Title of the paper:

Seismic Assessment of Bridge over Spencer Gulf at Pt Augusta

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Summary of the paper contents:

This paper briefly discusses the risk analysis & assessment process, and then on the detailed seismic assessment of the Bridge over Spencer Gulf at Pt Augusta, which was constructed in 1970 with a length of 530 m. Each of the 20 simply-supported spans of the bridge is approximately 26 m long and adjacent spans share supports via notched-ended girders.

The paper presents the seismic assessment analysis approach, the tools used, and the results. The structural modelling and seismic input are discussed. The results of the Ruaumoko 3D nonlinear time-history analysis are summarised. Identified vulnerabilities of the bridge include the inadequate flexural strength of pier walls, shear capacity in bearings and girder seat length. Proposed retrofitting schemes are described.

Note: Due to the late receipt of this paper, it was not possible, in accordance with DEST requirements, to subject it to an independent critical review by two experts from the field in which the material was written.

Seismic Assessment of Bridge over Spencer Gulf at Pt Augusta, South Australia

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Abstract

The Department for Transport, Energy & Infrastructure (DTEI) recently undertook an earthquake risk study of all of their bridges to identify those at risk from a large seismic event. The study considered the earthquake hazard (magnitude of earthquake & foundation response), the structural risk (susceptibility to damage, dependant on structure type & configuration), and the bridge importance (strategically important to freight movement and/or post-disaster response). Results of the study identified and ranked a number of bridges that were to be analysed in more depth to assess their performance in a large seismic event. One of these structures was a major bridge on National Route One crossing the Spencer Gulf at Port Augusta.

The Port Augusta Bridge, with a length of 530 m, was constructed in 1970 and consists of 20 simply-supported spans that share supports via notched-ended girders. Each span is approximately 26 m long and contains five prestressed concrete girders. Total replacement cost is estimated at approximately $30m.

The bridge was modelled and analysed using Ruaumoko3D software for a seismic event with an annual probability of exceedance of 0.0005. Modelling of the bridge included assessment of superstructure and substructure elements. Performance deficiencies were identified and retrofitting schemes developed.

Keywords: bridge, seismic assessment, time-history analysis, Ruaumoko, seismic retrofit

1 INTRODUCTION

Bridges are lifeline structures in the transportation system. While new bridges are designed with improved seismic standards and details, many existing bridges pose a risk of failure in a large earthquake. Several recent destructive earthquakes, particularly the 1989 Loma Prieta and 1994 Northridge earthquakes in California, and the 1995 Kobe earthquake in Japan, have caused significant damage to highway structures. These events gave impetus to a serious review and seismic assessment of existing bridges. Investigations have indicated that bridges designed and constructed prior to the development of modern seismic design guidelines may be vulnerable to damage, in particular due to deficiencies in the strength and ductility of bridge piers, shear capacity of bearings, and inadequate girder seat lengths at the abutments and piers.

The Department for Transport, Energy and Infrastructure (DTEI) in SA recently undertook an earthquake risk study of all of their bridges to identify those at risk from a large seismic event. The study considered the earthquake hazard (magnitude of earthquake & foundation response), the structural risk (susceptibility to damage, dependant on structure type & configuration), and the bridge importance (strategically important to freight movement and/or post-disaster response). Results of the study identified and ranked a number of bridges that were to be analysed in more depth to assess their performance in a large seismic event. One of these structures was a major bridge on National Route One crossing the Spencer Gulf at Port Augusta. The bridge was identified as deficient under earthquake loading under the AusLink Perth-Adelaide Corridor Strategy (draft, 2007).

The Port Augusta Bridge was constructed in 1970, consisting of 20 simply-supported spans that share supports via notched-ended girders. Each span is approximately 26 m long and contains five prestressed concrete girders. Total replacement cost is estimated at approximately $30m. The main objectives of the analysis were to:

- determine the dynamic response of the bridge elements to earthquake ground motions with an annual probability of exceedance of 0.0005;
- determine the capacity of these bridge elements;
- identify any deficiencies and design appropriate retrofit schemes.

