The seismic assessment and retrofit of the Shell Gully bridges

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ABSTRACT: The Shell Gully bridges, situated immediately north of the Terrace Tunnel are a vital link on the Wellington Urban Motorway providing access from the north to the CBD and beyond to the hospital and airport. Located in New Zealand’s capital and second largest city, and a highly seismic part of the country, Transit New Zealand assigned a high priority to ensuring the integrity of these critical structures against damage and loss of service from a major earthquake event.

This paper presents the detailed assessment of the bridges’ earthquake resistance and seismic retrofitting undertaken. The retrofitting has encompassed:

- improving the security of short link spans, interconnecting the structures, from falling;
- modifying the heavy fascia units along the sides of the bridges to prevent their dislodgement under shaking of the structures;
- isolation of a pier from potential downhill movement of the underlying slope; and
- strengthening and modification of the restraint of the high northern abutment to improve its seismic performance.

1 INTRODUCTION

New Zealand’s stock of bridges includes numerous structures that were designed before current seismic design procedures were introduced. For some years now Transit New Zealand has engaged in a programme of screening its State Highway bridges for seismic resistance and assessing for retrofit those bridges identified as being of highest priority. Linkage retrofitting has almost been completed on the highest priority bridges. Detailed assessments have been undertaken on the 50 bridges of highest priority for assessment. A number of especially important bridges have now been retrofitted, including the Thorndon Overbridge in Wellington and the Auckland Harbour Bridge. The Shell Gully bridges on the Wellington Urban Motorway, immediately north of the Terrace Tunnel leading into the CBD, are another such group of structures. This motorway link is vitally important as a post-disaster lifeline for Wellington city as it provides direct access into and through the city CBD from the north to the main hospital and airport to the south.

The Shell Gully bridges were designed with provision for future duplication of the Terrace Tunnel based on separate structures being provided to carry each carriageway of the motorway, but have only been constructed to the extent necessary to feed traffic into the single existing tunnel. Thus, currently, both carriageways of the motorway are carried on the same structures intended ultimately to carry the northbound carriageway. These comprise two continuous bridge structures end-to-end, interconnected to each other, to the abutments, and to an on-ramp from Clifton Terrace by short 2.4 m long link spans. Of the future southbound carriageway, the northern structure has also been constructed and connects to a bridge structure carrying an off-ramp to The Terrace, with these structures being similarly connected by link spans. Figure 1 illustrates the layout of these structures. Figure 2 shows the Clifton Terrace On-Ramp and southern part of the main motorway structures.
The main motorway carriageway structures comprise twin continuous prestressed T-beams constructed integral with piers comprised of twin column portal frames, with each column founded on a single cylinder foundation socketed into bedrock. The on and off-ramp structures comprise single continuous prestressed concrete T-beams, again constructed integral with the piers, which are single columns founded as before, except for the upper end structure of the Clifton Terrace on-ramp, which is a slab structure supported on piles.

![Figure 1: Layout of the Shell Gully Bridge Structures](image1)

![Figure 2: Main Motorway Structure and Clifton Terrace On-Ramp](image2)

An initial detailed seismic assessment had already been undertaken by the Holmes Consulting Group. Opus International Consultants were commissioned by Transit New Zealand to review the prior work, confirm the soil parameters assumed, investigate the stability of the western slope underlying the structures, design the recommended retrofit works and manage the associated physical works contract.

2 PRIOR WORK

The initial detailed seismic assessment identified:

- That the bridge was expected to remain essentially undamaged in a 150 year return event, other than for minor pounding damage at the link spans;
- That under a 475 year return period event:
  - unseating of the link spans was likely;
  - pounding between deck edge fascia units at the link span locations was likely to result in dislodgement of these;
  - pounding between the future southbound carriageway structure and underlying Clifton Terrace car park structure was likely, resulting in spalling concrete damage;
  - Flexural yielding in the tops and bottoms of the piers of the main carriageway bridge structures would occur with spalling of the cover concrete, and
  - Again, there would be pounding damage at the link spans;
• And that under the maximum credible event, in addition to the damage from a 475 year return period event, flexural yielding would also have initiated in the piers of the ramp structures.

