

Seismic performance of long cable-stayed bridge with various time history loading using finite element method

Nabila Huda Aizon¹, Azlan Adnan², Mohd Zamri Ramli³, Norazah Arjuna⁴
Asmawisham Alel⁵, and Muhamad Ali Muhammad Yuzir⁶

^{1,2,3,4,5} School of Civil Engineering, UTM, Skudai, Johor, Malaysia

⁶ Malaysia-Japan International Institute of Technology MJIT, Kuala Lumpur, Malaysia

*Corresponding author e-mail: nhuda.aizon@gmail.com

Abstract. Recently, numerous natural disaster events have occurred in various parts of the world. Bridges are one of the most critical structures to be examined to check for defects resulting from disaster events like earthquakes. Therefore, the serviceability of the structure is important to maintain the safety of users. Thus, the main objective of this work is to study the seismic response of a cable-stayed bridge under earthquake time history loading. The cable-stayed bridge components (deck, piers, pylons, and cables) were modelled using finite element modelling in three dimensional (3D). Two types of seismic analysis was implemented in this study; Free Vibration Analysis (FVA) and Nonlinear Time History (NLTH) direct integration analysis to obtain the bridge seismic performance under the earthquake loading. A total of eight earthquake loads in scaled Peak Ground Acceleration (PGA) values of 0.75g, 1.0g, and 1.5g to represent low, moderate and strong earthquake loads, respectively, were used for the analysis. First, eight mode shapes with component directional and period are presented for free vibration analysis. Meanwhile, the NLTH direct integration analysis results reveal the time detection of seismic cable-stayed damage levels under certain peak ground accelerations (PGA). It can be concluded that the long cable-stayed bridge is not safe under several earthquake loads due to different earthquake load profiles. The results are also important to aid further damage assessment of the cable-stayed type of bridge.

1. Introduction

Cable-stayed is constructed uniquely for aesthetic values and structural reasons. As one of the most complex long span bridges in recent times, it has become an attraction for any transportation project. They are the most cost-effective option for engineers when a longer span is required, but the span is short enough that the suspension bridge is economically impractical. Owing to their importance, they carry a large amount of traffic in their daily service for automobiles, trucks, bicycles, and pedestrians. As with any other bridge, they often subjected to severe conditions such as earthquakes, which could collapse some of their critical structural elements, particularly the cables. Therefore, proper management of this bridge is crucial since any extreme and hazardous event would not only threaten its users but also cause economic losses. The motivation of researchers to study the topic includes the higher repair costs and logistical importance of the bridge [1]. Furthermore, due to its flexibility and



Content from this work may be used under the terms of the [Creative Commons Attribution 3.0 licence](https://creativecommons.org/licenses/by/3.0/). Any further distribution of this work must maintain attribution to the author(s) and the title of the work, journal citation and DOI.

low inherent damping, cable-stayed bridge is highly sensitive to earthquake excitation; thus, the effect could not be ignored [2,3]. Moreover, the strong coupling between vibration modes makes the bridge sensitive to three-dimensional excitation [4,5].

Free vibration method, nonlinear time history analysis, nonlinear static analysis and response spectra analysis are the common analysis to study the seismic behaviour of structures. However, in this study, only free vibration analysis and the nonlinear time history were used. Some researcher strictly stated the study of vibration modes is essential in design and seismic effects assessment of cable-stayed bridge, regardless of the analysis method used [5]. There are two significant reasons to identify the dynamic behaviour of the structural system which is first, to assess the possibility of damage under extreme motion events, and second, a general overview of structural dynamic characteristic obtained can be used in future design and analysis [6]. The fundamental understanding that governs the dynamic response is derived from the equilibrium equation (1) of motion theory by [7].

