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Abstract

A lack of seismic consideration in the practice calculation design of BS 5400 was already known by engineers and specialists, but it is ignored since the authorities thought Malaysia was outside the seismic activity zone. Most bridge engineers in Malaysia have acquired their tertiary education from European nations like the United Kingdom (UK), New Zealand (NZ), and others who adhere to the British Standard 5400 code of conduct. It's a pity some officials haven't remembered that Malaysia is just a few miles away from Indonesia and the Philippines, both of which are regarded seismically active. This study proposes to extract a pier structural geometrical shape to be assessed under Ranau seismic event using non-linear time history analysis. Indeed, the purpose of this paper is to build up a Malaysian National Annex Standard for bridge seismic design based on Eurocode 8 Part-2 provisions, so that Malaysia can set its own criteria in terms of longitudinal reinforcement ratio and amount of confinement at various degrees of PGA intensity (30%g, 12%g, 9%g, 6%g, and 3%g). The results shows that the structural to withstand seismic loading against Ranau historical event, it needs 3.33% reinforcement ratio with minimum amount of confinement at 12%g intensity level is corresponded to Malaysia seismic hazard zonation map.

Keywords: Eurocode 8 Part-2, Design Standards, Longitudinal Reinforcement, Confinement, Ranau Earthquake

1. Overview of Malaysia's Code Practice and Challenges

According to the most recent information received by researchers, scientist, and designers, that the majority of bridge engineers in Malaysia are adopting the BS 5400 code as a design guideline for bridge projects. This is due to the fact that bridge engineer received their fundamental knowledge or tertiary education from European countries such as the United Kingdom, New Zealand, and other countries that follow the British Standard 5400 as a code of practice.

Despite the fact that the engineers or the specialist already aware that BS 5400 does not include seismic consideration in its practice calculation design, and chose to overlook this situation because, in their perspective, that Malaysia as a country is outside of the seismic activity zone. Indeed, seismic resistant design is a tool that is used to handle earthquake hazard and reduce the danger to human life and property damage. As a guideline for constructing structures to withstand potential catastrophic earthquakes in the future, seismic resistant design standards are often included as part of code regulations as a guideline for developing structures to withstand potential destructive earthquakes in the future.

Nevertheless, some of the authorities seem to have forgotten that Malaysia is in close proximity to neighbors, including Indonesia and the Philippines, both are considered as one the highest seismic prone areas surrounding Malaysia. While Penang Island, Kelantan, Perak, and Kedah cities experienced vibrations as a result of an earthquake measuring 4.3 Richter scale, this incident was caused by the Acheh earthquake in Indonesia. The earthquake's excitation motions have cracked several of the building's structural parts, such as columns, walls, and slabs. Following a statement by the Malaysian Meteorological Services and other sources, the reading value of an earthquake for Peninsular Malaysia is 0.075g (75 gal), and the reading value for Sabah is 0.16g (160 gal). These values are deemed minimal vibration by some engineers and are not a source of concern for the safety of bridge structures; but, for those who are concerned about it, these values can result in the collapse of our building or bridge if it occurs on a regular basis.

Malaysia, which has been affected from far-field and near-field earthquakes scenearips from both locally and globally distinct faults, recognizes that earthquakes can cause fatalities, pose a threat to the community and welfare, and cause significant damage to both public and private property. This is due to the fact that less than one percent of Malaysia's structures are seismically resistant, which is a major concern (Taksiah Abdul Majid, 2009). This is due to the fact that the most typical structural design practice in Malaysia is of the aseismic type, which means that seismic force is not taken into consideration.

Historically background for Malaysia seismicity between 1976 and December 2016, the Malaysian Meteorological Department (MMD) has reported a total of 6108 ground motions, ranging from 0.3 to 4.2 Mw, of which 623 are of local origin. There have been 623 local earthquakes, 446 in Sabah, 74 in Peninsular Malaysia, and 16 in Sarawak. With the Ranau earthquake of June 5, 2015, Malaysia had its most powerful earthquake in East Malaysia since 1976's Lahad Datu.

As a result of this scenario, structures and infrastructures are particularly vulnerable to the devastation caused by earthquakes. To reduce the impact of future earthquake disasters well before they occur, earthquake hazard mitigation is essential. For example, actively monitoring seismic activity and developing codes and zoning regulations are all important steps in ensuring a safe environment for the community. Therefore, a need to develop Malaysia practice bridge design code by following Eurocode 8 Part 2 with some modifications especially in design of bridge is much needed to protect bridge structure from the undesired damaging effect due to this natural disaster. As a result of EC8's Malaysian National Annex release in late 2017, Malaysia has implemented its first national code of practice for the seismic design of structures.

For a developing country such as Malaysia, investment in preparedness for a potential seismic disaster is critical in order to reduce the loss of lives and property, and thus the cost of recovery, compared to the alternative of ignoring mitigation. In Malaysia, a group of local academics has been working on generating seismic hazard maps for the country for the past two decades. Researchers have used the Probabilistic Seismic Hazard Analysis (PSHA) and earthquake data from the neighboring tectonic plate borders to create the maps.

The Malaysian Meteorological Department (MMD) established the first seismic hazard maps in the form of Peak Ground Acceleration (PGA) maps of Malaysia, which were created by converting seismic intensities into Peak Ground Acceleration (PGA) as shown in **Fig. 1**.

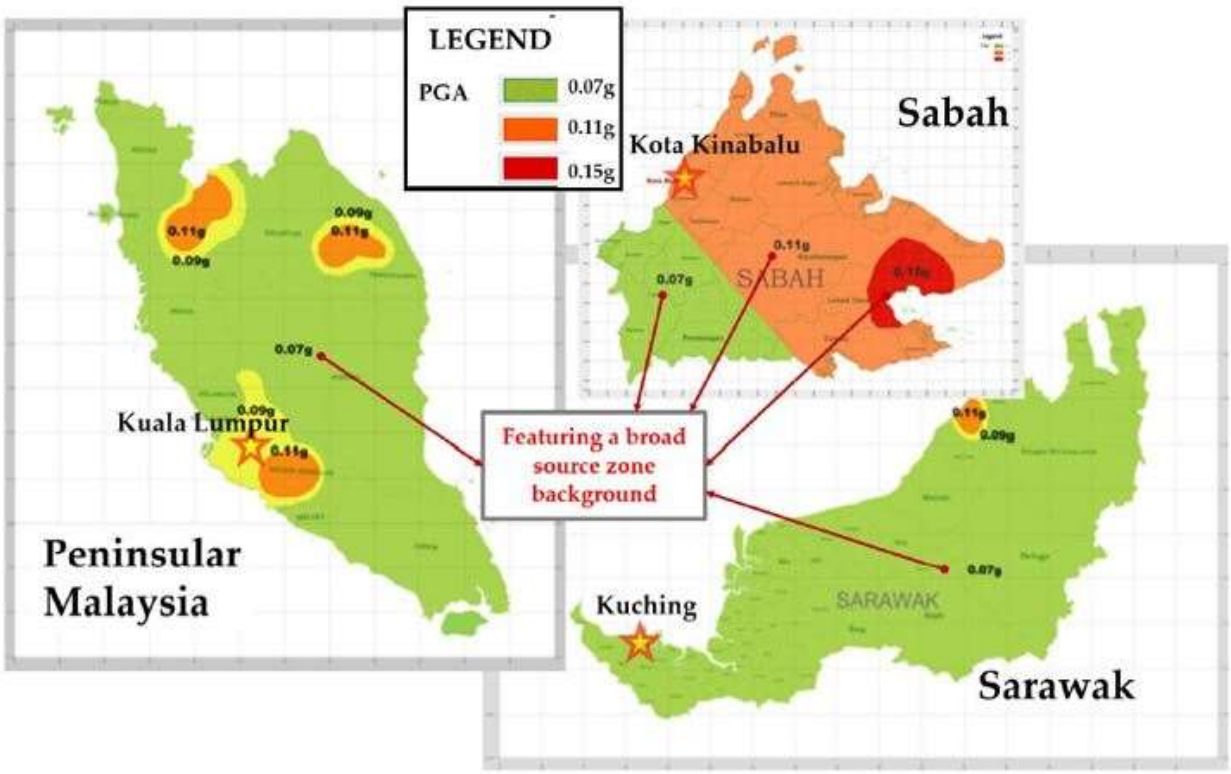

Fig. 1 Proposed Seismic Hazard Map for Malaysia

2. Confining Reinforcement: Purpose and Needs

Over the last few years, earthquake codes have been amended and upgraded. Numerous researchers have shown an interest in and a desire to see the standards for seismic design of bridges published and implemented (Malekly et al., 2010). The fundamental approach of the codes is to incorporates with "performance-based seismic design," because it is the most essential part of the code's implementation. Earthquake codes are changed and updated on a regular basis because of significant advancement in how ground motions are represented. The development of new techniques and procedures, and in particular the lessons learned from recent disastrous earthquakes, seismic codes are altered and updated on a periodic basis. Regions of low and high seismicity can be found in Europe, for instance. It has been stated that it has progressed more in the direction of performance-based seismic design (PBSD) of new structures than any other seismic code for new construction in the EC8.

Following a series of devastating earthquakes over the last two decades, there appears to be widespread consensus on the need to design structures with predictable seismic performance. Performance-based seismic design, on the other hand, necessitates the use of trustworthy methods to design structures in order to ensure that stated seismic performance targets are reached. Bridges frequently rely exclusively on the ability of columns or piers to withstand large displacements without collapsing in order to maintain structural integrity. While the design of reinforced concrete bridge columns or piers for specified flexural performance has gotten simpler in recent years, a reasonable approach to confinement is still required in some cases (De Domenico et al., 2019).

Besides, the increasing number of studies have focused on the concept of capacity design in structures, which is concerned with the goal of ensuring plastic deformation occurs in specified zones that exhibit markedly ductile behavior in order to avoid brittle failure mechanisms and encourage an overall ductile global collapse mechanism of the structure. In numerous nations including Italy, Europe, New Zealand and America, this concept is embedded in technical rules. Because concrete has a brittle behavior; but, when combined with transverse reinforcement, such as steel stirrups, it can exhibit a noticeably ductile behavior. The purpose of stirrups is to increase the confinement effect of concrete and to prevent the lateral expansion of concrete, hence altering the concrete stress-strain constitutive law and allowing for higher compression strains and higher strength levels.

Transverse reinforcement is used to avoid longitudinal bar buckling and shear failure, as well as to confine the concrete core to ensure deformability and ductility during the construction process. These three roles are generally handled separately in American, Canadian, and New Zealand design rules (ACI,2008, CSA,1994, NZS, 1995). According to Richart et al., (1928) it was anticipated that the compressive strength of a confined core of a column post spalling would be equivalent to the strength properties of the column's gross section prior spalling. This is because the ratio of concrete compressive strength to transverse reinforcement yield strength determines the amount of confinement reinforcement needed. The Canadian Standards Association (CSA) stipulates that the column section of a ductile structure must satisfy the appropriate confinement specifications as the American Concrete Institute (ACI). Note that these criteria were originally designed for normal strength concrete. The New Zealand Standard NZS 1995 differs from the preceding standards in that it was established specifically for seismic effects and took into account the axial stress on the column sections. Several researchers have shown the limits of these codes by demonstrating: First, the ACI Code and the CSA Specification confinement reinforcement requirements do not consider the expected axial force concentration, placing a priority on column flexibility. Second, the reinforcement criteria of the ACI, CSA, and NZS standards do not take into consideration; medium, high and also very high-strength concrete, and do not take into account high-yield steel except in limited cases or circumstancees (Sheikh and Khoury, 1993, Sheikh et al., 1994, Azizinamini et al. 1994, Watson and Park, 1994, Li et al., 1994, Bayrak and Sheikh, 1998).

In both Canadian and American bridge design specifications (CSA 1994 and ACI 2008), confinement parameters provide uniform confinement reinforcement. When concrete strength is raised, the amount of confinement reinforcement must be increased to keep column ductility constant (Paultre and Legeron, 2006). Furthermore, high axial load columns or piers may require a large amount of confinement reinforcement to ensure adequate ductility. Congestion in the reinforcing cages causes problems during the concrete pour due to the considerable amount of transversal steel employed. It is expected that improving transverse reinforcement yield strength will allow for less transverse reinforcement. Increasing confined steel yield strength does not necessarily increase ductility when the lateral strength is held constant (Azizinamini et al., 1994). So, confinement reinforcement in design codes must be changed, and new parameters or changes must be devised to logically account for the axial load ratio and ductility need.

In the event of an earthquake, this horizontal reinforcement helps to ensure that the system and component react ductilely. As demonstrated in **Fig. 2**, confinement reinforcement such as closed loop ties or spirals inhibits concrete dilation by providing lateral resistance. This boosts the concrete's overall strength and ductility. Alternative standards and specifications [AASHTO (1983), AASHTO (2013), Caltrans (2010), Priestley (2000), NZS (1995), ATC (1985), CEN (2005), CSA (1994)] suggest different strategies for determining the necessary quantity of transverse reinforcement for confinement reasons that should be discussed in detail.

Each of the individual factors such as variation in longitudinal reinforcement amount, applied axial force ratio, compressive strength, gross concrete area to core concrete area ratio, and material properties of the cross-section used in the design all influence the amount of transverse confinement reinforcement required for a particular design. Example of this variation is shown in **Fig. 3**, where different are compared by among each other in which only concrete compressive strength is increased while leaving the column diameter at 4

feet, the axial load ratio at 5 percent, the longitudinal reinforcement ratio at 2 percent, and a 3 in. horizontally covered with #5 bar are used.

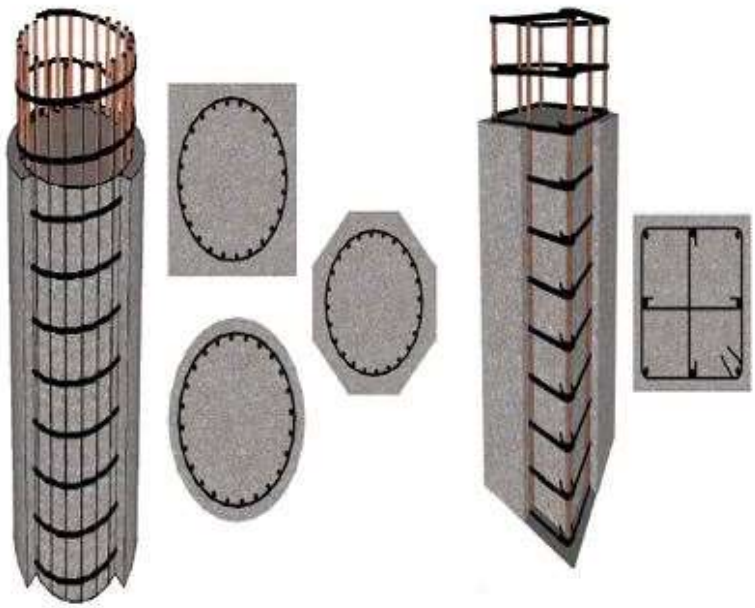

Fig. 2 Samples of confinement in seismically active areas

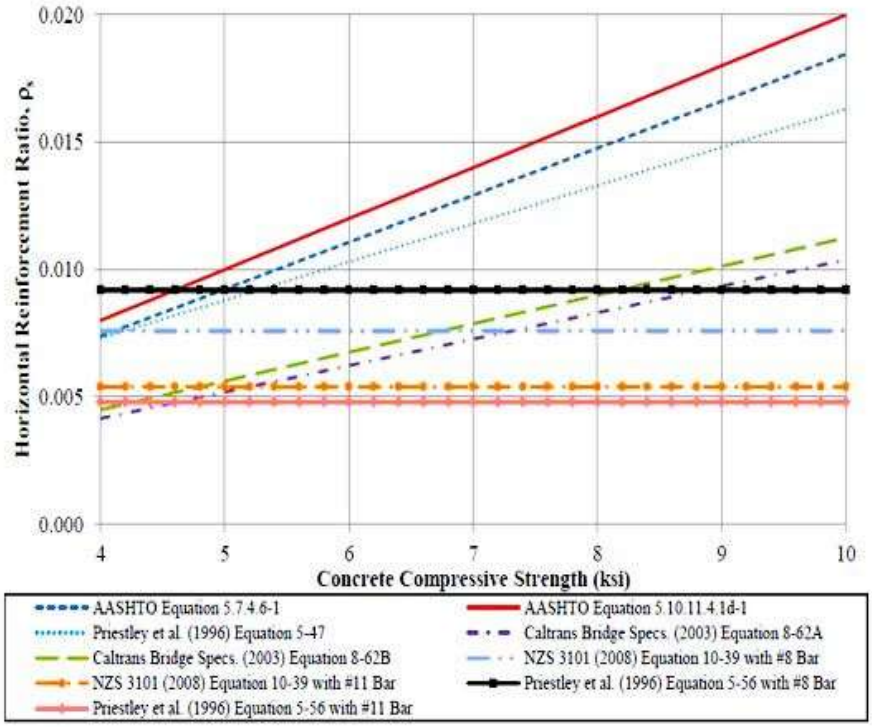

Fig. 3 Impact of concrete compressive strength on horizontal reinforcement ratio

Nevertheless, the structural integrity of structures such as bridges deteriorate over time, and they may no longer function as well as their originally performed. Due to an increasing number of cases of unsatisfactory performance, engineers began to investigate developing performance-based design principles as a replacement for, or at the very least as an alternative to, the current code-based techniques.

Regarding to this work. the existing imperfections of the current code are pointed to adopt a Malaysian National Annex Standards that follows EC8 with some modifications based on lessons learned from significant earthquakes in the recent few years. Thus, the aim of this work is as follow:

1. To investigate the amount of confining reinforcement for bridge seismic design at high seismic zone in Malaysia.

2. To determine the %reinforcement ratio in longitudinal rebars using EC8 based on Malaysia seismic prone area under different intensity measures.

3. Factors Affecting Confinement Reinforcement

The following parameters have been discovered to have an impact on the quantity of confinement reinforcement required:

- It is important to evaluate the real confining pressure or concrete compressive strength to tie strength because as yield strength increases, the quantity of confining reinforcement required decreases. High strength concrete may require additional constraining steel due to its brittleness.

- Columns with low compressive axial loads may not require as much confinement as columns with high axial loads. The effects of axial load are taken into consideration in the Canadian and New Zealand codes.

- The unconfined concrete in the cover part of the column spalls as a result of excessive load. Unconfined concrete in the column's cover portion begins to spall when the compressive strain in the concrete reaches a thickness of 0.003 to 0.005. When the area of unconfined concrete cover constitutes a larger fraction of the total concrete, the resulting loss will be significant.

- Researchers found that when the longitudinal reinforcement ratio is high, steel congestion concerns may arise. As a result, in these instances a larger column size is advised.

4. Bridge Design Criteria Listed in Seismic International Codes

It was around two decades ago that Ghobarah (2001) stated that agreement on defining performance thresholds, which matched damage states and design criteria, was needed. However, no agreement has been reached within the bridge community. Engineers and scientists use a wide range of performance-based design criteria. Current design codes and literature have been summarized and compared here. Keep in mind that this is not a detailed or comprehensive list of all seismic design codes, nor even a full description of all seismic design scenarios.

In 2014, the Canadian Highway Bridge Design Code incorporated performance-based design for the first time, and it has since gained widespread acceptance (CSA, 2014). According to the code, lower-level design is subject to a return time of 475 years, whereas upper-level design is subject to a return term of 2475 years. In lower-level design, there is no allowance for yielding. Extreme damage is permitted at the upper-design level of the design hierarchy. The steel strain, on the other hand, must not be greater than 0.05, and the core concrete must not be crushed.

According to Japanese Design Specifications for Highway Bridge a multi-level seismic design method is employed in Japan (Kuwabara et al., 2013). It distinguishes between two types of earthquake ground motions

(Level 1 and Level 2). Level 1 earthquakes are highly expected to occur during the bridge's design life, and the bridge's primary design criterion is that it sustains minimal damage. Earthquake of level 2 occurs when the probability of occurrence is high while the intensity is high at the same time. The purpose of the design is to minimize damage to the highly important bridges while also preventing the failure of less significant bridges.

The seismic risk consequence is evaluated using deterministic methods in the Japanese Design Specifications, but probabilistic approaches are used in the majority of design codes in the United States, Canada, the European Union, and China. The deterministic technique is typically used to analyze a single scenario, whereas the probabilistic method is used to consider numerous situations with different probability associated with each scenario. Despite the fact that the probabilistic technique is the most often used seismic risk analysis method, the deterministic method is crucial in seismically active areas such as in Japan, knowing that the greatest seismic event are likely to strike about once every 300 years.

In New Zealand Bridge Manual (NZT, 2016) there are three damage levels: No traffic disturbance is allowed under minor earthquake loads. Only emergency traffic must be accommodated during a design level earthquake. If the bridge is damaged by major earthquake, it must be temporarily repaired for emergency traffic. The design return period for bridges on primary routes is 2500 years, which is a long time in seismic terms. When it comes to typical bridges, the design return period for earthquakes is 1000 years long.

After a decade of debate, China finally adopted a two-level seismic structural bridge design concept in 2008 (China-MOT, 2008). The importance of the bridge is divided into four groups, ranging from Class A to Class D. Single span lengths of Class A bridges are greater than 150 m, while single span lengths of Class B bridges are less than 150 m. Class C and D bridges are those on less-traveled routes with shorter spans, and they are the most common type. As defined by the Chinese bridge design code, Class B bridges must have a return term of 75 to 100 years without damage at the lowest hazard level in order to be considered structurally sound. Class B bridges are designed to sustain a return duration of 1000 to 2000 years without collapsing at the highest degree of hazard without deterioration in structural integrity.

Eurocode uses a return duration of 475 years for upper-level design that is meant to ensure life safety. A 95-year return period is advised for low-level risks. In 2014, the American Concrete Institute (ACI 341) proposed a return period of 50 years for lower-level design and a return period of 2500-years for upper-level design.

When comparing different regulations and recommendations, it can be been realized that design principles are not compatible with one to another. Even when the damage states are referred to the same phrase (for example, minimum damage), the particular description may vary (different strain limits). Table 2 presents an overview of design criteria derived from a variety of different codes.

5. Damage Avoidance Design for Bridges

Bridges have generally performed well during earthquakes in the past. The earthquakes of 1989 Loma Prieta and 1994 Northridge, California, were both believed to have caused less than 5% damage to bridges (Basoz & Kiremidjian, 1998). According to Schanack et al. (2012), the majority of Chilean bridges had minimal damage after the 2010 earthquake. During the earthquakes that struck Christchurch in 2011, historic bridges with integral abutments performed admirably. However, it was revealed that the precast concrete bridges built by the city after the 1960s had considerable residual deformation, which led to the closure of the bridges. It is clear that the typical ductile design of structures has a limit in terms of seismic performance that may be achieved at a fair cost. The traditional ductile design attempts to dissipate seismic energy by including plastic hinges into the design, which are built into the structure. The use of performance-based design in ductile structures can help to enhance the management of damages, but it does not change the structural behavior in any significant way.

Researchers presented unique structures based on the damage avoidance design concept in order to substantially reduce earthquake losses in high earthquake prone areas. During the process of executing performance-based design, designers are not required to rely on prescriptive techniques included in design

rules (such as the application of force reduction factors); as a result, damage avoidance design may be an excellent alternative for unique buildings that are not generally ductile in nature (Skinner et al., 1976; Tyler, 1978; Mander and Cheng, 1997, Mander et al., 2009).

Base isolations are a widely acknowledged design technique for damage avoidance in structural engineering. Base isolation was first proposed more than a century ago (Buckle and Mayes, 1990), and it is still in use today. Breakle et al., (2006) identified several types of isolation bearings that were frequently utilized, including the lead-rubber bearing, the friction-pendulum bearing, and the isolator. Isolation bearings were first utilized in bridge retrofits, but they have now been used in a large number of new constructions as well. Isolation bearings isolate the vibrations of the superstructure from those of the substructures, hence reducing the force demands placed on the substructure. Increasing the fundamental periods of vibration is a common method for accomplishing this. Because of this, the components of isolation systems can move laterally in the event of a design earthquake, while maintaining acceptable stiffness under serviceability stresses and having adequate energy dissipation capacity. Base isolation has been demonstrated to be a highly effective method of reducing earthquake forces.

Based on the results of strong earthquakes (Hijikata et al., 2012; Kasai et al., 2013), base isolations have proven to be an effective method of seismic protection. The AASHTO Guide standards for seismic isolation design (AASHTO, 2014) can be considered one of the primary design specifications for bridge isolation in earthquakes.

In addition to the isolation, a post-tensioned rocking column, it is a revolutionary structural system that has undergone intensive research over the last few years. Post-tensioned rocking columns, as proposed by Mander and Cheng (1997), are designed to insulate buildings from seismic activity. Tendons were used to link the column to the footing and pier cap, which were both prefabricated. At the connection joints, no continuous reinforcing was applied since the joints were so small. Steel plates were installed on the ends of the column to prevent damage from swaying impacts. A number of scholars (Dawood et al., 2012; Sideris et al., 2014; Vassiliou & Makris, 2015; Zhang et al., 2016; Rahmzadeh et al., 2018) have conducted experimental and analytical studies that are similar to the ones described here. ACI-374 (2014) notes that rocking structures exhibit flag-shaped hysteretic behavior (Chou & Hsu, 2008), which can result in high displacement demands and protracted oscillations, resulting in low-cycle fatigue effects.

For rocking structures, researchers advise using extra dampening devices or energy dissipation bars (Cohagen, et al., 2008; Marriott et al., 2009; Metelli et al., 2011; Motaref et al., 2011; Thonstad et al., 2016). The additional components are designed to yield in the event of a earthquake, ensuring that the columns are not exceeded in capacity. Normally, energy dissipation reinforcement bars are left unbonded at the joints for a specified duration of time in order to avoid strain concentration induced by a significant joint opening. The regulations of the design code for rocking structures in bridges are not as comprehensive as those for buildings (Kurama et al., 2018). Damage avoidance design at joints in building structures has been thoroughly explored and is now incorporated into building code requirements (Mander et al., 2009). ACI-374 (2014) specifies acceptance criteria for moment frames that are not conventional in nature. The design technique for special frames with beams post-tensioned to concrete columns is contained in ACI-550.3 (2013), which is in accordance with the standards of ACI-374 (2014).

For damage avoidance design purposes, there are various different types that utilise unique structural systems, smart materials, or a combination of the two in order to achieve a sustainable bridge design. In most cases, these innovative materials were used in the plastic hinge regions or other key joint locations that were subjected to a great deal of seismic damage.

6. Code Specifications for Reinforcement Details.

There are several different approaches that are utilized all throughout the world to determine how much confinement is required, as well as which design elements are the most important.

The amount of parameters used in each approach, as well as the complexity of the equations, differed greatly from one another. Although this was the case, there was no widespread agreement on the most effective method for evaluating the appropriate amount of longitudinal and horizontal reinforcement when dealing with lateral stresses.

American Code

ACI specifies minimum area values for rectangular hoop reinforcement. These are denoted by Eq (1).

$$A_{sh} = 0.3\frac{sb_{ch}f_c^{'}}{f_{yt}}\left[\left(\frac{A_g}{A_{ch}}\right)-1\right] \qquad (1)$$

Where,

A_{sh} is the total area of hoop reinforcement,

$f_c^{'}$ is specified compressive strength of concrete,

f_{yt} is yield strength of transverse reinforcement,

A_g is gross area of concrete section,

A_{ch} is cross sectional area measured to the outside edges of transverse reinforcement,

s is center to center spacing of transverse reinforcement, and b_{ch} is the cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh} .

Where the transverse reinforcement ratio is shown as below:

$$\rho_s = 0.6\left(\frac{f_c^{'}}{f_{yt}}\right)[\frac{A_g}{A_{ch}}-1] \qquad (2)$$

Japan Society of Civil Engineers

Despite the lack of an equation for the spiral volumetric ratio or area of hoop reinforcement necessary for concrete columns, seismic requirements were included within the concrete design specifications. For spiral reinforced columns, Eq. (3) defines the reduced cross-sectional area of spirals, Aspe, which reduces the spiral area dependent on column spacing.

$$A_{spe} = \frac{\pi d_{sp}A_{sp}}{s} \leq 0.03A_c \qquad (3)$$

Where,

d_{sp} is the cross sectional diameter of spiral reinforced column,

A_{sp} is the cross-sectional area of spiral reinforcement,

A_c is the cross section of the column and s is the spacing of spiral reinforcement,

The transverse reinforcement ratio required for column design is:

$$\rho_s = \frac{4\pi A_{sp}}{s^2} \leq \frac{0.03\pi d_{sp}}{s} \qquad (4)$$

Canadian Standard Association Code (CSA)

As specified by the Canadian code, the overall cross-sectional area (A_{sh}) of reinforcing bars at the plastic hinge zone shall be regarded to be the greater of the values obtained from the following two equations:

$$A_{sh} = 0.30sh_c \frac{f_c^{'}}{f_y}\left(\frac{A_g}{A_c} - 1\right) \qquad (5)$$

$$A_{sh} = 0.12sh_c \frac{f_c^{'}}{f_y}\left(0.5 + \frac{1.25P_f}{\phi f_c^{'} A_g}\right) \qquad (6)$$

Where

$$\left(0.5 + \frac{1.25P_f}{\phi f_c^{'} A_g}\right) \geq 1.0$$

s is the vertical spacing of transverse reinforcement;

h_c is the core dimension of a tied column in the direction under consideration; and A_{sh} is the total cross-section of tie reinforcement.

The greater of the two equations below is the transverse reinforcement ratio at plastic hinges:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c^{'}}{f_y}\left(0.5 + \frac{1.25P_f}{\phi f_c^{'} A_g}\right) \qquad (7)$$

$$\rho_s = 0.12\frac{f_c^{'}}{f_y}\left(0.5 + \frac{1.25P_f}{\phi f_c^{'} A_g}\right) \qquad (8)$$

Where,

A_g is the gross cross-sectional area;

A_c is the area of the core of reinforced compression member;

$f_c^{'}$ is specified compressive strength of concrete;

f_y is yield strength of reinforcing bar;

P_f is factored axial load at a section at the ultimate limit state; and ϕ is resistance factor of concrete.

New Zealand (NZS3101)

According to the New Zealand concrete construction code, two formulae for transverse reinforcement ratio are relevant to places where ductile hinging is not expected (low seismic zone), while the other two equations are suitable to regions where ductile hinging is expected (moderate and high seismic zone). The following are the equations (9) through (12):

(A) For high seismic zoned (ductile hinging region)

$$\rho_s = \frac{2(1.3 - P_t m)}{3.3} \frac{A_g}{A_c} \frac{f_c^{'}}{f_{yt}} \frac{N^*}{\phi f_c^{'} A_g} - 0.012 \qquad (9)$$

Where,

A_g is gross area of column,

A_c is core area of column,

A_{st} is the total area of transverse reinforcement column reinforcement,

f_{yt} yield strength of transverse reinforcement,

N^* axial compressive strength load on column, and strength reduction factor 0.85, and f'_c is concrete strength, where $P_t = \frac{A_{st}}{A_g}$ and $m = \frac{f_y}{0.85 f'_c}$

$$\rho_s = \frac{2\sum A_b f_y}{96 f_{yt} d_b d''} \qquad (10)$$

Where,

d_b is diameter of reinforcing bar; and d'' is the depth of concrete core of column measured from center to center of rectangular or circular hoop.

(B) For low seismic zoned (non-ductile hinging region)

$$\rho_s = \frac{2(1.3 - P_t m)}{3.3} \frac{A_g}{A_c} \frac{f_c^{'}}{f_{yt}} \frac{N^*}{\phi f_c^{'} A_g} - 0.013 \qquad (11)$$

$$\rho_s = \frac{2\sum A_b f_y}{135 f_{yt} d_b d''} \qquad (12)$$

New Zealand's concrete structural code specifies that the cross-sectional area of rectangular hoop or tie reinforcement should be calculated as given in Eq (13).

$$A_{sh} = \frac{(1.3 - P_t m) s_h h''}{3.3} \frac{A_g}{A_c} \frac{f_c^{'}}{f_{yt}} \frac{N^*}{\phi f_c^{'} A_g} - 0.006 s_h h'' \qquad (13)$$

Where, s_h is the spacing between hoop sets, and h'' is the dimension of core of column at right angles to direction of transverse bars under consideration.

AASHTO LRFD Bridge Design Specifications

AASHTO (2013) specifies that the overall gross sectional area of a rectangular column with rectangular hoop reinforcement must fulfill in Eq (14).

$$A_{sh} \geq 0.30 s h_c \frac{f_c^{'}}{f_y} \left(\frac{A_g}{A_c} - 1 \right) \qquad (14)$$

Although, the required transverse reinforcement ratio is developed as shown in Eq. (15).

$$\rho_s = 0.24\frac{f_c^{'}}{f_{yt}} \qquad (15)$$

Eurocode 8 Part 2: BS EN 1998-2:2005

Enclosure of the concrete pier compression zone is required in potential hinge zones where the normalized axial force exceeds the limit: ηk = Ned/Acfck > 0.08. It is determined by the mechanical reinforcement ratio: ρ_w is the transverse reinforcement ratio equal to $\rho_w = \frac{A_{sw}}{S_L b}$ for rectangular sections and $\rho_w = \frac{4A_{sp}}{D_{sp}S_L}$ for circular sections.

Where;

A_{sw} is the total area of hoops or ties in the one direction of confinement.

S_L is the spacing of hoops or ties in the longitudinal direction.

b is the dimension of the concrete core perpendicular to the direction of the confinement under consideration, measured to the outside of the perimeter hoop.

Table 1 shows the amount of reinforcement for rectangular and circular cross section to be seismically designed using Eurocode 8 – Part 2: Bridges in case of ductile and limited ductile behaviors.

Table 1. Amount of reinforcement for rectangular and circular pier cross sections according to EC8-Part 2: Bridges Seismic Design Guidelines

Eurocode 8 – Design of structures for earthquake resistance – Part 2: Bridges		
Rectangular	$\omega_{wd,r} \geq \max\left[\omega_{w,req};\ 2/3\omega_{w,\min}\right]$ $\omega_{w,req} = \frac{A_c}{A_{cc}}\lambda\eta_k + 0.13\frac{f_{yd}}{f_{cd}}(\rho_L - 0.01)$	
Circular	$\omega_{wd,c} \geq \max\left[1.4\omega_{w,req};\ \omega_{w,\min}\right]$	
Seismic Behaviour	λ	$\omega_{w,\min}$
Ductile	0.37	0.18
Limited Ductile	0.28	0.12

Where, $\omega_{w,req}$ is required amount of reinforcement, ρ_L is the longitudinal reinforcement ratio, A_{cc} is cross-sectional area of the confined concrete core of the section, A_c is the area of gross concrete section.

Based on the aforementioned specifications guidelines that described the role of different parameters in designing the bridges to withstand against seismic loading are summarized in Table 2.

Table 2. Parameters impact on longitudinal and transversal confinement reinforcement in bridges pier design

Standard Specifications	Variables/Parameters					
	Material Compressive Strength	Steel Yielding Strength	Area Gross	Area Core	Axial Loading	Spacing between rebars
ACI						
JSCE						
CSA						
NZS						
AASHTO						
EC8						

7. Proposed Malaysia National Annex Reinforcement Ratio and Amount

7.1. Bridge Specifications, Modeling, and Analysis Approach

The selected structure is a prototype of a bridge located in Kuala Lumpur-Malaysia- called Samudera Bridge. The rectangular pier substructure is extracted from the selected bridge to be investigated under Ranau Earthquake that's happened on 2015 in Sabah in the Eastern part of Malaysia. The selected structure is a 3-span structure of total length equal to 82.50m with 10m wide prestressed concrete box girder deck has a 6% longitudinal slope and it is supported by two single column reinforced concrete piers of rectangular section (5.0m x 2.5m) and a clear height of 8.0m. The deck is monolithically connected to the piers and rests on the abutments through elastomeric bearings allowing movement of the deck in any direction. The bridge rests on firm soil and both piers and abutment have surface foundations.

A 28-day compressive concrete strength of 40MPa is required for the bridge piers. The longitudinal and transverse steel reinforcements have a yield strength of 500MPa. Steel has an elastic modulus of 200000 MPa while grade 40 concrete has a modulus of 35000MPa. Elastic elements with a modulus of elasticity of 1.0×10^{10} MPa. have been used to depict the cap beam elements and the vertical connection between the girder and the top of the pier. The bridge is assumed to be located at the highest seismic zone in Malaysia with a reference peak ground acceleration $a_{gR} = 0.16g$.

The bridge has been idealized as a multiple degree of freedom (MDOF) system and was developed as a three-dimensional model with each nodal point having six DOFs using CSI Bridge Software. Piers were modeled as "nonlinear Beam Column" components, in which the flexibility of the pier element is supposed to propagate over the entire element. The concrete and steel fibers used in the pier element were discretized. Suitable stress-strain relationships were used to simulate each fiber, which represented confined concrete, unconfined concrete, and reinforcing steel, among other things. **Fig. 4** shows the sectional view of the modeled bridge in idealized 3D model using FE platform.

The Samudra Bridge was designed approximately two decades ago according to the British Standards BS 5400: Part 2:1978 (British Standards Institution, 1978), BS 5400: Part 4:1990 (British Standards Institution, 1990), and BD 37/01 (the Highways Agency, 2001). For the record, seismic loading was not considered at the design stage because there was no such provision in the design code.

A detailed seismic assessment using Ranau ground motion is currently under way, and consequently relevant information concerning the longitudinal reinforcement ratio and amount of confinement of their piers is required in case of limited ductility and ductility. The purpose from using nonlinear time history analysis instead of response spectrum analysis in this case to show a realistic assessment that may ensure the seismic design requirements for the pier's longitudinal reinforcement ratio, and the amount of confinement. Consequently, this can ignore the combination of modal responses combination in X, Y, and Z directions Indeed, from this analysis a new modified formulas can be generated for Malaysia National Annex for bridge seismic design.

For the limit ductility assessment, the permanent loading actions is only considered in this analysis to obtain the required reinforcement for the rectangular pier section using the ultimate limit state load combination without considering the seismic actions. However, the ductility assessment considered several action effects during design including ground motion or seismic impacts. Table 3 describe the load combinations used in case both cases of ductility with its limitations. Refer to combination of actions in equations (6.10) to (6.12) in EN 1990.

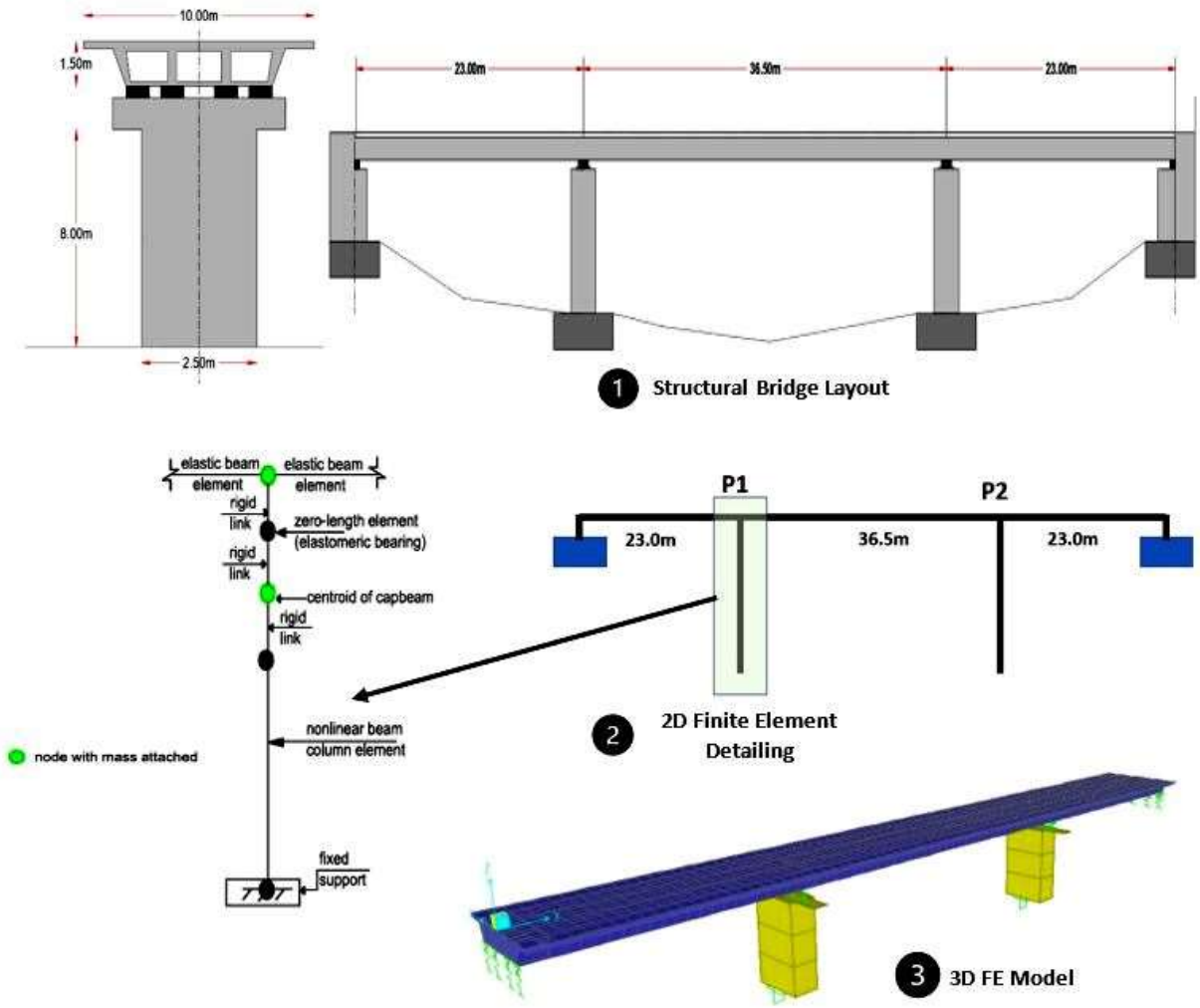

Fig. 4 Bridge 3D Model using Finite Element Software, and its sectional views.

Table 3. Load combination actions, and q-factor.

Type of Ductile Members	Seismic Behavior	
	Limited Ductile	**Ductile**
Vertical Piers in Bending	1.5	$3.5\lambda(\alpha s)$
αs : Ls/H is the shear span ratio of the pier, where L is the distance from the plastic hinge to the point of zero moment and h is the depth of the cross-section in the direction of flexure of the plastic hinges. For αs > 3.0, $\lambda(\alpha s)$ = 1.0, For 3.0> αs > 1.0, $\lambda(\alpha s)$ = Sqrt (αs /3),		

Load Combinations	Limited Ductility	Ductility
	$1.35G_k + 1.50Q_k$	$G_k+P_k+A_{ED}+ \psi_{21}Q_{1k}$

* G_k are the permanent loads, P_k is characteristics values of the prestressing after losses, $\psi_{21}= 0.2$, A_{ED} is seismic loads.

In this study, Ranau earthquake is considered in the selection of ground motion. The details of Ranau ground motion are as shown below and illustrated in **Fig. 5**.

Event	Ranau, Sabah (Station: KKM_HNE)
Year	2015
Magnitude	6.1
Rjb (km)	60 to 70
PGA (g)	0.123

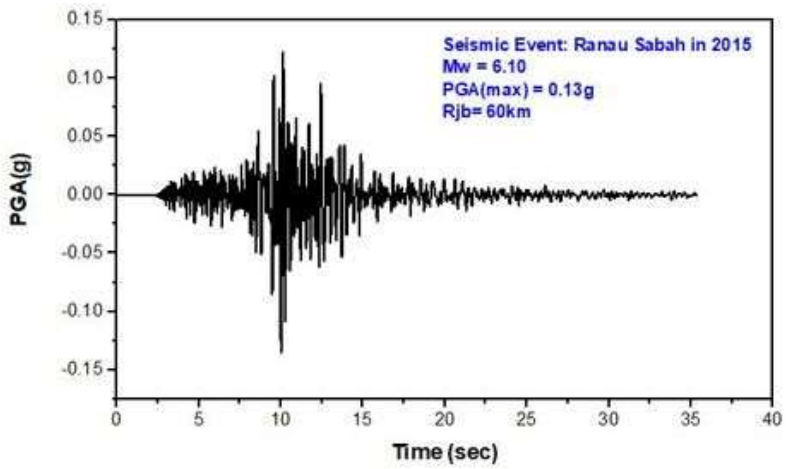

Fig. 5 Time history of Ranau ground motion seismic records in 2015

7.2. Modal Analysis

For any structure, the dynamics is governed by a simple equation of motion such as the one in (Eq. 16), with the damping coefficient assumed to be zero.

$$MU + KU = F(t) = 0 \qquad (16)$$

Mode shapes would be produced using the fundamental equation above, depending on the degree of freedom. Modal analysis is also very useful to recognize the boundary conditions and modeling quality since it allows you to see the harmonics that are obtained. In order to understand the possible behavior of any structure, modal analysis is essential. The frequency or time period of the analysis directly reflects the exactness of the numerical model and the boundary conditions of the structure. In fact, parametric investigations are the most important for reaching reliable conclusions. The characteristics of the first 4 modes are considered in the analysis as shown in Table 4. The shapes of the first 4 modes are presented in **Fig. 6.**

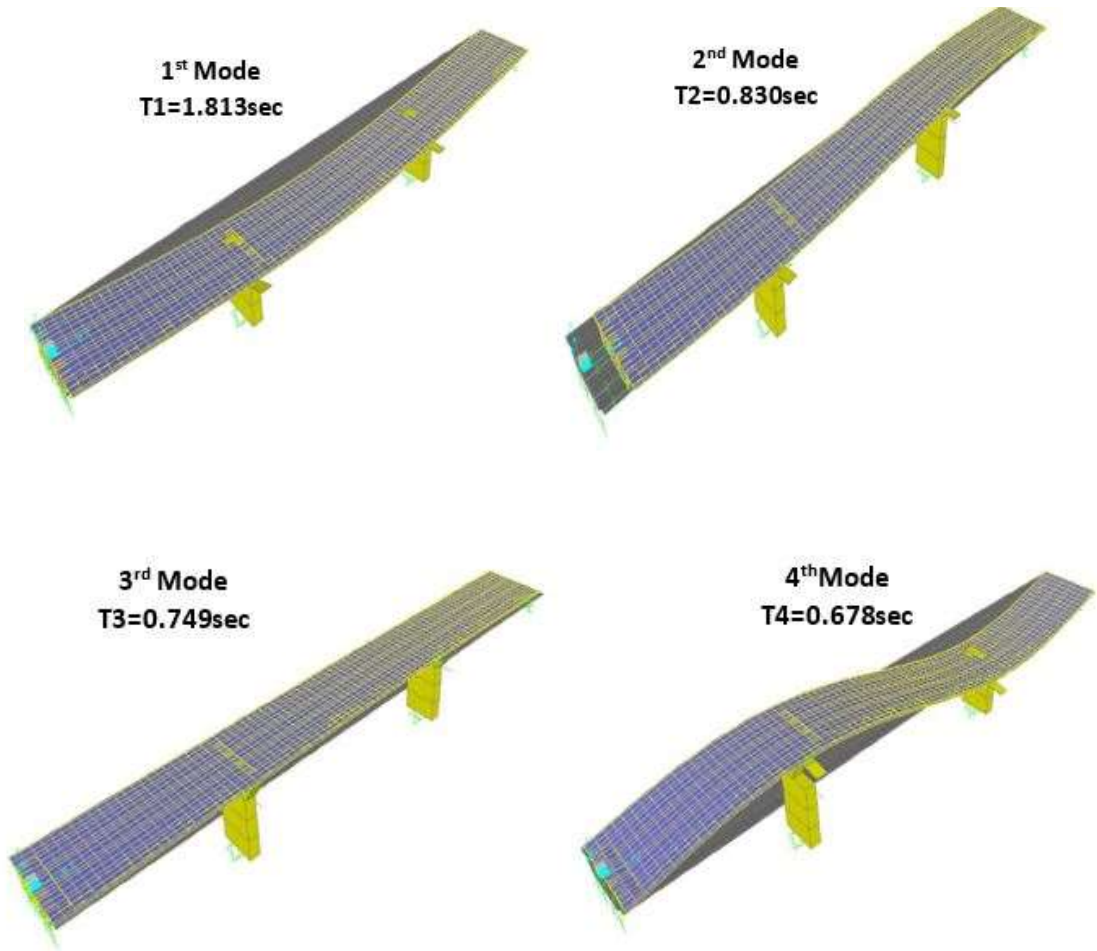

Fig. 6 Mode shapes for the selected bridge

Table 4. Mode shapes for the selected bridge.

Mode Number	Period in (sec)	Deformed Shape
1st Mode	1.813	Vertical Mode
2nd Mode	0.830	Longitudinal Mode
3rd Mode	0.749	Transverse Mode
4th Mode	0.678	Rotation Mode

7.3. Longitudinal Reinforcement –Ductility

After obtaining the seismic load combinations as described in Table 3 to have a ductile pier structure due to the combination of permanent gravitational loading and the selected ground motion records. The designed axial and moment internal forces are generated at different seismic intensity measures and clusters. The PGA as intensity measure is distributed into 4 seismic hazard classes such as; Medium - to - High having 12% of gravitational acceleration, followed by Medium prone area with 9%, g, after that in case of Low seismic prone with 6%, g, and eventually reaching to very low seismic zone with 3%, g. The designed axial load and the base moment in Y-direction are used in these 4 classes with maximum intensity of peak ground acceleration (PGA)max of each case. The reinforcement layers for the pier section 5.0m x 2.50m is represented in **Fig. 7**. The geometric properties of the rectangular concrete pier structural elements, with the input designed data are illustrated in Table 5 and Table 6, respectively. After performing the non-linear time history analysis, and generating the designed internal forces, Eurocode 2 and Eurocode 8 are used to obtain

the required longitudinal reinforcement in different seismic scaling scenarios of Ranau earthquake as shown in Table 7 and schematized in **Fig. 8**.

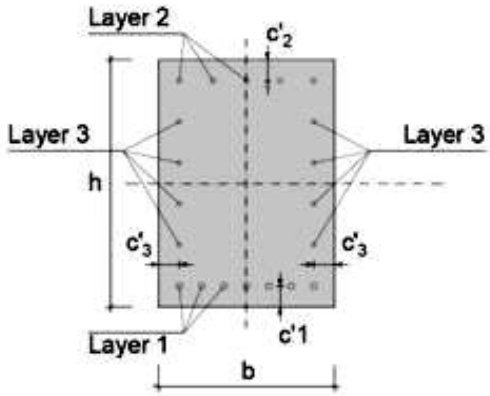

Fig. 7 Reinforcement Layer for the pier rectangular section

Table 5. Geometric properties for the selected concrete pier structural element

Distance from bar centroid of bottom reinforcement layer 1, 2, and 3 to bottom concrete edge (c')	75mm
Design value of reinforcement ultimate strain,	$\varepsilon_u = \varepsilon_y$
Coefficient taking account of long-term effects and loading effects on the compressive strength of concrete (for bending), α_{cc}	1.0
Concrete partial material safety factor γ_c	1.50
Reinforcement steel partial material safety factor γ_y	1.15
Area of gross cross-section	A = 12.5 m^2
Second moment of the area of gross cross-section (I)	: I = 26.041667 m^4
Total height of cross-section (H)	5.0 m
Total width of cross-section (B)	2.50 m

Table 6. Designed input data generated from Ranau earthquake at different intensity scales

PGA, (g)	**Intensity Classifications**	**Design value of the bending moment (M_{ED}), in (t.m)**	**Design value of the axial force (N_{ED}), in (t)**
12%	Medium-High	279350	1073
9%	Medium	257930	1073
6%	Low	85970	1073
3%	Very Low	17195	1073

Table 7. Total steel reinforcement and longitudinal reinforcement ratio for the selected Pier under Ranau seismic event with different seismic hazard classes.

PGA(g)	Area of Steel $As,_1$ (Layer 1)	Area of Steel $As,_2$ (Layer 2)	Area of Steel $As,_3$ (Layer 3)	Total Area of Steel Reinforcement	Longitudinal Reinforcement Ratio (ρ_L)
30%(g)	-	-	-	-	4%
12%(g)	694.20 cm^2	694.2cm^2	2776.79 cm^2	4165.19 cm^2	3.33%
9%(g)	749.60 cm^2	749.60 cm^2	2141.72cm^2	3640.92 cm^2	2.90%
6%(g)	490.08 cm^2	490.08 cm^2	1306.89 cm^2	2287.05cm^2	1.83%
3% (g)	201.77 cm^2	201.77 cm^2	645.65 cm^2	1049.19 cm^2	0.84%

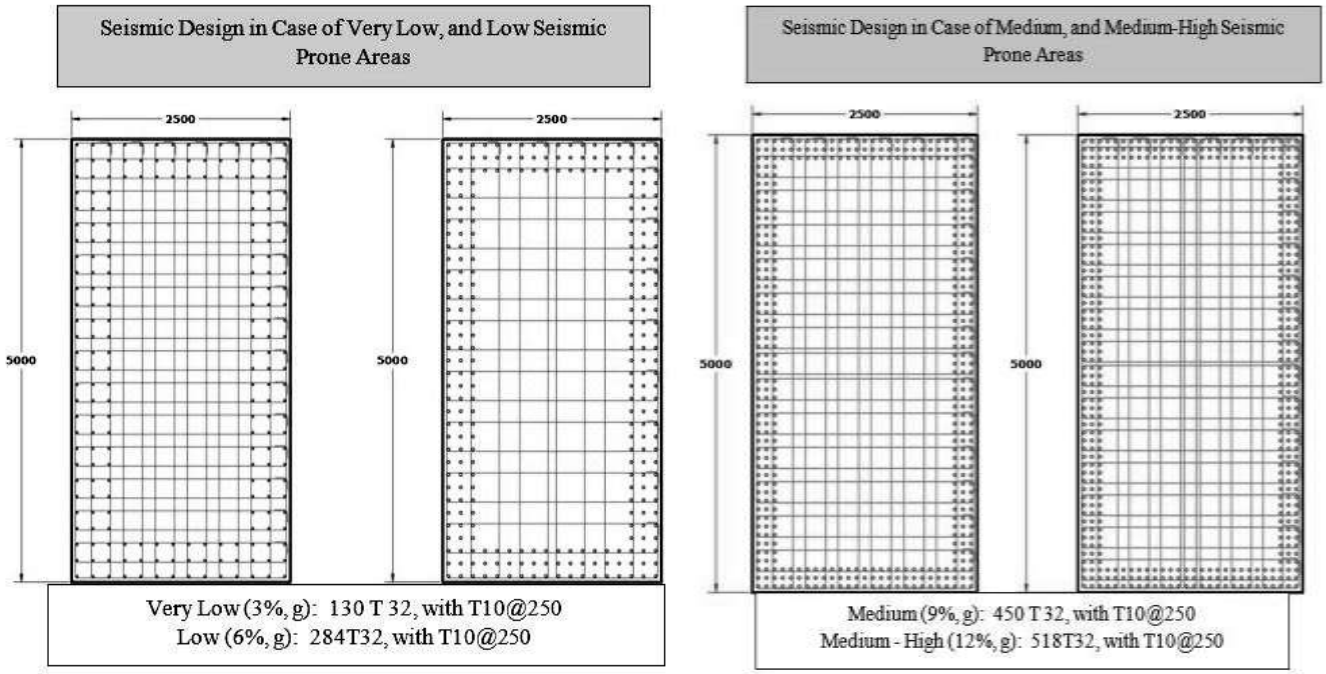

Fig. 8 Reinforcement detailing for the pier rectangular section in 4 seismic hazard scenarios.

After scaling Ranau ground motion into 4 seismic scenarios (Very Low, Low, Medium, and Medium-High) a range of longitudinal steel reinforcement is developed. In case of very low to low seismic hazard zone the design requirement of longitudinal rebars in the pier section must be with range of [0.84% to 1.83%] to have a ductile structure in low prone areas. On the other hand, in case of medium seismic zone, the structure needs 2.90% total longitudinal reinforcement at 9%, g intensity measure, as well it may increase to 3.33% when the structure is subjected medium-high seismic movements at 12%, g similar to that ground motion happened in Sabah East Malaysia in 2015. **Fig. 9** shows the range of reinforcement in different seismic hazard zones.

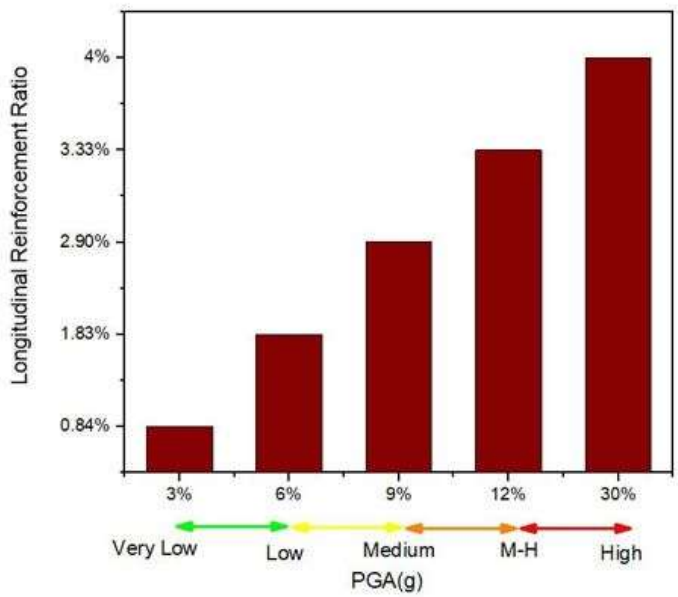

Fig. 9 Longitudinal reinforcement ratio in case of different seismic hazard zones.

8. Conclusion

According to the previous international codes and standards, it can be observed that the New Zealand, Canadian, and European codes, in contrast to the American and Japanese codes, take into consideration all of the variables and parameters that influence the confinement detailing for bridge seismic design. According to this perspective, Malaysia is a moderately seismically prone area bordered by nations with a high seismic zone, such as Indonesia, which follows the Eurocode in seismic design practices. After considering all of the parameters that influence confinement, as well as the fact that the majority of countries following EC8, it can be concluded that the values suggested in EC8 are the most appropriate for Malaysia.

From the nonlinear time history analysis obtained from Ranau seismic event, it can notice that the required longitudinal reinforcement for designing the pier structural element exceeds the minimum reinforcement ratio to be equal to 3.33%. While the amount of confinement is still the same as Eurocode 8 to have the minimum of 0.12.

Thus, the new formulas for Malaysia National Annex to use for seismic bridge design based on Ranau historical seismic event is as follow:

$$\omega_{w,req} = \frac{A_c}{A_{cc}} \lambda \eta_k + 0.13 \frac{f_{yd}}{f_{cd}} (0.023)$$

6.2.1.4(1)P: Type of Confinement Reinforcement

After performing the non-linear dynamic analysis (NL-DA) via simulating Ranau ground motion record the longitudinal reinforcement has been achieved for the pier structural element. The designed axial force N_{ED} and designed moment M_{ED} have been computed from the analysis.

In case of ductility the q-behavior factor must be calculated based on $3.5\lambda(\alpha s)$ to be more than 1.50 in case of moderate and high seismic zone. From this parametric analysis it can noticed that minimum reinforcement ratio to withstand seismic loading against Ranau it needs 3.33% as longitudinal rebars, with minimum amount

of confinement ($\omega_{w,req}$) of 0.12. Therefore, in case to have ductile superstructure such as pier it is recommended ρ_L=3.33% for moderate and high seismic zone.

6.5.1(1)P: Simplified verification rules for bridges of limited ductile behaviour in low seismicity cases

In case of limited ductility, the q-behavior factor must be less than or equal to 1.50 to be used for low seismicity zone whereases the seismic design and the confinement is not necessary to be included, however only the gravitational load represented by ultimate limit state (ULS) combination is used and neglecting the seismic loading impact on the structure. Thus, the confinement from transversal reinforcement and longitudinal reinforcement must follow the minimum reinforcement ratio of (ρ_L=1.0%) or less based on geometrical shape of the designed section.

The following are the required steel longitudinal reinforcement ratio for different seismic hazard scenarios:

1. In case of Very Low to Low seismic hazards (3%, g to 6%, g), the required reinforcement ratio must be with the range of [0.84% - 1.83%].
2. In case of Low to Medium seismic hazards (6%, g to 9%, g), the required reinforcement ratio must be with the range of [1.83% - 2.90%].
3. In case of Medium to Medium-High seismic hazards (9%, g to 12%, g), the required reinforcement ratio must be with the range of [2.90% - 3.33%].
4. In case of High seismic hazards (30%, g) and above, the required reinforcement ratio must be with the maximum of 4%, and some codes mentioned it may reach 8%.

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