Retrofit work recommended included:
• Increasing the seating lengths provided to the link spans to prevent premature loss of seating and drop-off in a moderate to large earthquake; and
• Installing “catch cables” to catch fascia units which may be susceptible to being dislodged in a moderate to large earthquake.

Assessment of the stability of the western batter slope underlying the structures was also recommended.

3 SITE SEISMICITY

Wellington is one of the higher earthquake risk regions in New Zealand, with several active faults which may affect the site, including the Wellington, Ohariu, Otaki Forks, Wairarapa and Lambton faults. The most active fault in the region, the Wellington Fault passes within 0.8 to 1.0 km of the Shell Gully site, with rupture of the fault expected to result in a magnitude 7.3 earthquake. The fault has a recurrence interval of between 500 and 770 years, with the most recent event estimated to have occurred between 340 and 450 years ago.

Site specific response spectra for a range of return periods were developed by the Institute of Geological and Nuclear Sciences Ltd (GNS), using their New Zealand National Seismic Hazard Model, for the initial seismic assessment. The Maximum Credible Event (MCE) for the site was taken as corresponding to an event with approximately a 1000 year return period. Peak ground accelerations estimated for this rock site, for the various return period events considered, were as follows:

<table>
<thead>
<tr>
<th>Earthquake Event</th>
<th>150 year return period</th>
<th>475 year return period</th>
<th>MCE 50th percentile</th>
<th>MCE 84th percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.22g</td>
<td>0.38g</td>
<td>0.48g</td>
<td>0.75g</td>
</tr>
</tbody>
</table>

4 SITE GEOLOGY AND GROUND CONDITIONS

The Shell Gully structures and motorway approaches are located in an old gully, which was partly infilled with alluvium.

The site is underlain by alternating sequences of sandstone and mudstone (argillite) of the Wellington Belt Greywacke bedrock. The sandstone exposed in the cutting to the east of the site is moderately weathered and is closely jointed, while the mudstone has finely laminated bedding features, is intensely deformed, and is extremely weak and fissile.

A cutting of up to 25 m in height was formed in the toe of the hillside slopes forming the western side of the gully. The cutting has been formed at an overall slope of about 40 degrees. Three lobes of fill or colluvium were exposed during cutting.

5 GEOTECHNICAL ASSESSMENT OF THE SLOPE STABILITY

The hillsides above the cutting are mantled with residual soils and colluvium. The slopes have remained stable in the recent geological past and have survived at least several ruptures of the Wellington Fault and the 1855 Wairarapa Fault event, without failing.
The overall cut slopes are not significantly steeper than the original hillside slope (30° to 40°) and most of the cut slopes are formed in rock where slope instability is not expected in large earthquakes. However instability of a portion of the cut slope could not be discounted in earthquakes with a return period of greater than 1000 years. In general, the piers and abutments for the main bridge are located some distance from the cut batter and are unlikely to be affected by instability or rockfall, but one pier of the Clifton Terrace On-Ramp could be affected.

6 ISOLATION OF CLIFTON TERRACE ON RAMP PIER 4C FROM SLOPE MOVEMENT

The Clifton Terrace On-Ramp pier 4C is sited near the toe of a section of cut slope where alluvium 3 m thick has infilled an old gully. Loss of stability and downslope movement of the alluvium in a large earthquake could result in earth pressures being applied to the pier that would exceed its lateral load capacity.

The risk was mitigated by excavating the loose material from around the pier and constructing a rock anchored wall 0.5m upslope of the pier, to isolate the pier from earth pressures due to downslope movement of the alluvium. The design used steel plate wall facing, but the contractor offered an alternative involving precast concrete units, forming a 4 m square box up to 3 m high around the pier, which was adopted.