The analysis applied to the Port Augusta Bridge conformed to the Australian Standard for Earthquake loads (AS1700.4-1993) and Australian Standard for Bridge design Part 2: Design loads (AS5100.2-2004). To be consistent with the bridge design approach, some modifications were introduced to model an earthquake with an annual probability of exceedance of 0.0005 (whereas AS1170.4 adopts 0.002). The program Ruaumoko3D (Carr, 2003), nonlinear time-history analysis software, was used to model and analyse the bridge as a grillage model. Three artificial ground motions were produced by the program SIMQKE (Carr, 2003), which generated statistically independent artificial acceleration time histories to match the specified response spectrum. The travelling wave effect on the response of bridges has also been investigated.

2 DESCRIPTION OF THE PORT AUGUSTA BRIDGE

The Port Augusta bridge is a 20 span bridge simply supported between piers with a total length of 530 m. It consists of a slightly curved portion from the Adelaide abutment to pier No. 7 and a straight portion from pier No. 8 to the Whyalla abutment. Each span is approximately 26 m long and shares a common support with the adjacent span via notched-end girders. The superstructure is comprised of five pre-cast, post-tensioned concrete T-beams and cast-in-place decks and diaphragms. Expansion bearings were used for girder-to-girder connections within the curved portion and also at piers No. 11 and 17. The remaining girder-to-girder connections are made through fixed bearings. The girder connections at the piers and Whyalla abutment are through fixed bearings and expansion bearings were used at the Adelaide abutment (Figure 1). The fixed bearing and expansion bearing details are shown in Figure 2. Wall piers were used from pier No. 1 to No. 8 and No. 14, supported by 2.4 m wide x 0.91 m deep pile caps and a group of sixteen 457 mm octagonal prestressed concrete piles. Six 610 mm octagonal prestressed concrete piles with crossheads were used for the other piers. The site consists of recently deposited shelly sands and clays up to several metres in depth overlying sands and clayey sands of Hindmarsh formation approximately 25 metres thick, which in turn overlies sandstone of Tent Hill formation.

