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Retrofit to improve earthquake performance of bridge abutment slopes

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ABSTRACT: Abutments of bridges have been found from seismic assessments to be vulnerable to damage due to ground instability, sometimes exacerbated by liquefaction and lateral spreading. The observed damage to bridges in the recent 2010 Darfield earthquake has also been associated with ground damage at bridge abutments. The small sized piles associated with older bridges have been assessed to be vulnerable to slope movements.

A variety of retrofit techniques have been developed, designed and constructed to mitigate abutment slopes at a number of bridges around New Zealand. These cost effective techniques have relied on a performance based assessment and retrofit design that was adopted for these bridge abutments. Retrofit techniques included ground improvement using drilled stone columns, strengthening using soil nailing and rock bolts and simple buttressing using earth or concrete blocks. These were chosen and tailored to suit the ground conditions and provide the performance appropriate for each bridge. Examples of their application to practical retrofit are illustrated through case studies of their application in New Zealand. Such retrofit contributes to more resilient lifelines, and thus the building of a more earthquake resilient society.

1 INTRODUCTION

The New Zealand Transport Agency has a programme of detailed seismic assessment and retrofit of bridges vulnerable to earthquakes, developed based on a seismic screening of all the state highway bridges in New Zealand (Opus International Consultants, 1998). As part of this programme, a number of prioritised bridges in various parts of the country have been investigated, assessed for their seismic performance, and strengthened where appropriate.

One of the critical vulnerabilities identified by the seismic assessments for a significant number of bridges is damage to their abutments due to earthquake induced ground instability of their abutment slopes or walls. The potential for ground damage is exacerbated when the soils at the bridge abutments are vulnerable to earthquake induced liquefaction and lateral spreading. The ground movements can cause damage to the abutment substructures and foundations.

The recent Darfield (Canterbury) Earthquake of 4th September 2010 has also highlighted the vulnerability of bridge abutments to ground damage. In that earthquake most of the observed significant damage to bridges was associated with ground damage in areas of poor ground, and in particular lateral spreading associated with liquefaction.

Retrofit of existing bridge abutments to mitigate the effects of earthquake induced ground instability poses a number of challenges. Any retrofit has to be carried out without compromising the ability of the bridges to continue to provide access to the community. Access to the buried sub-structures is very limited. Retrofit techniques have been developed and implemented to limit damage to the bridges and their substructures, minimise disruption of access across the bridges, and are illustrated in this paper using a number of case studies of retrofit design completed by Opus.

2 OBSERVATIONS FROM THE 2010 DARFIELD EARTHQUAKE

2.1 The Earthquake

The magnitude 7.1 Darfield (Canterbury) Earthquake occurred at 0435 hrs on 4th September 2010. The earthquake was associated with a 22 km long rupture of a fault (now named as the Greendale Fault), and the epicentre was located approximately 40 km west of Christchurch City, at a depth of 11 km. This was followed by numerous aftershocks that were centred closer to the city. The main shock caused Modified Mercalli MM VII to MM VIII shaking in the Christchurch area.

The earthquake caused extensive liquefaction and lateral spreading in some parts of Christchurch, and adjacent districts. There were rock falls near the Port Hills, damage to some road bridges, embankments and pavements, and landslides closer to the epicentre. There was also significant damage to buildings and some collapses, particularly of the older unreinforced masonry construction. Liquefaction and associated lateral spreading caused extensive damage to houses and other buildings as well as to roads, bridges and buried utilities in the affected areas.

2.2 Inspections

Bridges on the state highway network and Christchurch City local road network were inspected by Opus International Consultants' bridge and road network management staff immediately following the 2010 Darfield Earthquake. This was followed by inspection of the effects of ground damage in Canterbury by the first and second authors from Opus, and bridge inspections by Howard Chapman and John Wood, to report on lessons from the earthquake for the New Zealand Transport Agency. Subsidence was observed at a number of bridge approaches and was quickly reinstated. There was damage to some highways due to ground damage as a result of failure of steep highway embankments, liquefaction and lateral spreading, rock fall and overslips.

2.3 Performance of Bridges

The bridges on the state highway network performed very well. Damage was minor and repairable.