Buildability was a key component in the design of the shielding structure. A lightweight structure was needed for lifting and placing in tight conditions. The box consisted of four precast wall panels connected at the corners with in situ concrete. Adequate space for installation of rock anchors and stressing equipment was required. The concrete box includes a cover to prevent debris and water run off from entering the void around the pier.

7 ASSESSMENT OF ABUTMENT STABILITY

The abutment structures were not assessed in the prior work, and so the need for them to be assessed was identified and this was included in Opus’ commission. These structures include the following:

- The northbound carriageway northern abutment, which is discussed separately below.
- The southbound carriageway northern abutment, sited alongside the northbound carriageway abutment, but without any structural connection, which is a cantilevered reinforced concrete wall founded on reinforced concrete bored cylinder foundations. It supports a concrete link slab spanning between the abutment and the adjacent motorway bridge structure.
- The northbound and southbound carriageway southern abutments, which are reinforced concrete cantilever retaining walls on slab foundations alongside each other but not structurally connected. The walls are founded on fill and gully alluvium and the foundation slab is buried 0.5 m. The northbound carriageway abutment supports a concrete link slab spanning from the abutment to the end of the adjacent motorway bridge.
- The Terrace Off-Ramp southern abutment, which is a reinforced concrete cantilever wall on a relatively narrow foundation slab. It supports the end of the ramp bridge superstructure.
- The Clifton Terrace abutment structure, which extends from Everton Terrace to north of Clifton Terrace, comprises a precast concrete slab deck supported on two longitudinal reinforced concrete beams on reinforced concrete columns and bored cylinder foundations. The western (hill) side beam retains earth fill placed between the structure and the original hillside. The ground below this structure slopes down to the Clifton Terrace car park at 40° and is backfill placed over the original Clifton Terrace roadway.

The abutment walls were assumed to be sufficiently flexible or displaceable for active pressures to mobilise. Seismic earth pressures on the walls were calculated using the Mononobe-Okabe method. Critical accelerations based on active soil pressures were assessed at which sustained forward
movement of the walls would mobilise. Where the abutment walls were predicted to lose stability by sliding forward, predictions of the amount of forward movement were made based on the Newmark sliding block model. There is no routine procedure for predicting the amount of movement for walls losing stability by tilting, but estimates were made based on the approaches for sliding displacement by considering the forward movement of the centre of mass.

Wall top movements under the 50th percentile MCE event of up to 190 mm for the southern abutments, and 150 mm for the southbound carriageway northern abutment were predicted. For the Terrace Off-ramp abutment displacements of less than 150 mm were predicted due to the stabilising effect of the wing walls and restraint provided by the bridge superstructure. The capacity of these abutments’ structural elements, in all cases, exceeded the seismic acceleration at which sliding or rotational instability initiated.

While damage would occur to “knock off” elements at the ends of the link slabs, the link slabs displaced, and displacements of the abutments would occur, it was assessed that the abutments would retain their load carrying capacities for traffic following a major earthquake event and that seismic retrofitting of the motorway structures southern abutments, of the southbound carriageway’s northern abutment, and of the Terrace Off-Ramp abutment was not justified.

The Clifton Terrace abutment structure is likely to be loaded laterally by soil pressures acting on the crib retaining wall on its uphill side, while the fill material surrounding its supporting columns will provide little lateral support to the structure. This structure was assessed to have only sufficient capacity for static earth pressures likely to be acting on it, with little or no additional capacity to resist earthquake induced soil pressures. Further investigations have been recommended to enable clarification of the earth pressures acting on this structure. Seismic retrofitting of this structure has not been undertaken at this point in time, but may be justified following these further investigations.