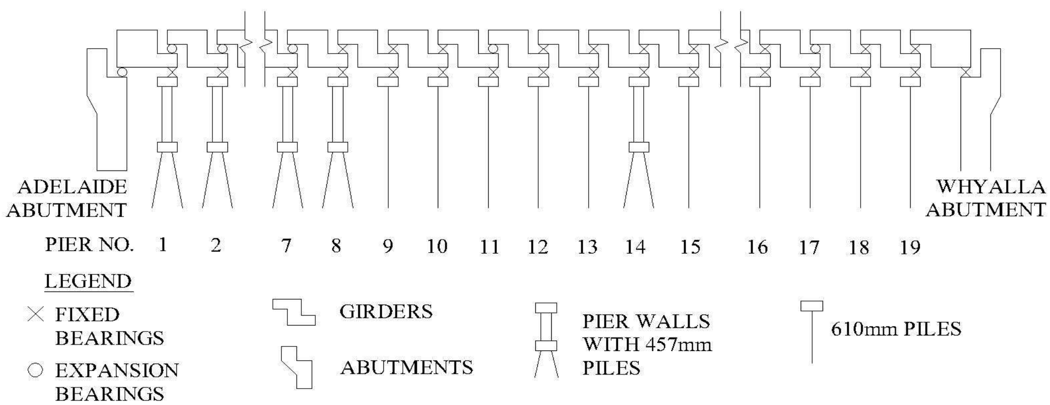


Figure 1 Port Augusta Bridge Profile

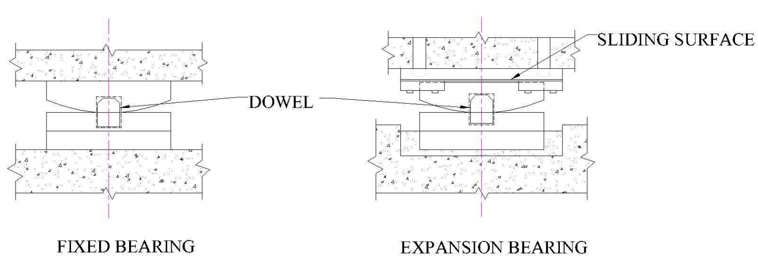


Figure 2 Bearings

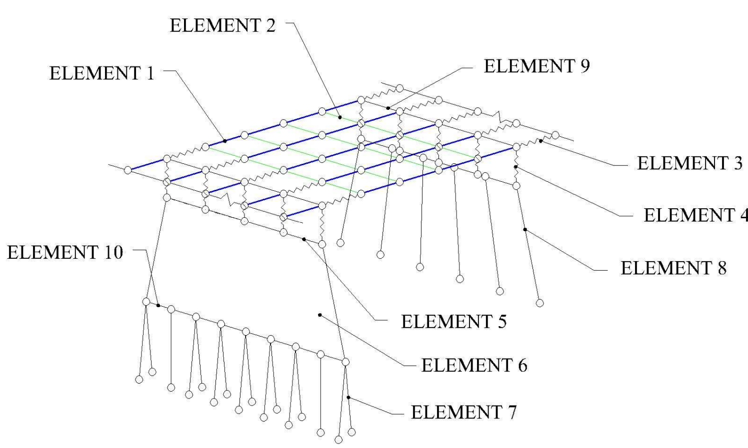
3 STRUCTURAL MODELLING AND SEISMIC INPUT

The bridge was modelled with the following assumptions:

- the bridge was straight;
- the superstructure, including girders, diaphragms, and deck, remains elastic during the applied earthquake (Priestley et al., 1996);
- the two abutments can be considered as two fixed points for modelling;
- the minimum earthquake wave travelling speed was 125 m/s.

According to AS5100.2, only the permanent effects are required to be considered when combined with earthquake load for ultimate limit state design. The mass of the superstructure was lumped at each node of the bridge model. For pier walls and piles, top half masses were lumped on the top end of each member and zero mass at the fixed base nodes. The Rayleigh damping model was used to model the damping exhibited by the structure in which the fractions of critical damping were assumed to be 5%. Earthquake motions in three directions - longitudinal, transverse and vertical - were analysed.

There are typically two ways to model the superstructure of a long bridge: a spine model with beam elements following the centre of gravity of the cross section along the length of the bridge, and a grillage of beam elements. For superstructures carrying wide roadways, the spine model may produce erroneous results, particularly when combinations of earthquake forces with gravity loads and live loads need to be investigated (Priestley et al., 1996). A simple spine model may not capture even gravity-load distributions to individual bearings and piers, since loads in the spine model are typically applied along the spine axis only. Therefore, a grillage model was employed to simulate the Port Augusta bridge superstructure as shown in Figure 3. The five longitudinal T-beams for each span were modelled by element 1, while element 2, applied at the centre and quarter points of each span, was used to model the diaphragm between beams. Element 2 was fixed to element 1 at each intersectional node.



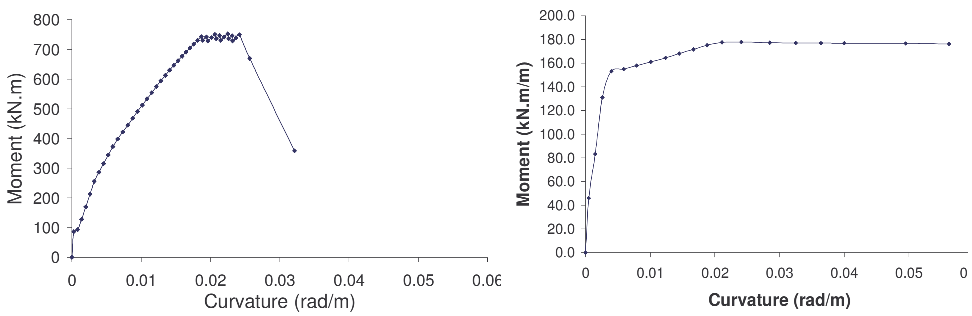
Element	Member
1	Girder
2	Diaphragm
3	G-G bearing
4	G-P bearing
5	Crosshead
6	Pier wall
7	457 mm Pile
8	618 mm Pile
9	Edge beam
10	Pile cap

Figure 3 Grillage model

Although detailed cracked section stiffness analyses can be performed for each superstructure element to determine the effective stiffness, it is recommended by Priestley et al. (1996) to calculate the gross-section stiffness I_g and reduce it to $I_e = 0.5\ I_g$ for reinforced concrete and assume no reduction (i.e. $I_e = I_g$) for essentially uncracked prestressed concrete superstructures, to give an effective flexural stiffness. The torsional rigidity J for grillage elements can also be determined from standard mechanics principles and can be considered as fully effective provided the torsional cracking moment is not exceeded.