The predominant damage to bridges was at the abutments of bridges. This was due to displacements of abutments and associated slopes due to ground shaking, particularly in areas of poor liquefiable ground. Very little damage was observed on bridges in the areas of more competent alluvial gravels, even in the epicentral area subject to stronger ground shaking (peak ground accelerations 0.4g to 0.7g). Liquefaction and lateral spreading led to significant damage to the abutments of some road bridges, and severe damage or collapse of some footbridges over the Avon River.

Notably the abutments of the South Brighton Bridge over the estuary and its approaches on Bridge Street were damaged. The bridge was quickly reopened to provide limited access to traffic after reinstatement of the approaches. Liquefaction and lateral spreading of the abutment slopes led to rotation of abutment, with some damage to the raked piles supporting the abutment, see Figure 1.

Figure 1. Damage to bridge abutment at South Brighton Bridge in 2010 Darfield Earthquake

3 SEISMIC VULNERABILITY OF HIGHWAY BRIDGE ABUTMENTS

Detailed seismic performance assessments carried out on a range of state highway bridges across New Zealand have shown bridge abutments to be one of the common areas of vulnerability due to ground displacement or failure. Often the ground displacements associated with the abutment slopes also have the potential to cause damage to the first piers closest to the abutments.

Existing bridge abutments have been assessed to be vulnerable to ground displacements in a range of situations, such as:

- Case 1 - Failure of steep rock bluffs, slopes or cuttings on which the bridge abutment is founded, under strong ground shaking.
- Case 2 - Lateral outward displacement of steep approach embankment slopes, sometimes underlain by weak soils or steep river banks, due to strong ground shaking.
- Case 3 - Lateral spreading of approach embankments built on saturated sand / silt susceptible to earthquake induced liquefaction and lateral spreading.
- Case 4 – Lateral outward displacement of abutment walls, under strong earthquake ground shaking.

The abutments of the old bridges are often supported by small sized piles, typically 400 mm square precast concrete piles. The piles are sometimes raked to provide additional lateral capacity. These slender piles are vulnerable to damage from the expected ground displacements. The ground failure or displacement has the potential to cause extensive damage to bridge abutments, their piles and elastomeric bearings, and possibly lead to collapse.

A variety of retrofit techniques have been developed, designed and implemented for these bridges. The retrofit has been designed to limit the extent of ground displacements during large earthquakes, and hence damage, to achieve an acceptable level of performance for the bridge. Such a performance based retrofit approach has enabled practical and economic strengthening works for these bridges.

The techniques are illustrated using actual case histories of retrofit for New Zealand bridges during the past 5 years.

4 PELORUS BRIDGE – ROCK ANCHORING AT BRIDGE ABUTMENTS

The strengthening of the Pelorus Bridge illustrates the use of rock anchoring to secure the abutment walls and a rock bluff below one of the abutments. The retrofit raised the performance of the bridge from failure in 50 to 300 year return period earthquakes to adequate performance in a 1000 year return period event. The Pelorus Bridge is located of State Highway 6 between Nelson and Blenheim, in the Pelorus National Park, Marlborough. The two span single lane bridge with a footpath is a concrete decked Callender Hamilton steel truss structure supported on abutments and a central pier constructed of unreinforced concrete, see Figure 2.

Figure 2. SH 6 Pelorus Bridge

The Nelson abutment and central pier of the bridge are founded on indurated sandstone bedrock, while the Blenheim abutment is founded on outwash gravels underlain by bedrock at shallow depth. It was assessed that the unfavourably oriented defects in the jointed rock mass at the Nelson abutment are likely to lead to planar rock slope failure under strong earthquake shaking, with a return period as low as 50 years. The cast in situ concrete abutment walls were also vulnerable to large displacements in 250 to 300 year return period earthquake shaking.

Rock anchors were chosen as the most economic method to retrofit both abutment walls as well as the rock bluff below the Nelson abutment, given the presence of good bedrock at shallow depth. Both concrete abutments were strengthened using a concrete overlay, and anchored back using rock anchors comprising double corrosion protected 32 mm diameter 1030 MPa Macalloy bars grouted into 150 mm diameter holes, see Figure 3. The wing walls were also tied back using anchors between opposite wing walls on each abutment. The rock blocks in the bluff below the Nelson abutment were rock bolted using double corrosion protected 25 mm 500 MPa Reid bars grouted into 130 mm diameter holes. The Macalloy and Reid bars were chosen because of their ductile post-yield behaviour.