$$M\ddot{u}(t)+C\dot{u}(t)+Ku(t)=m_x\ddot{u}_{gx}(t)+m_y\ddot{u}_{gy}(t)+m_z\ddot{u}_{gz}(t) \quad (1)$$

where [M], [C] and [K] are the matrix of mass, damping and stiffness, respectively. The u, \dot{u} , and \ddot{u} are the relative displacements, velocities, and accelerations with respect to the ground m_x, m_y , and m_z are the unit Acceleration Loads; and $\ddot{u}_{gx}, \ddot{u}_{gy}$, and \ddot{u}_{gz} are the components of uniform ground acceleration. To calculate the seismic response, researcher used the Open Sees platform and performed NLTH direct integration analysis with a specified time step to be 0.01s on the time history data [8]. Another researcher used SAP2000 to aid her analysis on dynamic bridge response with a time step of every time history data to be 0.05s [9]. The nonlinear structures like cable-stayed bridge are suggested to deploy the time history analysis due to external excitations to obtain the stresses and deformations [10]. From the literature, a different step-by-step integration scheme can be adopted for the integration of equations of motion. For problems with complicated nonlinearities, direct integration methods are preferable despite longer time it needs to finish an analysis. Besides, direct integration methods are always in higher demand, but the choice of one method over another is firmly problem-dependent [11].

Prior to objective of this study, previous research has shown the remarkable attention on dynamic behaviour and seismic response of cable-stayed bridge. However, although extensive research has been carried out on the matter, the lack of full-scale 3D cable-stayed model for earthquake simulation and numerical analysis is identified. Furthermore, most structure analysis field studies in the Malaysia region have not taken seismic effects into consideration. Even though Malaysia is not located in pacific ring fire area, Malaysia is known to be surrounded by an active and moderately active tectonic plate, with the nearest threat a far-field ground motion from Sumatera earthquakes. Consequently, the long period cable-stayed bridge structure could be in a high risk by this type of ground motion. Therefore, the seismic performance study of cable-stayed bridge is very important for low and moderate seismicity areas like Malaysia.

2. Finite element modelling

The bridge has two pylons (east pylon and west pylon) with 143 m height from the pile-cap, two rows (left and right) of semi-fan cable system on each pylon (84 cables in total), and 22 concrete piers along the bridge. Deck are consisting of composite (in the middle span) and concrete box girder with 3.2 m in depth and 25 m in width for four lanes; the box girder rigidly connects with the two pylons and spans to each side. The bridge has a total span length of 1708 m. The cable is stayed at distance of 12m from each other with a carriage width of 3.65m at each of the 4 lanes. The geometry of the bridge is shown in Figure 1.

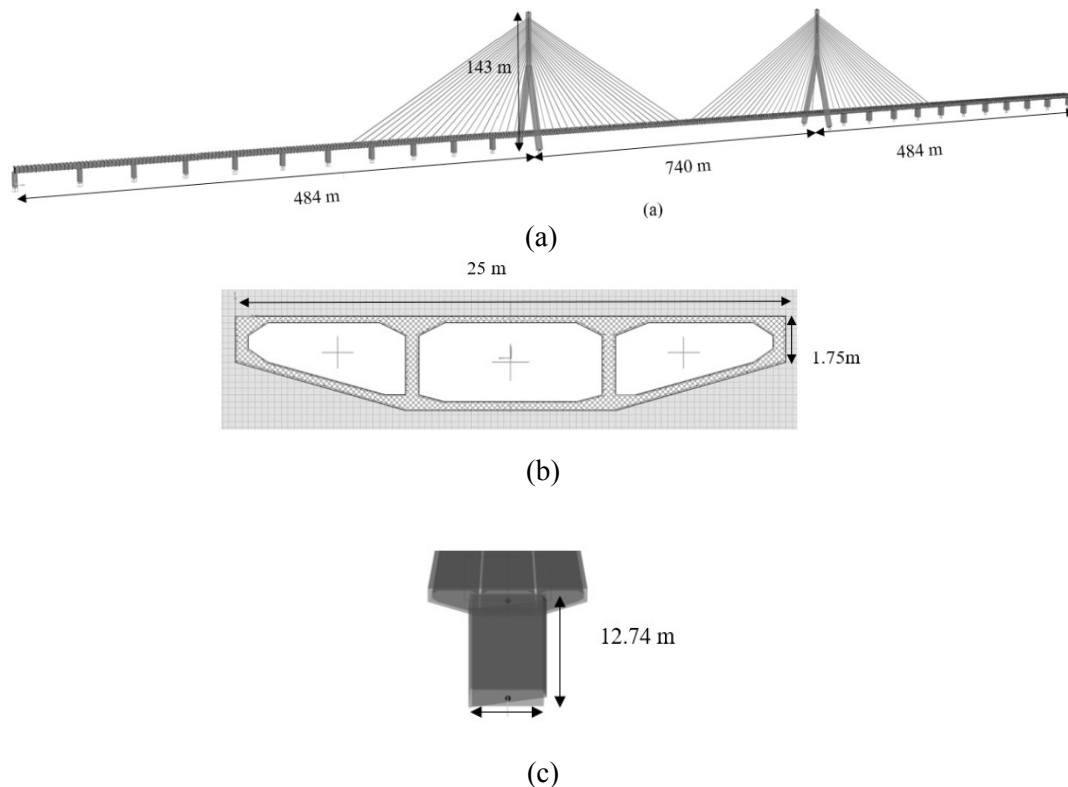


Figure 1. Geometry of the bridge: (a) Schematic of the bridge elevation, (b) Cross-section of the deck, (c) Cross-section of pier

The numerical analysis computer programme is used for FVA and NLTH direct integration analysis to achieve the objective. Free vibration analysis is performed to obtain the natural frequencies and periods. The natural periods and mode shapes are the most important factors to identify the dynamic characteristic of structures. The natural vibration periods of the building were first determined using an eigenvalue analysis. The eigenvalue analysis gives the natural frequencies and modes of a system. The term “natural” is used to indicate each of these vibration properties to emphasize the fact that these are natural properties of the structures in free vibration, and they depend solely on its mass and stiffness properties. The number of modes chosen are based on mass participation factor of more than 90% [12]. The loading involved are dead load and eight scaled earthquake time history loading.

The output of Free Vibration Analysis obtained the failure mode shape, period and natural frequencies of bridge structure. Each mode shape and natural frequency obtained from the result is captured and studied. The procedure should split into either deformation-controlled or force-control by applying the component force versus deformation curves as shown in Figure 2. The cable-stayed bridge seismic performance in terms of damage level is based on FEMA 356, which consists of Immediate-occupancy (IO), Life-Safety (LS), and Collapse-Prevention (CP) identified for each scaled PGA time history case.

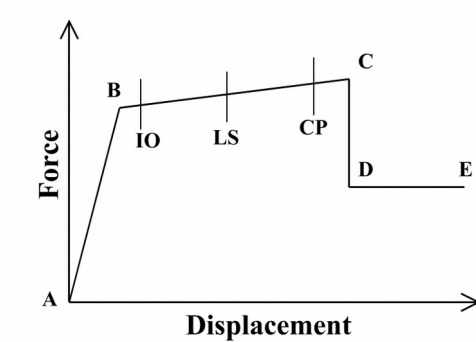


Figure 2. Typical load – deformation relationship and target performance levels [12]

B transition can be explained as damage onset, IO: damage is relatively limited; the structure retains a significant portion of its original stiffness and most if not all of its strength. LS: substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur. CP: at this level the building has experienced extreme damage; if laterally deformed beyond this point, the structure can experience instability and collapse [12]. Figure 3 shows the damage level and performance ranges based on the real damage of structure condition.

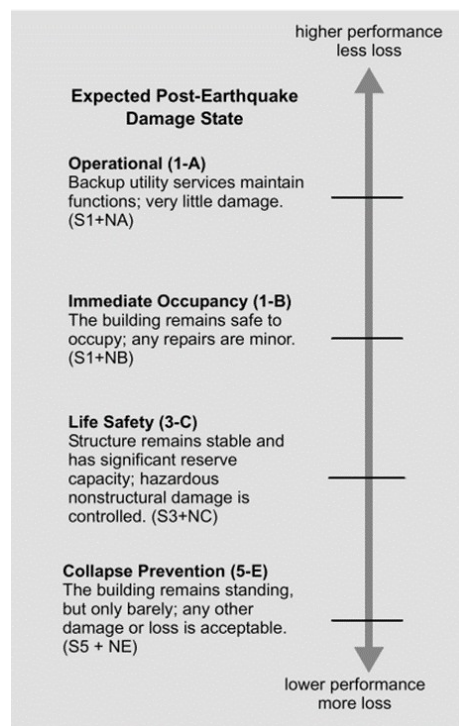


Figure 3. Building performance levels based on damage (FEMA, 2000)