Piles were modelled as 3D concrete beam-column members using a one-component model (Carr, 2004), which idealised a reinforced concrete beam as a perfectly elastic mass-less line element with non-linear rotational springs at the two ends to model the potential plastic hinges. The bi-linear hysteresis rule (Carr, 2003) was employed for the hinge spring stiffness to represent the elastic and inelastic behaviour of the member.

The effective piles stiffness EI_e was determined from section moment-curvature analyses as $EI_e = M_y/\Phi_y$, where M_y and Φ_y represent the ideal yield moment and curvature for a bilinear moment-curvature approximation (Priestley et al., 1996). The effect of concrete confinement was also considered using Mander et al.'s model (1989). The result of the section moment-curvature analysis of the piers under a static axial load is shown in Figure 4. The effective stiffness reduction in shear was considered proportional to the effective stiffness reduction in flexure (Priestley et al., 1996). The torsional moment of inertia was multiplied by a factor of 0.3 to get the effective torsional moment of inertia for these bridge piles after Singh and Fenves (1994).



(a) 610 mm Pile (b) Pier Wall Section (transverse axis)

Figure 4 Moment-curvature relationship for 610 mm pile and pier wall section

For simplicity, the effective fixity approach (Reese et. al., 1974) was applied to model soil flexibility effects on pile shaft systems. In this approach, the equivalent depth to fixity, d_f, was determined; i.e. the depth of a fixed-base column with soil removed that produced the equivalent top-of-pile lateral displacement as the embedded pile . The d_f value was found to be about 2.4 m and 3.7 m beneath the top face of the sands and clayey sands layer for 610 mm and 457 mm piles respectively.

Pier walls were modelled as quadrilateral finite elements and assumed to be fixed at the top of pile caps.

Bearings were modelled by 3D spring elements. The spring stiffness for a fixed bearing in the longitudinal direction was based on the idealised shearing deformation of the central stainless dowel by $k_{dowel} = G_s A/d$, where G_s is the steel shear modulus, and A & d are cross sectional area and diameter of the dowel respectively. In the transverse direction, the same stiffness was used for a fixed bearing, while the spring stiffness for an expansion bearing was determined using a similar method applied to the edge restraint bolts of the expansion bearing top plate.

Two different joint models were applied to the fixed and expansion bearings. The elastic hysteresis rule was assumed for fixed bearings. For expansion bearings, a hertzian contact spring with a slackness hysteresis rule was used in the longitudinal direction as shown in Figure 5. This hysterisis rule permitted the definition of two different contact gaps (i.e. slackness length) for both positive and negative directions, beyond which a spring stiffness model could also be defined. For the model, a gap between two adjacent girders of 25 mm was set as the positive gap. A very large spring stiffness derived from the concrete compressive stiffness was then used to model the case when one girder drifted beyond the 25 mm gap and hit the adjacent girder. In the negative direction, the expansion bearing movement limit of 100 mm was applied as the negative gap. In order to prevent girders "dropping off" during the analysis, an idealised restrainer cable was applied to connect the two adjacent girders, and the slackness length of the cable was set at 100 mm. The tensile stiffness of the restrainer cable was then used for the bearing stiffness beyond the negative gap. In this way, the program can calculate the relative displacements in the joint, the pounding forces of two adjacent girders, and the tensile force in restrainer cables if activated during the analysis.

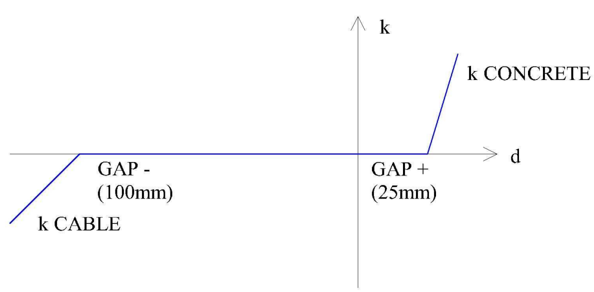


Figure 5 Hertzian contact spring with slackness hysteresis rule

4 EARTHQUAKE RESPONSE OF THE BRIDGE

The purpose of the nonlinear time history analyses was to assess the seismic vulnerability of the bridge. Seismic evaluation is typically based on a quantitative assessment of the "capacity" of, and "demands" on, individual components, where capacities include member resistances and displacement capabilities, and demands include force effects and displacement effects.

Six models were analysed for the Port Augusta Bridge. The first three models investigated three different artificial earthquake ground motions, which were representative of a possible severe earthquake, without wave effects. The other three models were applied with the travelling wave effects. All of the six models were run in three different earthquake directions: longitudinal, transverse and vertical.

The main vulnerabilities are summarised in Table 1. It should be noted that the table reports the maximum values from the six models, and therefore the maximum force does not necessarily occur concurrently with the maximum displacement. 'Exp' and 'Fix' refer to expansion and fixed bearing respectively. 'G-G(A)' refers to girder-to-girder or girder-to-abutment. The maximum displacement among the five girders is reported, while the total transverse shear is summed across the whole transverse section. The capacity of each element is listed at the bottom of Table 1.

Pier No.	G-G(A) bearing type	Pier wall bending moment	Longitudinal displacement in G-G(A) joints		Longitudinal total force in G-G(A) joints		Transverse total shear
		(kNm/m)	(mm)	(mm)	(kN)	(kN)	(kN)
			drift away	drift towards	drift away	drift towards	
Ade Abut	Exp	-	39	25	0	7350	741
1	Exp	94	27	26	0	1930	954
2	Exp	111	36	26	0	2160	905
3	Exp	104	49	26	0	2445	708
4	Exp	102	43	26	0	2855	824
5	Exp	103	33	27	0	3455	846
6	Exp	103	71	27	0	3155	845
7	Exp	98	71	27	0	4315	738
8	Fix	143	-	-	114	1630	218
9	Fix	-	-	-	77	1310	186
10	Fix	-	-	-	39	1970	217
11	Exp	-	102	27	216	3530	656
12	Fix	-	-	-	40	2250	107
13	Fix	-	-	-	79	1890	117
14	Fix	172	-	-	302	1300	205
15	Fix	-	-	-	40	2040	149
16	Fix	-	-	-	19	2490	83
17	Exp	-	77	27	0	4240	610
18	Fix	-	-	-	186	3030	73
19	Fix	-	-	-	364	3120	34
Why Abut	Fix	-	-	-	3140	2540	99
Capacity	Exp	123	100	25	cable	varies	410
	Fix		-	-	769	769	769

Table 1 Summary of Earthquake Response for Port Augusta Bridge

The analyses identified:

- bending moment capacity issues with pier walls 8 and 14
- pier 11 large girder longitudinal displacement associated with "drifting away" motion
- the colliding force at the Adelaide abutment was greater than the capacity of the abutment backwall
- the longitudinal shear force in the fixed bearings was greater than the dowel capacity
- inadequate shear capacity of expansion bearings in the transverse direction.

The capacities of the remaining bridge elements were found to be adequate. The ductility of each member was also checked and found to be sufficient.

5 SEISMIC RETROFIT SCHEMES

Several retrofit proposals were developed to address the bridge response vulnerabilities. The objective of the retrofit schemes is to increase the capacity of the bridge members to satisfy the demand. Carbon fibre reinforced polymer (CFRP) near surface mounting techniques was used to improve the ultimate strength of pier walls. Longitudinal and transverse restrainers were used to prevent any failure in bearings. Restrainer cables with 100 mm slackness were used to prevent girders dropping off piers.

Near surface mounted (NSM) CFRP vertical strips were designed to retrofit pier walls No. 8 and 14. The thickness, modulus of elasticity and rupture strain are 1.2 mm, 165 GPa and 1.7% respectively. The CFRP-to-concrete debonding strain, calculated based on Seracino et al.'s generic model (2005), was used as the ultimate strain in the retrofit design. Vertical NSM CFRP strips with the dimensions of 1500 mm long by 25 mm wide by 1.2 mm thick, with a centre to centre spacing at 450 mm, were designed for both sides of pier walls as shown in Figure 6. The moment-curvature relationship for the retrofitted pier wall is shown in Figure 7. It can be seen that the ultimate flexural strength of the retrofitted pier wall has been improved by approximately 40%, while the stiffness remains similar up to the yielding point (M_y = 154 kN.m). Three courses of externally bonded (EB) horizontal CFRP wet lay-up fibre strips will assist the confinement of concrete and prevent damage or spalling of cover concrete, in which the vertical CFRP strips are mounted.

