Pertanika J. Sci. & Technol. 24 (2): 273 - 283 (2016)

SCIENCE & TECHNOLOGY

Journal homepage: http://www.pertanika.upm.edu.my/

Seismic Response of a Light Rail Transit Station Equipped with Braced Viscous Damper

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ABSTRACT

The use of the Light Rail Transit (LRT) system is currently preferred because LRT is sustainable, improves travel options and facilitates swift mobility in urban areas. Hence, the structural stability and safety of this public transportation system against seismic occurrences are indispensable. Given that these structures cannot be considered conventional frames because of their complex architectural design, focussing meticulously on reliable seismic design codes and structural rehabilitation techniques is vital for the design of the lateral resistance system. One Malaysian LRT station is considered in this study, and the seismic response of this train station when equipped with supplementary viscous damper devices is evaluated. Thus, the LRT station is modelled through finite element simulation. The methods of seismic analysis are limited to linear seismic analyses, namely, response spectrum and time history analyses. Results derived in this study show a significant improvement in structural response when the station is fitted with dampers; approximately 40% reduction in displacement is observed at the top joint of the roof. Furthermore, the lateral base shears decrease by approximately 70%.

Keywords: Brace viscous damper, time history analysis, light rail transit, response spectra

INTRODUCTION

The general use of Light Rail Transit (LRT) is preferable because LRT is sustainable, improves travel options and facilitates swift mobility in urban areas. Hence, the safety of this type of public transportation against seismic occurrences is indispensable. Considering that LRT systems cannot be erected on conventional frames because of their architectural complexity, focussing meticulously on reliable seismic design codes

Article history:
Received: 8 January 2015
Accepted: 4 February 2016

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ISSN: 0128-7680

and structural rehabilitation is vital. The misconception by Malaysians that the country is free from earthquake disasters has become the main concern of engineers in Malaysia in analysing and designing structures without accounting for the possibility of an earthquake and thereby, even going for retrofitting of the existing structures.

Peak Ground Acceleration (PGA) plays an important role in design and earthquake load determination. Given the fact Malaysia does not have any reliable and economic seismic code, local engineers utilise the specifications of available international codes to determine the seismic loads to be applied. The direct application of international standards, such as the Uniform Building Code UBC 1997 or the American Assocation of State Highway and Transport Officials Standard Specifications for Highway Bridges AASTHO 1996, to determine such loads has resulted in high construction costs because of the overestimation of PGA for Malaysia in the absence of sufficient seismic data for Malaysian conditions.

REVIEW OF LITERATURE AND STUDY SCOPE

The Lam stochastic model was introduced to simulate the elastic response spectra (5% critical damping) of large-magnitude long-distance earthquakes generated by the Sunda Arc subduction source in Indonesia (Lam et al., 2009). A new set of attenuation relationships for PGA, peak ground velocity and response spectrum analysis in rock sites caused by distant Sumatran-subduction earthquakes have been derived for Singapore and Peninsular Malaysia based on synthetic seismograms that account for source and path effects (Megawati et al., 2003; 2005). A previous study investigated potential effective factors, such as site ground response, variation in the water table and soil properties, in the response of a structure to multi-directional earthquake loading (Yang & Yan, 2009). The spatial effects on the seismic responses of large structures with two-line support were studied. The 3D variation in ground motion was modelled with an empirical coherency loss function. Numerical outcomes indicated that horizontal multi-support excitations have a large amplification effect on the seismic responses of the trussed arch (Su et al., 2006; 2007).

In the present study, the seismic response of an LRT station located in Kuala Lumpur, Malaysia was evaluated. Time history analysis was performed using the Structural Analysis Programme (i.e. SAP-2000) by modelling structures 'with' and 'without' the brace viscous damper. In other words, in this study the damper element was introduced as a supplementary energy dissipation device to the existing station's frame and the seismic behaviour of the mentioned frame was assessed. The efficiency of deployment of SAP-2000 in dynamic analysis was established based on the results derived (Behnia et al., 2013).