With the anchors in place, the abutments and underlying rock are secured against movement or failure in 0.36g peak ground accelerations with a 1000 year return period. The anchors were designed using capacity design principles to ensure ductile failure by yielding of the anchor bars under seismic overload, rather than failure in a brittle manner by either rock-grout bond or grout-bar bond failure. This was achieved by providing greater strength reduction factors for these more brittle failure mechanisms, and the ductile elongation of the bars accommodating some displacement, and allowing associated reduction in earth pressures on the walls.

(a) Nelson Abutment (b) Blenheim Abutment (c) Wing Wall

Figure 3. Rock anchor strengthening of abutments at Pelorus Bridge

5 MOHAKA BRIDGE – ABUTMENT SLOPE STRENGTHENED USING SOIL NAILS

The retrofit of the southern abutment of the Mohaka Bridge illustrates the use of post-grouted soil nails to reduce the displacement of the abutment slope during strong earthquake shaking. The strengthening programme also included substantial strengthening of the steel superstructure.

The Mohaka Bridge is on a straight alignment with a vertical sag curve on State Highway 5 between Napier and Taupo, see Figure 4.

The structure comprises three steel truss main spans of 57.6 m, 82.3 m and 57.6 m lengths, and two steel beam land spans of 7.6 m and 10.7 m lengths at the northern and southern ends respectively. Both the main spans and the land spans have reinforced concrete decks. The bridge was designed by the Ministry of Works and construction was completed in 1962. A description of the design and construction of the Mohaka Bridge is given by Bewick (1964).

The geology of the Mohaka Bridge site is tertiary sandstone and siltstone beds of Miocene age, with uniaxial compressive strengths of the order of 1 MPa to 10 MPa.

Figure 4. SH 5 Mohaka Bridge

The northern abutments were assessed to perform well with minimum displacement in 500 year and 1000 year return period events, but possibly significant displacements in a 2,500 year return period events (0.72 g peak ground acceleration). The northern abutments of both the land span and main spans are reinforced concrete slabs founded on bedrock. Bewick (1964) reports of the discovery of cracks, inferred to be open joints in the bedrock, which led to the incorporation of additional mitigation measures during construction. These comprised the addition of two reinforced concrete shafts to 6.1 m depth into rock at the northern main span abutment (B), installation of an array of rock dowels to tie the slab into rock, and drilling and pressure grouting of 9 holes. These additions during construction have significantly enhanced the likely performance during earthquakes.

Significant damage to the southern main span Abutment E was considered to be unlikely given that its reinforced concrete slab foundation was supported by four heavily reinforced concrete columns socketed into bedrock. These columns had been added during construction, to replace the original design of 27 raked piles, after the test pile was not able to achieve sufficient penetration.

The southern abutment slope was underlain by fill and weak alluvium deposits, with potential for large displacements of up to 450 mm in a 500 year return period event (0.41g peak ground acceleration). Such displacements had the potential to cause significant damage to the four slender reinforced concrete 400 mm octagonal driven piles supporting the southern land span Abutment F and the bearings, and possibly lead to collapse in larger events.

A number of options including drainage, stone columns, H-piles and soil nails were considered. Soil nails were chosen as the most economical and practical solution to reduce the amount of slope displacement and hence damage to the bridge foundations. Stone columns and H piles would be difficult to install given the limited head room below the bridge, and the presence of raked piles.

The soil nails were installed to stabilise the ground between Abutment E and Abutment F, and comprised 32 mm Reid Bars (500 MPa yield strength) at 2 m horizontal and 1.4 m vertical spacing. The soil nail heads and the ground face were supported using a 150 mm thick reinforced shotcrete facing, see Figure 5. Drainage holes were also installed.

However, given the variable poor alluvial ground conditions, ordinary soil nails would not give adequate capacity and would give large displacements. The soil nails were *post-grouted* at 300 to 500 psi grout pressures through nodes in a post-grout tube grouted in during the initial grouting of the double corrosion protected bar assembly into the holes. This *post-grouting* technique for soil nails was originally developed for the Wellington Inner City Bypass project (Brabhaharan, 2007), and was adapted to suit this site. Soil nail pull-out tests were carried out to confirm design parameters and their suitability to give an adequate level of earthquake performance. The post-grouted soil nails reduced earthquake induced displacements of the abutment slope in a 1000 year earthquake, to less than 50 mm, which can be tolerated by the existing sub-structure.

Pier D of the bridge was also protected with rock bolts installed to stabilise the steep slope below its foundations.

