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DESIGN CONSIDERATIONS FOR A BASE ISOLATED STRUCTURE WITH TRIPLE-FRICTION-PENDULUM ISOLATORS: ISTANBUL SABİHA GÖKÇEN INTERNATIONAL AIRPORT TERMINAL BUILDING

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ABSTRACT

This paper reviews the key design considerations for the performance-based seismic design of the Sabiha Gökçen International Airport (SGIA) Terminal Building in Istanbul, Turkey that utilizes seismic isolation with triple-friction-pendulum devices. Currently, SGIA Terminal Building is the largest seismically isolated structure in the world with an area over 200,000 square meters and 296 seismic isolators. The performance objectives are established as Operational Level for a Design Basis Earthquake and Immediate Occupancy for a Maximum Considered Earthquake. A site-specific seismic hazard study is conducted to derive the design response spectra and spectrally matched time-history pairs. Various nonlinear triple-friction-pendulum models are investigated, and a parallel discrete spring model is used in the analysis and design. An equivalent lateral force procedure is used for the estimation of the total base shear and maximum isolator displacements. Response spectrum method is used for code compliance and performance based design. Time-history analyses are conducted for stability checks and verification. It is observed that selected isolator model is efficient in capturing the nonlinear behavior of the isolator, and different analysis procedures give similar results verifying the overall design process. The results show that, the isolated building met and surpassed the performance objectives while achieving significant reduction in the base shear, story drifts and floor accelerations.

Introduction

Sabiha Gökçen International Airport (SGIA) is one of the two major airports serving Istanbul with an annual passenger capacity of 5 million. In 2006, Turkish government decided to increase the capacity of SGIA to 15 million passengers through addition of a new international terminal due to the unexpectedly increased number passengers traveled since it has been opened in 2001. The construction of the new terminal was started in 2008, and the building was opened in October 31 2009.

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It is very well known that Istanbul is a seismically active region of Turkey and major earthquakes occur periodically. As for any other crucial infrastructure, uninterrupted operation of SGIA is essential after a major earthquake, and therefore, the highest seismic performance levels are established for the performance-based design of the Terminal Building as follows:

1. Operational Level for the Design Basis Earthquake (DBE),
2. Immediate Occupancy Level for the Maximum Considered Earthquake (MCE).

In general, above performance objectives are very stringent, and a fixed-based structure that meets these criteria would be very expensive and infeasible. Therefore, a base isolation system is used for the seismic protection of the Terminal Building. Recently introduced triple-friction-pendulum (TFP) device by Earthquake Protection Systems (EPS) is selected as the isolator after comparing the cost and performance of various types of bearings proposed for the Terminal Building.

The new SGIA Terminal Building is a steel structure with a rectangular plan 160 meters by 272 meters. The total building height is approximately 32.5 meters. The building consists of 4 stories above and a basement floor below the isolation plane, where the isolators rest on cantilevered reinforced concrete columns. Typical floor heights are 6 meters at the ground floor and 5 meters at the upper levels. The SGIA terminal is the world's largest base-isolated structure with a total floor area more than 200,000 square meters and 296 seismic isolators (initial design concept was 160,000 square meters and 252 isolators). The gravity system of the superstructure is composed composite floor system and composite steel columns, and the lateral system is steel moment frames in both major directions. The Terminal Building is modeled and analyzed in SAP2000 structural analysis program. Currently, the Turkish seismic code for buildings, TEC 98/07 has neither a guideline for performance-based design of structures nor requirements for analysis and design of seismically isolated buildings. Therefore, ASCE 7-05 was selected for the basis of performance-based design of the base isolated Terminal Building.

This paper reviews these key design components and provides the results of and discussion on the performance-based design of the SGIA Terminal Building. A brief review of the seismic hazard study is given. Then, the isolator design and modeling is explained in detail where various nonlinear isolator models are compared. The analysis procedures are explained and the tabular and graphical results are presented. Finally, the performance verification is explained.

