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Seismic vulnerability analysis of Bankstown's West Terrace railway bridge

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Abstract. This paper highlights a case study that investigates the behaviour of existing bridge, West Terrace Bridge, induced by horizontal seismic loading. Unfortunately the lack of past information related to seismic activity within the NSW region has made it difficult to understand better the capacity of the structure if Earthquake occurs. The research was conducted through the University of Western Sydney in conjunction with Railcorp Australia, as part of disaster reduction preparedness program. The focus of seismic analyses was on the assessment of stress behaviour, induced by cyclic horizontal/vertical displacements, within the concrete slab and steel truss of the bridge under various Earthquake Year Return Intervals (YRI) of 1-100, 1-200, 1-250, 1-500, 1-800, 1-1000, 1-1500, 1-2000 and 1-2500. Furthermore the stresses and displacements were rigorously analysed through a parametric study conducted using different boundary conditions. The numerical analysis of the concrete slab and steel truss were performed through the finite element software, ABAQUS. The field measurements and observation had been used to validate the results drawn from the finite element simulation. It was illustrated that under a YRI of 1/1000 the bottom chord of the steel truss failed as the stress induced surpassed the ultimate stress capacity and the horizontal displacement exceeded the allowable displacement measured in the field observations whereas the vertical displacement remained within the previously observed limitations. Furthermore the parametric studies in this paper demonstrate that a change in boundary conditions alleviated the stress distribution throughout the structure allowing it to withstand a greater load induced by the earthquake YRI but ultimately failed when the maximum earthquake loading was applied. Therefore it was recommended to provide a gap of 50mm on the end of the concrete slab to allow the structure to displace without increasing the stress in the structure. Finally, this study has proposed a design chart to showcase the failure mode of the bridge when subjected to seismic loading.

Keywords: railway bridge; seismic analysis; vulnerability; Earthquakes; ballast-top bridge

1. Introduction

Railway bridges are complicated in nature as they have to accommodate not only passing

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trains, but also other essential systems such as overhead wiring structures, derailment protection structures, walkways and so on. There are many type of railway bridges including transom bridges, ballas-top bridges, direct-fixation slab-on-girders bridges and concrete viaducts. In this study, the bridge of interest has been designed to cater dual ballast-top rail tracks using direct-fixation concrete slabs resting on steel trusses. This research was conducted to study the effects of lateral seismic loading and interaction of a steel truss to concrete slab. A major earthquake has the potential to be catastrophic in nature and cause damage to lives, property and all aspects of the environment. Research conducted by Geoscience Australia (Leonard *et al*. 2012) has indicated that Australia has been excited by copious minor earthquakes in recent years. The gravest of these was the Newcastle earthquake in 1989. Fortunately, New South Wales has only experienced 10 earthquakes in the past 12 years and they have not been of severe magnitude to affect the integrity of Sydney structures. Unfortunately the lack of information regarding seismic activity within the Sydney region and use of outsourced guidelines has become a major risk and has made it difficult to observe and analyse the failures induced in a structure. Therefore this problem presents the opportunity to study the behaviour of a structure under lateral seismic loading through the use of 3D Finite Element Modelling (FEM). The structure under study is the West Terrace Bridge located in Bankstown, NSW. The aim of this research is to evaluate whether the existing design conditions are adequate for a range of lateral seismic loadings. The results derived from the simulation have the potential to provide additional information on stress distributions, reinforcements and failure modes for retrofitting.

Leonard and Luntz (2007) examined the uncertainty of predicting earthquakes in Australia. One of their main concerns was that there was not an adequate amount of data to be updated to the Australian earthquake hazard map. The 1991 published earthquake hazard map has made the assumption that earthquake activity over the next 500 years will be similar to activity over the previous 30-100 years. Although these assumptions are proved correct by certain “active” regions in Australia, this is not the case for most of Australia. Using the same assumptions that were used to create the 1991 earthquake hazard map, a map created for 1988 would rate both Newcastle and Tennant Creek (in NSW Australia) as low risk. Newcastle and Tennant Creek were the most damaging and largest earthquakes in the past 50 years. With these serious issues pointed out, Leonard and Luntz (2007) have outlined components for an earthquake risk model, which requires a diverse set of data to be fully functional. What was also pointed out was the fact that the historical record of Australian earthquakes spans only the last 100-200 years since European settlement. This provides a problem in creating a seismic hazard map, as there is a lack of recorded 5.5 or greater magnitude earthquakes (McCue *et al*. 2008).

Similarly, Gibson (2010) has examined the risk of earthquakes in Australia with the associated prevention and management plans. What is noteworthy is that the examination questions the validity of information being used in building codes and warning systems. Australia experiences very few earthquakes and even rarer experiences very infrequent large earthquakes; as a result the earthquake hazard is quite low. Therefore by considering this assumption a moderate risk criterion is adopted for building codes. The 1989 Newcastle earthquake was a magnitude 5.6 earthquake and was deemed a moderate risk. However the earthquake cost the lives of 13 people and had an estimated cost of over 2 billion dollars. Therefore, in order to reduce the risk to society and the surrounding environment, it is important to understand how the structure interacts and behaves as a result of the lateral loading so that the individual components of the structure can be studies and addressed.

This numerical study was carried out by using advanced nonlinear finite element modelling

Fig. 1 Bankstown West Terrace Railway Bridge

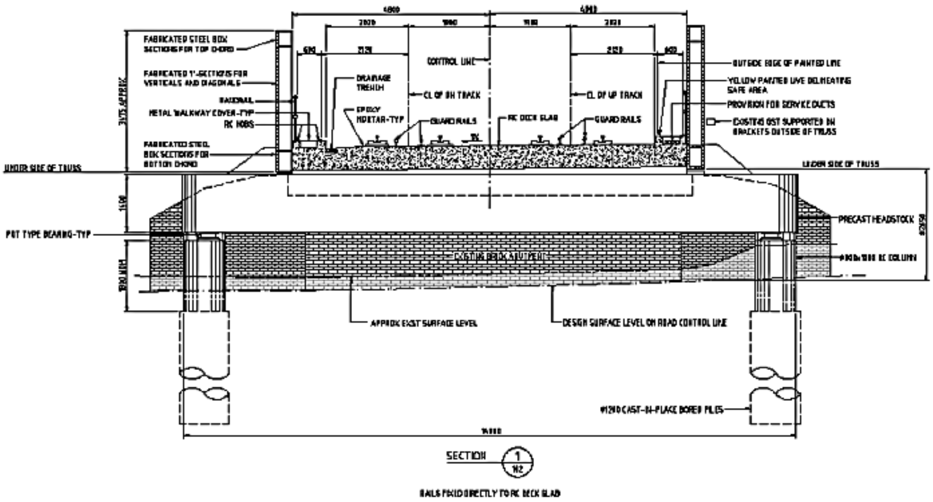


Fig. 2 Mesh grid of topographic model

using 3D elements. Fig. 1 shows the real existing bridge structure, which was adopted for this case study. The emphases of this study are placed on:

- Effect of lateral seismic loading on the bridge structure, and
- Interaction between the truss and concrete slab when subjected to lateral seismic loading.

2. Description of the existing bridge

The bridge under study consists of a main concrete slab, which has a span length of 18.6 m and width of 9.6 m. Fig. 2 illustrates the cross section of the bridge structure. The structure consists of ballast, concrete sleepers, rail pad, rail, a concrete slab and two arch trusses. Of relevance to the study is the shear connection between the concrete slab and steel trusses.

The main focus of the study is to analyze the behavior of the concrete slab and adverse effect on the steel trusses under increasing cyclic horizontal loading. The seismic loading ranges from 82.24 kN through to 2961 kN; this data is in accordance with the performed calculations set out by AS5100 and in conjunction with earthquake loading cycles set out by AISC (2005). Similarly, the boundary conditions where changed from a simply supported pin-roller configuration to a roller-roller configuration in the parametric study to observe the effects on the structure. The FEM modelled the longitudinal and lateral displacement of the arch trusses and concrete slab. Furthermore the analysis was simplified using a pressure loading only on the left face of the concrete slab. Simulation of the seismic event was carried out to demonstrate the intensity of an event by multiplying the maximum load achieved in that year return interval at a certain percentage through specific time steps.

3. Finite element modelling

Three-dimensional solid elements and two-dimensional truss elements, were used to model the bridge in order to achieve an accurate result using the finite element software ABAQUS. The accuracy of the analysis is dependent on the constitutive laws used to define the mechanical behaviour in materials such as concrete, steel reinforcement and steel, the constitutive laws are represented by the stress-strain relationships of the materials. In this paper, the mechanical behaviour at pre-seismic and post-seismic loading has been considered.

3.1 Structural steel

The behaviour of structural steel is an important consideration when analysing the West Terrace Bridge. The steel truss is connected to the concrete slab via shear stud connections and is an integral part in the analysis of the West Terrace Bridge. When steel is within its linear region it will behave elastically and return to its initial form when released. Research by Mirza (2008), Mirza and Uy (2009, 2010) and Mirza *et al*. (2016) stated that the ultimate tensile stress, plastic strain and ultimate strain values for steel can be calculated in Table 1.

In order to be able to run a finite element analysis, material input properties are required. The model itself contains two different properties that need to be taken into consideration when

Table 1 Stress-strain relationship for structural steel

Properties	**Steel truss**	**Reinforcement**
σ_{us}	$1.28_{\sigma ys}$	$1.28_{\sigma ys}$
ε_{ps}	$10\ \varepsilon_{ys}$	$9\ \varepsilon_{ys}$
ε_{us}	$30\ \varepsilon_{ys}$	$40\ \varepsilon_{ys}$

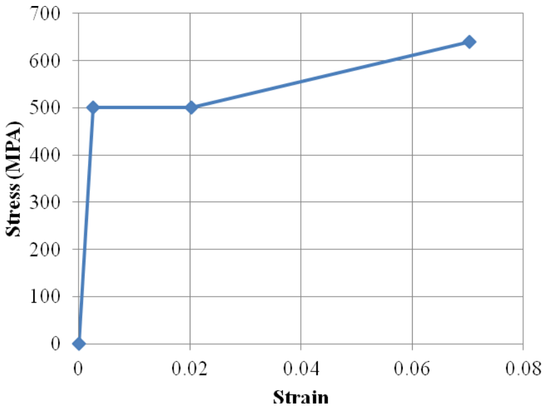


Fig. 3 Stress-strain relationship for steel truss

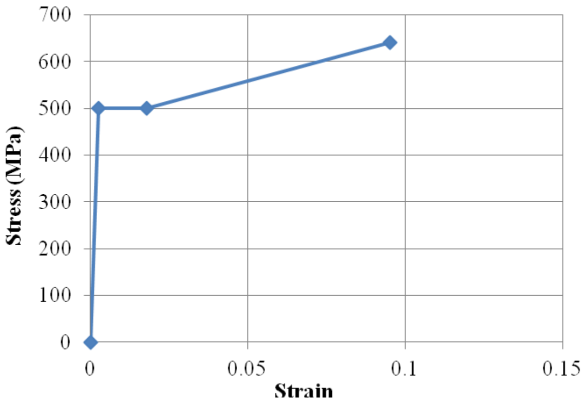


Fig. 4 Stress-strain relationship for steel reinforcement

creating the model structure. For the steel truss structure a typical stress strain relationship was considered. The steel truss was modelled into ABAQUS with the following properties in Fig. 3.

3.2 Steel reinforcement

The general stress-strain characteristics of steel reinforcement are assumed to be similar to structural steel as illustrated in Fig. 4. The behaviour demonstrates that it is initially elastic after which yielding and strain hardening develop. According to Loh *et al*. (2003), the stress-strain relationship for structural steel is represented as a simple elastic-plastic model with strain hardening. The mechanical behaviour for both compression and tension is assumed to be similar.

3.3 Concrete

It is vital to recognize the characteristics of the concrete material and identify its stress-strain relationship. Accurate modelling of concrete under seismic loading requires accounting of the following phenomena Légeron *et al*. (2005):

- Cracking in tension
- Confinement effect in compression
- Cyclic behaviour

The concrete slab being modelled in this study contains reinforcements and is connected to the steel truss via shear stud connections. In the study conducted by Baskar *et al*. (2002) they stated that due to the presence of a large number of shear studs in concrete, it behaved differently from plain or reinforced concrete. Carreira and Chu (1985) recommended the use of plain concrete where stress of a concrete member in compression is assumed to be linear up to a stress of 0.4*f'c*. Past this point stress is represented as a function of strain shown in Eq. (1).

$$\sigma_c = \frac{f'_c \gamma (\varepsilon_c / \varepsilon'_c)}{\gamma - 1 + (\varepsilon_c / \varepsilon'_c)^{\gamma}} \tag{1}$$

where

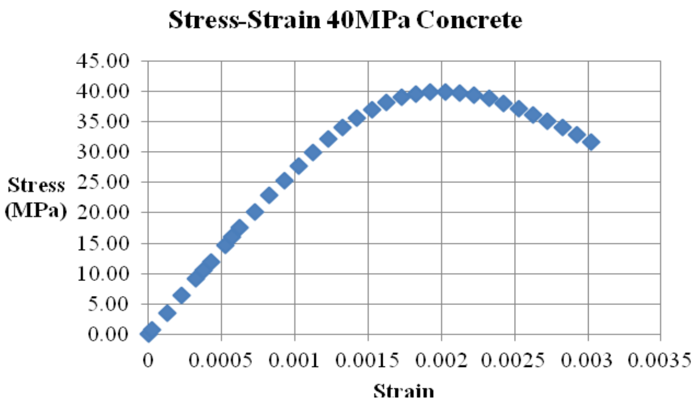


Fig. 5 40MPA concrete stress-strain diagram

$$\gamma = \left| \frac{f_c'}{32.4} \right|^3 + 1.55 \tag{2}$$

$$\varepsilon_c' = 0.002 \tag{3}$$

For concrete in tension Mirza (2008) mentioned that the tensile stress is assumed to increase linearly until the concrete cracks. Once cracking transpires the tensile stresses decrease linearly down to zero at a strain of 10 times the strain at cracking. It is also important to note that Légeron, *et al*. (2005) mention that when the crack in concrete is totally closed the concrete stiffness is no longer affected by the previous cycle in tension.

Stress-strain relationship's based on Carreira and Chu (1985) studies will be utilised within the analysis of the West Terrace Bridge and will provide a foundation for the stress-strain relationships required for the ABAQUS modelling (Singh and Zheng 2003, Liang *et al*. 2005). This stress was then used to compute the maximum compressive strength of the concrete throughout each seismic event, furthermore, it will encourage and provide accuracy in the ABAQUS modelling and results. Young's Modulus was calculated to be 28,069 MPa and the corresponding stress-strain graph is demonstrated in Fig. 5.

3.4 Finite element type, mesh, boundary conditions and loading conditions

In order to achieve an accurate result of the concrete slab and steel sections of the bridge the structure will use three dimensional brick elements and truss elements when assembling the bridge for simulation as shown in Fig. 6. Different types of finite element have been used where a 2-node linear 3-D Truss (T3D2) for the steel reinforcements and an 8-node linear brick (C3D8R) for the steel truss and concrete slab (Griffin *et al*. 2014b, 2015). The steel arch truss was fixed to the concrete slab as shear connectors through tie constraints. Similarly the individual members of the truss were fixed onto the bottom chord and arch through tie constraints. As a result of this, the model has constraints for translation and rotation of the boundary elements.

The concrete slab was considered to lay 300 mm on top of two precast headstock at either end of the bridge and was considered as simply supported. The boundary condition for left support was

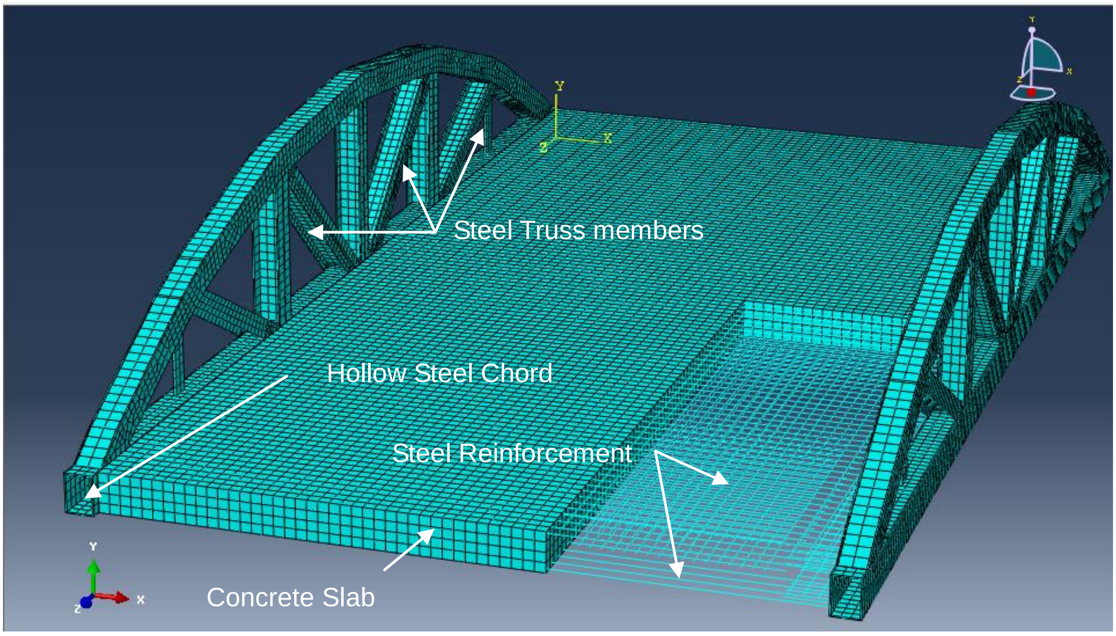


Fig. 6 Finite element mesh model

Table 2 Sensitivity analysis results

Mesh Size	Displacement
125	40.1
140	40.05
150	**39.85**
175	41.5
225	48.5
275	48
325	48
375	3.10E+03
475	3.10E+03
550	3.10E+03

set as the roller support with 'Displacement/Rotation' fixed for U1 and U2 (or x and y direction, respectively). The right support was set as a pin support with Symmetry/Antisymmetry/Encastre' type selecting XSYMM, U1=UR2=UR3=0. Similarly for the parametric study the boundary condition for left and right support was set as the roller support with 'Displacement/Rotation' fixed for U1 and U2.

The mesh is an important aspect of the FEM analysis because it determines the quality of the results as either accurate or inaccurate. In the model the size, shape and fineness of the mesh was arranged to provide an accurate result within a reasonable computation time. When the mesh was too fine the time to analyze the model was inefficient and not practical, moreover, it was found that a finer mesh did not necessarily mean a more accurate result. Ultimately, the conclusion drawn from sensitivity analysis indicated that a well-shaped mesh provides an accurate representation of results. The relationship between accuracy and mesh size was demonstrated in the sensitivity analysis by comparing displacement in mesh size to calculated displacement. Table 2 indicates the results obtained from the sensitivity analysis and also helped define the mesh sizes used for the structure.

The mesh size chosen was of 150 due to the displacement falling within a 10% margin of error from the calculated displacement. The loading conditions were applied and analyzed under varying earthquake year return intervals (YRI) in the horizontal axis of the structure. These where done in YRIs of 1-100,1-200,1-250, 1-500, 1-800, 1-1000, 1-1500, 1-2000 and 1-2500. Similarly a parametric study was undertaken considering a change in the support combinations and boundary conditions. The earthquake loadings where applied as indicated by AS5100.2 Bridge design (Standards Australia 2004).

4. Methodology

Horizontal Design Earthquake Force (*Fc*) can be obtained from two methods found in AS 1170.4 (Standards Australia 2004a). Using equations below, *Fc* values can be derived for both the General and Simplified method. Then, ultimately they will be compared with the Horizontal Design Earthquake (Hu*) obtained from AS 5100.2 (Standards Australia 2004b).

General method:

$$Fc = \text{afloor}\,[\,Ic \,.\, ac \,/\, Rc\,]\,Wc \quad \text{but} \quad < 0.5Wc \tag{4}$$

Simplified Method:

$$Fc = [\,kp \,.\, Z \,.\, \text{Ch}(0)\,]\,\text{ax}\,[\,Ic \,.\, ac \,/\, Rc\,]\,Wc \quad \text{but} \quad > 0.05Wc \tag{5}$$

Where afloor is the effective floor acceleration factor at height of the component of mass

Fc is the horizontal design earthquake force on the part of component

kp is the probability factor appropriate for the limit state under consideration

Z is the earthquake hazard factor which is equivalent to an acceleration coefficient

Ch(0) is the Bracketed value for the spectral shape factor for the period of zero seconds

ax is the height amplification factor at height, hx of the component centre of mass

Ic is the component importance factor

ac is the component amplification factor

Rc is the component ductility factor

Wc is the seismic weight of the part or component.

It is found that the Simplified method is preferable because the equation takes into account the total height of the structure. On this basis, Fc calculation is adequate when compared with the Horizontal Design Load (Hu*). The calculations imply that the load capacity of the structure is less than the design capacity for the greatest seismic intensity as demonstrated in Table 3.

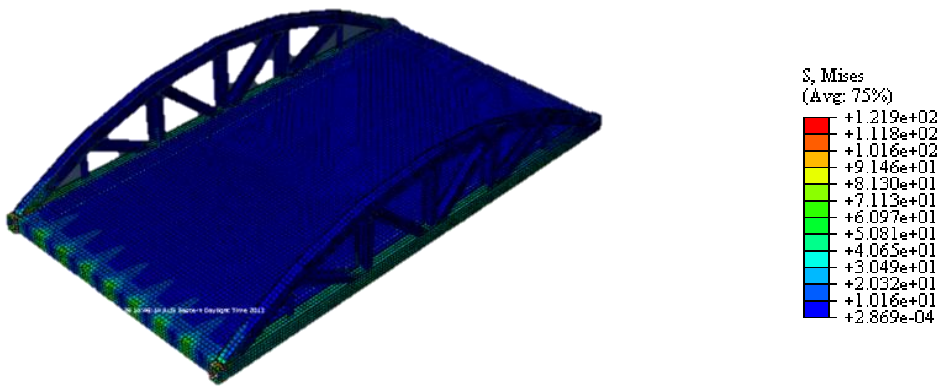
5. Results and discussion of pin-roller supported bridge

The existing West Terrace Bridge in Bankstown was designed to have an allowance of 30 mm of horizontal displacement and 5 mm vertical displacement. It is noted that there are no gaps at

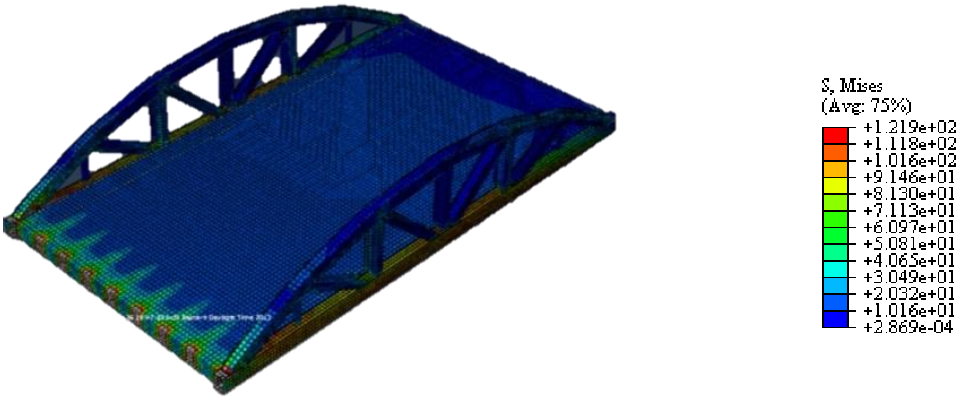
Table 3 Calculated results for simplified method

Seismic recurrence interval*	**Probability factor (AS1170.4)**	**Calculated horizontal design seismic load (kN)**	**Design Load (kN)**
1/100	0.50	125.70	1645.07
1/200	0.70	175.90	1645.07
1/250	0.75	188.50	1645.07
1/500	1.00	251.30	1645.07
1/800	1.25	314.20	1645.07
1/1000	1.30	326.70	1645.07
1/1500	1.50	377.00	1645.07
1/2000	1.70	427.30	1645.07
1/2500	1.80	452.40	1645.07

*associated with Earthquake Year Return Interval (YRI)



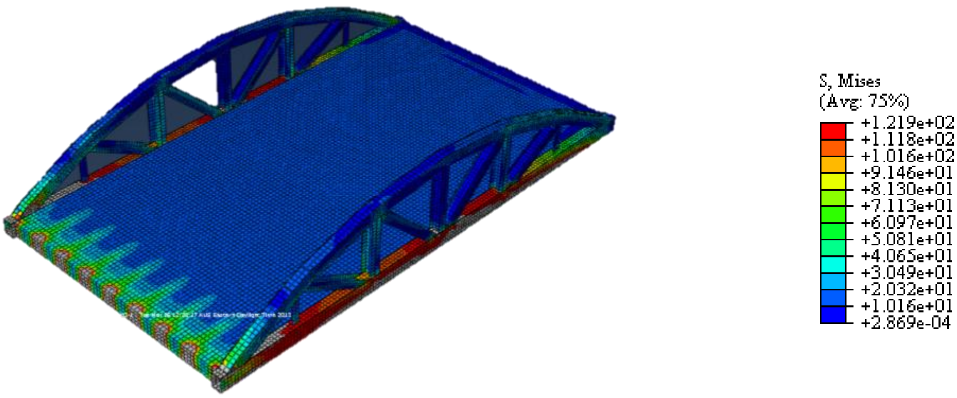
(a) 50% (left) and 100% (right) Stress distribution for 1/100YRI



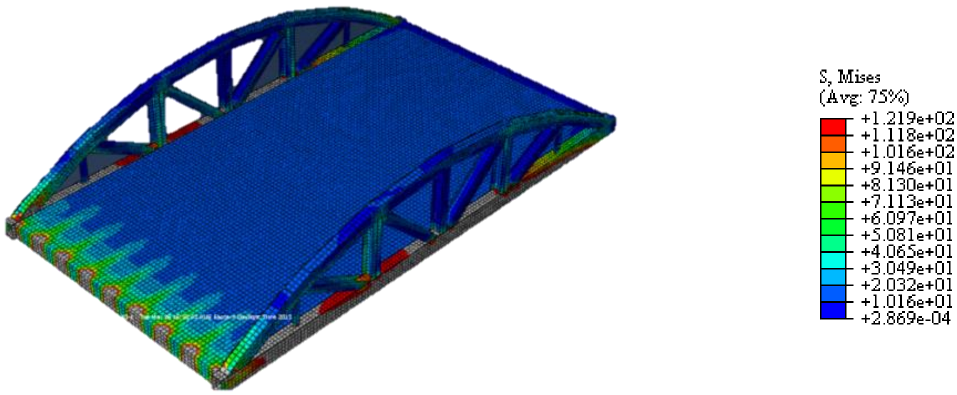
(b) 50% (left) and 100% (right) Stress distribution for 1/800YRI

Fig. 7 Finite element mesh model

either end of the slab connections to allow for horizontal displacement. Also, the steel arch truss was designed to be a non-load bearing member therefore none of the stresses experienced in the



(c) 50% (left) and 100% (right) Stress distribution for 1/1000YRI



(d) 50% (left) and 100% (right) Stress distribution for 1/2500YRI

Fig. 7 Continued

concrete slab should be transferred to the steel.

The stress distribution throughout the structure when for a 1-100 year seismic event is illustrated in Fig. 7(a). It is observed that a horizontal displacement of 8.83 mm and a vertical displacement of 0.215 mm lead to a stress increase in the structure. There is a transfer of energy from the slab, due to the displacement of the concrete, through to the steel truss. It can be seen that there is an increase in stress on the bottom chord of the steel. The stress increase can be correlated to the corresponding transfer of energy from the concrete slab through to the shear connectors. The steel truss remains in the elastic region as is indicated. It is important to emphasise this transfer of energy into the truss because the section is a non-load bearing member. Similarly, there is a minimal increase in stress inside of the concrete slab, which would be considered negligible compared to the stress increase in the steel truss, thus indicating that the concrete is still within the elastic region.

The stress distribution throughout the structure when for a 1-800 year seismic event is illustrated in Fig. 7(b). The figure demonstrates that the highest stresses transferred onto the truss occur within the first 14 m from the point of loading. A horizontal displacement of 22.06 mm and a vertical displacement of 0.537 mm lead to a much higher stress distribution on the structure. The stress is distributed to some parts of the concrete surface whilst the stress is distributed throughout

Table 4 Horizontal and vertical displacement responses to Earthquakes

Pin-Roller Configuration		
Period	Horizontal Displacement (mm)	Vertical Displacement (mm)
1/100	8.83	0.215
1/200	12.36	0.301
1/250	13.24	0.321
1/500	17.65	0.430
1/800	22.06	0.537
1/1000	22.92	0.548
1/1500	26.46	0.642
1/2000	30.00	0.731
1/2500	31.59	0.750

the entire bottom chord for the steel truss. Similarly, the simulation indicates that the bottom chord leaves is no longer in the elastic region and has moved towards the plastic region The simulation results indicate that the concrete slab remains under 40% of the ultimate capacity indicating that the section remains elastic.

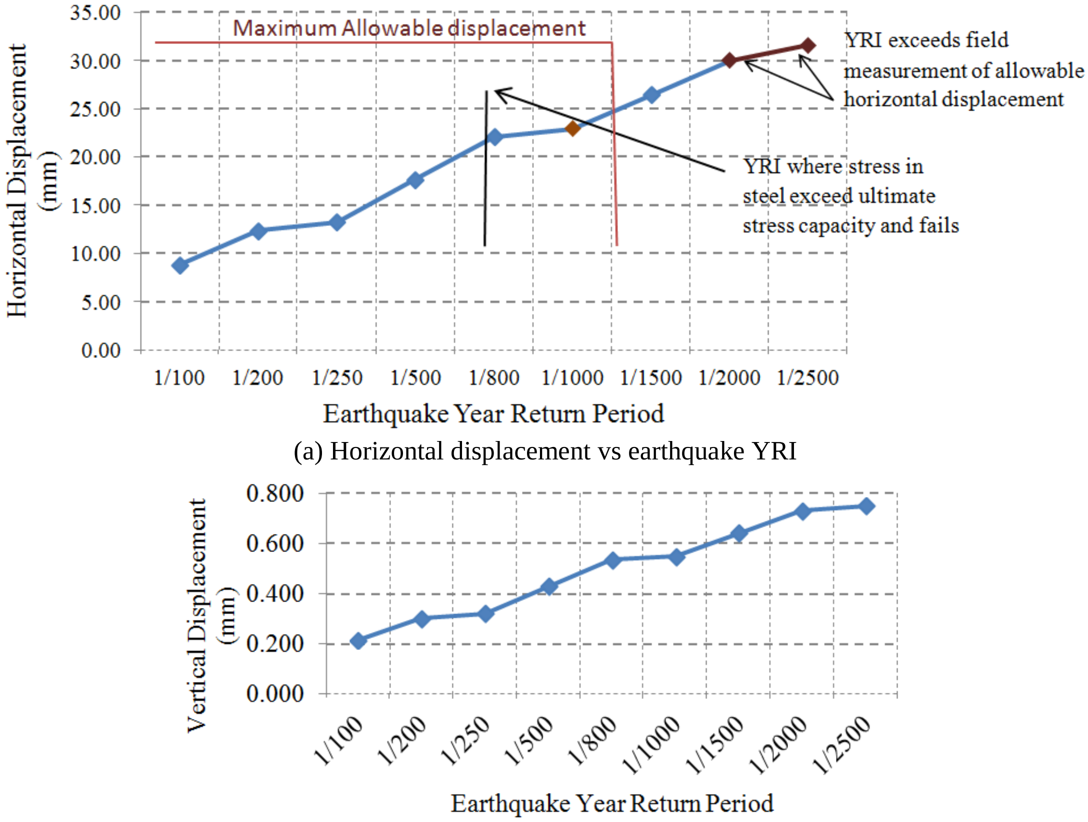
The stress distribution throughout the structure when for a 1-1000 year seismic event is illustrated in Fig. 7(c). A horizontal displacement of 22.96 mm and a vertical displacement of 0.548 mm were observed from the simulation. It is noticed that an increase of 4.1% in horizontal displacement and 2.1% in vertical displacement, compared to the displacements in of 22.06 mm and 0.537 mm respectively for the YRI of 1-800, leads to structural failure. The simulation indicates that the bottom chord leaves is no longer in the plastic region and has surpassed ultimate stress capacity. The stress is distributed to the majority of the concrete surface and the entire bottom chord for the steel truss. The simulation results indicate that, although there is a noticeable change in stress, the concrete slab remains under 40% of the ultimate capacity indicating that the section remains elastic.

The stress distribution throughout the structure when for a 1-1000 year seismic event is illustrated in Fig. 7(d). An observed horizontal displacement of 31.59 mm and vertical displacement of 0.75 mm was recorded. The figure demonstrates that there is a dramatic increase in stress on the bottom chord of the steel truss. Once the earthquake loading reaches 50% loading the bottom chord reaches ultimate. There is a significant increase of stress in the bottom chord of the truss the simulation indicates that the bottom chord is no longer in the ultimate stress capacity region but has exceeded it. The simulation results indicate that the concrete slab remains under 40% of the ultimate capacity suggesting that the section remains elastic.

The results in Table 4 indicate that the earthquake load applied is directly linked to the horizontal displacement experienced in the structure, as the earthquake YRI increases the displacement also increases.

5.1 Horizontal displacement for pin-roller configuration

Fig. 8(a) reveals that failure did not occur in the concrete slab at any stage instead the failure occurred when the stress in the bottom chord of the steel truss exceeded the ultimate capacity, this occurs at the YRI of 1-1000 with a horizontal displacement of 22.92 mm. The FEM results



(a) Horizontal displacement vs earthquake YRI

(b) Vertical displacement vs earthquake YRI

Fig. 8 Seismic response spectra of West Terrace railway bridge

demonstrate that the horizontal displacement remained within the limitation of 30 mm provided by the field observations. From the YRI of 1-1000 to 1-2500 the stress in the bottom chord of the steel exceeded the ultimate capacity indicating failure within all of these YRIs. Similarly, at the YRI of 1-2500, the horizontal displacement exceeds the field observations by 1.59 mm with a total of 31.59 mm.

It was observed that the design charts in Fig. 8 do not follow a linear relationship. The results obtained from the simulation illustrate the capacity of the West Terrace Bridge to withstand stresses incurred on the structure due to the horizontal and vertical displacements. The maximum horizontal displacement of 31.59 exceeded the field observations of 30mm, thus, requiring a gap of, 1.5 *x* the maximum horizontal displacement, 50 mm whereby the structure is able to displace without creating excessive stresses on the structural members. In summary, although the horizontal displacement does not exceed the field observation until 1-2500, the steel trusses are not adequately designed and will fail in the event of an YRI of 1-1000. Critical to the study is that the concrete section does not exceed 40% of the ultimate capacity indicating that it is well within the plastic region even in 1-2500 YRI.

5.2 Vertical displacement for pin-roller configuration

The simulation results in Fig. 8(b) reveal that the vertical displacement remains within the 5 mm limitation provided by the field observations. The maximum vertical displacement only reaches 15% of the allowable deflection. Consequently, it was found that the stresses induced by the vertical displacement on the structure do not impact the structure as much as the stresses induced by the horizontal displacement. Similarly the vertical displacement is less than the calculated displacement imposed by the dead load. Ultimately, the vertical displacement induced by the horizontal displacement remains within the limits provided by the field observations.

6. Results and discussion for parametric study (roller-roller supported bridge)

For the YRI of 1-100 through to 1-2000, the structural behaviours remain within the established field measurements but once the Earthquake event reaches the YRI of 1-2500 the structure surpasses the field measurement of 30mm, as tabulated in Table 5. This means that an adjustment needs to be made to the bridge to account for the 0.65 mm displacement, which exceeds the field measurement. However, it is found from the simulation that the steel truss reaches ultimate stress capacity at YRI 1-1500. As a result, it is very important to limit the energy transfer from the concrete slab into the steel truss. It is also critical to note that the concrete section does not exceed 40% of the ultimate capacity indicating that it is well within the plastic region even subjected to 1-2500 YRI.

6.1 Horizontal displacement for roller-roller configuration

The FEM results demonstrate that the concrete slab remains in the plastic region and does not fail at any earthquake YRI but there is a noticeable stress increase in the concrete when compared to the pin-roller configuration by 66.6%. As illustrated in Fig. 9, the FEM results also demonstrated that structural failure could occur when the stress in the bottom chord of the steel truss exceeded the ultimate capacity, and this could then happen at the YRI of 1-1500 with a horizontal displacement of 25.54 mm. The FEM results show that the horizontal displacement remains within the limitation of 30 mm provided by the field observations until the bridge is subjected to an Earthquake with a YRI of 1-2500. From the YRI of 1-1500 to 1-2500, it is

Table 5 Horizontal displacement responses to Earthquakes

Roller-Roller Configuration	
Period	Horizontal Displacement (mm)
1/100	8.51
1/200	11.92
1/250	12.77
1/500	17.03
1/800	21.29
1/1000	22.14
1/1500	25.54
1/2000	28.95
1/2500	30.65

important to note that the stress in the bottom chord of the steel has exceeded the ultimate capacity indicating structural failure when subjected to Earthquakes with all of these YRIs. Similarly, at the YRI of 1-2500, the horizontal displacement response exceeds the field observations by 0.65 mm with a total of 30.65 mm.

The FEM results illustrate that the West Terrace Bridge has certain capacity to withstand stresses incurred on the structure due to the horizontal displacements. As discussed earlier, the maximum horizontal displacement of 30.65 would thus require a new design gap of 45 mm whereby the structure is able to displace without creating excessive stresses on the structure. From this study, although the horizontal displacement did not exceed the field observation until 1-2500 YRI, the bridge structure is not adequately designed and the steel truss components will fail in the event of an earthquake YRI of 1-1500.

6.2 Vertical displacement for roller-roller configuration

The results for horizontal displacements are demonstrated in Table 6. It can be observed that the structure displaces less with a roller–roller support configuration and has a better stress

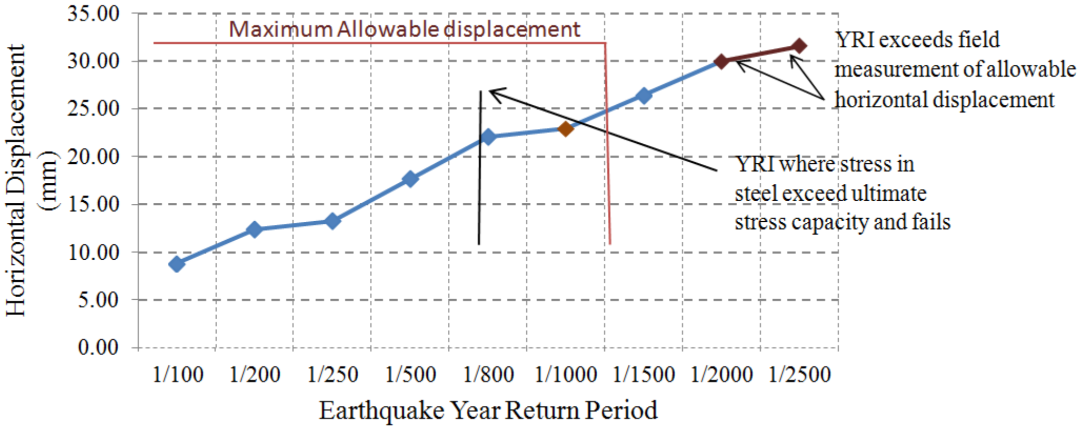


Fig. 9 Earthquake vulnerability with field allowance

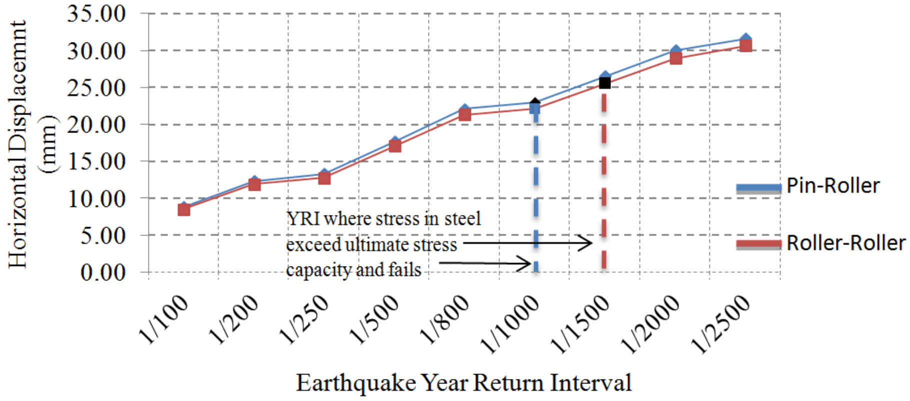


Fig. 10 Comparison of horizontal displacement design graph

Table 6 Comparison of support combinations and displacement for varying YRI's

Horizontal Displacement responses (mm)		
Period	Roller-Roller	Pin-Roller
1/100	8.51	8.83
1/200	11.92	12.36
1/250	12.77	13.24
1/500	17.03	17.65
1/800	21.29	22.06
1/1000	22.14	22.92
1/1500	25.54	26.46
1/2000	28.95	30.00
1/2500	30.65	31.59

distribution throughout the whole bridge. Ultimately the simulation, as shown in Fig. 10, demonstrates that changing the configuration from a pin-roller to a roller-roller allows the structure to bear a greater amount of earthquake loading before failure occurred in the steel truss members. Based on this study, the pin-roller configuration could potentially fail at a lower earthquake YRI, specifically at YRI 1/1000 compared to the YRI of 1/1500 for the roller-roller configurations due to a greater increase of tensile stress experienced in the steel chord.

The simulation also indicates that the von-mises stresses in concrete slab elements do not exceed 40% of the ultimate stress capacity for the maximum earthquake loading in combination with the maximum YRI of 1/2500. It can also be observed that structural failure could occur in the bottom chord of the steel truss. This occurs at the 1-1000 YRI for the pin support combination and at the 1-1500 YRI for the roller support combination.

7. Conclusions

The numerical modelling of the West Terrace Bridge revealed that the concrete slab and steel truss can experience significant increases of stresses and displacements within the structural members for each incremental earthquake Year Return Interval (YRI), which could potentially lead to structural failure in the steel chord. The numerical analyses of the structure were performed to simulate the behaviour of the railway bridge under varying horizontal earthquake loads. The Year Return Interval (YRI) ranges from 1-100 through to 1-2500 with the earthquake loads varying from 82.24 kN through to 2961 kN; this data is in accordance with the performed calculations set out by Australian Standard for Bridge Design (AS5100) and in conjunction with earthquake loading cycles set out by American Institute of Steel Construction. The displacements induced by the earthquakes were determined by applying a horizontal earthquake load as a cyclic pressure at the end face of the concrete slab. Although the design calculations indicated that the bridge structure had been designed adequately, the simulation actually demonstrated that the bridge structure will collapse when the YRI surpasses the loading equivalent to a 1/1000 event. Similarly the parametric study demonstrates that a roller support configuration will increase the resultant load equivalent to a 1/1500 event and thereby distribute the seismic load more effectively throughout the structural system. As a result, it can be seen that the calculations and displacements from the design method are insufficient to determine the progressive stage at which the bridge structure fails. Ultimately, the use of numerical simulations can provide better demonstration of the progressive collapses of the bridge structure with the variation of support conditions, for both the pin-roller and roller-roller support configurations.

From the FEM results it has been observed that the bridge structure can fail in both the pin-roller and roller-roller configuration due to the fact that there is not enough space for displacement to occur. Therefore, it would be recommended that the resilient behaviour of the railway bridge be studied prior to the design of construction gap, on the opposite side of the loading, (i.e., of 10 mm, 20 mm, 30 mm and 50 mm). Future studies will focus on different connection types between members and steel truss-concrete slab on the bridge as the connection has strong impact on the stress redistribution of the structural components and their consequent failure modes.

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