8 STRUCTURAL ASSESSMENT AND STRENGTHENING OF THE NORTHERN ABUTMENT

The northbound carriageway northern abutment structure is a reinforced concrete earth retaining wall up to 8.2 m high founded on reinforced concrete bored cylinder foundations. It is stabilised against forward rotation by a reinforced concrete deadman anchor beam buried under the roadway some 18 m back behind the wall, tied to the abutment wall by a series of longitudinal reinforced concrete tie-beams. The abutment supports a reinforced concrete link slab spanning to the northern end of the main motorway carriageway bridge structure.

Assessment of the abutment performance revealed that forward rotation of the wall could be expected to initiate at a peak ground response acceleration of about 0.23g due to passive failure of the soil in front of the deadman anchor beam. Predictions of the amount of forward movement of the top of the wall under the 50th percentile MCE event, by various methods, ranged from 80 to 230 mm. Consequences of this forward movement are likely to be displacement of the link span and some disruption of the road surface behind the abutment.

Over the width of the road carriageway, the depth of overburden above the deadman is relatively constant, but where the deadman extended into the berm on the western side of the motorway to restrain the abutment wingwall, the depth of overburden increased dramatically. The effect of this was to create a “hard spot” at the western end of the deadman, which, under earthquake response, would resist movement while the rest of the abutment wall system was sliding and rotating forward. This would cause a redistribution of loading in the wall tie-back elements, which could result in overloading and failure of the tie-back beam at the western end, and then lead to progressive failure of other tie-back beams and complete instability of the wall.

Complete stabilisation of the abutment wall for the 50th percentile MCE event by rock anchoring was investigated. This approach required limiting the stiffness of the rock anchor restraint to the wall in
order to minimise the level of earthquake loading attracted by the wall. Behind the wall the bed rock surface drops away, requiring anchors to be inclined steeply in order to intersect the bed rock. Positioning and spacing of the anchors was also dictated by the structural capacity of stiffening ribs of the abutment wall and head room limitations for drilling. All in all, this approach involved some significant uncertainties and construction risk, and was estimated to cost in excess of $600,000.

The adopted solution was to accept the likelihood of some forward rotation of the abutment wall, but to excavate overburden above the deadman on the berm down to a level consistent with that over the width of the carriageway to eliminate the restraint “hard spot” and provide the deadman anchor system with a consistent level of performance under earthquake over its length. FRP reinforcement glued on to the face of the abutment wall was also applied to increase the flexural capacity of a diagonal stiffening rib supporting the abutment wingwall, which was found to otherwise lack sufficient capacity.

9 SUPPLEMENTARY STRUCTURAL ANALYSIS

Supplementary structural analysis was undertaken to investigate the effects of the possible variation in foundation soil properties and to confirm the findings of the prior work in the light of these variations.

Each of the six independent frames was modelled as a three-dimensional space frame using the structural analysis software, SAP2000 (version 8). Modelling of the bridge structure and materials was based primarily on the recommendations given by Priestley et al. Piers, abutments and superstructure were modelled using cracked section properties.

Soil support at the piers was modelled using the Elastic Continuum method. This method was considered to provide a more realistic modelling of the foundation stiffness, and provided a stiffer response and thus lower displacement than the simpler Winkler spring (springs acting normal to the axis of the pile) method used in the prior work.

Seismic displacements were predicted for individual frames by modal analysis for upper and lower bound soil stiffness. These displacements were used to determine transverse response, for example at the interface between the Clifton Terrace On-Ramp and the motorway structures.

Inelastic time-history analyses were used to investigate the interaction and relative displacements between adjacent frames and between frames and their adjacent abutments under longitudinal earthquake response.

From the above analyses the required overlap at bridge link spans was determined from equations proposed by Priestley et al, which include allowance for travelling wave effects. This was in the order of 0.6 m for link spans between frames. At the abutments, the required overlap ranged from 0.3 m to 0.8 m.

