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Seismic Resistant Design of Structures in the Low to Medium Seismicity Countries: Flexible Structures for Economic Construction

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Abstract

Earthquake hazard mitigation aims to reduce the impact of future earthquake disasters. It is proposed to adopt the use of flexible structures in seismic resistant design for Malaysia. In this study pushover analysis is performed on a bridge structure to illustrate how flexible structures fulfil the performance objectives of surviving an earthquake, and help keep the construction cost of seismic resistant structures at a reasonable level. Two types of cross section of the Samudera Bridge are subjected to monotonically increasing lateral forces until a target displacement is reached. First, pushover analysis on the original cross section is performed as a control to observe the seismic performance of the structure at the site. Next, pushover analysis is performed on the modified cross section, which represents the flexible structure, to study its seismic performance and the economic impact of introducing flexible structures as an approach to control the construction cost of seismic resistant structures. The study seems to show that desired ductility for satisfactory seismic performance may be achieved by using flexible structures, and at the same time the increase in the construction cost of flexible structures is reasonably low and acceptable by the industry.

Keywords: Seismic Resistant Structures, Flexible Structures, Low to Medium Seismicity, Pushover Analysis

Introduction

Seismic hazard in Malaysia is irrefutable although Malaysia is sited within the stable Sunda shelf. This is evident from records of ground motions originated locally and from seismically active neighbouring countries whose active plates are at least 350 km away from Malaysia. It has been learnt from the 2001 Gujarat Earthquake that seismic waves can spread and travel a long distance; in the case of the 2001 Gujarat earthquake, 600 km from the epicentre (Bendick *et al*., 2001). It has also been observed that a high density earthquake can pose an impact to sites located up to 700km away from an earthquake epicentre (Megawati *et al*., 2005).

Hence, earthquake mitigation to reduce long-term hazard, for example by developing building codes and zoning regulations to protect the safety of public and private properties against the damaging effects of ground motions (Godschalk *et al*. 1999, Quarantelli 2003; Hermon, 2014) are indispensable. However, while the structural engineering community in Malaysia is receptive to performing earthquake design of structures, project owners have the perception that seismic resistant design will cause a hefty increase in the construction cost. Such perception has generally left project owners in a dilemma as to whether they should proceed with seismic design or otherwise.

Seismicity of Malaysia

Peninsular Malaysia is sited within the relatively stable crust of Indosina-Sundaland, approximately 500 km from the Sumatran subduction zone, the most active plate tectonic margins in the world (Peterson *et al*. 2004), and situated at least 350 km from the 1650km long Sumatran fault. Peninsular Malaysia belongs to

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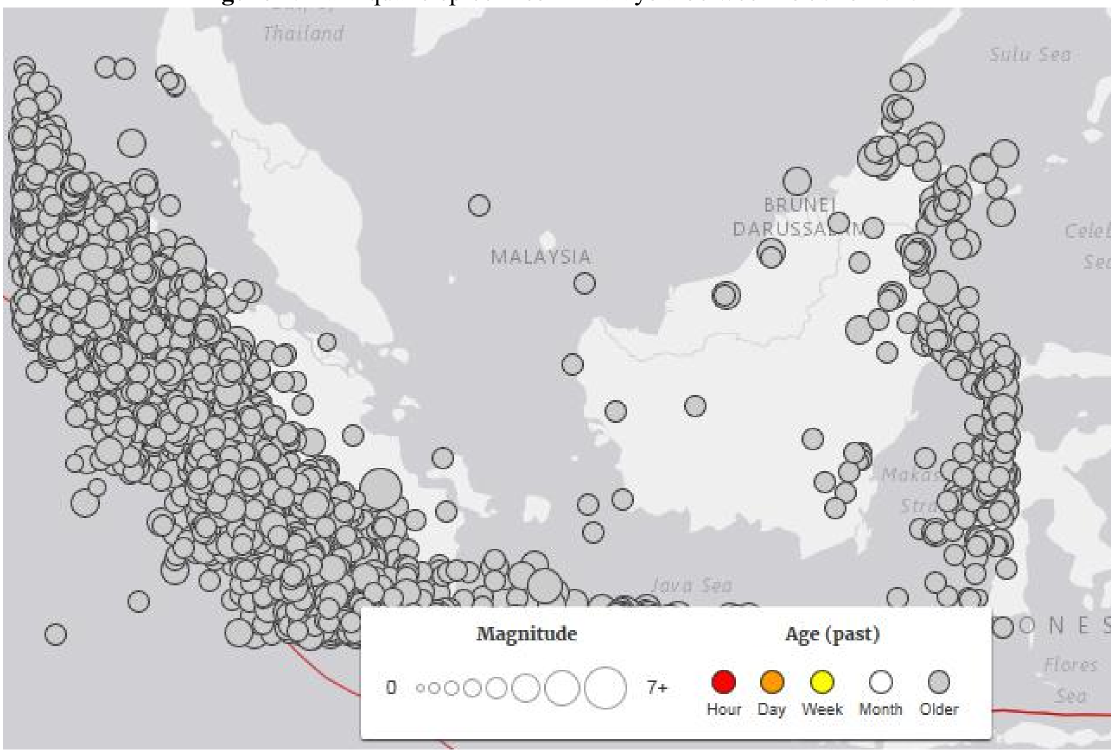
the low seismicity zone with no record of large, damaging earthquake. However, recent evidence has shown that several segments of the Bukitinggi fault zone are active and is a potential of future earthquakes (Mustaffa Kamal Shuib *et al*., 2017).

East Malaysia is located on the South-eastern Eurasian plate and is bordered by the tectonically active Philippine Plate and the Pacific Plate. Earthquakes in Sabah occurred as a result of collision between three tectonic plates, namely the Philippine and the Pacific Plate which move westward, colliding with the Eurasian plate. East Malaysia is classified as moderately active in seismicity and has experienced earthquakes of local origin, with maximum magnitude of 5.8 on Richter scale. Records also show that East Malaysia has been hit by strong ground motions originated from the Southern Philippines, Straits of Macassar, Sulu Sea and Celebes Sea.

Observation of earthquake events by the Malaysian Meteorological Department (MMD) show that between 1976 and December 2016 a total of 6108 ground motions have been recorded, of which 623 are of local origin with magnitudes ranging from 0.3 to 4.2 M_w. From a total of the 623 recorded local earthquakes 446 were recorded in Sabah, 74 ground motions were observed in Peninsular Malaysia, while 16 earthquakes were recorded in Sarawak. The most talked about earthquake in Malaysia is the June 5, 2015 Ranau earthquake which is the strongest to affect East Malaysia since the 1976 Lahad Datu earthquake. This earth. Records of earthquake epicentre, with minimum magnitude of M2.5, in Malaysia and those originated from neighbouring countries are as shown in Figure 1.

Figure 1. Earthquake epicentres in Malaysia between 1976 to 2016



(courtesy of United States Geological Survey)

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Earthquake Hazard Mitigation

Having been affected by both local and distant ground shakings, Malaysia acknowledges that earthquakes may cause casualties; have the potential to threaten the public safety and welfare; and destruct public and private properties. Such a concern is attributed to the fact that less than one percent of the buildings in Malaysia are seismic resistant (Taksiah Abdul Majid, 2009). This is because the common structural design practice in Malaysia is of the aseismic type whereby seismic force is not accounted for. With this situation buildings and infrastructures are vulnerable to the destructive effects of earthquake events. Hence, earthquake hazard mitigation is important to reduce the impact of future earthquake disasters well ahead of the disaster events, for example by actively monitoring seismic activities, developing building codes, and zoning regulations in securing a safe environment for the community.

Seismic Resistant Design

Seismic resistant design is a tool used to address earthquake hazard and minimizes risk to life and properties. Seismic resistant design requirements are typically presented in building code provisions as a guideline to designing structures for protection from potential destructive earthquakes in the future. For a developing country such as Malaysia investment in preparation for potential earthquake disaster is important so that loss of lives and properties can be minimized, and hence, the cost of recovery is lower than if mitigation is disregarded.

Over the past two decades a group of local researchers in Malaysia have worked on developing seismic hazard maps for Malaysia using the Probabilistic Seismic Hazard Analysis (PSHA) and earthquake data from the surrounding tectonic plate margins. The first seismic hazard maps in the form of Peak Ground Acceleration (PGA) maps of Malaysia were developed by the Malaysian Meteorological Department (MMD) in 2007 by converting seismic intensities into PGA. Currently PGA maps developed by local researchers are still under review by the government of Malaysia (Malaysian Standards Department) before they can be incorporated in the Malaysian National Annex to MS EN 1998-1: Eurocode 8: Design of Structures for Earthquake Resistance. It should also be noted that while there have been efforts by certain parties and researchers to construct earthquake design spectrum for Malaysia the government, as the authority, has yet to render the proposed design spectrum as suitable for use in Malaysia. Hence, engineers, practitioners and regulators do not have a confirmed guide that can be used in the design of earthquake resistant structures.

In view of the above mentioned situation a need arises to propose a simple approach which can be used as a design guide for seismic resistant structures which would meet the performance objectives of surviving an earthquake. The aim is to ensure that structures possess adequate ductility so that they can deform well into the inelastic range with acceptable stiffness reduction and without significant strength loss. Such behaviour is favourable because it enhances the seismic performance of a structure, even though damages are allowed to occur. However, a bigger challenge is to address the concern of some parties with regard to the increase in the construction cost due to the introduction of seismic resistant design in the industry. This study therefore has a two-prong objective, namely to demonstrate that seismic resistant design can result in flexible structures with lighter cross sections having the desired ductility and at the same time able to keep the construction cost at a reasonable level. The Samudera Bridge in Kuala Lumpur has been selected for seismic performance investigation using the non-linear static procedure. Figure 2 depicts the layout, elevation, and pier section of the bridge system.

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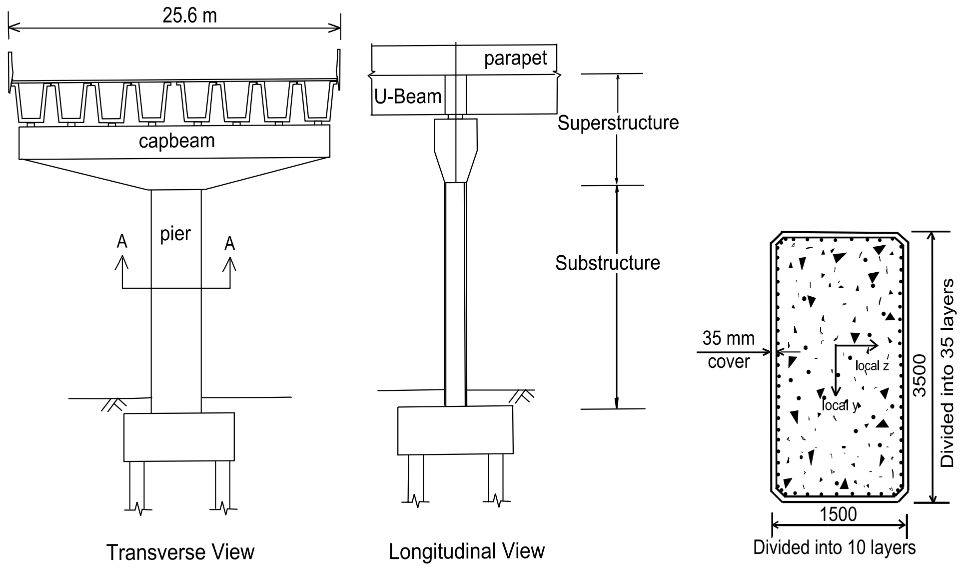

Figure 2. Sectional view of the Samudera Bridge: transverse view (left) and longitudinal view (right).

Seismic Performance Investigation

It is widely accepted that the performance of a structure during a large earthquake event depends very much upon the structure's properties beyond the initial yield point (Mo, 2003). Ductile structures are known to possess this attribute and are able to resist deformations in the inelastic range and thus, have higher chances of survival when subjected to large deformations caused by large earthquakes. Engineers now understand that they must design a structure to have ductility, rather than strength, in order to survive earthquake forces.

Following the growing interest in the performance based concept in seismic design an investigation will be carried out to evaluate the seismic performance of an existing multi-span reinforced concrete bridge structure which has been designed aseismically by employing the non-linear pushover analysis (NSP). The procedure for conducting NSP are described as in the following steps:

1. Develop model of the selected bridge as idealized in Figure 3. This is a one-pier system which represents the multi-span bridge system chosen for investigation;
2. Identify input information such as the target displacement, lateral load profile, and selection of an observation point to monitor the development of base shear capacity and displacement of mass curve;
3. Load the structure with maximum gravity load before subjecting it to monotonically increasing displacements in small increasing increments of 0.2mm, to a maximum displacement of 500mm; and
4. Examine the pushover curve (base shear versus displacement) for ultimate displacement at Ultimate Limit State.

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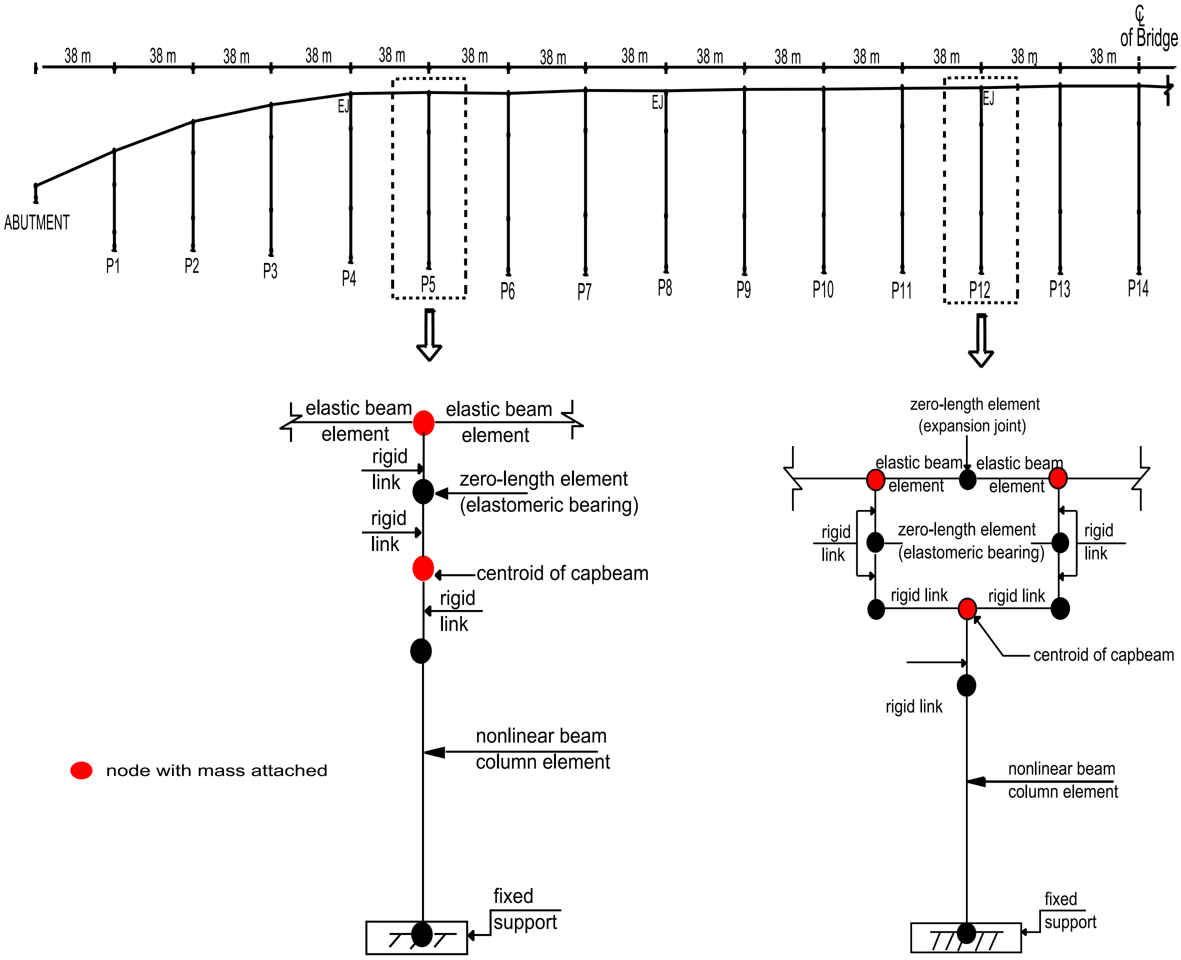

Figure 3. The Samudera Bridge as modeled and idealized in OpenSees

Specification of the Bridge

The Samudera Bridge, located approximately 15 km away from the Kuala Lumpur city centre, has been chosen as the subject to study the seismic performance and economy impact of seismic design of bridges in Malaysia. The Samudera Bridge is a 28-span, six-lane dual carriageway viaduct structure with varying pier heights, between 4.5 m and 10.4m (height measured to the neutral axis of the capbeam). The substructure consists of rectangular piers of size 3500x1500 mm.

The Samudera 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.

Both piers are reinforced with basic longitudinal reinforcements equivalent to 68T40. Fixed piers have been detailed with an additional 20T40 and 36T32 longitudinal steel bars (total main bar is 88T40 + 36T32). Confinement of piers is provided by high yield rectangular hoops, of 10 mm diameter, spaced at 250

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mm and 150 mm in fixed and free piers, respectively. The sectional detailing and cross sections through piers are as illustrated in Figure 4.

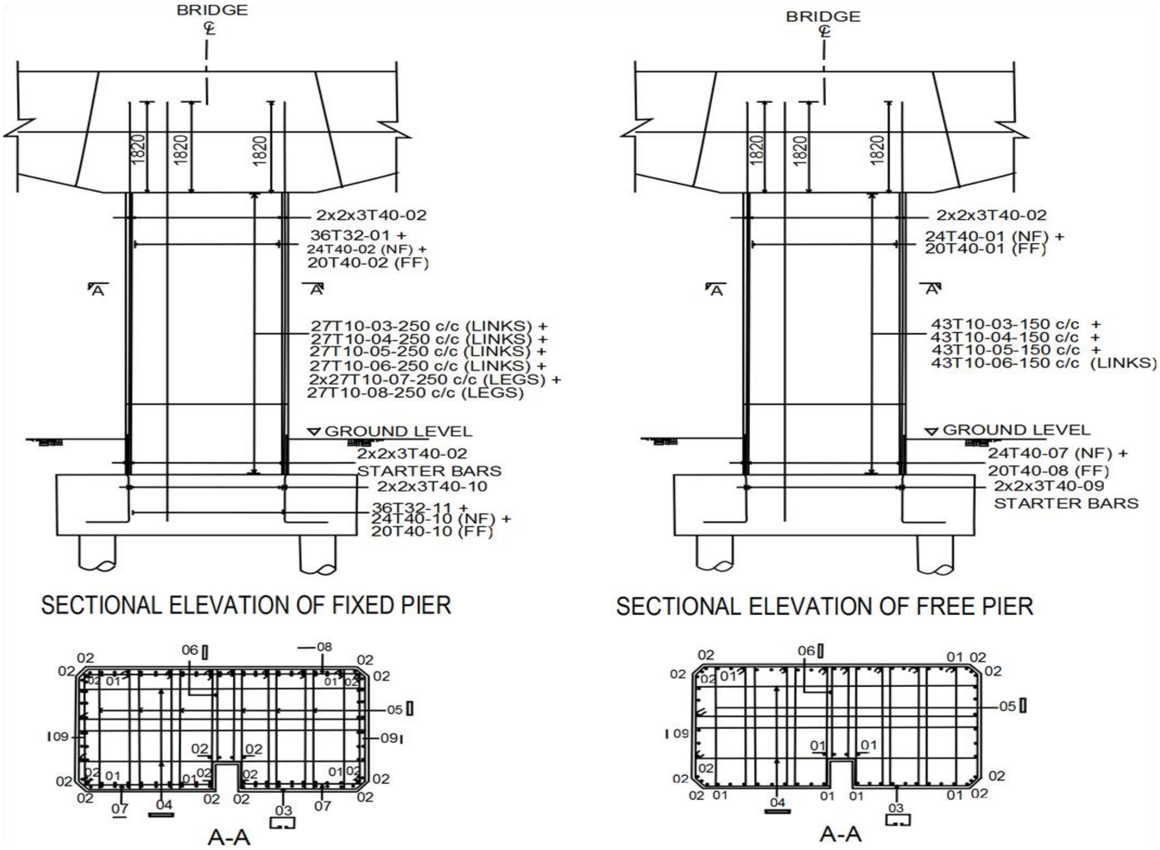

Figure 4. Sectional detailing of fixed (left) and free (right) piers.

The substructure consists of 27 rectangular cantilever piers, seven of which are of fixed type labelled as P2, P6, P10, P14, P18, P22, and P26. Fixed piers are equipped with bearings, which allow rotation about the longitudinal axis. The remaining piers are of free type where longitudinal movement and rotation about the longitudinal axis is allowed. For the purpose of this study the fixed pier is chosen as the subject for investigation because it has smaller amount of transverse reinforcement in comparison to the free pier. The as-built drawings on pier detailing show that the fixed piers are confined with 10 mm diameter transverse bars at 250 mm spacing. The free piers, on the other hand, have been provided with a richer amount of transverse reinforcement (T10 bars at 150 mm spacing), almost double the amount in fixed piers.

This study will employ the use of the original pier section and a modified section, which is flexible, but is ductile enough to survive seismic forces. Furthermore, to illustrate how seismic resistant design may not necessarily end up with skyrocketing construction cost, this study has chosen to educate readers that the use of flexible piers in a bridge may help in keeping the construction cost of seismic resistant structures at a reasonable level.

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Material Specifications

The bridge piers have a 28-day compressive concrete strength of 40 N/mm^2. The yield strength of both longitudinal and transverse steel reinforcements is 460 N/mm^2. Elastic moduli for grade 40 concrete (concrete strength 40 N/mm^2), and steel are 31000 N/mm^2 and 200000 N/mm^2, respectively. The capbeam elements and the vertical connection between the girder and the top of the pier have been represented by elastic elements having modulus of elasticity of 1.0x10^{10} N/mm^2.

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 the OpenSees software (McKenna and Fenves, 2006). Masses were lumped at the deck and the neutral axis of the capbeam. The piers have a 28-day compressive concrete strength of 40 N/mm^2. The yield strength of both longitudinal and transverse steel reinforcements is 460 N/mm^2. Elastic moduli for grade 40 concrete (concrete strength 40 N/mm^2), and steel are 31000 N/mm^2 and 200000 N/mm^2, respectively. The capbeam elements and the vertical connection between the girder and the top of the pier have been represented by elastic elements having modulus of elasticity of 1.0x10^{10} N/mm^2. The bridge structure had been constructed on limestone area, thus, pier bases were assumed fixed. The structural damping ratio of the bridge system was assumed at two percent (2 %).

Piers were modelled as “nonlinearBeamColumn” elements, whereby plasticity is assumed to spread in the entire pier element. The pier element was discretized as concrete and steel fibers. Each fiber was modelled with appropriate stress-strain relationships representing confined concrete, unconfined concrete and reinforcing steel. Confined concrete properties have been estimated based on the constitutive model proposed by Mander *et al.* (1988). “Concrete02” was selected to model both the concrete core and cover. The pier reinforcements were modelled as “Steel02”, and yielding of reinforcements would occur at a strain value of 0.0023. Figure 4 is an illustration of the idealized model of the bridge system.

In this study the ultimate limit state has been defined as the following: a reinforced concrete section is said to have reached the ultimate limit state as the strain in steel reaches 0.0023, while the concrete will fail when its peak compressive stress drops to a value of 80%. At the ultimate limit state, steel reinforcement would yield and crushing of concrete is expected. A detailed description of the procedure used in this study is as discussed below:

a. **Choice of pier**: Pier P18, also called the ‘original’ section, is selected for study mainly because it has lower transverse reinforcement than the free pier. The ‘original’ pier section has a rectangular cross-section 3500 mm x 1500 mm (depth x width). The transverse and longitudinal views of the bridge system; and the idealized pier section are as shown in Figure 1. The total longitudinal reinforcement provided in the modified section is 88T40 + 36T32. The pushover curves of the ‘original’ section of Pier P18 and that of the ‘modified’ section of Pier P18 are established. The modelling of pier P18 is explained in detailed in Section 3.2.3.

b. **Modified section**: Only one section shall be investigated. A flexible pier can be achieved by making the original section smaller. The limit of section reduction considers the provision in BS5400: Part 4 (1990). This guideline was used because the Samudera Bridge was designed to this design standard. Thus, consider the slenderness limit of:

$$l_o = 100b^2/h \quad \leq 60b \qquad (1)$$

where l_o, h and b are column clear height, the larger dimension of a column, and the smaller dimension of the column, respectively. This limit suggests a modified section of size 2800x1500 mm.

The revised pier reinforcement ratio in the modified section is based on the provision stated in BS 5400: Part 4 (1990) i.e. the minimum percentage of longitudinal reinforcement is 0.4%, but longitudinal reinforcement should not exceed 6%. The total longitudinal reinforcement provided in the modified section is 48T40 + 56T32.

c. **Calculate the confined concrete properties for modified section**: The confined concrete compressive strength and its ultimate strain are calculated based on the model proposed by Mander *et al.* (1988). This model suggests that the modified section has a confined concrete strength of 48 N/mm^2, and crushes at an ultimate limit state of 0.025.

d. **Conduct pushover analysis on the modified section**: Perform pushover analysis on the modified section in the longitudinal and transverse directions, separately. Apply gravity loading, followed by a lateral load equivalent to 5% of the superstructure weight. The pushover analysis shall be conducted to a maximum displacement of 500 mm with monitoring node at the deck level. Next, establish the force-deformation relationship of the pier. Observe and identify the displacement at the ultimate limit state. The ultimate limit state shall be defined at 80% its peak compressive strength. Yielding of steel reinforcement shall be taken to occur when the strain reaches 0.0023.

e. **Estimate the ductility:** The ductility of the original and modified sections shall be estimated in order to evaluate the 'flexibility' of the modified section. Flexible section should be able to deform larger than the original section. The relationship is as shown in Figure 5.

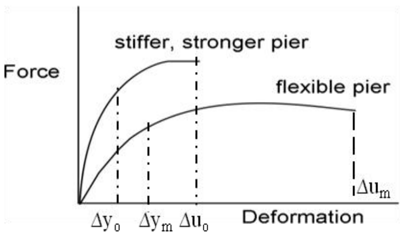

Figure 5 Force-deformation relationship, obtained from pushover analysis, used to estimate ductility

The estimate of displacement ductility factor μ_Δ is as given below

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \qquad \text{...} \quad (2)$$

where:

Δ_{yo} = yield displacement of the original section (mm)
Δ_{ym} = yield displacement of the modified section (mm)
Δ_{uo} = ultimate displacement of the original section (mm)
Δ_{um} = ultimate displacement of the modified section (mm)

f. **Run dynamic analysis to determine displacement response**: Compare the displacement response of the original and modified sections recorded by the dynamic analysis. If the responses are approximately equal, then the modified section can be assumed to have similar ability to resist seismic forces, as that by the original section. Thus, economically, the modified section may be used as an alternative section to the larger, stiffer original section.

g. **Calculate the amount of saving or additional spending:** The impact on the economy can be determined as a percentage of the total cost of the bridge structure. The prices of building materials such as steel reinforcement in piers and concrete shall refer to the contract document of the project

"Menaiktaraf Jalan Serta Pembinaan Bertingkat di Persimpangan Jalan Taman Samudera dan Perusahaan kecil di Batu Caves" (PWD, 2003).

Results of Discussion

Pushover analysis was performed on the original section, and the modified section. Figure 6 represents the ductility of the original and modified sections respectively. From Figure 6, it can be observed that the modified section is more ductile, as shown by the larger displacement capacity. The ductility factor of the original section is 2.86, while the modified section 2.57. This supports the fact that the modified section is more ductile than the original section.

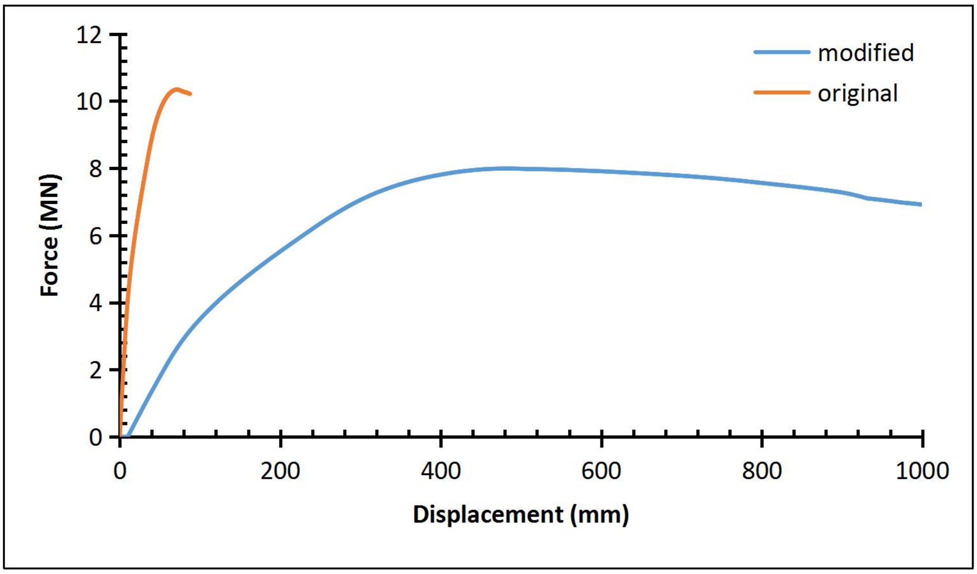

Figure 6. Comparison of ductility factor between the modified and original sections

Dynamic analysis is performed on the entire bridge system using the Kobe input motion. The modified section recorded a maximum displacement response of 205 mm, while the original section recorded 250 mm. The displacements recorded using dynamic analysis did not differ too much, and thus the modified section can be used as an economical alternative pier section in the bridge system. Most people perceive seismic resistant design as an expensive solution to mitigate seismic risk. This is true a few decades ago when construction of seismic resistant structures generally used bulky and stiffer structural elements. However, in recent years, the engineering community has understood the role of flexible elements in seismic resistant structures. Flexible elements have gained popularity due to their ability to withstand seismic forces with smaller sections, which are ductile and more economical.

This study has attempted to use the flexible pier element as a strategy to illustrate how smaller sections can survive an earthquake excitation, and offer an economical solution to the construction industry. The contract document prepared for the Samudera Bridge project is used as a reference for the cost of material. The main materials involved in the calculation of savings or spending, due to the implementation of the seismic resistant design are concrete and reinforcements. The currency used in the calculation of addition or savings in the construction cost is the Malaysian Ringgit or MYR.

The cost reference for concrete and reinforcements are MYR 230 per cubic metre of concrete and MYR 1.80 per kg of steel reinforcements. The total reduction of concrete volume from 27 piers due to downsizing of pier size is 233 cubic metre. This is translated as a saving of MYR 54,000. With smaller pier sections, less longitudinal reinforcements are used and as a result, there is a reduction of 19679 kg of T40 bars and 6816 kg of T32 longitudinal bars. As such, there is a further savings of MYR 48,000. In fixed piers, transverse reinforcements have been added to enhance their seismic performance, and due to that, an additional 920 kg T10 bars have been used, which marks a spending of MYR 1655. It can be demonstrated that by using flexible pier sections in the construction of the bridge, a savings of MYR 100,000 is possible.

Apart from these savings, smaller pier sections require the use of smaller foundations and pile size. Thus, flexible sections would result in further savings. However, one must realize that due to a reduction in pier size, displacement response may increase, and thus fail-safe mechanism may be required. Fail-safe mechanism secures bridges from unexpected large earthquakes. It is important to consider the installation of fail-safe mechanism in bridges and an allocation for it should be made in the construction of bridges. This budget can be taken from the savings on material cost, calculated earlier, and can be added when necessary.

Since there has been no reference, in Malaysia, of the cost of fail-safe mechanism to protect a bridge from the damaging effects of seismic forces, this study will consider the cost of elastomeric bearings to estimate the cost of fail-safe mechanism. Noting that fail-safe mechanisms shall be provided at locations of expansion joints, they will be installed at piers P4, P8, P12, P16, P20 and P24. Based on the highest price of a piece of 600 (W) x600 (L) x200 (H) mm elastomeric bearing, which is MYR 1500, a pier requires a cost of MYR24,000 for fail-safe mechanism. Thus, the total cost to protect the bridge from unexpected large earthquakes, which may result in large displacement response, is approximately MYR150,000. This cost is calculated based on the material cost of the year 2003 and may vary due to price change in the market.

Assuming the calculation above, the cost of seismic resistant design for the Samudera Bridge is 0.4%. However, this is only the price of material alone, and after an addition of the installation cost of fail-safe mechanism, the total cost of introducing seismic resistant design and construction may be estimated at 0.5%, but not to exceed one percent. It is noteworthy that although the use of flexible piers in bridges may seem appealing, one should realize that flexible piers may experience larger displacement response under an earthquake excitation. Thus, flexible piers should be considered together with the application of fail-safe mechanism. In fact, fail-safe mechanism must be compulsory in all seismic regions, be it of high, moderate or low seismicity. This is because earthquakes cannot be predicted, thus, preparation for the worst is necessary.

Conclusions

Essentially, seismic resistant design does not necessarily mean allocating a large budget in construction. Rather, a reliable and smart resistant design may keep the construction cost at an acceptable level. Of course, important structures may need extra strengthening but, basically an additional cost involved in seismic resistant design is due to providing and installing fail-safe mechanisms. Based on the illustration of savings in material cost and an additional budget for fail-safe mechanism, calculated in section 6.0, it is observed that the cost, which will be incurred to implement seismic resistant design, is less than one percent of the bridge structure cost. It is important to note that for a low seismicity region, flexible sections may be used to achieve ductility; however, designers must be aware that flexible sections may result in larger displacement response during an earthquake. Thus, fail-safe mechanisms must be provided. It is only natural to use the savings resulted from using flexible sections as an allocation for fail-safe mechanism.

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