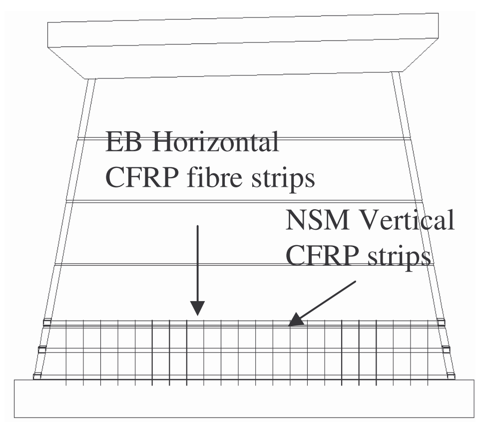


Figure 6 Layout of NSM CFRP retrofitting pier wall No. 8

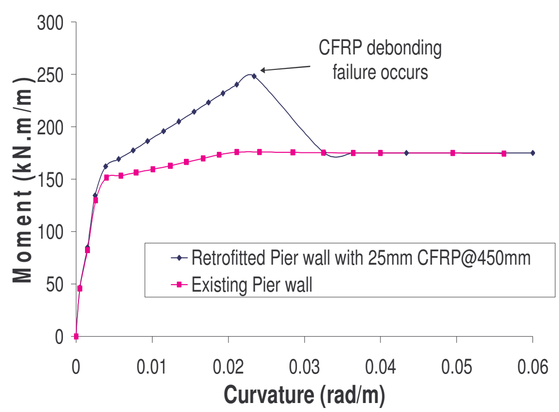


Figure 7 Moment-curvature relationship for the retrofitted pier wall

To address the transverse shear capacity deficiency in expansion bearings, a shear wall will be constructed on top of the pier between the third and fourth girders. The shear wall will carry the transverse shear concurrently with the expansion bearing transverse shear restraint bolts.

To prevent any failure of the G-G(A) fixed bearings in the longitudinal direction, rubber blocks will be inserted between adjacent girder ends, allowing small girder rotation in the longitudinal plane.

Restrainer cables will be used to tie up the adjacent girders on pier No. 11, to prevent them “dropping off” after moving beyond the available seat length. Additionally, during construction work, the available travel length in other expansion bearings will be measured and additional restrainers installed if necessary.

Restrainer brackets at the bottom of girders in the end spans will transfer loads from girders to the abutment main wall, preventing any failure of abutment backwalls or the fixed bearings.

6 CONCLUSIONS AND RECOMMENDATIONS

The selection of the best retrofitting schemes for the Port Augusta Bridge required the potential deficiencies of the bridge to be evaluated by the nonlinear time-history analysis program Ruaumoko (Carr, 2004). Retrofitting techniques that could overcome these deficiencies were identified and assessed for their feasibility and effectiveness. If the proposed retrofitting scheme altered element stiffness and consequently structural response, the model should be reassessed using the modified properties of the retrofitted element.

Modelling piles with the effective fixity approach and assuming fixed boundary conditions for abutments may be slightly unconservative in terms of predicting the displacement (McDaniel, 2006). It is recommended that the effects of soil-structure-interaction be considered and investigated in the seismic modelling of bridges.

An alternative approach to seismic retrofitting is to decrease the bridge seismic displacement demand, rather than increase capacity. Investigations into the use of friction and viscous dampers to improve bridge seismic performance may be justified.

The risk of not installing seismic retrofits on identified high-risk structures is that bridges that are strategically significant to the freight industry, the transport system performance and post disaster relief may be damaged during a severe earthquake. Using an assessment regime where bridges are assessed in terms of importance and vulnerability to damage will help to mitigate the risk and ensure that limited funds are used effectively.

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