2.1. Earthquake Excitation Data

Eight different earthquake time histories have been tabulated, each with scaled Peak Ground Acceleration (PGA) values: 0.75g, 1.0g and 1.5g specifically for nonlinear direct integration time history analysis. The earthquake loads are chosen randomly and divided into 3 epicentre distances; short distance (0km-70km), moderate distance (70km-300km) and long distance (300km-700km). The five earthquake loads are adopted from the Pacific Earthquake Engineering Research (PEER) database website while the other earthquake loads such as Aceh Earthquake, 2004, Padang Earthquake, 2009 and the latest Ranau Earthquake are adopted from Malaysian Meteorological Department (MMD). The

local earthquake time histories are obtained from seismic stations in Kota Kinabalu from Ranau earthquake events in June 2015. Bridge damage level is based on FEMA 356 [12], which consists of Immediate-Occupancy (IO), Life-Safety (LS), and Collapse-Prevention (CP) from the Pacific Earthquake Engineering Research (PEER) database website. Table 1 shows the earthquake profile details used in this study. The table includes the eight different earthquakes with the frequency content, distance, Peak Ground Velocity (PGV), and Peak Ground Acceleration (PGA) from epicentre.

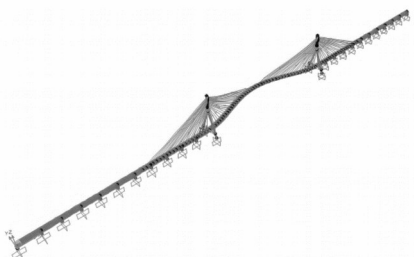
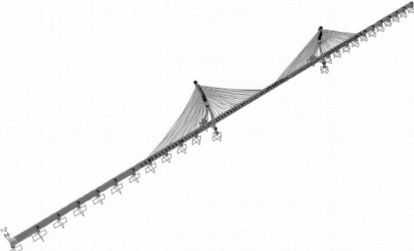
Table 1. Earthquake Profile Details

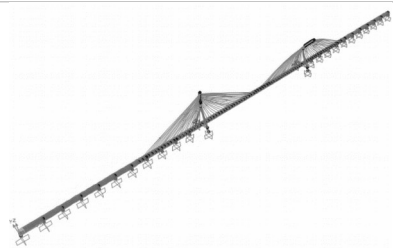
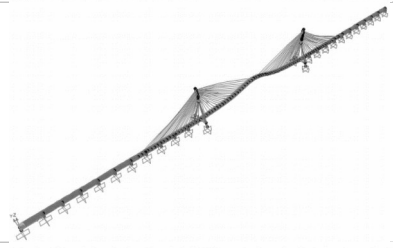
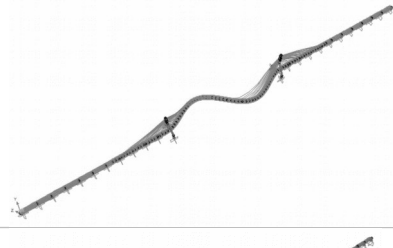
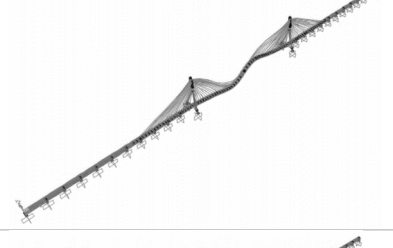
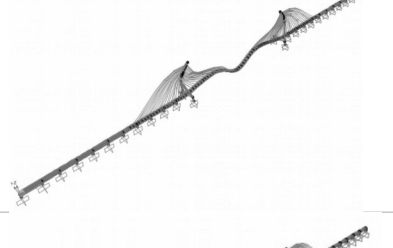

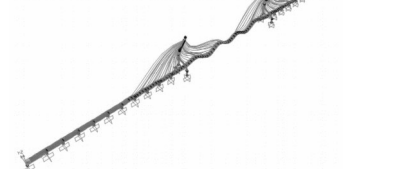
No.	Name of Earthquake	Year	Magnitude (M_w)	PGV (m/s)	PGA (g)	Frequency content, s	Epicentre Distance (km)
1.	New Zealand	1987	6.60	0.2167	0.255	2.515	16.1
2.	Ranau	2015	5.2	0.0410	0.1325	3.784	57
3.	Loma Prieta	1989	6.93	0.1735	0.120	1.025	87.9
4.	Northridge	1994	6.69	0.0454	0.064	1.587	144.7
5.	Kobe, Japan	1995	6.90	0.0429	0.034	0.806	158.1
6.	Chi Chi, Taiwan	1999	7.62	0.0776	0.017	1.831	169.9
7.	Padang	2009	7.60	2.6E-6	2.3E-5	13.196	470
8.	Aceh	2004	9.1	1.2E-6	1.4E-4	14.05	755

3. Result

From the free vibration analysis, the data presented shows all the possible main mode shapes and natural period. The first 8 modes and the 17th mode are identified as the dominant mode and the modes shapes with the structural period by directional are shown in Table 2. Meanwhile, the nonlinear analysis result with damage level is tabulated in Table 3.

Table 2. Mode shapes with component directional and period

Mode	Period (sec)	Direction	Mode shape
1	3.6959	Deck, y-direction (transverse)	
2	2.9441	Pylon, torsional	

3	2.9439	Pylon, torsional	
4	1.5994	Deck, z-direction (vertical)	
5	1.3617	Deck, y-direction (transverse)	
6	1.3024	Deck, z-direction (vertical)	
7	1.0177	Pylon, Couple, Torsional	
8	0.8842	Pylon, Couple, torsional	
17	0.4198	Pier, longitudinal	

As shown in Table 2, the dominant mode shape generated from this type of bridge is in transverse direction of the deck with period 3.6959s. Second and third dominant mode are appeared in pylon with torsional movement. Most mode shapes show for the pylons. Meanwhile, the first pier

mode shape occurred at 17th mode showing the longitudinal direction. Figure 4 shows the period distribution of modal analysis. The periods decrease gradually as the number of modes increases. Only the 7 first modes have periods above 1s. After 1s, many other modes dramatically decline near to 0s. It also can be seen from the table that every mode shape is dedicated to one dominant component, such as pylon mode shapes, deck mode shapes or pier mode shapes. This observation can be found by going through all of the mode shapes one-by-one.

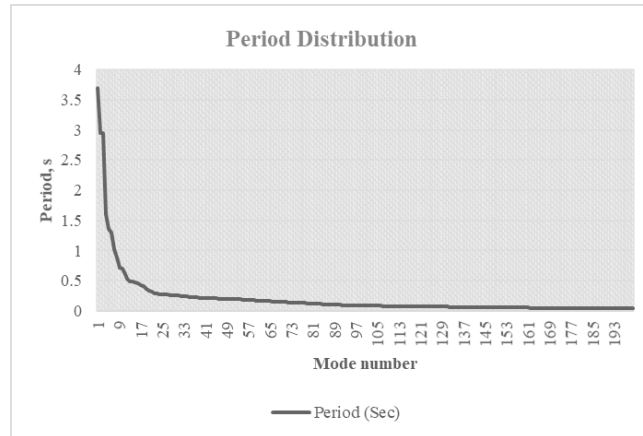
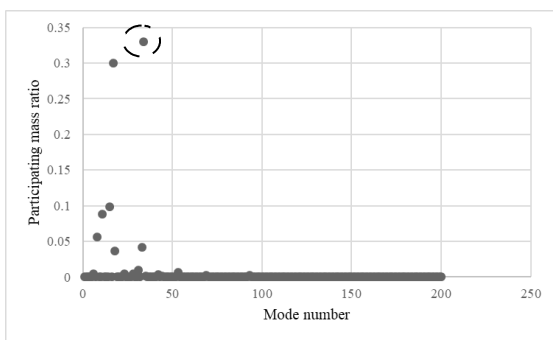
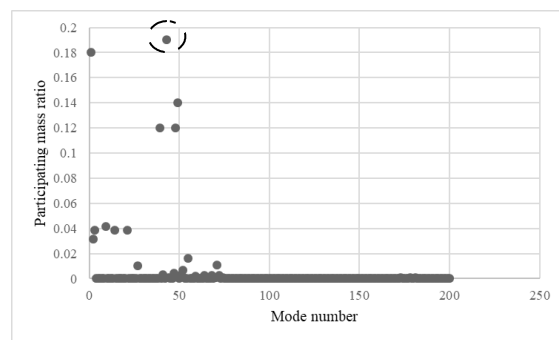


