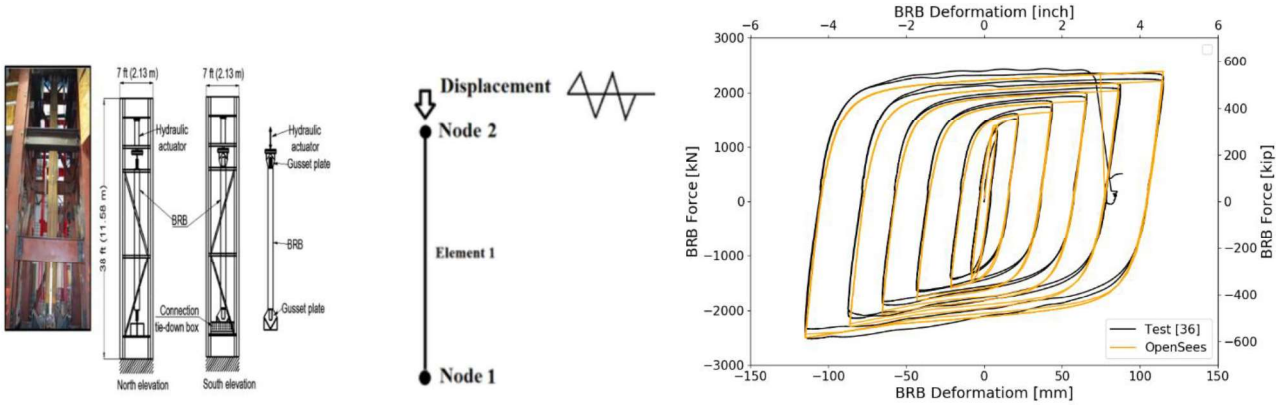
A 3D finite element model of the MTB^2 was created using the *OpenSees* platform [8]. In this case, columns and beams were modeled using force-based beam-column elements with elastic sections that have four Gauss-Lobatto integration points each. To simplify the model and avoid numerical instability, linear transformation was used to transfer the element stiffness and resisting force from the local-coordinate system to the global coordinate system. A rigid diaphragm assumption is invoked to constrain the in-plane stiffness of the frame as is commonly observed in in-situ conditions of flooring systems. The connections in the BRBFs were simulated as pinned, while the shear-fuse moment frame connections were modeled with lumped inelastic moment-

rotation hinges. Concentrated point masses were assigned to the center of the floor in the in-plane direction at each level for simplicity. Gravity analysis was performed, and base reactions verified against hand calculations to ensure the accuracy of the assigned nodal masses. Transformation constraints were used to handle the boundary conditions when carrying out nonlinear static and dynamic analysis.

The BRB members were simulated with truss elements, to ensure no moment transfer occurs at their connections to the primary frame. BRB components were calibrated by comparing to experimental test specimens [4][10]. In this study, a numerical model of a BRB quasi-static cyclic test was subjected to the cyclic axial displacement loading protocol used in an actual experiment [10]. A comparison of the numerical and experimental hysteretic response is presented in Fig. 4. It was found that the *OpenSees* BRB model accurately captured the BRB cyclic test behavior [10]. The final BRB material model, which is a combination of the *Steel02*, *Pinching4*, and *Fatigue* material is assigned to a corotational truss element. This corotational truss element is defined from workpoint-to-workpoint and has a cross-sectional area of the design BRB core equal to 1.15 in^2 (29.12 mm^2) Since the BRB element in the numerical model is assigned between work-point, an equivalent BRB stiffness is used to consider both the yielding core and end sections. The modulus of elasticity of the BRB material in the numerical model was modified to reflect the core stiffness, as per Eq.(1) [10].

$$E_{wp} = E_s \frac{L_{wp}}{L_{core} + 2L_{end}\frac{A_{core}}{A_{end}}} \qquad [1]$$

Nonlinear static and dynamic analyses were conducted to evaluate the performance of the building with the three configurations. The nonlinear static analysis captures the weak links and failure modes of the structure with increasing intensity of the earthquake input motion. Leslie (2009) [11] compared the effects of different load patterns including Uniform distribution, equivalent lateral force distribution, response spectrum analysis, and upper-bound pushover analysis when conducting nonlinear static analysis; and observed that for all four loading types, the performance points are remarkably close. Therefore, in the present, the equivalent lateral force distribution was adopted. To perform the dynamic analysis, each configuration is analyzed with the same suite of seven earthquake input motions, as previously discussed. Classical Rayleigh damping is used with the assumption of 2% damping for the first two translational modes. For each earthquake of the suite, its ground motion time histories are used to define four load cases which are longitudinal shaking (E-W, 1-DOF), transverse shaking (N-S, 1-DOF), combined longitudinal and transverse shaking (E-W and N-S, 2-DOF), and finally a coupled vertical-horizontal ground shaking (3-DOF) which includes the measured vertical component from that associated receiving station and is scaled equally to the horizontal ground motion seeds. This sequence of load cases was defined to track the attainment of performance objectives and compare overall structural behavior as different ground motion components are included in each load case. This is particularly important when analyzing the base of the structure to determine the implications of coupled vertical-horizontal ground motions on the reactions at column baseplates for the different configurations. Results from both the nonlinear static and dynamic analysis will be presented in Section 5.0, following description of the complementary ETABS model.



(a) (b) (c)

Fig. 4 – (a) Experiment Setup; [4] (b) OpenSees Model for the BRB component Test; [10] (c) Comparison of *OpenSees* BRB model and test results [10]hm

4.2 ETABS Numerical Model

As noted, a 3D finite element model of the MTB^2 was also developed within the *ETABS* platform [9]. Similar to the OpenSees model, frame elements with appropriate section properties were used. To conduct modal and linear analyses, the shear fuses in the SMF frames are not explicitly modeled, but rather the beam-column connections are assumed fully-fixed with no moment releases, which is an accurate representation of the rigid-plastic behavior of the shear fuses exhibited in component testing. Results from the eigenvalue analyses utilizing both models, for the three structural configurations are shown in Fig. 6. This figure displays the first two modes in the principal direction of the given LFRS and demonstrates that a more than two-fold increase in fundamental period has been achieved via the varying LFRS and in the different directions, $T_1 \sim 0.25 - 0.5$ sec. In linear-elastic analyses, the BRBs in the BRBF are modeled as frame elements, using appropriate section properties and stiffness modifiers determined by the designer and manufacturer. These are adequate modeling simplifications for the linear analyses of the MTB^2 for determination of the stiffness and associated modes as no performance objectives are being tracked yet. The floor system consisting of steel plates and drop-in decking and in ETABs was modeled by thin shell elements with no sectional properties. The shell is defined in the numerical model to act as rigid diaphragm, idealizing the in-situ condition of the floor diaphragm. Concentrated point masses were originally applied at the center of the floor diaphragm at each level to simplify the modeling of the additional mass from the floor components. Then, an evenly distributed mass was applied to the surface area of the shell elements to better represent the actual distribution of mass at the individual floor levels for more accurate modeling in *ETABS*.

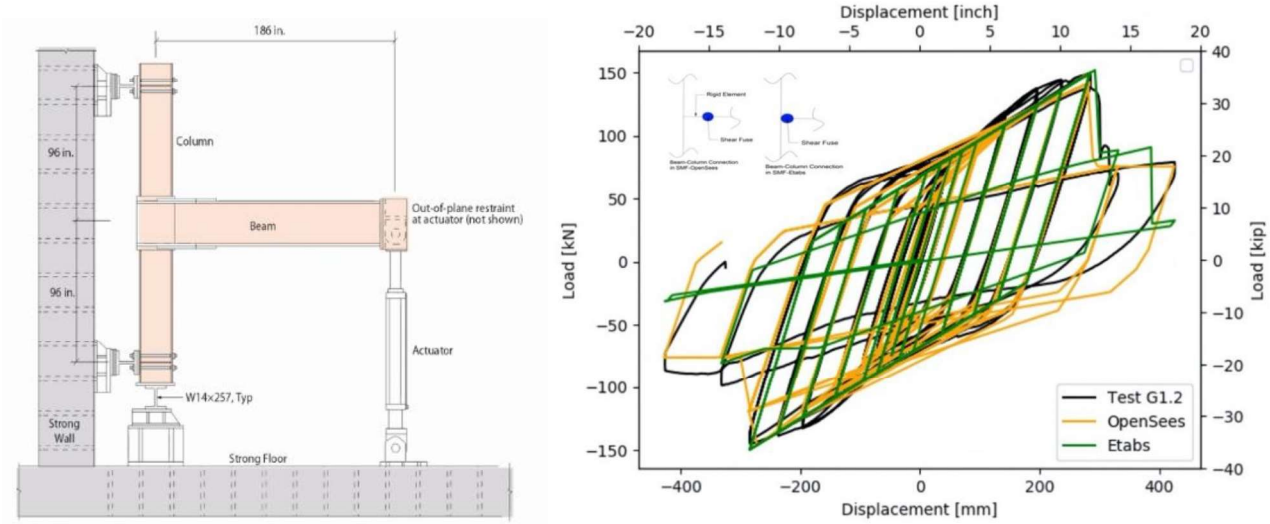
Beyond the eigenvalue analyses, the assumption and use of frame elements for the BRBs with additional stiffness modifiers becomes inadequate. As with the BRBs, the idealized fully-fixed assumption of the shear fuses in SMF frames also becomes inadequate for the purpose of accurately and precisely capturing the nonlinear response of the fuse regions to their performance targets. In this regard, similar to the OpenSees analysis, validation of the numerical model physical test specimen response was deemed necessary, a sample of such analysis for the moment frame shear fuse (SMF) connections is provided in Section 4.3.

The BRBs are composed of two sections: and elastic element and a yielding element. For nonlinear analyses, these are modeled separately where the elastic element is modeled on either side of the yielding element with appropriate sectional and linear elastic properties. The plasticity is considered within lumped springs, which connect the elastic elements at fixed nodes and are modeled as multi-linear plastic axial springs using backbone curves, which were calculated based on the recommendations of the manufacturer. The shear fuses for the SMF frames are modeled as zero-length, multi-linear plastic rotational spring elements using a backbone curve derived from physical testing and validated prior to integration in the *ETABS* numerical model.

Nonlinear static pushover analysis in accordance with ASCE 41-16 is performed in ETABS and the results of the analysis are discussed in Section 5.0. Uniformly distributed dead loads determined from tributary areas and assumed equally distributed floor mass are applied to the floor beams for this analysis. Although not the subject of this paper, it was found that nonlinear hinges could be used to represent the yielding elements in the nonlinear static procedure; however, when advancing to NLTH analysis it was also determined that hinges could not reliably represent the cyclic behavior of the fuses as well as the springs; thus, springs are incorporated in all nonlinear analyses. Lastly, a direct-integration, NLTH analysis using Newmark's constant average acceleration method is conducted in *ETABS* for the MTB^2 using the same assumptions and procedure as the OpenSees model, discussed in Section 4.1.

4.3 Numerical Validation of Yielding Shear Fuses

Before incorporating the shear fuses of the SMF frames into either of the two numerical models for dynamic analyses, validation of the numerically modelled component against physical testing must be proven. The physical tests were conducted by Reynolds and Uang (2019) [6]. Both ETABS and *OpenSees* were used to validate the shear fuse model of the SMF frames. In *ETABS*, numerical models using frame elements exactly replicating three different physical tests of shear fuses in various configurations and size were created and zero-length rotational springs were defined at the beam-to-column connection. Estimated backbone curves created from the test data were used to define the multi-linear plastic behavior of the rotational springs and it was determined that the *ETABS* model could accurately replicate the physical test results provided by the manufacturer. Numerical analyses of the physical tests were also conducted in *OpenSees* to validate shear fuse behavior. To better capture the connection rigidity observed in experimental testing, the beam-column joints in SMF were modeled in *OpenSees* with rigid elements connected to shear fuses modeled using zero-length elements with hysteretic material using the same backbone curve as defined in *ETABS*, as shown on the upper left in Fig.5(b) The comparison between the numerical models and the experimental data from test G1.2 is shown in Fig. 5. These results show that ETABS and *OpenSees* can accurately capture the hysteretic behavior of the replaceable shear fuses prior to failure. The ETABS model predicts failure slightly before *OpenSees* or the experimental tests but still accurately captures pre- and post-peak behavior of the shear fuse which is the primary concern of this study for use in modeling the MTB^2.



(a) (b)

Fig. 5 – (a) Experiment Setup [5];[6] (b) Comparison of ETABS and OpenSees modeling of replaceable shear fuses from Test G1. [5];[6].

5.0 Numerical Modeling

Results from the eigenvalue analysis from each numerical model are presented in Fig. 6. It can be observed that the first two modes of each configuration between the two models are in good agreement. This lends confidence towards the reliability of both numerical models, at a minimum under linear elastic response. Results from the nonlinear static pushover analysis is presented in Fig. 7. The final point of each curve indicates the displacement at which the structure becomes unstable and fails due to incipient attainment of a performance limit of one or more yielding component. Fig 7. shows good agreement between the pushover curves between the two models, for each configuration and direction, well into the nonlinear response of the models.

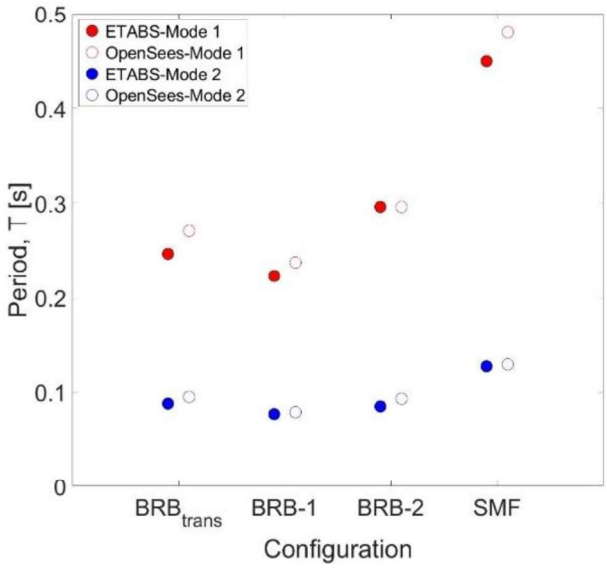


Fig. 6 – Estimated period of first two modes in the direction of given LFRS of each MTB^2 configuration.

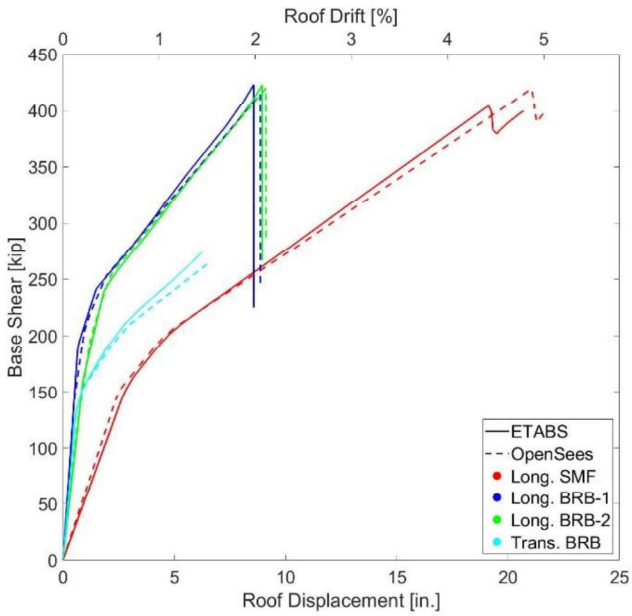


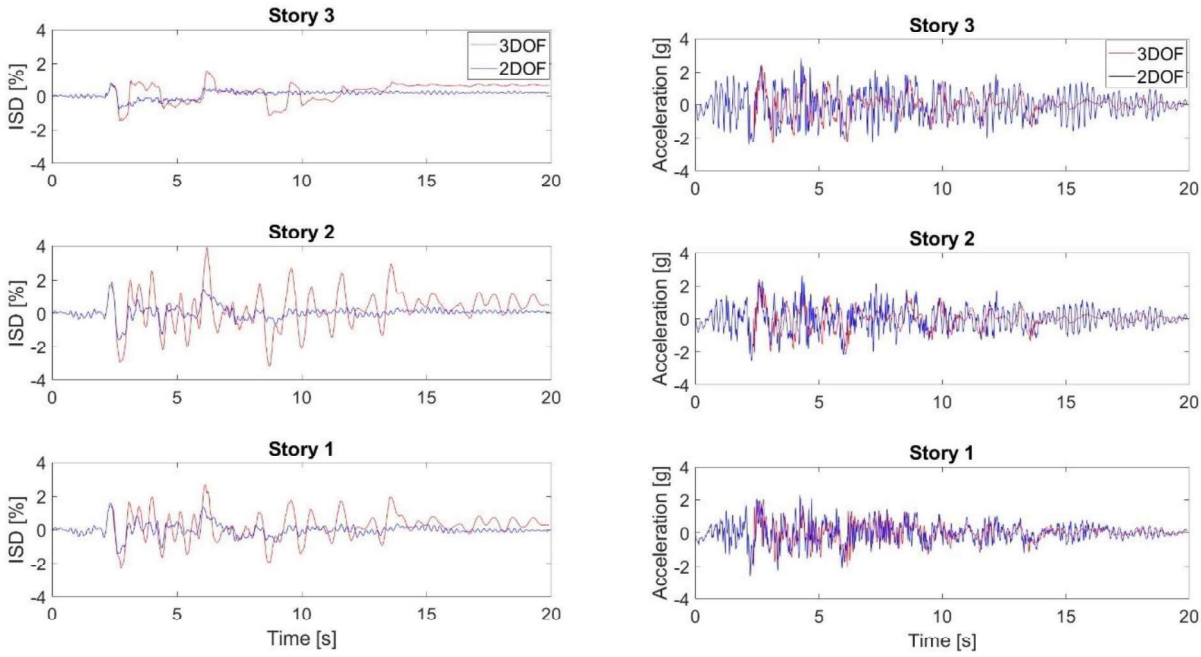
Fig. 7 – Nonlinear static pushover analysis for MTB^2 configurations comparison between *ETABS* and *OpenSees* numerical analyses.

SMF configuration pushover curves show the most appreciable displacement ductility ($\mu_{\Delta,capacity} \sim 8$) when compared to the other three pushover curves, which is consistent with this type of structural system. The model also shows that the SMF configuration is the softest of all configurations, while the remaining configurations have higher, and relatively similar, initial tangent stiffnesses. Likewise, this observation is consistent with the design assumptions and anticipated differences in response of a BRB-framed versus a moment-framed LFRS. Utilizing the pushover analyses, progression of limit states, or performance objectives, can be observed so designers can be conscious of when and where replacement of fuses is likely to occur after a seismic event. In this regard, the nonlinear static pushover analysis reveals that over all conceived configurations, failure of ductile fuses in either direction will first occur on the second story. Although one may generally anticipate that initial failures would occur on the first (ground) level, their performance limits are attained almost *after* limit state attainment of the second story fuses.

NLTH were conducted using both *OpenSees* and *ETABS* using the same scaled earthquake motions and sequential load cases, i.e., considering 1D, 2D, then 3D input motion cases, for each of Northridge (RRS) and Kobe (KJM000) motions and for all three configurations of the MTB^2. Fig. 8 shows the transverse (NS) story level interstory drift (ISD) and acceleration time histories for the 2DOF and 3DOF input cases of the Northridge (RRS) for the SMF configuration. Upon investigation, it was found that there is negligible difference in story level accelerations as the analysis progressed from 1DOF to 2DOF input motion, therefore only 2DOF is included in this plot as representation. In Fig. 8, however, it is seen that the 3DOF shaking case is where the ISD and acceleration time histories divert from the 2DOF case. This is because the inclusion of the raw vertical ground motion forces the BRBs in the transversely oriented BRBFs to surpass their desired PO, to the point of failure. This failure of the fuse causes a redistribution of force and significant reduction in stiffness in the longitudinal direction which explains the excessive floor level drift seen in Fig. 8 for the 3DOF input case. It is also notable that that the residual story level ISD are more pronounced for the 3DOF input motion case, compared with the 2DOF analysis case. In the longitudinal (EW) direction, it was found that the inclusion of the vertical ground motions was not sufficient to fail the replaceable shear fuses in these frames, so the differences between displacement responses from 1DOF up to 3DOF loading are negligible.

Analyses showed inclusion of the vertical ground motion in the Kobe sequence had no effect in forcing the longitudinal fuses beyond their PO to failure, similar to the 3DOF Northridge sequence. However, unlike the Northridge sequence, the Kobe analysis showed that the inclusion of the vertical ground component does not cause failure in the BRBs of the transverse frames. The results for the BRB-1 and BRB-2 configurations are similar compared the SMF configuration in that 3DOF Northridge sequence caused failure in the transverse BRBs of those configurations as well.

The effect of coupled-vertical horizontal ground motions on the base shear of the MTB^2 was also investigated. It was determined from the analyses using both numerical models that when evaluating the unidirectional base shear, that is either in the transverse or longitudinal direction, there is negligible difference between the sequential directional earthquake motion cases. However, when considering maximum resultant base shear by including both orthogonal components for the 2DOF cases, as anticipated, the maximum base shear when 2-dimensional horizontal input motions are imposed are larger than the unidirectional input motion case. But when coupling vertical ground motions with the horizontal components, it is determined that the inclusion of the vertical ground motion results in negligible difference in maximum resultant base shear. It is important to note however that, although the maximum resultant base shear in the 3DOF cases is similar to the base shear of the 2DOF cases, the axial forces experienced by the columns are significantly higher than that of the 2DOF cases.



(a) (b)

Fig. 8 – Comparison of 2DOF and 3DOF input motion cases for 1994 Northridge (RRS) for the SMF ETABS model: a) interstory drift and b) acceleration time histories calculated in the transverse direction. (1 in. = 2.54 cm).

6.0 Conclusions

In the present paper, the design and modeling of a modular test bed building, referred to as the MTB^2, is discussed. Design and modeling decisions are discussed, and evaluation of a component-level model used in numerical modeling of the overall structure presented. The MTB^2 is intended to be a community resource for use by researchers at the Large High-Performance Shake Table (LHPOST6) at UC San Diego. As a 3-story modular steel building, it facilitates use of a variety of lateral force resisting systems, while also being designed to directly attach to the LHPOST6. Results from the analysis of two different numerical models are presented herein in an effort to guide the design of the MTB^2. Notably, dynamic nonlinear time history analysis will be useful in guiding the motion scaling of the physical MTB^2 planned for testing in Fall 2021. These results have led to important discussions regarding motion scaling to achieve the desired performance objectives of the structural fuses. In addition, floor level acceleration histories are a result of importance as these will be used to create floor level acceleration response spectra for the eventual inclusion of hung or mounted nonstructural component systems. ISD response results discussed in Section 5, will be used to create interstory drift envelopes for all motions of the suite to design and implement drift sensitive nonstructural components to the test plan. Running sequential analyses to determine these spectra and other envelopes allows the researchers to quantify the effect of coupled vertical-horizontal ground motions on the aforementioned components which will be included in physical testing.

Comparing results from the NLTH using the Kobe (KJM000) and Northridge (RRS) motions indicate that the addition of the vertical component of the Northridge (RRS) Earthquake had a more significant effect on the transverse fuses, causing these elements to surpass their prescribed PO, whereas including the vertical component of Kobe (KJM000) was not. This highlights the importance of performing coupled vertical-

horizontal analysis in the design of buildings as drift sensitive systems as well as overall structural health could be compromised if significant plastic redistribution and failure of fuses occur.

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