MATERIALS AND METHODS

Detail of the Case Study

The structure of the study station is supported by three reinforced concrete (RC) column piers attached to the foundations located on the road shoulders. The spacing between the supporting column piers is orthogonal to the carriageway. The guide way structure varies between 13.0 and 16.5 m and is repeated every 12.0 m along the viaduct piers, as shown in Figure 1. The station consists of steelwork, fabricated plated steel sections spanning continuously between the supporting RC piers and simply supported precast pre-stressed hollow core slabs with over 12 m spans. The steel girders are supported on laminated elastomeric bearing pads to reduce the risk of damage to the RC piers caused by the rotation of the beams being loaded. A series of one-way and two-way RC slab frameworks spanning onto in situ RC beams are in turn supported by RC columns. The design is independent of the viaduct structure to prevent dynamic loads from the rail from propagating into the station structure. The roof is pinned or fixed to a curved truss frame, stable on its own plane by virtue of moment continuity throughout, and exhibits 1 to 3 degrees of static indeterminacy. The resistance of the base is improved with the use of a continuous plate girder that effectively acts as the base tie. On-elevation lateral stability in the orthogonal direction to the plane of the frame is provided by tie-bracing or moment frame trusses or beams. Roof diaphragm action is ensured with the use of on-plan tie-bracing and roof sheeting. For the stations constructed with reinforced concrete, the rigid connection between the RC girders and piers ensures a portalised framing on-elevation lateral stability configuration. On-elevation lateral stability between the concourse level and the platform is provided by the inherent-moment connections between the in situ RC columns and beams. The building is founded on RC pile caps and in situ bored RC piles. Tie-beams between these pile caps are employed to resist the pile out-of-tolerance moments and ensure robustness and rigidity. These tie-beams also resist the bending action caused by vehicular impact loading. In order to evaluate the dynamic behaviour of LRT station time history as well as response spectrum analyses (RSA) was performed. Time history analysis shows the response of a structure over seismic time history record. Furthermore, the results were separated for each direction of motion to assess the seismic behaviour of structure with the existence of a damper. This study was performed for evaluation of an elevated light rail transit station. In the case of an underground station, a few design considerations and parameters such as soil pressure should be included during modelling. Therefore, the evaluation of seismic response of an underground station is out of this study scope.

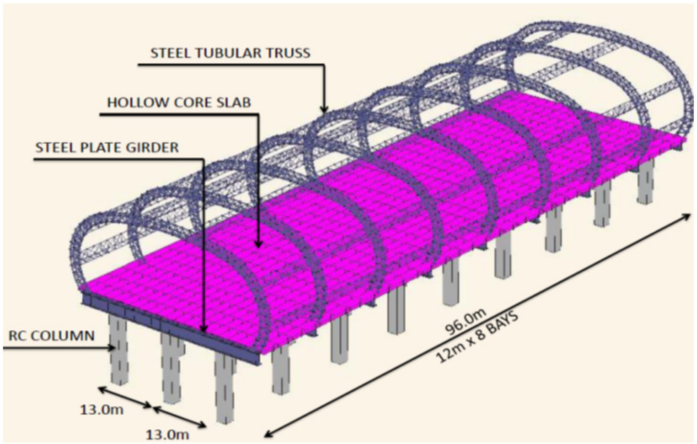


Figure 1. Structural layout of the LRT station.

Software Model Detail

This research was carried out by employing the modelling techniques and capabilities of SAP-2000. As illustrated in Figure 2, the model comprised a 3D space frame model founded on fixed supports. Two models were created to represent the station 'with' and 'without' dampers so as to analyse the different impact. The models were analysed through linear dynamic time history analysis.

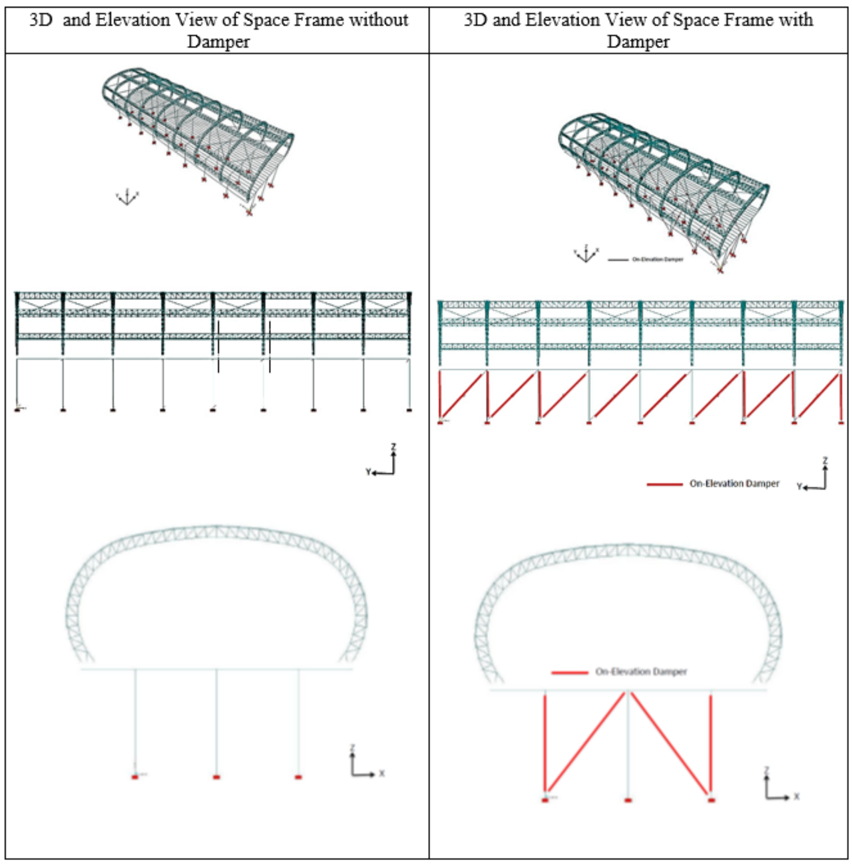


Figure 2. 3D space frame models 'with' and 'without' dampers.

Loading Details

The material densities adopted in the calculation of loads were based on British Standards, schedule of weights of building materials (BS 648:198). Live load reduction in accordance with loading for buildings, code of practice for dead and imposed loads (BS 6399-1:1995) was inapplicable because of the relative low-rise nature of the station buildings. Wind loads were calculated in accordance with Loading for buildings, Code of practice for wind loads (BS 6399-2) and Malaysian Standards, code of practice on wind loading for building structure (MS 1553:2002). Vehicular impact loads were considered in the design of the primary station piers over the vehicular trafficked roads according to British Standards, the design of highway bridges for vehicle collision loads (BD 60), which stipulates an equivalent static load of 1000 kN at 3.0 m and 500 kN at 1.5 m above the carriageway level parallel to traffic direction and half of the above values for the orthogonal direction. All structures were designed for the worst combination of loading. Different load combinations were considered to fulfil the ultimate limit state and serviceability limit state criteria, which are based on structural use of concrete, Code of practice for design and construction, BS 8110 and structural use of steelwork in building, BS 5950. Since the station consisted of a variety of structural elements such as RC frame and steel girder and was subjected to the various types of load such as wind, earthquake and moving loads, use of different design codes was inevitable.

Construction Materials and Structural Control Specifications

In the reinforced concrete elements, longitudinal reinforcement bars (Type 2 deformed grade T460), that is, threaded rebar with minimum characteristic yield strength of 460 N/mm^2, were utilised. Shear links were of grade R250, that is, plain rebar with minimum yield strength of 250 N/mm^2. The mesh reinforcement, BRC, had minimum characteristic yield strength of 485 N/mm^2. The grades and properties of the hollow core slab were based on the specifications of the renowned prefabricator, Eastern Pretech (Malaysia) Sdn. Bhd. The properties of the damper are damping coefficient of 490 KN m/s, brace stiffness of 98066 KN/m and brace-damper mass of 1360 KNs^2/m; these properties were based on a previous study (Panah et al., 2008).

Seismic Analysis and Software Modelling Procedure

A general overview of different analysis methods is presented in Figure 3. Modal analyses were performed to determine the modes of vibration of the LRT station. The modes were utilised as the basis of modal superposition in the time history analysis. In the modal analyses, the structural system of the LRT station was relatively regular and suitable for either single-dominant modal response analysis or multi-modal response analysis. To evaluate the response of all modes of vibration with a significant contribution to the global response, the following conditions were considered and should be satisfied for each direction as stated in the Euro code EC8.

Firstly, the effective modal masses for the modes considered should be at least 90% of the total mass of the structure. Secondly, all modes with effective modal masses greater than 5% of the total mass must be considered. Modal analysis, which defines the fundamental modes inclusive of participating masses was established for both un-damped and damped structures. A linear time history analysis of the LRT station was performed with the El Centro 1940 earthquake as 3D excitation (X, Y and Z). The displacement and base reactions were the benchmark for the comparison. The process of seimic evaluation of LRT station is shown in Figure 3.

RESULTS AND DISCUSSION

Four parameters were compared to check the response of the proposed models in linear static and dynamic analyses. The three parameters are the three natural vibration periods of the sample buildings, participating mass ratios, displacements of the buildings and base shear of the buildings. Understanding the modelling features is critical to obtaining an idea of the situation before viewing the results of all the analyses. Figure 4 shows the essential elements that pertain to the important features of the model.