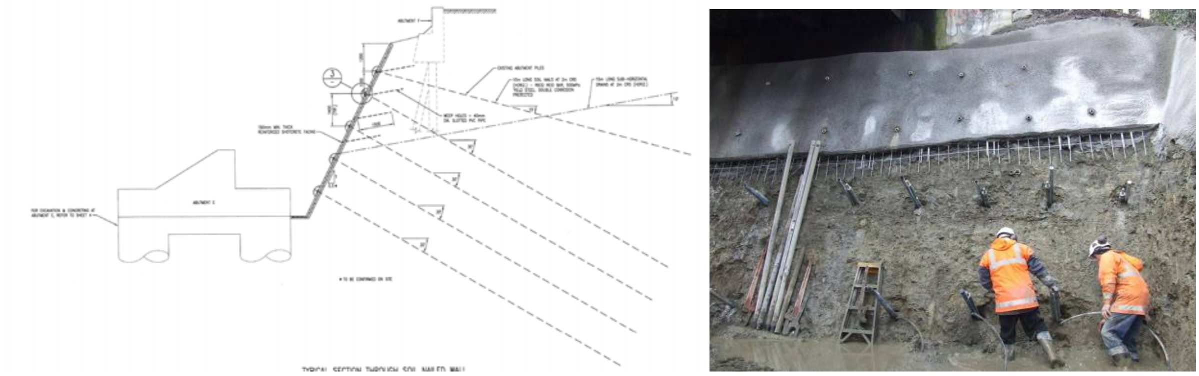

Figure 5. Mohaka Bridge south abutment strengthened using post-grouted soil nails

6 SH 1 OTAKI RIVER BRIDGE – ABUTMENT STRENGTHENING USING BUTTRESS

The Otaki Bridge retrofit illustrates an integrated seismic and scour strengthening for the bridge abutments, using conventional riprap and concrete blocks to buttress the abutment slopes, provide scour protection, and provide walkway access.

The Otaki River Bridge carries State Highway 1 across the Otaki River approximately 70 km north of Wellington and is on the only western access route across the river and north of Wellington.

The two lane bridge spans approximately 208 m across the Otaki River, see Figure 6. The site is underlain by recent alluvial deposits, comprising well sorted flood plain gravels, cobble and boulders.

Figure 6. SH1 Otaki River Bridge and its north abutment

A seismic and scour assessment of the bridge indicated that the bridge abutments were prone to scour damage, and significant displacement of the abutment embankment slopes in large earthquake events. The displacements were assessed to be 200 mm to 400 mm at the north abutment, and 50 mm to 150 mm at the south abutment, in a 1000 year return period earthquake shaking (peak ground accelerations of 0.69g). The bridge sub-structure at the abutments comprised four reinforced concrete square columns supported by reinforced concrete octagonal piles, and was assessed capable of tolerating the displacement at the southern abutment slope, but not the larger displacements at the steeper northern abutment. Additional piles at the abutments are being considered to resist the loads from the bridge superstructure.

Discussions with Greater Wellington Regional Council indicated that river protection works for that section of the river and construction of a walkway along the northern bank of the river was under consideration. An integrated solution was developed that provided scour protection for the bridge abutment, river protection works, and a walkway under the bridge for recreational use. Riprap protection of the river bank at the north and south banks and a concrete block walkway at the north bank were constructed in 2010, see Figure 7. The buttressing works will reduce the earthquake displacements at the abutments to less than 100 mm in a 1000 year return period earthquake shaking.

Figure 7. Rip rap and concrete blocks provide scour and earthquake protection at Otaki River Bridge

To avoid the imposition of additional lateral loads on the piers, space was provided between the concrete blocks and pier walls, to allow for limited displacement of the buttress.

The integrated scour and earthquake protection achieved an economical solution to earthquake retrofit of the abutments at Otaki River Bridge.

7 COBHAM BRIDGE ABUTMENT RETROFIT USING DRILLED STONE COLUMNS

The abutment strengthening of Cobham Bridge illustrates the use of drilled stone columns to provide ground improvement to mitigate the risk of liquefaction, lateral spreading and abutment slope failure.

The State Highway 3 Cobham Bridge provides access across the Whanganui River, just upstream of its confluence with the sea, and is located southwest of the city centre. The 275 m long, 9 span bridge was designed in 1959 by the Ministry of Works, and constructed in 1962, see Figure 8. The abutments are supported on raked prestressed concrete piles.