10 RETROFIT OF THE LINK SPANS

The link spans consist of short reinforced concrete deck panels spanning 2.4m between the adjacent bridge frames or abutments and are supported on bearing pads sitting on a 0.4m wide seat. Each link span is made up of rectangular panels 1.6m wide connected using a stressing cable.

The assessment showed that greater seismic movements needed to be catered for to prevent unseating of the link spans. Unseating could allow them to drop out, effectively closing the motorway and creating a hazard to people and property below. In the event that adjacent frames move towards each other, a “knock off” element allows the link spans to remain intact.

The retrofit consisted of a series of steel corbels fitted below the link spans and fixed to the ends of the main bridge beams or abutments. The steel corbels have a plan footprint of 0.50m wide by 0.53m or
0.34m long depending on location and were spaced up to 1.7m apart. The corbels allow the adjoining bridge spans to move further apart without the link spans dropping out onto the Clifton Terrace car parks and ensure the motorway remains useable.

Steel corbels were connected to the ends of the bridge beams or abutments with four epoxy grouted bolts, and were required to be fixed and positioned in a matrix to:

- Allow the MCE displacements to occur, in both the longitudinal and transverse directions, without the corbels impacting each other
- Ensure fixings did not conflict with existing reinforcement, T-beam prestress anchors or recesses.

The steel corbels were positioned snugly against the underside of the link spans allowing for the insertion of an expanding foam gasket. A protective coating was applied to both the steel corbels and fixings.

11 RETROFIT OF THE DECK EDGE FASCIA UNITS

The fascia units are approximately 5 m long precast concrete units providing an architectural finish to the edges of the bridge deck. They are also used for the collection of storm water from the deck.

At the link spans, these fascia units were cut back to allow the design longitudinal movement between the adjacent bridge frames to occur without the fascia units impacting on each other, which could cause their fixings to break, allowing the units to fall into the car park and onto the pedestrian routes below. The fascia panels were cut back to provide a gap of 0.6 to 1.1m, depending on the movement requirement at the particular link span location. At the Terrace Off Ramp/ Southbound carriageway link span, where it was not detrimental to the bridge aesthetics, a fascia unit was completely removed.

Thin aluminium fascia covers were installed to replace the removed sections of panel. A sliding arrangement allowed the movements expected in an earthquake without impact of the fascia units. The aluminium fascia covers were required to provide aesthetic continuity of the bridge and provide a channel for the flow of storm water. The aluminium fascia covers were connected to the deck with grouted steel rods. Secondary connections using stainless steel cables were also put in place to catch the covers if the bolted connection failed during a seismic event.

Lateral seismic movement of the frames was assessed to be sufficient for the fascia units of the adjacent structures to impact at the junction between the main motorway bridge and Clifton Terrace On-Ramp. A 17 m long steel catch net was attached to the underside of the bridge decks at this location to prevent debris falling to the ground.

Additional work was identified during construction. The U shaped fascia units are bolted to the bridge deck by four bolts. Some of the concrete fascia units at the link span locations were showing signs of fatigue failure or complete loss of the fixing bolt connections due to the higher vibrations induced by traffic loading at these locations. The opportunity was taken to inspect and replace faulty connections.
CONSTRUCTION

The retrofit work, undertaken by Riverside Construction Limited, commenced in July 2003 and was completed one month ahead of schedule in December 2003. There was no disruption to vehicular traffic using the motorway, but pedestrian access and the car park under the bridges was affected by the works. The cost of the physical works contract was approximately $430,000 dollars.

CONCLUSION

The Shell Gully bridge structures have been assessed and retrofitted to withstand the 50th percentile MCE event, a level of shaking intensity expected to exceed that generated by a Wellington fault event. This vital lifeline link for emergency traffic into the Wellington CBD and through the CBD to the hospital and airport to the south is now much less vulnerable to a major earthquake event. Following such an event, these structures are expected to be repairable and able to be restored to full design capacity and returned to normal service.

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REFERENCE: