Seismic Performance of a Tall Diagrid Steel Building with Tuned Mass Dampers

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Abstract— The steel diagrid structural system is a recent load bearing and lateral resisting structural system for tall building structures that is relatively unexplored in the western United States. One possible reason for the little use of diagrid systems in earthquake prone regions is the lack of guidelines and application examples illustrating the design and analysis of these structures. In this work, a 72-story prototype building is used as an example for which the design and analysis of the diagrid system is performed. To mitigate the possible large displacement and base shear demands that these structures may undergo under seismic events, two new design solutions consisting of one or two friction tuned mass damper (TMD) units are explored. In the first solution, a TMD is placed on the top four stories of the building and is tuned to reduce the contribution of the fundamental mode of vibration of the structure, in both horizontal directions. In the second solution, a double TMD system is added at mid-height of the building, in which a second TMD unit in tuned to the second period of the structure.. Using a nonlinear finite element model of the tuned mass damper, the effectiveness of the friction mass damper design is studied. The mass damper system consists of a concrete tank containing sand or water. The tank is placed in between the building reinforced concrete structural core and the exterior steel diagrid system. This mass damper is connected to the structure using friction pendulum isolators that are chosen due to their ability to undergo large deformations. The models are then subjected to accelerograms from historical shallow crustal earthquakes and subduction zone earthquakes. Parametric studies are carried out to understand the influence of different parameters of the mass damper design in improving the seismic performance of the building. Improvement of the seismic performance is assessed in terms of minimization of inter-story drift ratios, base shear forces, as well as floor absolute accelerations. The results show that the single TMD system can reduce significantly the peak base reaction and inter-story drift envelopes. Addition of the second TMD provides further improvements in terms of reducing the peak base reactions, while also producing notable reductions in peak absolute floor accelerations, which are not observed when only one TMD unit is used.

Keywords— **Cascadia Subduction Zone Earthquakes, Diagrid, Earthquakes, Friction Pendulum Isolator, Nonlinear Structural Analysis, Seismic Design, Steel Structures, Tall Buildings, Tuned Mass Damper.**

I. INTRODUCTION

THE FIRST tall buildings were built in United States of America in the late nineteenth century [1]. There is no set definition for what constitutes a tall building, but often buildings that are 14 stories or more are considered as tall buildings (50 m or taller). At present times, many tall buildings have been built around the world and The Council on Tall Buildings and Urban Habitat contains information on more than ten thousand tall buildings [2]. Several structural systems have also been developed to realize mankind's dream in pursuing new heights and allow for the design of tall buildings. Out of many structural systems, the outer steel diagrid structural system with reinforced concrete inner core is but one of them. The diagrid structural system gets its name from the diagonal columns that form triangular trusses. Diagrid is an abbreviation for "diagonal grid" of trusses. The diagonal trusses are connected by horizontal rings (steel beams), which provide support for the floors and column buckling restrains. The diagonal members carry gravity loads as well as lateral loads, and thus steel is typically used in diagrid structures. The main difference between conventional steel exterior-braced frame trusses (X, K, V, and Chevron type braces) is that in the diagrid structural system almost all vertical columns are typically eliminated. The interior reinforced concrete core typically consists of walls and is designed not only to resist lateral loading, but also to allow for the vertical communication and transport of people (elevator shafts and stairwells), heating, ventilation, and air conditioning, plumbing, electrical, and fire protection systems.

The diagrid structural system is known for its redundancy, continuous and uninterrupted load paths, and is thus considered to be a very efficient structural system [3]. However, these efficiencies also come with drawbacks. Most new structures that have been designed and built using this system are lighter and more flexible than conventional tall building systems, and thus can suffer large displacements, especially under wind and seismic loading. Some prime examples of this kind of structures are the Hearst Tower, in New York City, the China Central Television (CCTV) Headquarters in Beijing, China, and the Tornado Tower in Doha, Qatar.

The diagrid structural system is relatively new and unexplored in the Western United States and other high-seismic regions of the world, and hence engineers lack the guidelines and examples that can be used to promote the design of tall buildings using such a structural system. This paper addresses this gap in the literature, as the main objective of this paper is to provide an example in the design and analysis of diagrid structural systems for seismic regions, by discussing and studying the sensitivity of solutions using tuned friction mass dampers to mitigate seismic demands on the structure.

In this study, a prototype 72-story building is developed. For the seismic design, the focus is placed on mitigating large displacements and shear forces that may appear in these structures. First, a system using a single tuned mass damper (TMD) unit placed at the top of the building is explored. The mass damper is connected to the structure with friction pendulum isolators, which are chosen due to their ability to undergo large deformations. A parametric study is carried out in order to optimize the mass damper design in terms of improving the seismic performance of the building structure. Second, the performance of a double tuned mass damper system is also investigated. In this system, an additional TMD unit is installed at mid-height of the building. In all, this paper provides a first example, which serves as guidance into design of diagrid structures in regions prone to seismic loading, including single or double tuned mass damper systems.

The building studied herein is designed for a location in downtown Seattle, Washington, USA and thus both shallow crustal earthquake motions as well as subduction zone earthquake motions are used in the analysis. Shallow crustal motions are typical of what is generated in intracontinental faults, while subduction zone motions are often generated in intercontinental faults [4]. Since a preliminary deaggregation of the seismic hazard for this site showed that the seismic hazard is contributed equally by crustal and subduction earthquakes, the same number of records (7+7) is used in this study. According to past research [5], the peak displacement demands obtained from using both types of motions are mostly identical, but the subduction zone motions induce a much larger number of inelastic excursions indicating that structures excited to these long-duration motions must be carefully designed to avoid low-cycle fatigue.

II. Solutions for Improving Seismic Performance of Diagrid Steel Structures

Two main systems have been proposed to date to improve the seismic performance of the diagrid system, including the use of base isolation or viscous dampers. In a first alternative, Arup (2009) [6] proposed a diagrid structure combined with a base isolation system as a method for reducing the potential for damage induced by earthquake shaking. In this solution, a 20-story office building was completed in 2006 in Sony City, Japan. The isolation solution was effective since the period of the base isolated building was shifted and the seismic lateral forces applied to the structure were substantially reduced. Base isolation typically adds 5% of the construction cost [7], even after considering the reduction in structural material in the superstructure. The extra cost arises due to the extra floor structure that needs to be constructed in between the building and the foundation in support of the base isolators. Furthermore, design of services and elevator shafts, passing through the isolation requires careful design for allowing for the lateral movement between the foundation and the isolated structure. In a second alternative, Lago et al. (2010) [8] proposed a vertically distributed isolation system. In this solution, the diagrid exterior structure was isolated from the main seismic mass of the building interior along the height of the structure. The distributed isolation was achieved by attaching viscoelastic dampers between floor diaphragms and horizontal rings of the diagrid structure. Lago et al. showed that this system has the potential to significantly reduce the damage to the architectural façades.

Even though the two systems described in the previous paragraph are unique and have several advantages for mitigating seismic demands, they are not suited for very tall building structures. The base isolation system is only effective for relatively stiff structures, since the period of the base isolated structures is typically set in the 2.0 sec to 3.0 sec range [9]. Tall building structures are typically very flexible and often have fundamental periods close to and above 5.0 sec and therefore the base isolation system is not effective. Following similar discussions, Lago et al. also stated that the vertically distributed isolation system is not effective for tall building structures. Based on numerical results, the authors showed that for a 20 story building the dampers had already experienced a stroke on the order of 0.8 m. Any form of extrapolation to the prototype 72-story building studied herein, would translate roughly to the need for dampers with approximately 4 m in length, which is beyond the scope of the proposed solutions.

Herein, a third alternative for mitigating seismic demands is proposed for use in tall buildings consisting of a diagrid structural system, which makes use of tuned mass dampers (TMDs) to mitigate lateral motion due to earthquake excitations at the base. Even though the particular system being proposed is new and the application of this new solution in diagrid systems has never been proposed, the concept of using TMD units have been applied in many skyscrapers built around the world. Examples are: (i) Taipei 101 in Taipei, Taiwan; (ii) One Wall Centre Hotel in Vancouver, Canada; and (iii) Shanghai World Financial Center in Shanghai, China. The TMD systems installed in these three buildings are all unique. Taipei 101 featured the heaviest TMD in the world with 660 metric-tons; the One Wall Centre fosters a tuned liquid (water) damping system; and the Shanghai World Financial Center holds a double TMD system. In these three building designs, the TMDs were installed at the top of the buildings and were shown to successfully mitigate the effects of the lateral loading. The TMD concept in this paper is somewhat similar to the used in the One Wall Centre Hotel [10]. Further explanation of the concept and its modeling details are provided in the following sections.

III. Tuned Mass Dampers and Implementation

Tuned mass dampers (TMDs) have been studied extensively by many researchers (e.g. Chopra 2001 [11], Inaudi and Kelly 1992 [12]). TMDs are placed in structures to improve their performance by providing counteracting (out-of-phase) forces that mitigate the vibration response of the original structure. Earlier studies included implementation of single mass damper units to mitigate wind-induced vibrations of building structures [13]. In the literature (e.g. Sadek et al. 1997 [14], Hadi and Arfiadi 1998 [15]), researchers have tuned the mass dampers by adjusting the stiffness and damping of the device or the mass of the TMD unit. In most cases in which TMDs have been used in buildings, these were placed near the top of buildings. The utilizations of multiple TMD units have also been discussed. To the authors's knowledge, the pioneering work by Xu and Igusa (1992) [16] proposed the first system with multiple damped oscillators and showed that multiple TMD units can be more effective than a single TMD with the same mass in mitigating vibrations induced motion (displacements). Chen and Wu (2001) [17] showed that multiple dampers are strictly necessary if the objective is to also reduce peak floor absolute accelerations of the building structure to impulsive (seismic) loading. Nonetheless, Lucchini et al. (2013) [18] concluded that the effectiveness of the TMD solutions consisting of two units is reduced if the uncertainty in the characteristics of the earthquake are considered.

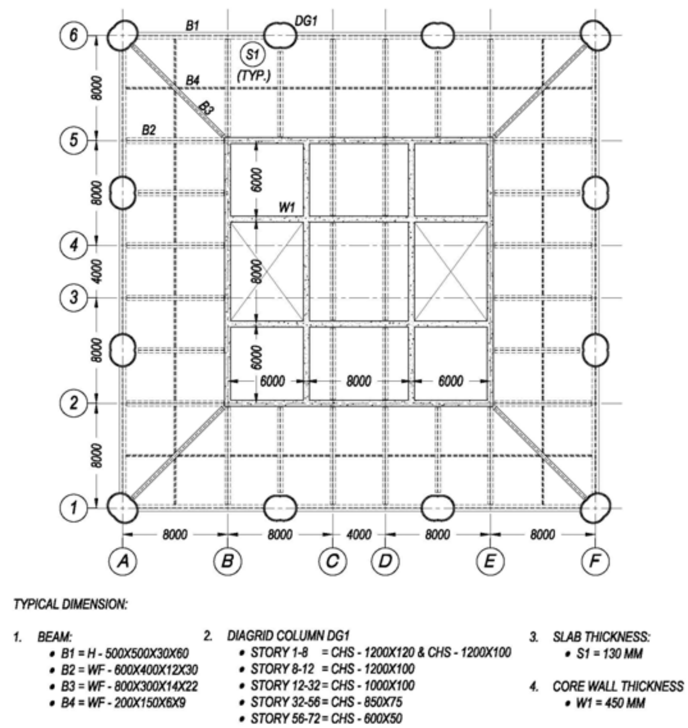
IV. Friction Pendulum Isolators

The TMDs solutions proposed in this study are supported on friction pendulum system (FPS) isolators [19]. The FPS isolators consist of a spherical sliding surface, which realize a pendulum system with a fundamental period that is related essentially to the length of the pendulum and radius of curvature of the pendulum. This is one of the unique characteristics of the FPS isolators, in which the fundamental period of vibration is essentially independent of mass. The dynamic response is strictly related to the friction characteristics of the sliding (curved) surfaces. In the interest of conciseness, the reader is directed to [19] for more details on FPS isolators, which are well-established for use in mitigating seismic demands in building and bridge structures. In this study, the computational model used to characterize the dynamic response of the FPS consists of a gap in the axial direction coupled the friction properties for two shear deformations with post-slip stiffness in the shear directions due to the radius of the sliding surfaces, and linear effective-stiffness properties in the torsional deformation. This friction model is based on the one proposed by Wen (1976) [20] and Park et al. (1986) [21]. The pendulum local axis of 1, 2, and 3 correspond to the global Z, X, and Y direction, respectively. More details on the modeling approach used are described by Ramadhan (2013) [22].

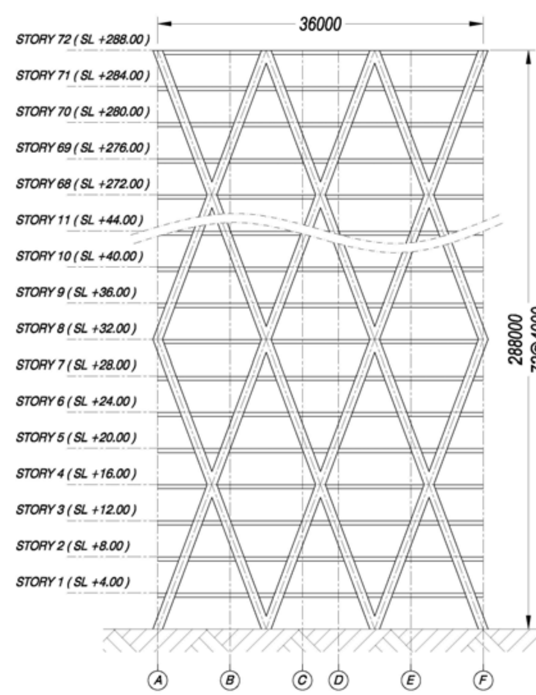
V. Methodology

A. Building Design

A 72-story prototype building with uniform floor height of 4 meters was designed following current US codes and standards for component design verification. This building is assumed to be located in Seattle, Washington, USA. The assumed latitude and longitude are 47°36'17.43"N and 122°19'51.88"W, respectively. As shown in Fig. 1, the building has a 36m×36m floor plan and floors are supported by diagonal columns that cross every four floors. With this configuration, the diagonal columns form isosceles triangles with an angle of 69°. This is the optimal configuration for slender diagrid structures greater than 60 stories according to empirical studies carried out by Moon (2008) [23]. All beams, except horizontal rings in floor diaphragms, are designed to carry gravity load only, and thus are designed and modeled as pin-connected at both ends.



(a)



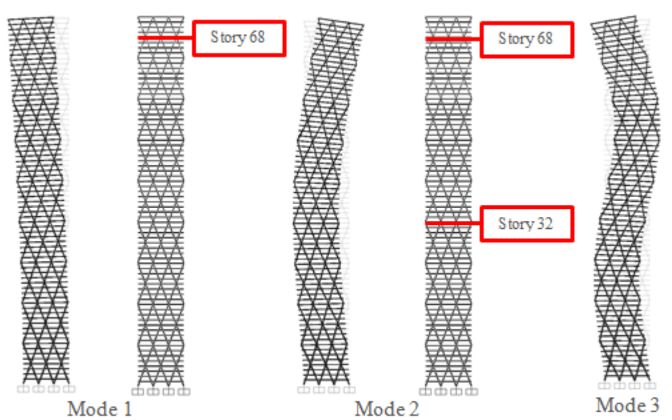
(b)

Fig. 1. Prototype building drawing: (a) plan view, (b) truncated elevation view; all dimensions are in mm.

A two staged design and design verification was performed: (1) In a first stage, the building was designed not considering the effects of the TMDs. Models of the building were developed and dimensions of all components of the building were first estimated using engineering judgment. Final dimensions were obtained through an iterative design process, in which the forces and displacements in the building were obtained using the response spectrum method for the prototype building (without the TMD units); (2) In a second stage of the design, the friction TMD system was incorporated to the model and design obtained in the previous model. In this second stage, nonlinear models were considered for the TMD units, and therefore the response spectrum method was not valid for use. Instead, nonlinear time-history response analysis was performed as described next in this methodology section. It is worth noting that the finite element models used are also described next.

B. TMD Unit Design

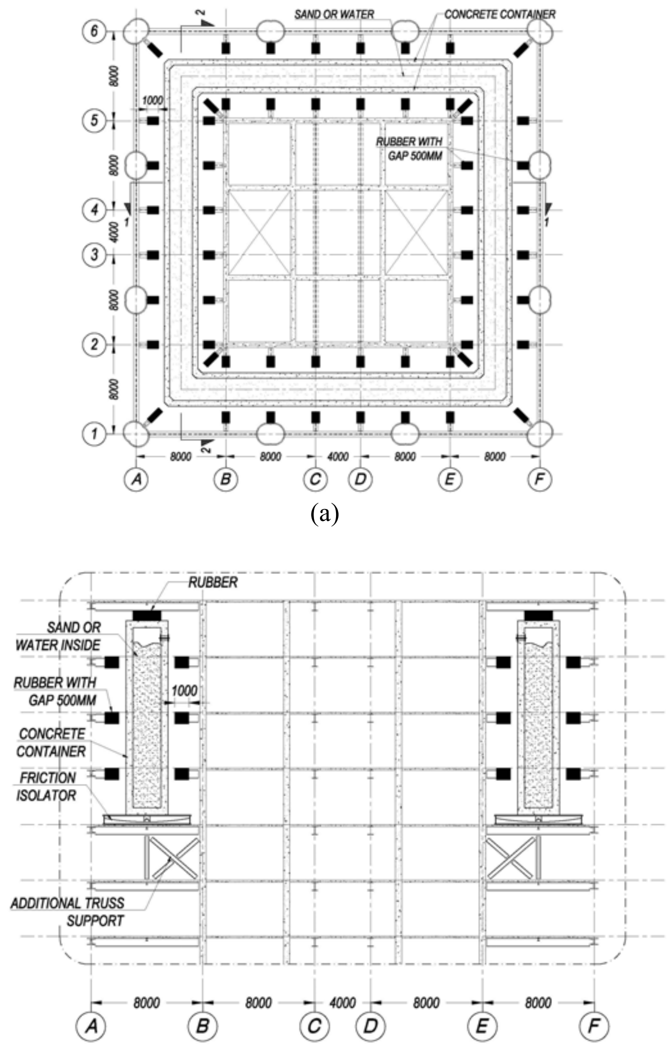
The TMD units consist of a concrete container with sand or water inside it, which is connected to the main structure using friction pendulum isolators. A similar concept to the one proposed herein can be found in One Wall Centre Hotel in Vancouver, Canada, which holds a tuned water damping system at the top level of the building [10]. The volume of sand or water can be adjusted according to optimal mass obtained from modeling and analysis. The reference model has the friction TMD unit placed at story 68 to alleviate the response from the first mode of the structure as shown in Fig. 2a.



(a) (b) (c)

Fig. 2. Mode shapes of the original structure and mass dampers placement: (a) Mode 1 and solution with only one TMD unit, (b) Mode 2 and solutions with two TMD units, (c) Mode 3.

The top friction TMD unit extends from story 68 to story 72 to provide improved load transfer from the mass damper to the stiffer floors, that is, floors at which the diagonal columns of the diagrid cross. It also serves as a room for the additional required mass that is provided by the sand or water. This TMD unit is illustrated in Fig. 3. For the model with two friction TMD units, the first unit is the same as the reference model, while the second is placed at story 32, extending to story 34. The second TMD unit is illustrated in Fig. 4. Implementing the second mass damper unit aims at reducing the contribution of the second lateral mode of vibration (see Fig. 2). Due to the usual shape of the seismic design response spectrum (as well as the shape of the response spectra of the ground motions considered) base shear forces due to the second mode (or even the third mode) have significant contributions to the floor accelerations and to the total base shear, as confirmed in the results section.

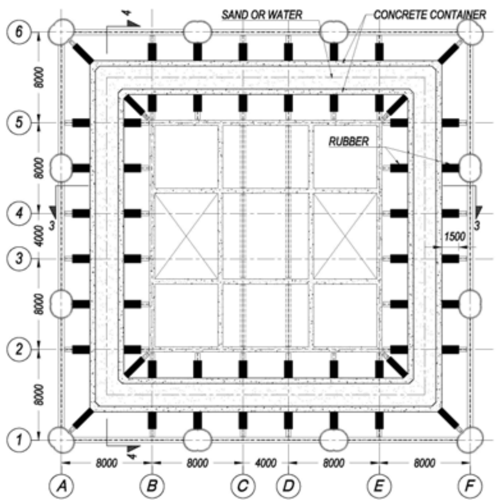


(a) (b)

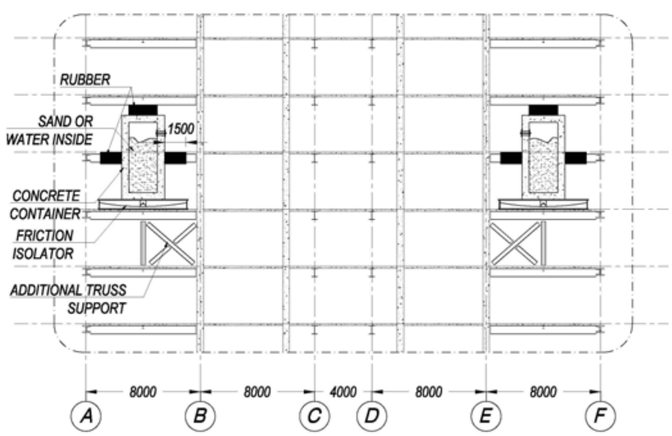
Fig. 3. First TMD unit: (a) plan view at story 69 to 71, (b) cross section 1-1. All dimensions are in mm.

Protective shock absorbers (rubber bearings or an equivalent system) are placed between the outer horizontal rings of the exterior diagrid and the TMD units. These bearings are also placed between the TMD units and the inner reinforced-concrete core. In the reference model, the TMD unit (at story 68) has absorbers – with thickness of 1 meter – placed in stories 69, 70, and 71. Initial gaps of 500 mm are provided, and absorbers are only engaged after the gaps are closed. For the model with double TMD units, the second TMD unit (at story 32) has absorbers – with thickness of 1.5

meters – placed at story 33. The second absorbers do not have gaps. This allows for tuning of the period of the second mass damper to be close to the second mode of the building. Absorbers are also placed above the container to prevent impact due to overturning. Lastly, additional stiff truss beams are provided at the floor below the TMD units, to transfer the vertical loads directly to the inner core. The design of the braces was done to ensure that the solution was possible.



(a)



(b)

Fig. 4. Second TMD unit: (a) plan view at story 33, (b) cross section 3-3. All dimensions are in mm.

C. Ground Motion Selection

The selection and scaling of horizontal component ground motion acceleration time histories are crucial to produce meaningful results and adequate comparisons of the responses of structures subjected to these ground motions. The selection performed herein has the primary objective of producing acceleration histories which are consistent with the seismic hazard at the site.

Seven (7) crustal earthquakes, listed in Table I, were chosen and scaled from the 2011 PEER Ground Motion Database. The target spectrum is based on ASCE 7-10 [24] with design earthquake spectral response acceleration parameters, S_{ds} = 0.911g, S_{d1} = 0.529g, and T_L = 6 sec. The target design response spectrum is shown in Fig. 5 and Fig. 6. The Pacific Earthquake Engineering Research Center (PEER) Center makes a ground motion database available and has a web tool linked to it for selection and scaling of acceleration time histories [25]. In the selection of the earthquake records, some parameters have to be input in the web tool. Readers are referred to the PEER manual for details on all the variables described next. The range of earthquake moment magnitudes is set to M_w = 6 to 7.25. $D_{5\text{-}95}$, which is the time duration for the intensity to rise from 5% to 95% is set to 0 to 300 seconds. Joyner-Boore distance (R_JB) and the rupture distance (Rrup) is set to 0 to 20.5 km. The range of average shear wave velocity in the top 30 m of soil (Vs30) is set to 190 to 350 m/s. The scale factor, for linear scaling of the ground motion records, is limited to 1/3 to 3.0. Lastly, the root-mean-squared errors (RMSE) between the ASCE7-10 target response spectra and the geometric mean of 5%-damped linear response spectra obtained for two orthogonal directions are used as a measure of goodness-of-fit. The RMSE are assessed in periods ranging between 1.0 second and 10 seconds, which covers a range below 0.2 and 1.5 of the fundamental period of the building structures analyzed herein. Fig. 5 shows the target spectrum as well as geometric mean of the selected response spectra of scaled acceleration time histories. The records selected for the crustal shallow motions are listed in Table I. It is worth noting that these records have the lowest root-mean-squared errors between the target response spectra and the response spectra of all ground motion records, and had the lowest usable frequency with a maximum value of 0.12 Hz (8.33 s).

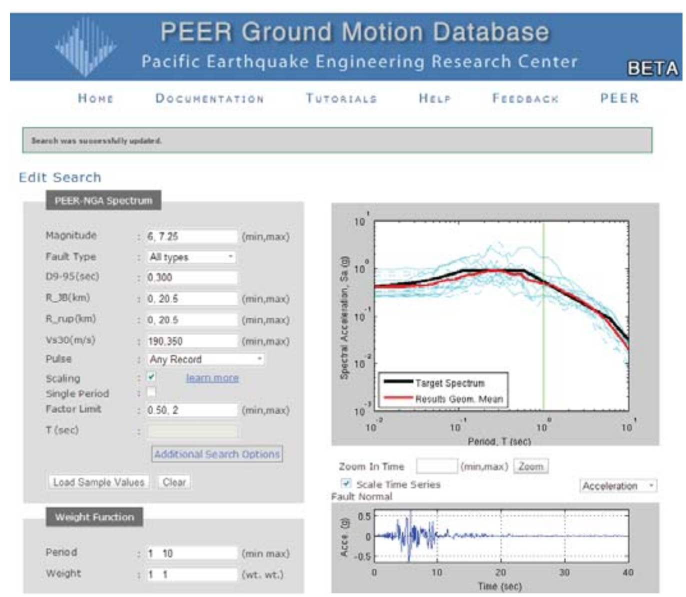


Fig. 5. Input parameter for selecting and scaling the earthquake time series in PEER ground motion database.

Seven (7) subductions earthquakes, shown in Table II, were chosen and scaled from Japan [26] using the method described in [27]. Those 7 earthquake records are obtained from the "2011 Tohoku Earthquake" of March 11, 2011. The response spectra for all selected earthquakes are shown in Fig. 6. It is worth noting that prior to this earthquake event, only very limited data was available for large magnitude earthquake motions, especially for the very large period range of interest of the building structure in analysis. Thus, all motions were selected from a single earthquake event, which may induce some bias in the results. From the database of Japanese ground motions used, there are a very small number of records that could be used for the analysis in this study due to the very large fundamental period of the tall structures. A minimum value for the lowest useable frequency was set to 0.12 Hz (highest useable period is T = 8.33 s), which allows for analyzing these tall building structures will a large degree of confidence. Often, two different components of the same earthquake need to be filtered using different corner frequencies. Thus, the records shown in Table II are applied to the two horizontal directions of the building, which was the basis for selection of the ground motions. Finally, it is worth noting that the records for the subduction earthquakes chosen herein have larger RMSE than the shallow crustal motions.

TABLE I
Properties of selected crustal earthquake records obtained from [25].

Event	Imperial Valley-06	Imperial Valley-06	Duzce-Turkey	Imperial Valley-06	Northridge-01	Imperial Valley-06	Northridge-01
Year	1979	1979	1999	1979	1994	1979	1994
Station	El Centro Array #8	Brawley Airport	Duzce	El Centro Array #10	Newhall - W Pico Canyon Rd.	Parachute Test Site	Newhall - Fire Sta
NGA#	183	161	1605	173	1045	187	1044
Scale Factor	1.194	1.986	0.743	1.479	0.906	2.000	0.911
RMSE	0.015	0.027	0.039	0.046	0.248	0.267	0.271
Pulse (FN/FP)	Yes/No	Yes/No	No/Yes	Yes/Yes	Yes/Yes	No/No	Yes/No
Tp (FN/FP)*	5.4 / --	4 / --	-- / 5.6	4.5 / 2	2.4 / 2.2	-- / --	2.2 / --
Magnitude	6.53	6.53	7.14	6.53	6.69	6.53	6.69
Mechanism	Strike-Slip	Strike-Slip	Strike-Slip	Strike-Slip	Reverse	Strike-Slip	Reverse
Rrup (km)	3.9	10.4	6.6	6.2	5.5	12.7	5.9
Corner Freq**	0.12	0.12	0.1	0.12	0.12	0.12	0.12

*TP is pulse period in seconds as defined by Baker (2007) [28].
*lowest usable frequency.

TABLE II
Properties of selected subduction zone earthquake records obtained from [26].

Event	Tohoku Earthquake	Tohoku Earthquake	Tohoku Earthquake	Tohoku Earthquake	Tohoku Earthquake	Tohoku Earthquake	Tohoku Earthquake
Date	March 11, 2011	March 11, 2011	March 11, 2011	March 11, 2011	March 11, 2011	March 11, 2011	March 11, 2011
Station	1103111446-AOMH16-EW-Surface	1103111446-FKSH10-EW-Surface	1103111446-FKSH16-NS-Surface	1103111446-FKSH20-EW-Surface	1103111446-IWTH11-NS-Surface	1103111446-IWTH24-NS-Surface	1103111446-TCGH16-NS-Surface
File Name	AOMH16EW2	FKSH10EW2	FKSH16NS2	FKSH20EW2	IWTH11NS2	IWTH24NS2	TCGH16NS2
Scale Factor	3.000	3.000	3.000	1.252	3.000	2.806	1.573
RMSE	0.606	0.520	0.667	0.823	0.778	0.173	0.356
Magnitude	9.000	9.000	9.000	9.000	9.000	9.000	9.000
Corner Freq	0.120	0.120	0.120	0.100	0.080	0.090	0.100

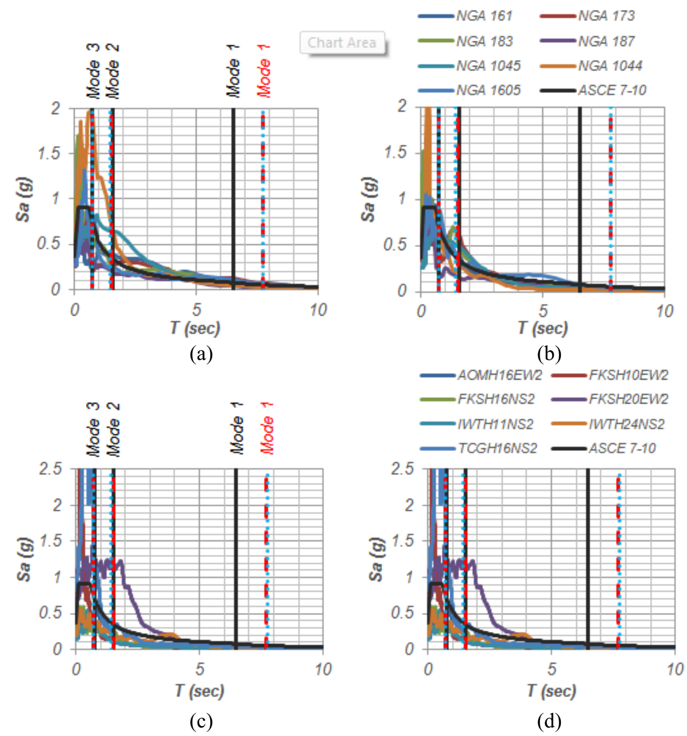
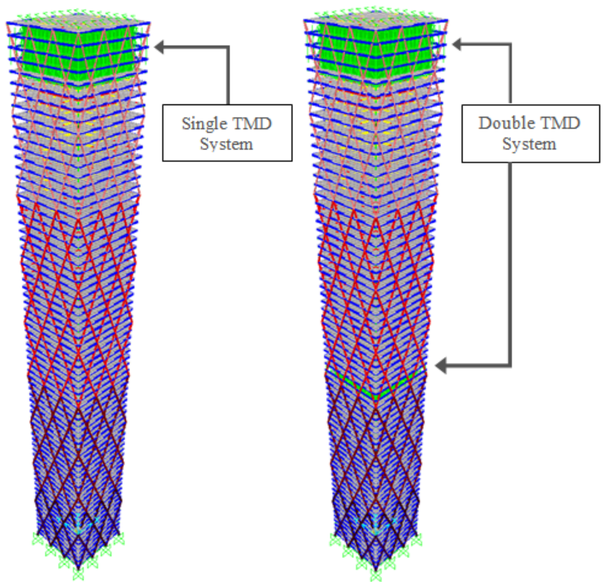


Fig. 6. Response spectrum of each tested ground motions: (a) crustal – fault normal applied in X-dir, (b) crustal – fault parallel applied in Y-dir, (c) subduction – applied in X-dir, (d) subduction – applied in Y-dir; straight line legend: (i) without TMD [–], (ii) reference model (with TMD) [- - -], (iii) with double TMD units [···].

D. Modeling

Structural modeling corresponds to the creation of a mathematical model that can be solved and analyzed. In this paper, SAP2000 [29] is the software used to analyze all building structural models. Sap2000 uses the finite element method to model all structural elements in the building. The diagrid structure is modeled using linear elastic frame elements with geometric and mechanical properties defined based on Fig. 1a. The inner reinforced concrete wall core is modeled using linear elastic shell elements. The effective Young's modulus of the walls is taken as 35% of the nominal concrete modulus E_c. Floor diaphragms consist of a grid of steel beams supporting a concrete slab, which are modeled as rigid diaphragms for lateral loading. The effective Young's modulus of the floor slabs are taken as 25% of the nominal concrete modulus in accordance to ACI 318-11 [30] section 10.10.4.1. Additional plan eccentricities in this regular prototype structure were not explicitly considered. The SAP2000 model of the prototype building consists of 8536 frame elements, 60275 joints, and 57624 shell elements. The total number of degrees of freedom in the model is exceeds 360,000.

For modeling purposes, the concrete container (with its content) is represented by a single wall, which is placed at mid-distance between the outer diagrid and the inner core. The single wall is modeled as linear elastic shell elements with thickness and mass defined as to represent the container and its content. It should be noted that sloshing effects of the water surface is neglected as it will not have a considerable effect in seismic loads and performance of the TMD. In the reference model, the ratio between the mass of the TMD unit and the mass of the main structure is approximately 4.7%. For the model that has two TMD units, the second TMD unit has approximately half the mass of the first.



(a) (b)

Friction Isolator Properties:
Linear analysis properties*:
- !"#$%! "#$%%&' ()) !
- !"#$&#'$(# "%*+&' ()) !
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Nonlinear analysis properties*:
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- !#$&#'$(! "$*$&' ()) !
- -./01 2+32#!"%*%4!
- -5.6 2+32#!"%*%7!
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* The positive local axis 1 is parallel to the positive global Z axis, the positive local 2 axis is parallel to the positive global X axis, and the positive local 3 axis is parallel to the positive global Y axis. k_e is effective stiffness and k is stiffness.

(c)

Fig. 7. Building model in SAP2000: (a) with single friction TMD unit, (b) with double friction TMD units, (c) friction isolator properties.

The material nonlinearity in this model was only explicitly considered in the links provided between the TMD and the main structure. A total of 392 links were placed in the model for each TMD unit. These nonlinear links simulate: (i) the friction pendulum isolator; (ii) the shock absorbers; and (iii) gap springs. First, in the definition of the links for the friction pendulum isolators, the model by Scheller and Constantinou (1999) [31] was chosen as the preferred model. Full details on the behavior of the FPS modeled using the friction isolator is available in the referred publication. The properties for all friction isolator links in the reference model are listed in Fig. 7. Second, linear springs were provided for the shock absorbers, with stiffness values of 1 kN/mm. Lastly, two types of gap links were provided – in parallel to each other – representing the shock absorbers. The first gap link corresponds to a gap of 500 mm followed by a linear elastic stiffness branch. The second gap link has a gap of 1500 mm, followed by a large stiffness to model contact between the TMD and exterior structure. The stiffness of the first gap links is 1 kN/mm with 500 mm opening (707.1 mm for diagonal links). The stiffness of second gap links is 2449 kN/mm (1725 kN/mm for diagonal links) with 1500 mm opening (2121.3 mm for diagonal links). The stiffness of the second gap link is proportional to the length of the beam member which the rubber is attached to. The behavior of absorbers for the second TMD unit are almost the same as the first, except the first gap links are replaced by uniaxial springs with stiffness values of 1 kN/mm.

E. Analysis Methodology

The nonlinear finite element analysis of the models is divided into three stages. In the first stage a linear static analysis of gravity loads and wind loads (for design verification only) are applied to the building. The second stage involves performing an eigen analysis to compute natural frequency, mode shapes, and mass participation ratios of the building models following a gravity load analysis. In the design verification of the prototype building (without TMDs), the third stage corresponds to performing a response spectrum analysis, using the appropriate target spectrum for analysis of mainly steel structures as the input. For models with TMD units, the third and final stage includes the nonlinear time-history analysis for computing the response of the building to the applied earthquake acceleration ground motions time-histories. Duration of analysis was at least the duration of the accelerograms and integration time step of 0.005 second was used for computing the building's responses to shallow crustal motions. On the other hand, the building's responses to subduction earthquake records are computed with analysis duration of 350 seconds and integration time step of 0.005 second. These time steps used herein provide sufficient accuracy in the displacement responses for the structure with the large fundamental periods, while the duration of the analysis is done to allow for the free-vibration period post-earthquake. Constant acceleration Newmark integration was used for numerical time integration, and Newton-Raphson was used to solve the nonlinear system of equations. Structural constant damping is set to be 2% for all modes.

F. Parametric Studies

There are 4 variables addressed in this research: friction coefficients of the TMD unit, height distribution of the TMD, mass of the TMD, and number of TMD units. In each model, 7 shallow crustal motions and 7 subduction zone motions are applied to the models. Each of the crustal motions has three components of acceleration time series and they are assigned randomly in either X- or Y- directions. Vertical components of motion were also used. The properties of each of earthquake record are listed at section V.C.

In all, the number of nonlinear time history performed for this parametric study (tornado type analysis) is 98 [14×(1+2+1+1+2)] analyses. The perturbations from reference models are:

(A) 3 levels of friction coefficient values for parametric studies are: (i) f_{slow} = 0.01, f_{fast} = 0.02; (ii) reference (f_{slow} = 0.04, f_{fast} = 0.06); (iii) f_{slow} = 0.08, f_{fast} = 0.12;

(B) 2 levels for height distribution of the TMD: (i) 4 stories (reference); (ii) 2 stories;

(C) Mass of the TMD (3 levels): (i) reference; (ii) increase by 20%; (iii) decrease by 20%;

(D) Number of TMD units (2 levels): (i) Single TMD system, (ii) Double TMD system.

VI. Results

A. Design Verification

There are two design aspects that are verified for the reference structure containing the TMD, which are (i) demand over capacity (D/C) ratios of the steel diagrid exterior members, and (ii) peak displacements of the TMD unit. For the member design check, the 7 crustal ground motions are averaged and incorporated in the design combinations following ASCE 7-10. The design check was performed based on AISC 360-05 [32]. A peak D/C ratio of 0.90 was obtained over all diagrid members. The TMD has limited movement. For this design, the displacement limit is ±1.5m. From the reference design the absorbers are proven to safely limit the TMD movement.

B. Comparison with Reference Model

TABLE III
Natural period and mass participation ratios of the main structure: (a) without TMD and (b) reference model (with one TMD unit)

(a) Without TMD

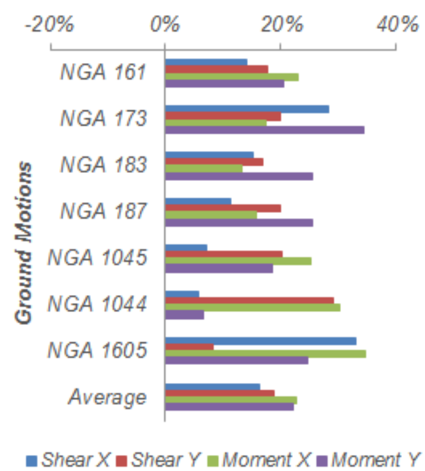
Mode	Period-X	Period-Y	UX	UY	RX	RY
1	6.487	6.546	0.600	0.600	0.970	0.970
2	1.546	1.549	0.210	0.210	0.028	0.027
3	0.740	0.732	0.069	0.070	0.003	0.003

(b) Reference model (with TMD)

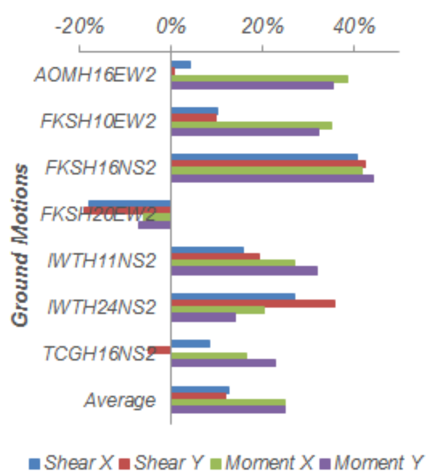
Mode	Period-X	Period-Y	UX	UY	RX	RY
1	7.718	7.765	0.424	0.429	0.760	0.753
2	1.506	1.509	0.196	0.195	0.023	0.023
3	0.729	0.723	0.065	0.066	0.003	0.002

The improvements in terms of seismic response from the prototype building model to the reference model with one TMD unit can first be examined from the comparison of modal parameters and mass participation factors shown in Table III. By placing the TMD at the top of the building, the mass participation ratios of the first mode shape in the X- and Y-direction are decreased by 29.3% and 28.5%, respectively. From this preliminary observation, it is reasonable to expect a significant (about 30%) reduction in the base shear for earthquake ground motions that excite mainly the first mode of the building. Minimal seismic performance improvements are expected for any records that mainly excite the building at the higher modes.

Fig. 8a shows that the utilization of a TMD system provides improvements in the peak base shear response to all shallow crustal motions, averaging (over both directions) 17.6% in reduction. Significant improvements can be seen for a few earthquake motions, with reductions of approximately 30% in base shears and base overturning moments. These improvements are related to the fact that these ground motion records excite mainly the first mode, which is the mode to which the TMD is tuned to. On the other hand, poor improvements can be seen at the base shear for NGA 1045 FN (X-dir), NGA 1044 FN (X-dir), and NGA 1605 FP (Y-dir), since those earthquakes have large contributions to the building response from higher modes.



(a)



(b)

Fig. 8. Improvements peak base reactions from basic diagrid building: (a) shallow crustal earthquakes, (b) subduction zone earthquakes.

Observing the result of subduction earthquakes, the average (over both directions) improvement of peak base shears is 12.2%. This percentage is lower than the one obtained for the shallow crustal because the setting of the TMD system is not suitable for one of the earthquake tested, that is the FKSH20EW2. However, significant improvements can still be seen at some of the earthquakes, with the reductions in maximum base shears of 40%.

Fig. 9 shows the geometric mean of peak inter-story drift, peak floor displacements, and peak absolute floor acceleration X- and Y-direction responses. It is worth noting that even though the building model (steel diagrid and reinforced concrete core) is linear, the displacements obtained from these analyses are expected to be identical to the ones that would be obtained using a nonlinear building model because the period of the building is relatively large and the "equal displacement" rule applies [11]. As seen in Fig. 9, the reference structure (with TMD) reduces the floor displacements (on average over all floors) by 19.8% for crustal earthquakes and 22.5% for subduction earthquakes. It also provides inter-story drift improvements of 17.5% for crustal earthquakes and 21.8% for subduction earthquakes. However, this TMD system is not as effective to reduce the floor accelerations as it only provides average reductions of 8.1% for crustal earthquakes and less than 1% for subduction earthquakes. This is because floor accelerations are usually controlled by higher modes which are not affected by this TMD system.

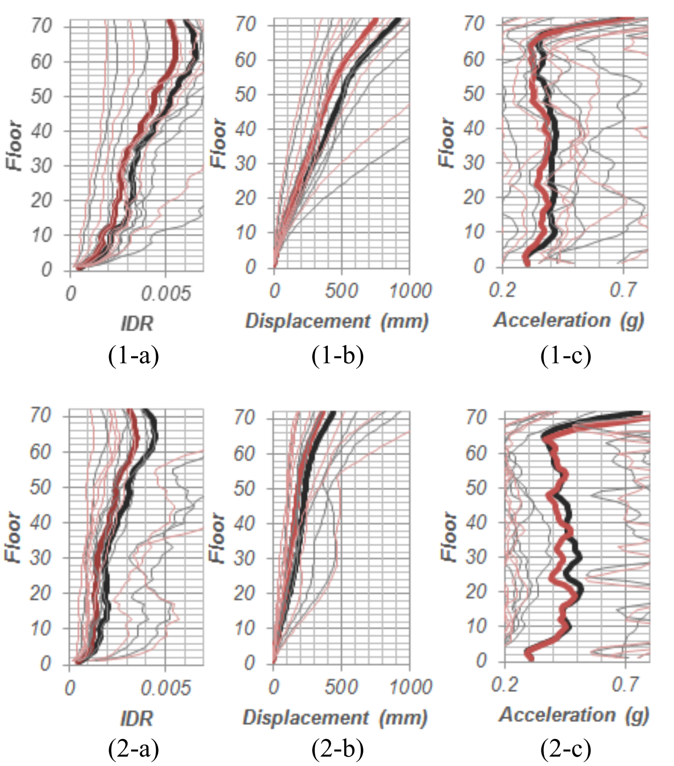


Fig. 9. Geometric means of envelope responses due to (1) crustal earthquakes and (2) subduction earthquakes for: (a) inter-story drift (IDR), (b) floor displacement, (c) absolute floor acceleration; thin lines are the responses of each model (grouped by colors) due to individual earthquake; legend: (i) **black line** is without TMD; (ii) **red line** is with TMD**.**

C. *Variation of FPS Friction Coefficient*

The parameters used in the reference structure with the TMD unit are f_{slow} = 0.04 and f_{fast} = 0.06. To study the effect of changing the friction coefficients of the friction isolators two new levels of friction coefficients are introduced: low friction (f_{slow} = 0.01, f_{fast} = 0.02); and high friction (f_{slow} = 0.08, f_{fast} = 0.12). The friction coefficient of f_{fast} = 0.12 can be produced by the friction of two lubricated hard steel materials [33].

From observation of the base shears in Fig. 10, it can be seen that the structure performance is improved when friction is increased. This conclusion had also been reached elsewhere [34]. In shallow crustal motions, improvements can be seen at almost all tested ground motions. Significant reductions in base shear can be seen in NGA 161 FP (Y-dir) and NGA 173 FP (Y-dir) in which additional reductions of approximately 13% from the reference model. However, increasing the friction does not significantly affect the overall improvements in peak base shear due to subduction zone earthquakes. Nevertheless, maximum additional reduction of 14% in base shear is still observed in subduction zone motion's response. On the other hand, lowering the friction coefficients results in smaller friction forces than those required to counteract the seismic forces and therefore, the results in the observed response are worse.

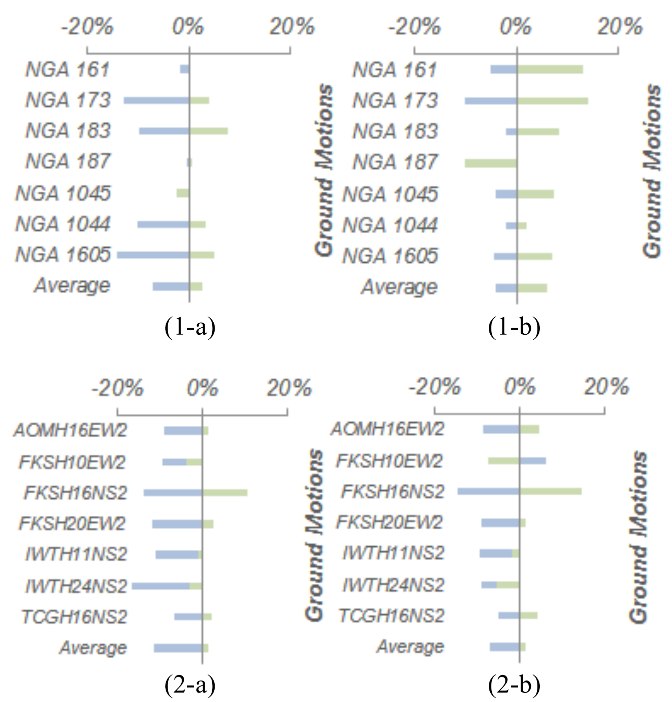


Fig. 10. Tornado plots for peak base reactions due to variations of the friction coefficient about the reference model: (1) crustal shallow motions and (2) subduction zone motions; (a) base shear X, (b) base shear Y; legend: (i) **blue bar** is low friction, (ii) **green bar** is high friction.

It is worth noting that the envelopes of inter-story drift, floor displacement, and peak absolute floor acceleration due to shallow crustal motions are not sensitive to changes in friction coefficient, as can be seen in Fig. 11. Increasing friction introduces negligible changes in those floor responses. Also, by reducing the friction, the observed floor responses due to crustal shallow motions only increase by approximately 3%. However, for subduction-zone motions, the floor displacements are still not sensitive to changes in the friction coefficient, but the inter-story drift ratios and absolute floor acceleration are quite sensitive. High friction provides significant absolute floor acceleration reductions of 10.5% while low friction increases the inter-story drift by 7.9%.

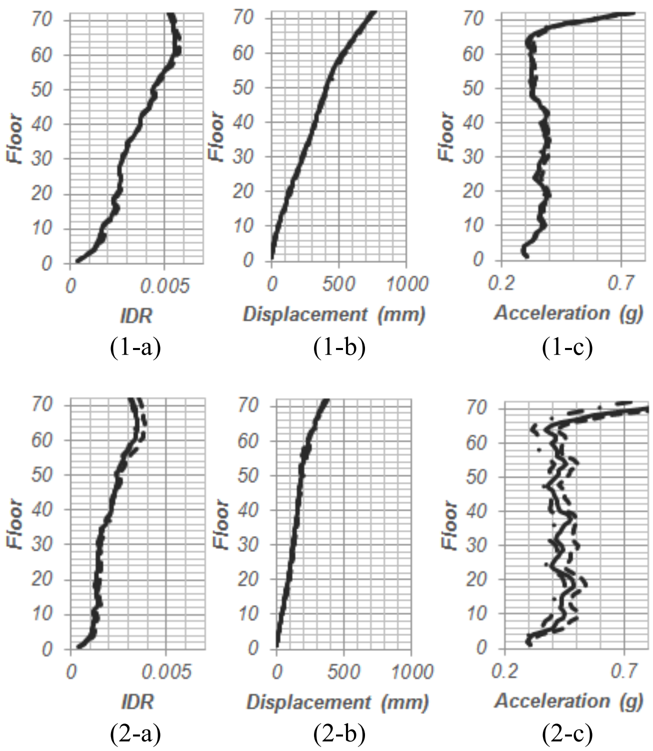


Fig. 11. Floor responses due to variations in friction coefficient: (1) shallow crustal motions and (2) subduction zone motions; (a) inter-story drift, (b) floor displacement, (c) floor absolute acceleration; legend: (i) low friction [- - -], (ii) reference [–], (iii) high friction [! !!!!].

D. Variation in Height Distribution and Mass of TMD Unit

As stated in the methodology section, the configuration for the TMD system needs the mass damper to be extended for four floors for optimal load transfer to the exterior diagrid structure. Fig. 12 illustrates the sensitivity of base reactions change in height and mass of the TMD unit. It can be seen from this figure that concentration of the same mass over half the height results in an increase in base shear forces (average of both directions) of approximately 3.8% for shallow crustal motions and a decrease of 1.6% in subduction zone motions compared to the reference model. No significant changes are observed in displacement and acceleration floor responses due to crustal earthquakes compared to the reference model (with one TMD unit) as ! 3% differences were observed in Fig. 13. In subduction zone earthquake's responses, the floor drift and displacement also show insignificant changes, but the absolute floor acceleration decrease by 6.0% from the reference model.

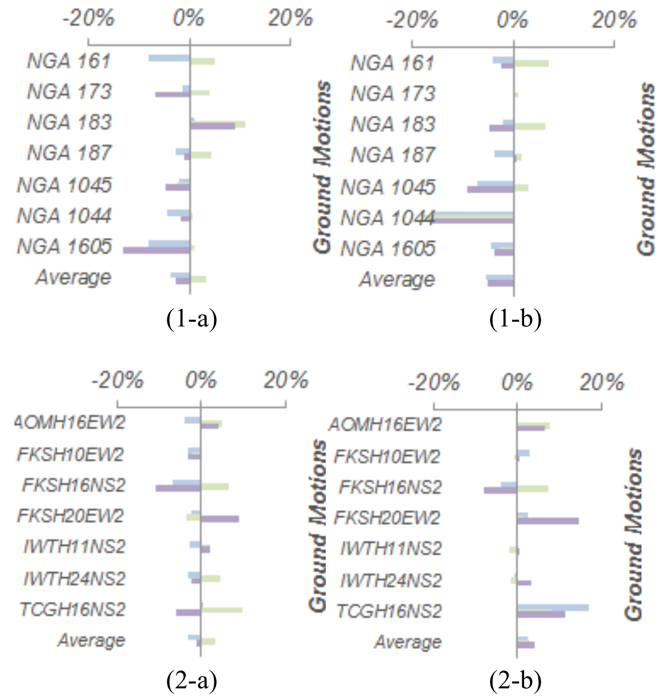


Fig. 12. Tornado plots for peak base reactions due to variation in height and mass of TMD about the reference model: (1) shallow crustal motions and (2) subduction zone motions; (a) base shear X, (b) base shear Y; legend: (i) **blue bar** is decrease mass, (ii) **green bar** is increase mass, (iii) **purple bar** is reduce height.

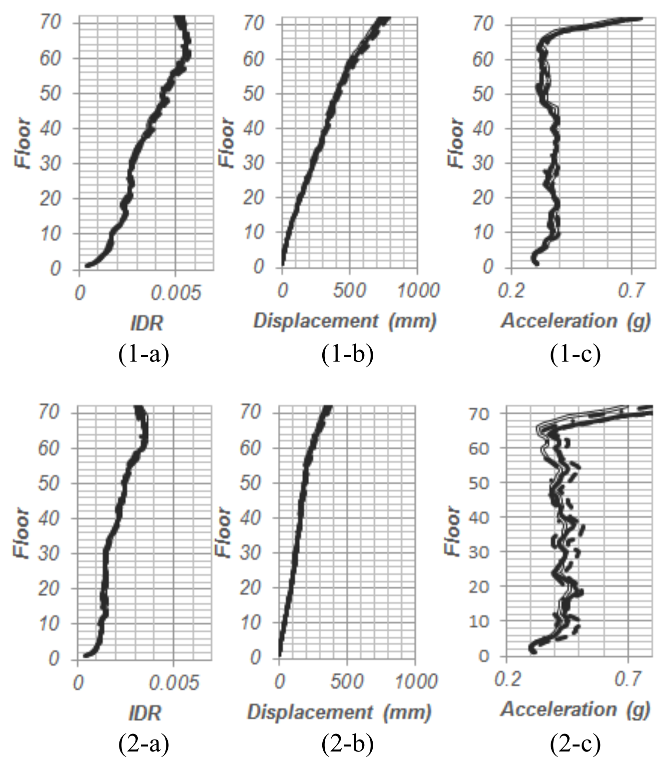


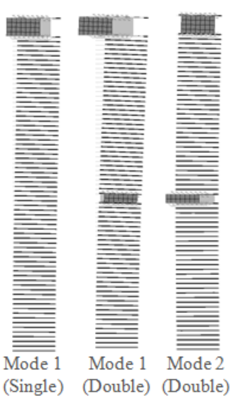
Fig. 13. Floor response due to variation in height and mass: (1) shallow crustal motions and (2) subduction zone motions; (a) inter-story drift, (b) floor displacement, (c) floor absolute acceleration; legend: (i) decrease mass [- - -], (ii) reference [–], (iii) increase mass [! !!!!]; reduce height [=].

Finally, Fig. 12 also shows the results obtained by maintaining the height and friction parameters fixed to the values of the reference model as the mass was varied by two levels (+20% and -20%). By reducing the mass by 20%, the overall base shears improvements for X and Y directions due to shallow crustal motions decrease by 3 and 5%, respectively, compared to the reference model. On the other hand, the average base reaction changes due to subduction zone motions are still irregular. Nonetheless, decreasing the mass of the TMD is still decreasing the overall base reactions improvements compared to the reference model. Additionally, increasing the mass by 20% corresponds to overall base shear improvements of only 3% from the reference for shallow crustal motions and 4% for subduction zone motions. As seen from Fig. 13, the changes in mass are also not affecting the displacement and acceleration floor response for shallow crustal motions. The observed floor responses only fluctuated by approximately 3.3%. However, the changes are reasonably significant in subduction earthquakes. While floor displacements do not differ much, increasing the mass by 20% increase the overall floor absolute accelerations improvement by 7.6% while decreasing the mass will do the contrary.

E. Comparison with Double TMD System

TABLE IV
Fundamental periods and mass participation ratios of the main structure with double TMD system.

Mode	Period-X	Period-Y	UX	UY	RX	RY
1	7.727	7.775	0.427	0.431	0.761	0.754
2	1.433	1.435	0.142	0.140	0.017	0.017
3	0.728	0.722	0.065	0.066	0.003	0.002



Single TMD

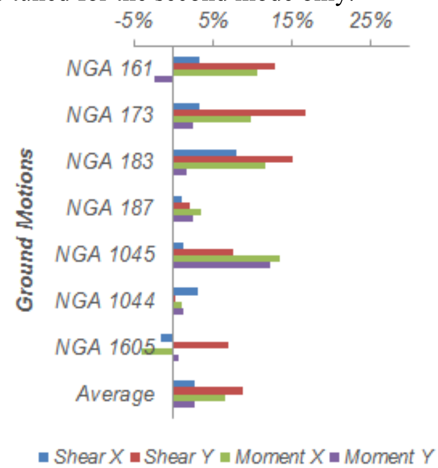
Mode	Period-X	Period-Y	UX	UY	RX	RY
1	4.982	4.996	0.197	0.191	0.213	0.221

Double TMD

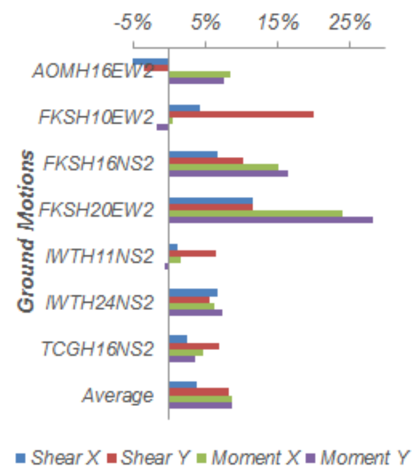
Mode	Period-X	Period-Y	UX	UY	RX	RY
1	4.994	5.008	0.198	0.192	0.212	0.219
2	1.843	1.844	0.054	0.054	0.006	0.006

Fig. 14. Mode shapes and mass participation ratios of each mass damper in the two models.

The modal parameters and mass participation factors are still useful to estimate the expected improvements in terms of seismic response of the model with the double TMD system. TABLE IV lists the three main periods of vibration of the structure in the X- and Y-direction. Fig. 14 shows the mode shapes and mass participation ratios of each mass damper in the double TMD system in comparison the single TMD system (reference model). It can be seen that for the model with the additional TMD in story 32, the mass participation of the first mode increases slightly, while a decrease of the mass participation ratio for the second mode is observed. It is clear, that the inclusion of the second TMD unit influences mainly the participation of the second mode, and thus this TMD unit is said to be tuned for the second mode only.



(a)



(b)

Fig. 15. Improvements in peak base reactions of double TMD system from reference model (single TMD system) due to: (a) shallow crustal motions, (b) subduction zone motions.

Fig. 15 shows that the utilization of double TMD system results in improvements in the peak base shear responses for almost all tested ground motions. For the shallow crustal motions, the double TMD system gives additional improvements of 5.7% to peak base shear responses (averaging in both directions) over the reference model and also resulting in a reduction of 23.3% of total peak base shear reduction. Significant improvements are observed for NGA 161 FP, NGA 173 FP, and NGA 183 FP with approximately 15% of additional improvements in peak base shear over the reference model. In the response generated by subduction zone motions, a similar trend is observed. The double TMD system reduce the overall peak base shear responses by 6.0% with maximum reduction of approximately 20%. However, for one particular earthquake, that is FKSH20EW2, although the double TMD system reduces the peak base shear responses by 11.6% over the reference model, it still resulted in 7% higher peak base shear compared to the prototype building that has no TMD installed.

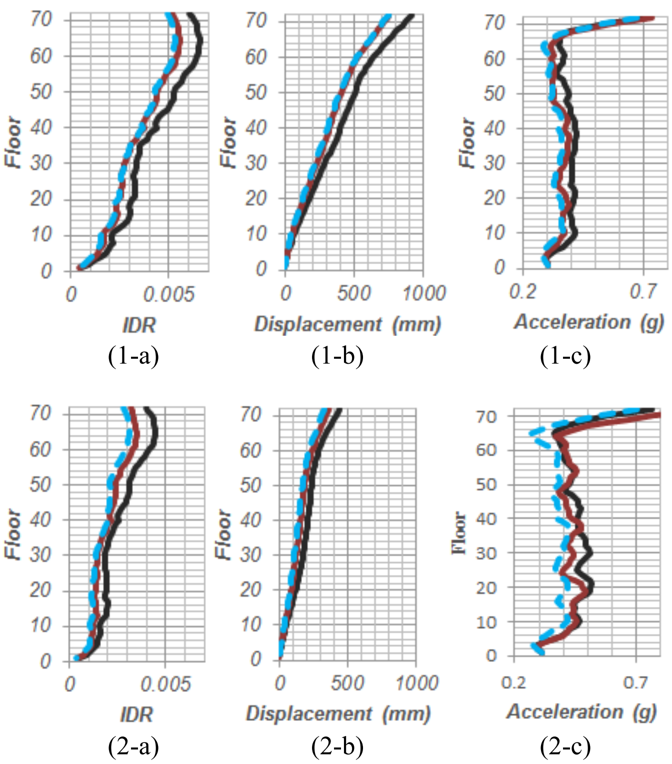


Fig. 16. Geometric means of envelope responses due to (1) crustal earthquakes and (2) subduction earthquakes for: (a) inter-story drift (IDR), (b) floor displacement, (c) absolute floor acceleration; legend: (i) **black line** is model without TMD; (ii) **red line** is model with single TMD system (reference model)**;** (iii) **blue dashed line** is model with double TMD system.

Fig. 16 shows the geometric mean of peak story-drifts, peak floor displacements, and peak absolute floor acceleration responses. All responses show that the double TMD system has the smallest responses. In responses due to shallow crustal motions, the floor drifts and displacements improve by 4.3% from the reference model, while the floor absolute accelerations provide improvements of 5.1%. For the subduction zone motions, the double TMD system reduces the inter-story drift and displacement responses of reference model by 9.5%. Furthermore, floor absolute accelerations are also decrease by 13.1%, which is a notable improvement from the reference model.

VII. Conclusions

In this paper, a structural system known as diagrid structural system is used in the design of a prototype 72-story building. This building is assumed to be located in Seattle, Washington, in the United States, which is a region of moderate-to-high seismicity that is exposed to both shallow crustal faults and the Cascadia Subduction Zone earthquakes, both of which can produce significant and intense ground shaking. The building solution is assessed with and without the use of tuned mass dampers (TMD) supported on friction pendulum system (FPS) isolators. The main conclusions of the paper are:

1. The buildings analyzed were subjected to an equal number of crustal and subduction motions (in total 14 motions). An equal number of crustal and subduction zone motions were selected since this approximated well the deaggregation of the seismic hazard at the site. Results presented were separated by types of earthquake motion to allow for a clear distinction between results obtained for both types of motion. Overall, from analysis of peak deformation and peak force response parameters, trends for both types of earthquake motions are identical. Results indicate that the effect of duration of the motions does not play an important role in the analysis. However, it should be noted that duration of the motion may be important in the analysis of shear and fatigue critical steel connections as the ones present in the diagonal elements, although such assessment was beyond the scope of this paper.
2. The design using one TMD unit was shown to improve the behavior of the prototype structure by reducing the base shear forces and overturning moments by as much as 20% on average and more than 30% for some motions. Peak interstory drifts and peak displacements were also reduced significantly. However, the peak floor acceleration responses were only modestly affected and can be said to have remained essentially unchanged.
3. Due to the configuration of the diagrid structural system, and since the levels where the diagonal elements cross were designed to be stronger and stiffer every four floors where the diagonal elements crossed, the TMD system was most effective if the mass damper is extended over four floors. This allows for optimal load transfer to the exterior diagrid structure.
4. Inclusion of a second TMD unit showed small improvements in the displacement and force responses when compared to the model using one TMD unit only. However, reductions in accelerations were observed when the second TMD unit was added.

The parametric study performed on the model with one TMD unit showed the following main points:

1. As the friction coefficients in the FPS are increased, the forces and accelerations in the building are reduced. These results seem to indicate that the most effective system is the one with larger friction. However, in most earthquakes, the changes in the displacements and inter-story drifts were negligible, mainly for subduction zone motions. Thus, these results seem to be competing in terms of the efficiency of the added friction.
2. Changes in the mass of the TMD unit by 20% did not correspond to significant changes in the response. This supports the conclusion that the response of TMD units supported on FPS are mainly sensitive to the friction parameters and are not very sensitive to the mass.

In summary, this document presented a design example and a TMD solution which can be incorporated into the design of a tall building using diagrid steel structures.

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