Figure 4. Modal Period distributions of modes

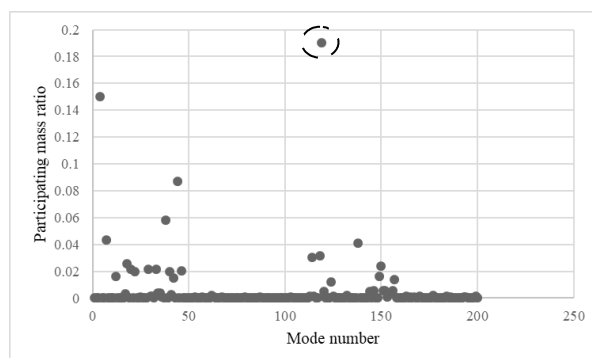
From the result, the maximum modal participating mass ratio in longitudinal (U_x) direction is 0.33 at the 34th mode shape with period of 0.2425s. The highest modal participating ratio for transverse (U_y) was recorded at the 43rd mode with value of 0.19 at 0.2132s. Meanwhile in vertical (U_z) direction, with ratio of 0.19 found at the higher mode, is 119th mode with 0.07176s period. The longitudinal, transverse and vertical direction distribution of modal mass participation of 200th mode is shown in Figure 5. From the figure presented, it can be concluded that better mass participation ratios can be found in higher number of modes (circled in dotted lines)



(a) x-direction



(b) y-direction



(c) z-direction

Figure 5. Modal mass participation distribution of 200 modes

The summary of time detection of seismic cable-stayed damage level from NLTH direct integration analysis is tabulated in Table 3. Items include the randomly varied scaled PGA 0.75g, 1.0g and 1.5g for 8 earthquake loads and the time for each damage level (B, IO, LS, CP) to start occurred.

Table 3. NTHA direct integration analysis result with damage level

NO.	TIME HISTORY	NO. OF DATA	DAMAGE DETECTION TIME (sec)				
			B	IO	LS	CP	END
1.	Northridge 0.75g	683	2.55	25.7	NA ^a	NA ^a	34.1
2.	Northridge 1.0g	733	2.00	22.3	NA ^a	NA ^a	36.6
3.	Northridge 1.5g	538	1.95	22.3	NA ^a	NA ^a	26.85
4.	Kobe 0.75g	837	7.0	29.3	NA ^a	NA ^a	41.8
5.	Kobe 1.0g	647	6.9	17.5	NA ^a	NA ^a	32.3
6.	Kobe 1.5g	674	6.65	13.8	31.2	NA ^a	33.65
7.	Loma Prieta 0.75g	46	0.15	0.7	1.65	1.95	2.25
8.	Loma Prieta 1.0g	44	0.15	0.65	1.15	1.65	2.15
9.	Loma Prieta 1.5g	38	0.15	0.65	1.05	1.25	1.85
10.	Chi-chi, Taiwan 0.75g	848	2.6	30.95	41.05	42.35	42.35
11.	Chi-chi, Taiwan 1.0g	505	1.65	17.25	NA ^a	NA ^a	25.2
12.	Chi-chi, Taiwan 1.5g	437	1.15	13.5	17.8	19.85	21.8
13.	Aceh, Indonesia 0.75g	1601	46.25	NA ^a	NA ^a	NA ^a	80
14.	Aceh, Indonesia 1.0g	1601	9.6	NA ^a	NA ^a	NA ^a	80
15.	Aceh, Indonesia 1.5g	1601	1.9	NA ^a	NA ^a	NA ^a	80
16.	Padang, Indonesia 0.75g	1601	0.5	NA ^a	NA ^a	NA ^a	80
17.	Padang, Indonesia 1.0g	1601	0.45	54.8	NA ^a	NA ^a	80
18.	Padang, Indonesia 1.5g	1061	0.2	12.55	NA ^a	NA ^a	80
19.	New Zealand 0.75g	278	3.4	7.85	NA ^a	NA ^a	13.85
20.	New Zealand 1.0g	293	3.4	7.55	NA ^a	NA ^a	14.6
21.	New Zealand 1.5g	222	3.25	7.5	7.9	NA ^a	11.05
22.	Ranau, Malaysia 0.75g	287	4.35	10.6	NA ^a	NA ^a	14.3
23.	Ranau, Malaysia 1.0g	227	4.35	9.85	NA ^a	NA ^a	11.3
24.	Ranau, Malaysia 1.5g	211	4.25	9.65	NA ^a	NA ^a	10.5