Site-Specific Probabilistic Seismic Hazard Study

A site-specific probabilistic seismic study is conducted by Erdik *et al*. (2008) to obtain the seismic hazard for the DBE and MCE events (Figure 1). Next Generation Attenuation relationships are used and directivity effects are included in the study. As an example, the following parameters are used for the estimation of MCE level response spectrum, which further increased by an estimated %20 for longer periods to account the directivity effects: M = 7.5, R = 20 km, V_s = 500 km/s, strike-slip fault, vertically dipping fault plane, random horizontal component, ε = 2.1 and 2% exceedance probability. For the time-history analysis seven pairs of ground motion records from three earthquakes are selected, where the selected earthquake locations have seismological characteristics similar to the SGIA site. These are 1992 Landers (M = 7.3), 1999 Kocaeli (M = 7.4) and 1979 Imperial Valley (Mw = 6.5) earthquakes. Selected ground motion data are spectrally matched to DBE and MCE events to obtain the DBE and MCE level ground acceleration time-histories as explained in Chapter 21 of ASCE 7-05.

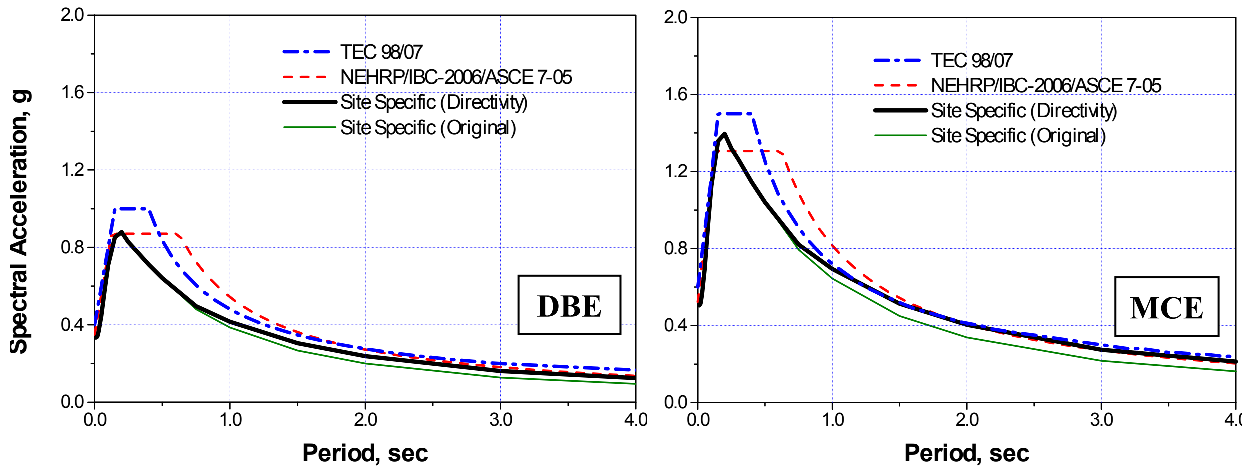
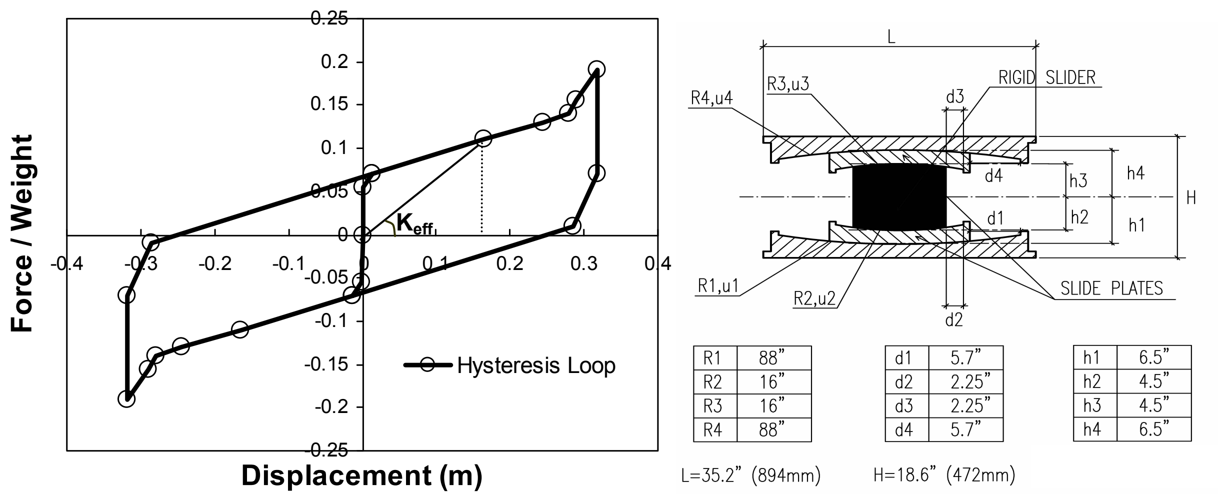