Figure 8. Cobham Bridge across the Wanganui River

A detailed description of the bridge, seismic assessment and retrofit is presented by Brabhaharan et al (2009). Geology of the area is alluvium comprising sand and silt of Holocene Age, deposited in an estuarine environment. The Cobham Bridge site has the potential for liquefaction, and consequent lateral spreading. Liquefaction will almost certainly lead to slope failure, and lateral spreading of the embankments, either during earthquake shaking or in the hours afterwards while the pore water pressures in the liquefied soils remain high. The abutment structure is expected to fail under large forces from the embankment fill.

A novel triangular shaped area of ground improvement behind each abutment, using stone columns and wick drains (see Figure 9) was adopted to protect the abutment while allowing lateral spreading or failure of the approach embankment slope away from the abutment. This design philosophy recognises that provided that the structure is protected from failure, the approach embankment could be relatively quickly reinstated by earth moving machinery after a major earthquake. This approach optimised the area of ground improvement, leading to a cost effective solution.

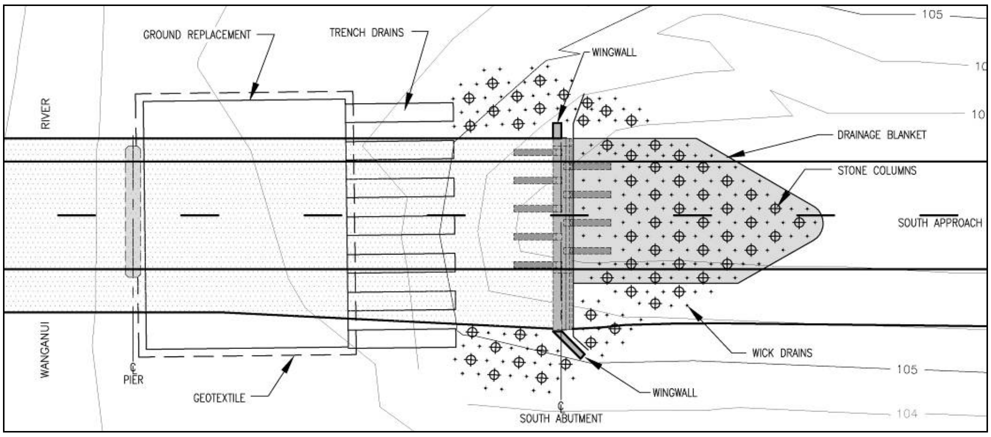

Figure 9. Layout of stone column ground improvement at south abutment of Cobham Bridge

Wick drains were also installed to the full depth in between the stone column locations prior to the construction of stone columns in each area. The purpose of the wick drains was to facilitate dissipation of the pore pressures generated in the ground during vibration of the stone columns, enabling densification of the ground, and providing additional drainage of the ground in between the stone columns, in the event of a rise in pore pressures during an earthquake.

A new construction approach was adopted, comprising drilled and compacted stone columns. Holes were drilled to 600 mm diameter with casing providing temporary support. A central steel probe was lowered into the hole, clean 5 mm to 50 mm size aggregate was poured into the hole in stages, as the casing and probe were withdrawn, with compaction by concurrent vibration. This enabled implementation on an active highway without causing failure of the embankment during construction. Areas in front of abutments were improved by trench drains to facilitate porewater pressure dissipation and avoid liquefaction, and replacement of liquefiable sands with dense gravels, see Figure 9.

8 CONCLUSIONS

The assessment of the seismic performance of a variety of state highway bridges have highlighted the vulnerability of abutments to damage from slope and abutment wall displacement, sometimes exacerbated by liquefaction and lateral spreading. This was reinforced by the recent 4th September 2010 Darfield Earthquake in New Zealand, where the limited damage to bridges were generally to their abutments, and some bridge abutments were damaged by liquefaction and lateral spreading.

A variety of seismic retrofit techniques have been developed and implemented over the past five years to effectively and economically strengthen abutments and associated embankment slopes. These range from simple rockfill / concrete block buttresses, to rock anchoring and post-grouted soil nailing, to drilled stone column and wick drain ground improvement implemented in a novel triangular layout.

A key feature of these approaches is that economical retrofit solutions were achieved by focussing on the performance of the abutments where the slope displacements are reduced but not eliminated.

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