^a NA = Not Applicable (Damage level not occur)

From the table, it can be seen that damage level CP occurred on cable-stayed bridge model due to Loma Prieta 0.75g, Loma Prieta 1.0g and Loma Prieta 1.5g, Chi Chi 0.75g, Chi Chi 1.5g. Meanwhile, the damage level did not occur under the other earthquake loads such as Aceh 0.75g, Aceh 1.0g, Aceh 1.5g and Padang 0.75g.

4. Discussion

Free vibration analysis or modal analysis is an important analysis to characterize the dynamic behaviour of the cable-stayed bridge. This study generates 200 mode shapes to achieve more than 90% of modal participation ratios suggested by [12]. It is important to check if the accuracy of the number of mode shape chosen is adequate to represent the structure mode shapes. The fundamental mode of this cable-stayed bridge has long period of 3.69s with dominant transverse direction. However, the highest modal mass participation of transverse direction can be found at 43rd mode with a 19% contribution compared to the first dominant mode at 18%. This result suggests that higher contribution possibly occurs in a higher number of modes rather than a few first modes. The result obtained agreed with previous work [13].

It can be seen that the first dominant mode shape generated from this type of bridge is at the transverse direction of the deck with period 3.69s. Second and third dominant mode appeared at pylon with torsional movement. Most mode shapes show significant deformation at the pylons. Meanwhile, the first pier mode shape occurred at 17th mode showing the longitudinal direction. Previous researchers mentioned that the nonlinear cable-stayed response in coupling with the instability effect of axial compression has substantial effects in deck and pylon [14,15]. Meanwhile, another researcher explains the nonlinear effect, which is either geometrical or material nonlinearities to the dynamic response of cable stayed bridge. Geometric nonlinearities present in any cable-stayed bridge could be due to; cable sags which controls the axial elongation and the axial tension; the action of compressive loads in the deck and in the tower and; the effect of relatively large deflections of the whole structure due to its flexibility [13].

The importance of the seismic performance is to assess the bridge health under earthquake effects. The performance level indication of bridge is using the FEMA 356 such as IO, LS and CP. In this study, the label B was used to indicate the transition from safe to damage level. According to the results of the nonlinear time history analysis, no level of damage occurred due to Padang and Aceh earthquakes based on scales of PGA tested (0.75g, 1.0g, and 1.5g). This is mainly due to its earthquake profile, which is far from the epicentre. The statement is supported by previous work [9]. Also, the longer fundamental periods make the structure different from other structures and greatly influence the dynamic behaviour.

5. Conclusion and Recommendation

From the objective, this study discussed the behaviour of the cable-stayed structure under eight earthquake loadings using the nonlinear time history analysis. The piers and pylon leg of the bridge are fixed-supported at the foundation base. A total of 10 joints of observation points were monitored, which mainly involved critical components, including the top pylon, middle pylon, middle deck, and the pier. This study used a total of 200 modes, capturing more than 90% mass participation in U_x , U_y , and U_z direction. The fundamental frequency of the bridge response due to the free vibration of the bridge model was 0.27Hz, while the natural period was 3.69s. It can be concluded that, in mode shapes represented earlier, the deck and pylon mode shape has dominated compared to other components.

The performance levels of cable-stayed bridge structure due to earthquake ground motion data are based on FEMA 356 such as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) level [12]. Meanwhile, the B level is a transition from a safe level to the first damage level, IO. According to the results of the nonlinear time history analysis, all PGA scaled for Loma Prieta and Chi-Chi earthquake (except for Chi-Chi 1.0g), the cable-stayed bridge is threatened until the CP damage level. Other time histories such as all PGA scaled for Northridge, Ranau, New Zealand (except New Zealand 1.5g) and Kobe (except Kobe 1.5g) are only devastated with IO damage level. There was no level of damage that occurred due to Padang 0.75g. Surprisingly, the Aceh earthquake was absent from any damage level despite being tested with scales of PGA 0.75g, 1.0g, and 1.5g. The results are influenced by ground motion parameters such as distance from the epicenter, frequency content, magnitude scale, and peak ground acceleration value.

In order to improve the study of performance of cable-stayed bridge under earthquake loading, several recommendations suggested for further study are as follows; first, numerical results presented in this study are adequate, however, the study could be improved by some experimental tests or in situ monitoring (sensor placement to the bridge); second, for further research, the cable-stayed behaviour could be studied in terms of seismic performance under retrofitting effects. The retrofitting should include laboratory test and analytical analysis of this full-scale model of cable-stayed bridge.

References

- [1] Atmaca B, Yurdakul M and Ateş Ş 2014 Nonlinear dynamic analysis of base isolated cable-stayed bridge under earthquake excitations *Soil Dyn. Earthq. Eng.* **66** 314–8
- [2] M. P B, Yozo F and Ajai S 1993 Estimation Curve for Modal Damping in Stay Cables with Viscous Damper *J. Struct. Eng.* **119** 1961–79
- [3] C. C K, L. M Y, C. C C, C. L L and C. C C 2004 Lessons Learned from the Damaged Chi-Lu Cable-Stayed Bridge *J. Bridg. Eng.* **9** 343–52
- [4] Kuleli M and Elnashai A 2013 Cable-Stayed Bridges Subjected To Near-Fault Vertical Motion *4th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering* ed M Papadrakakis, V Papadopoulos and V Plevris (Kos Island, Greece) pp 1941–57
- [5] Camara A 2019 Seismic behavior of cable-stayed bridges: a review *MOJ Civ. Eng.* **4** 161–9
- [6] Clemente P, Çelebi M, Bongiovanni G and Rinaldis D 2004 Seismic analysis of the Indiano cable-stayed bridge *13 th World Conference on Earthquake Engineering*
- [7] Chopra A K 2000 *Dynamic of Structures: Theory and Application to Earthquake Engineering* ed L Curless (Parentice Hall)
- [8] Li C, Li H N, Hao H, Bi K and Chen B 2018 Seismic fragility analyses of sea-crossing cable-stayed bridges subjected to multi-support ground motions on offshore sites *Eng. Struct.* **165** 441–56
- [9] Suryanita R 2014 *Integrated bridge health monitoring, evaluation and alert system using neuro-genetic hybrids*
- [10] Mohammad Hashemi Yekani and Bahar O 2017 A new methodology for health monitoring of cable-stayed bridges; identifying the major features sensitive to damage/failure *Int. J. Eng. Appl. Sci.* **4** 1–17
- [11] Seed F K A, Ahmed H H, Abdel Raheem S E and Abdel Shafy Y 2013 Dynamic non-linear behaviour of cable stayed bridges under seismic loadings *Life Sci. J.* **10** 3725–41
- [12] FEMA356 2000 Prestandard and Commentary for the Seismic Rehabilitation of Buildings *Rehabil. Requir.* 1–518
- [13] Morgenthal G 1999 *Cable-Stayed Bridges - Earthquake Response and Passive Control*
- [14] Bruno D, Greco F and Lonetti P 2013 Static and Dynamic Nonlinear Modelling of Long-Span Cable-Stayed Bridges **1** 3–27
- [15] Aparicio A C 2006 Seismic Behaviour of Cable-Stayed Bridges: Astate-of-the-Art Review *4th International Conference on Earthquake Engineering* (Taipei, Taiwan)

Acknowledgments

Authors wishing to acknowledge assistance, encouragement from colleagues, special help by technical staff of Universiti Teknologi Malaysia (UTM), Skudai, Johor, Malaysia and also financial support from Ministry of Education of Malaysia by MyPhd fund.