Figure 1. Code specified vs. Site Specific Response Spectra curves for 5% damping

Isolator Bearing Design

In general, isolator design is an iterative procedure, where the structural performance determines the isolator parameters, which in turn affect the overall structural performance. The isolator design requires estimation of three parameters: target isolated building period, T_{eff}, axial load on the isolators W, and the level of the earthquake excitation for DBE and MCE hazards. In practice, it is recommended that $T_{\mathrm{eff}} \geq 3 \cdot T_0$, where T_0 is the fixed base period. Based on this information, the manufacturer designs the isolator and provides the following information to the engineer: Geometry of the isolators (R_i, d_i, h_i, H, L), effective damping for DBE and MCE (β_{eff}), friction coefficients (u_1, u_2, u_3, u_4), effective stiffness (K_{eff}), isolator displacement capacity and hysteretic model curve (Figure 2).



(a) (b)

Figure 2. (a) Idealized nonlinear hysteresis curve for the SGIA TFP isolators, (b) geometry of the SGIA TFP isolators (Zayas *et al*. 2008 and accompanying reports)

Table 1. Upper and lower bound friction, stiffness and damping properties

	DBE		MCE	
	Lower	Upper	Lower	Upper
u_1	0.060	0.075	0.065	0.080
u_2	0.054	0.068	0.059	0.072
u_3	0.054	0.068	0.059	0.072
u_4	0.060	0.075	0.065	0.080
K_{eff} (kN/m)	2,900	3,630	2,503	2,871
β_{eff} (%)	38%	38%	30%	30%

The superstructure of the Terminal Building has a fixed base period of 0.8 seconds. Average vertical load on isolators is approximately 5350 kN. The TFP bearing (by EPS), with a theoretical period of 3 seconds and displacement capacity of 345 mm, is selected on the basis of performance and cost. The effective damping provided by the isolators is 38% and 30% at DBE and MCE events, respectively. An idealized nonlinear shear-displacement curve obtained from the tests is used in the modeling of TFP isolators (Figure 2-a). Geometry of the proposed bearings is shown in Figure 2-b. Further, there are uncertainties in the isolator properties due to aging and contamination effects and variation in the production tests. Therefore, two sets of isolator properties, lower and upper, are provided by the EPS. These properties for the SGIA TFP devices are shown in Table 1 and taken into consideration in the analysis and design.

Nonlinear Hysteretic Modeling of TFP Isolators

In the linear procedures (equivalent lateral and response spectrum methods), effective stiffness and damping are the only isolator parameter used in the analysis. For nonlinear time-history analysis, a Parallel Discrete Spring Model (PDSM) is developed (Figure 3). PDSM has three types of nonlinear elements that are connected in parallel and explained as follows:

1. Hysteretic Element: This element simulates the lateral stiffness and energy dissipation of the isolators. In SAP2000, this element is further modeled as two nonlinear springs that are connected parallel, which is explained in detail in the following.
2. Gap Element: This element simulates the vertical stiffness (compression) and resistance to the uplift (no resistance) by the isolators.
3. Hook Element: This element simulates the boundary conditions (stopper) under the ultimate horizontal displacement limits of isolators.

The hysteretic element in PDSM cannot be adequately captured by SAP2000. Also, user-defined multi-linear curves cannot be used since this approach will ignore the bi-directional interaction (softening) effects of the isolators. Various hysteric models that are constructed from available SAP2000 link elements are investigated as shown in Figure 4. The first two bearing are recently developed by and uses multiple friction pendulum (FP) elements that are connected parallel and serial (Sarlis *et al*. 2009). The third model implements two rubber bearing elements that are connected parallel. The fourth model uses multi-linear plastic element, and the fifth and sixth models use friction bearing and rubber bearing elements with modified parameters so to reflect the TFP bearing behavior as efficient as possible. Clearly the first three models provide provides an accurate estimate of the nonlinear behavior of the TFP bearings from a design perspective.

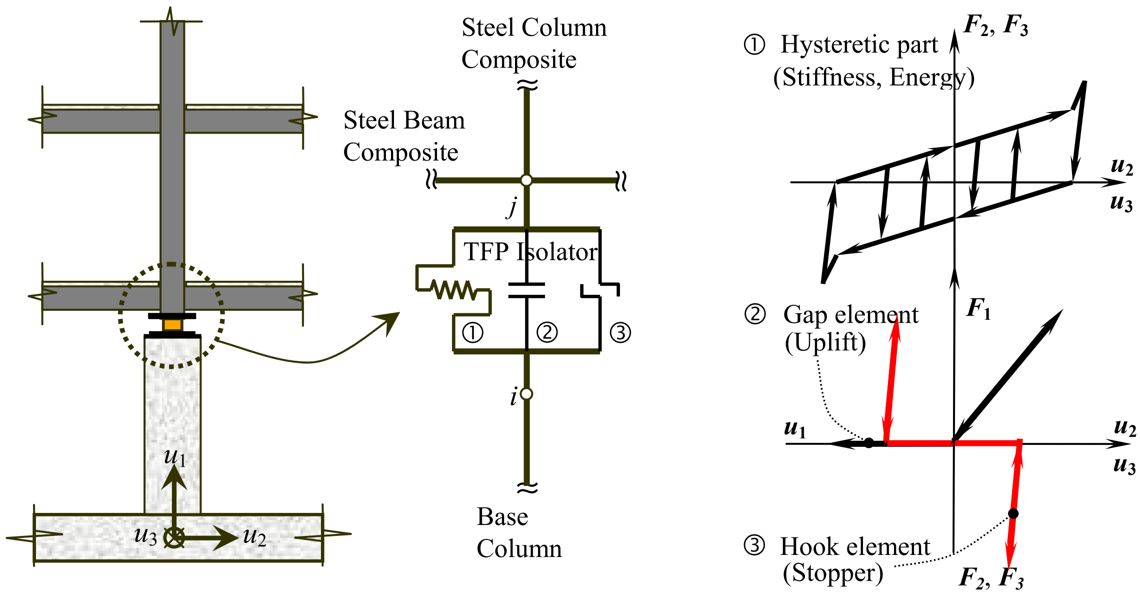


Figure 3. Nonlinear Parallel Discrete Spring Model for the TFP isolators

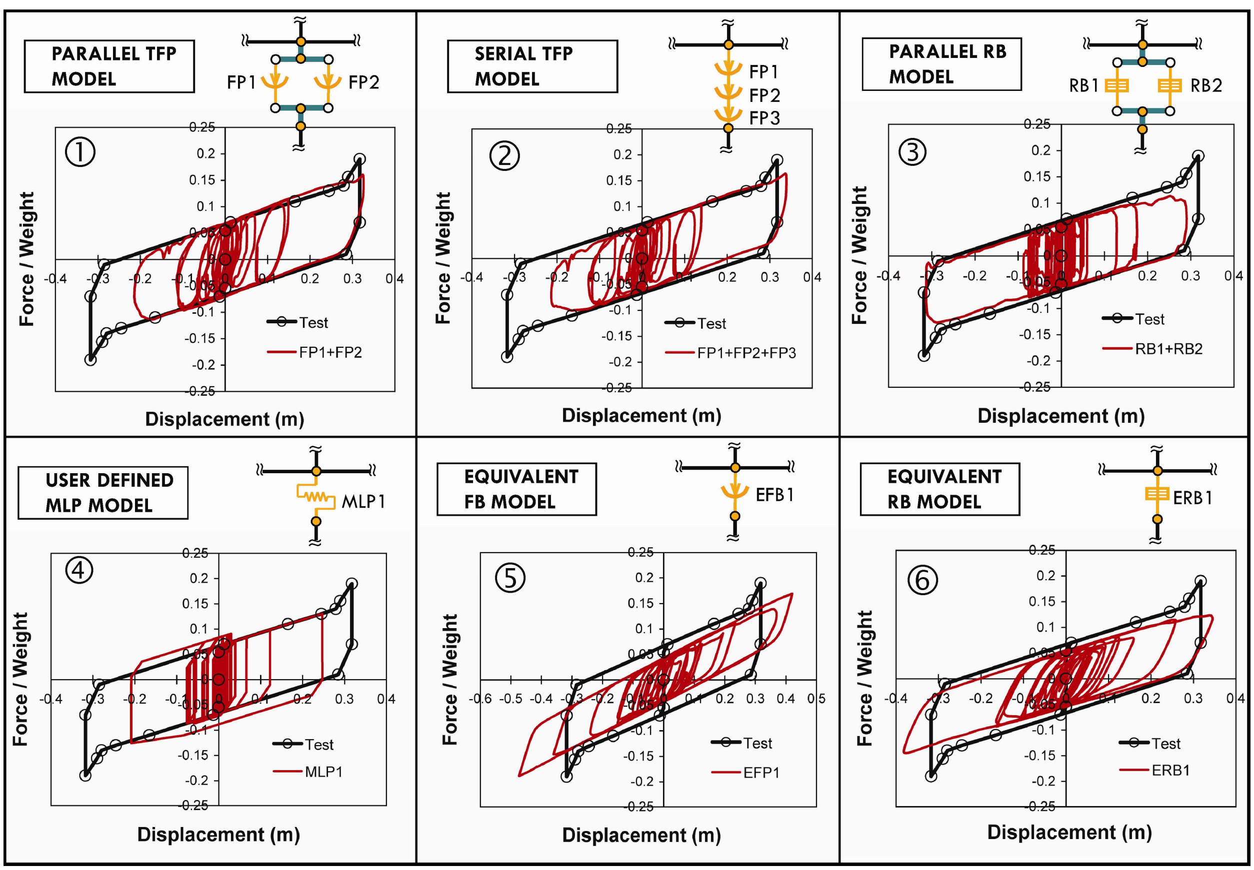


Figure 4. Comparisons of the TFP hysteresis response curves using different approaches

Analysis Procedures and Results

Equivalent lateral force, response spectrum, and nonlinear time-history analysis Procedures are used in the performance-based design of SGIA. Results obtained from each analysis procedure are compared to verify the overall results.

Equivalent lateral force procedure of ASCE 7-05, Chapter 17 estimates the total base shear and maximum isolator displacements efficiently and is used for preliminary analysis in the early stages of the design and brief verification of the results obtained from other procedures. The results of this procedure are given later in this section.

The response spectrum analysis forms the basis of the structural design and compliance with ASCE 7-05. The guidelines given by Article 4.5 of the AASHTO Standard Specification are used for the linear dynamic response spectrum analysis of the isolated SGIA building with the following adaption: (a) The isolation bearings are modeled by the use of their effective stiffness, K_{eff}, which is determined at the design displacement, (b) The ground response spectrum is modified to incorporate the effective damping, β_{eff}, of the isolated structure. In order to obtain a response spectrum for a damping ratio that is different than the damping of the original response spectrum curve, which is 5%, the equation Eq.1-13 of ASCE 41-06 is used. The scaling factors for SGIA are estimated to be 2.04 for DBE (38%) and 1.82 for MCE (30%) event. It should be noted that the modified portion of the response spectrum is only used for the isolated modes, *i.e.*, scaling factor is only applied to the portion of the response spectrum curve with periods greater than $0.8T_{\mathrm{eff}}$. The response spectrum curve with 5% damping is used for the other modes. Final response spectrum curve that is used for the isolated structure is often called composite spectra.

The Nonlinear Time-History Analysis is conducted as a verification analysis over and beyond ASCE 7-05 compliance requirements. Only the “average” response parameters are used for verification purposes. Seven ground motion record pairs obtained from the site-specific hazard study are used as excitation, where both components of the each pair are applied simultaneously to the model. The maximum displacement of the isolation system was calculated from the vector sum of the orthogonal displacement at each time step. Further, this analysis is repeated by switching the components in principle directions. Maximum averaged story shears and their minimum and maximum envelopes obtained from the time-history analysis are plotted in Figure 5(a). Also presented are the result of equivalent lateral force method at the isolation level and the response spectrum analysis results, which show perfect correlation. The base-shear obtained from the time-history procedure (averaged) is slightly smaller than the shear obtained from the other procedures. It is also observed that even though the averaged values (over the records and principal directions) are in good correlation with the response spectrum results, there is a significant difference between the minimum and maximum envelope. Maximum story shear distributions in transverse direction from individual record pairs are shown at Figure 5(b). Maximum averaged story displacements from the time-history analysis together with minimum and maximum envelope are plotted in Figure 6(a). Averaged values also show good correlation with the results obtained from the other procedures. Again, significant deviation among the story displacement envelopes (obtained from individual records) is observed Figure 6(b).

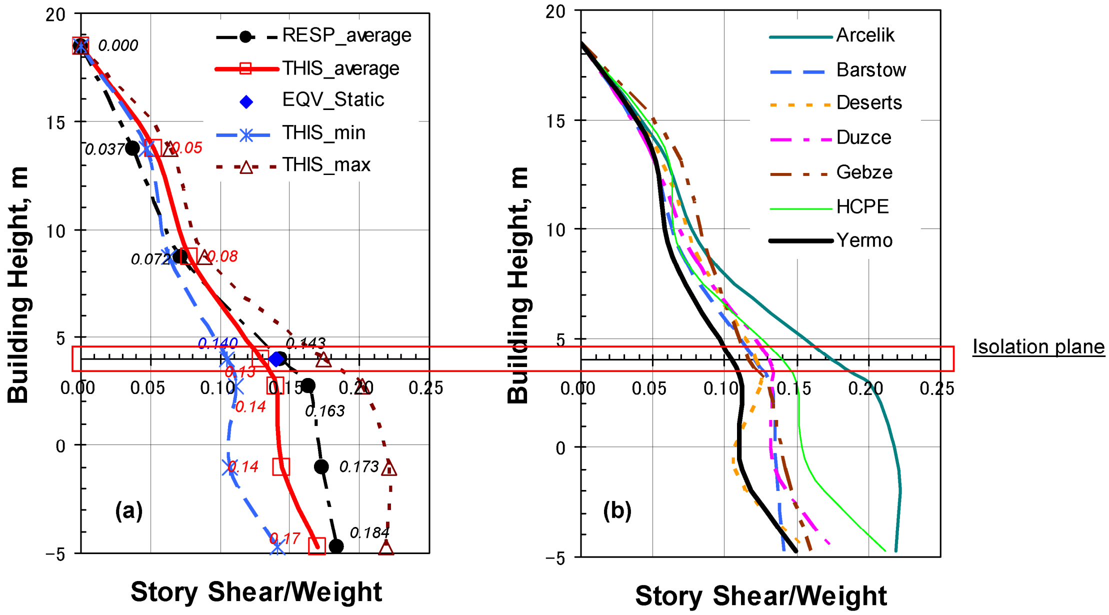


Figure 5. (a) Maximum story shears from the time-history analysis (MCE), (b) maximum story shears in the transverse direction by the individual record pairs (MCE)

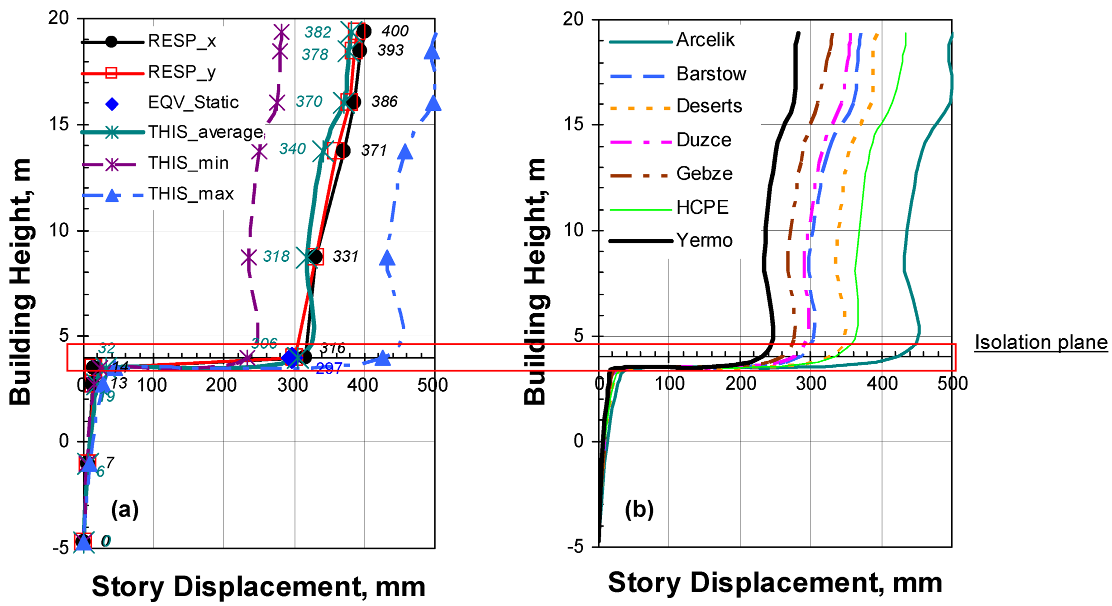


Figure 6. (a) Maximum floor displacements from the time-history analysis (MCE), (b) maximum story displacements in transverse direction by individual record pairs

The results show that hazard study, selection and number of records used in the analysis may affect the analysis results, design, and structural performance significantly. The authors believe that current code provisions for seismically isolated buildings are not specific on the selection of time-histories. Base shear coefficients and isolator displacements obtained from the analysis procedures are presented in Tables 2 and 3.

Table 2. Comparisons of the base shear coefficient using different analysis procedures

Base Shear (*W*%)	**DBE**		**MCE**	
	Lower	Upper	Lower	Upper
Equivalent Lateral Force Procedure (ASCE 7-05)	0.093	0.103	0.137	0.140
Response Spectrum Procedure	0.098	0.101	0.139	0.145
Nonlinear Time-History Procedure (Averaged)	0.091	0.095	0.130	0.135

Table 3. Comparisons of isolator displacements using different analysis procedures

Isolator Displacement (mm)	**DBE**		**MCE**	
	Lower	Upper	Lower	Upper
Equivalent Lateral Force Procedure (ASCE 7-05)	139	110	297	235
Response Spectrum Procedure	145	115	296	234
Nonlinear Time-History Procedure (Averaged)	130	104	267	211

A detailed treatment of the analysis procedures and further discussion can be found in an earlier publication of the authors (Atila *et al*. 2009)

Structural Performance

In this study, time-history and response spectrum procedures are used for the performance evaluation of members and for stability check of the system (particularly for the MCE event). Building performance is quantified in terms of story drift and member-based demand capacity ratios at DBE and MCE events. Nonlinear acceptance criteria for the structural components defined by FEMA 356 and ASCE 41-06 are implemented. Initially, nonlinearity is assumed to be limited to the isolators (*i.e.*, the superstructure is elastic). When the results indicate a demand larger than the elastic limit of a member, a nonlinear link element is assigned for that member. If the member performance does not meet the acceptance criteria, the section is modified. Analysis is then repeated until all the members satisfy the performance criteria.

Accidental torsion, which is an important aspect of the structural model, is also considered in the analysis. For a base-isolated building, ASCE 7-05 requires that accidental torsion effects should result a minimum of 10% increase in the maximum diaphragm displacement that of an analysis without accidental torsion. It is observed that accidental torsion affects did not yield a significant increase in the stress levels except for a minor group of members (corner elements), increase in the member stresses was in the order of 30%, yet, all were within the acceptable limits.

Response sensitivities due to change in the isolator properties are also included in the analysis and the performance evaluation. Upper bound properties are used in member based capacity checks and lower bound properties are employed for the deflections. It was observed that the upper bound isolator properties result in a decrease up to 20% in the isolator displacements and approximately a 5% increase in the member forces.

Another observation is that maximum story drift from the individual records (not averaged) is less than 0.3% for DBE and 0.5% for MCE level hazard, which can be compared to FEMA 356, Table C1.3, "Steel Moment Frames" drift limits of 0.7% for Immediate Occupancy (IO) and 1% for Life Safety (LS). Also, the maximum isolator displacement for MCE hazard is found to be 297 mm, which is less than the isolator displacement capacity, 345 mm.

A design stress check based on AISC 1999 (LRFD) showed that the structure behaves elastically (D/C < 1) under the DBE hazard using the response spectrum and time-history analysis where upper bound isolator properties are used and accidental torsion effects are included in the analyses. Stress check for the MCE level hazard is also performed using the response spectrum and time-history analysis, which is not averaged (note that this stress check is not required by ASCE 7-05), and the maximum results are within the following limits:

- Majority of the columns < 1.3 (FEMA 356 *m*-factor for IO is 2.0)
- All Beams < 2.0 (FEMA 356 *m*-factor for IO is 2.0)

No specific procedures exist for averaging the stress ratios of the time history results from the seven MCE pairs. However, if the maximum stress ratios from the time history analysis were scaled using average base shear of 13%, divided by maximum base shear of 16% from the individual time-history analysis, it could have been easily argued that columns would be still within their elastic limit, while beams are expected to experience a moderate amount of inelastic behavior. This shows that the design presented herein will still meet the performance objectives even if the maximum response values from the time-history analysis (not averaged) are used.

Conclusions

There are several conclusions derived from the performance-based design of the SGIA Terminal Building. It is concluded that the results of the equivalent lateral force, response spectrum and time-history analysis procedures are all in good agreement. For DBE hazards, the base shears obtained from the three procedures are approximately 10% of the seismic weight. For MCE hazards, the averaged base shears obtained from the time-history analyses is slightly smaller than the other two methods (%13 vs %14 of the seismic weight). In addition to parallel discrete spring model, recently proposed SAP2000 TFP models are efficient in capturing the nonlinear behavior of the isolators and can be used in the performance-based design. It can also be concluded that the TFP isolator displacement capacity is adequate for the Terminal Building since the maximum isolator displacement is 297 mm, which is less than the isolator allowable limit of 345 mm. Further, maximum inter-story drift for the DBE and MCE hazards are much smaller than that is required by ASCE 41-06 showing the efficiency of the base isolation system. Design stress check showed that the structure behaves elastically (D/C < 1) under the DBE hazard using response spectrum and time history analysis when the upper bound isolator properties are used and accidental torsion effects are included. Finally, the seismically isolated structure met and surpassed the performance objectives, while achieving an 80% reduction in the base shear (relative to the fixed-base building model), significant decrease in the story drift (75% - 80%) and floor accelerations (65% - 80%).

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