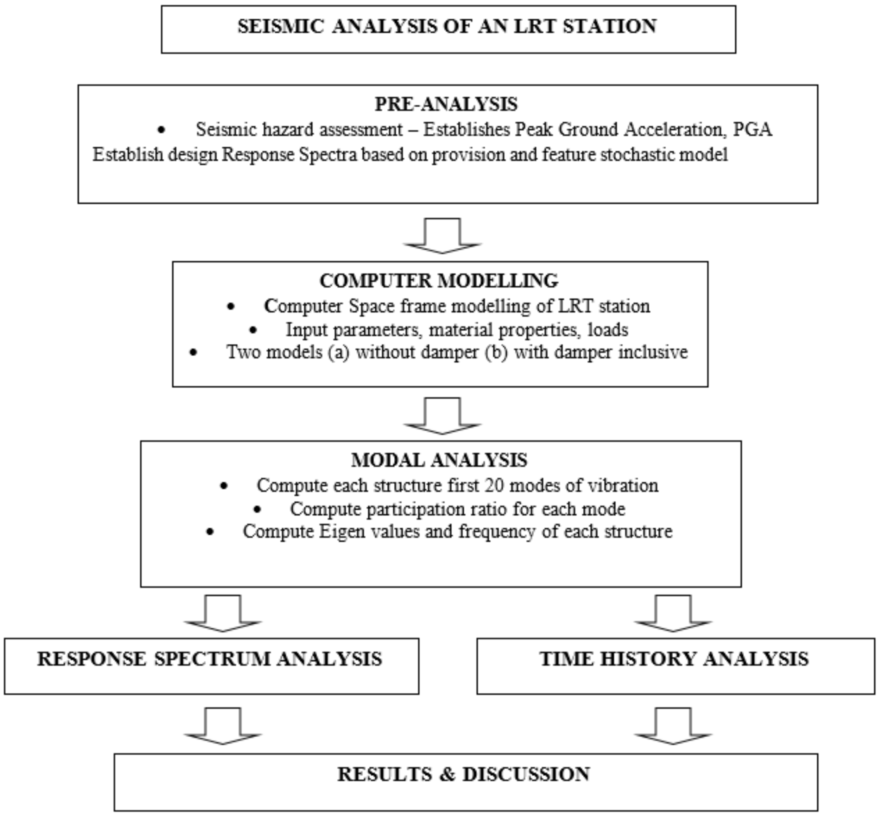


Figure 3. Seismic analysis procedure of the LRT station

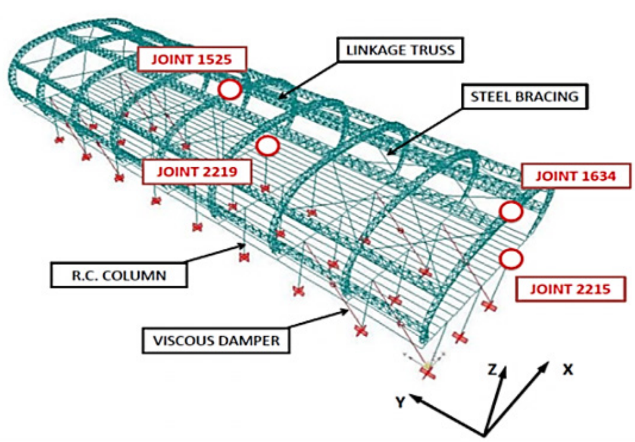
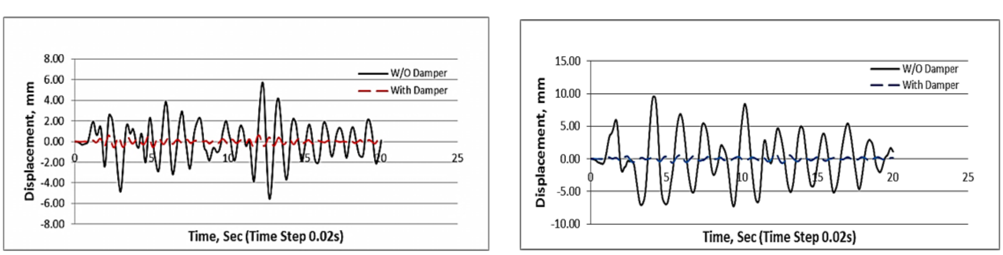


Figure 4. LRT station FE 3D model

Modal and Linear Time History Analyses

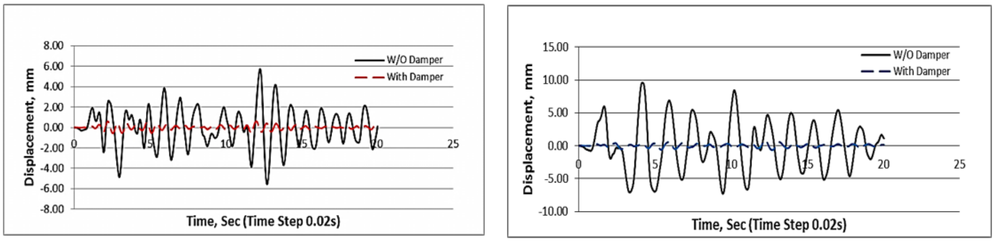
Twenty Eigen modes were selected during pre-analysis and their equivalent frequencies and periods were calculated. The dominant period for the un-damped case was 1.594 s, whereas for the damped case it was 0.938 s. Time history analysis was performed in X, Y and Z directions for the station building structures. The displacement and base shear results were evaluated for both models (damped and un-damped). The acceleration time record of the El Centro earthquake was applied directly to the base of the station building structure for both models in all the three directions. The excitation in the lateral directions (X and Y) produced significant responses. By contrast, the excitation in the vertical direction (Z) produced no significant change in the responses. The time history response at each joint was derived from the analysis. The results showed a limited response of the LRT station for 20 seconds of the entire duration of the earthquake, which was 50 s. However, the critical components of the earthquake fell within the first 20 s. Apart from the maximum responses, a time-dependent function was also obtained; hence, a dynamic response can be plotted with respect to time. Similarly, four joints were selected through Response Spectrum Analysis (RSA) to display the response to time history excitation. Figures 5 and 6 show the displacement response to earthquake excitation in X and Y directions in relation to the damped and un-damped models at different locations. The provision of viscous dampers caused a significant decrease in response. Two joints at the top of the tubular roof structure (Joints 1525 and 1634) exhibited approximately 40% reduction for the damped condition even though the damper was not physically connected to any part of the roof structure. Furthermore, two joints at the top of the RC column or piers (Joints 2219 and 2215) showed approximately 90% reduction for the damped condition. Contrary to the roof joints, the column at Joint 2215 was directly connected to the damper and thus allowed for high dissipation of energy, which in turn reduced the displacement significantly. Figure 7 shows the base shear time-dependent graphs obtained through time history analysis in X and Y directions for the two models, namely, the damped and un-damped structures. Comparison of the un-damped and damped structures indicated more than 60% reduction in Y direction and 71% in X direction in accordance with the overall base shear.



a) X direction displacement

b) Y direction displacement

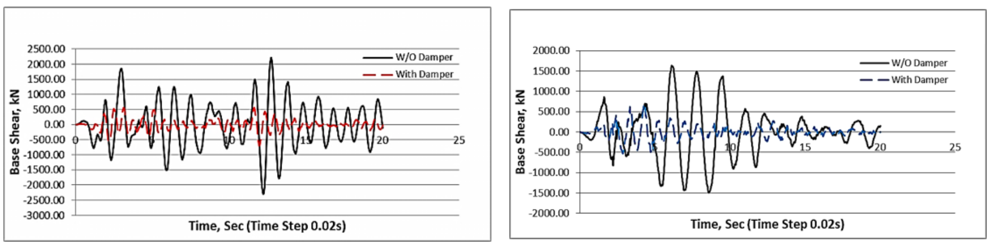
Figure 5. Time history displacement for Joints 1525 and 1634, Roof Top Node



a) X direction displacement

b) Y direction displacement

Figure 6. Time history displacement for Joints 2215 and 2219, RC Column Top Node



a) X direction base shear

b) (a) Y direction base shear

Figure 7. Time history of base shear

Modal Analysis

Modal analysis is a prerequisite for both analysis cases. Based on the abovementioned conditions, 20 Eigen modes were selected during pre-analysis. Their equivalent frequencies and periods were calculated. The dominant period for the un-damped case was 1.594 s whereas for the damped case it was 0.938 s.

Response Spectrum Analysis

The response spectrum curve is plotted in Figure 8. The RSA of the structure was performed for the un-damped and damped LRT station structures and the results were separated for each direction of motion. Differential displacements in all three global directions obtained from the comparison of the damped and un-damped structures are shown in Table 1 together with the corresponding percentage of reduction for ease of comparison.

The displacements were confined to four predefined joints within the model, which were used as the basis for discussion.

1. Two joints at the top of the tubular roof structure (Joints 1525 and 1634) exhibited more than 40% reduction for the damped condition even though the damper was not physically connected to any part of the roof structure.
2. Two joints at the topmost point of the RC column or piers (Joints 2219 and 2215), which were connected to the damper element (damped model), showed more than 80% reduction compared to the un-damped model.

Comparison of the four predefined design response spectra showed more than 75% reduction in overall base shear (Table 2). Generally, the introduction of dampers to the structure reduced the inherited frequency of the structure, which was inversely proportional to the natural period of vibration, and affected the overall equivalent static earthquake load applied to the structure, which in turn, reduced the base shear.

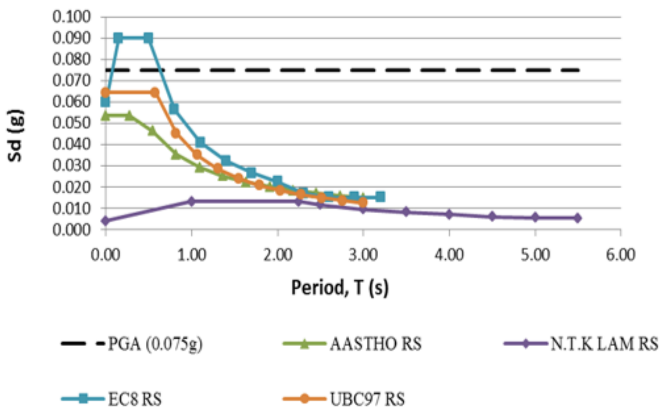


Figure 8. Summary of the Seismic Design Response Spectrum Curve

Table 1
Displacement in the Damped and Un-damped Structures

Response Spectrum Curve		Un-damped Structure dx (mm)	Un-damped Structure dy (mm)	Un-damped Structure dz (mm)	Damped Structure dx (mm)	Damped Structure dy (mm)	Damped Structure dz (mm)	Reduction (%) dx	Reduction (%) dy	Reduction (%) dz
JOINT 1525	ASSHTO	1.463	2.171	0.563	0.691	0.971	0.555	52.77	55.27	1.42
	UBC97	1.769	2.274	0.742	0.968	1.202	0.732	45.28	47.14	1.35
	EC8	2.177	2.728	1.043	1.212	1.473	1.029	44.33	46.00	1.34
	LAM.S	0.613	1.248	0.095	0.158	0.373	0.093	74.23	70.11	2.11
JOINT 1634	ASSHTO	1.361	2.184	0.43	0.722	0.989	0.429	46.95	54.72	0.23
	UBC97	1.642	2.288	0.559	1.009	1.224	0.559	38.55	46.50	0.00
	EC8	2.017	2.745	0.778	1.261	1.499	0.772	37.48	45.39	0.77
	LAM.S	0.574	1.255	0.101	0.168	0.38	0.093	70.73	69.72	7.92
JOINT 2215	ASSHTO	0.626	1.36	0.006	0.031	0.016	0.002	95.05	98.82	50.87
	UBC97	0.753	1.419	0.007	0.043	0.02	0.003	94.29	98.59	43.61
	EC8	0.925	1.701	0.008	0.054	0.025	0.005	94.16	98.53	36.38
	LAM.S	0.265	0.786	0.002	0.007	0.006	0.001	97.32	99.20	61.35
JOINT 2219	ASSHTO	0.787	1.362	0.007	0.032	0.015	0.003	95.93	98.90	57.14
	UBC97	0.945	1.42	0.008	0.045	0.019	0.005	95.24	98.66	37.50
	EC8	1.16	1.703	0.011	0.056	0.023	0.006	95.17	98.65	45.45
	LAM.S	0.334	0.787	0.003	0.007	0.006	0.001	97.90	99.24	66.67

Table 2
Base Reaction for Damped and Un-damped Structures

Response	Un-damped Structure			Damped Structure			Reduction (%)		
Spectrum Curve	Fx (kN)	Fy (kN)	Fz (kN)	Fx (kN)	Fy (kN)	Fz (kN)	Fx	Fy	Fz
ASSHTO	251.84	190.42	44.73	54.29	32.39	44.92	78.44	82.99	0.43
UBC97	302.59	198.59	58.99	76.05	40.13	59.18	74.87	79.79	0.32
EC8	371.51	238.16	82.96	95.25	49.31	83.22	74.36	79.29	0.32
LAM.S	106.84	109.99	7.52	12.43	12.36	7.52	88.36	88.77	0.11

CONCLUSION

The seismic behaviour of a structure in terms of displacement and base shear was determined. The numerical results for the two models (un-damped and damped) clearly indicate that the dampers reduced the seismic response of structures in an extremely efficient manner. The horizontal displacements of the joint at the top of the structure were reduced by 40% for both the RSA and time history cases. Consequently, the horizontal displacements of the joint at the top of the column or pier were reduced by 80% and 90% for the RSA case and time history cases, respectively. Lastly, the lateral base shears were reduced by 75% and 65% for the RSA and time history cases, respectively. The benefits of damper application in the LRT station were demonstrated by the comparative data provided above. Therefore, it can be inferred that an improvement in the performance of the structure during earthquake excitation was observed from the modelling study.

ACKNOWLEDGEMENTS

This work received financial support from the Ministry of Higher Education of Malaysia under FRGS Research Project Nos. 5524254 and 5524256 and was further supported by University Putra Malaysia under Putra Grant No. 9415100. The support provided is gratefully acknowledged.

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