COMPARISON OF PREDICTED AND OBSERVED SEISMIC PERFORMANCE OF KEKERENGU AND TIROHANGA BRIDGES DURING KAIKOURA 2016 EARTHQUAKE

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ABSTRACT

On November 14, 2016 at 12:02 a.m. local time, the M_w 7.8 Kaikoura earthquake occurred along the east coast of the upper South Island, New Zealand. The earthquake affected a relatively large area and significant impacts occurred to the horizontal infrastructure in the region. This paper focuses on the effects of the ground shaking on two bridges - Tirohanga Stream Bridge and Kekerengu River Bridge along State Highway 1 (SH1). As a result of this earthquake, a new bridge and associated embankments have been constructed at the Tirohanga site and the Kekerengu Bridge has been repaired. One year prior to this earthquake, the same two bridges were the subject of a geotechnical and structural seismic assessment initiated by the New Zealand Transport Agency (NZTA) as part of a programme for assessing the seismic performance of bridges on national strategic routes. This paper compares the predicted seismic performance of the bridges prior to the earthquake and their observed performance during the Kaikoura earthquake. It provides lessons learned for young geotechnical engineers to consider for seismic assessments of existing structures and valuable insights into their performance, and how uncertainties can be accounted for.

1 INTRODUCTION

Tirohanga Stream Bridge (Tirohanga Bridge) and Kekerengu River Bridge (Kekerengu Bridge) are located on the eastern coastal margin of the upper South Island, sandwiched between the uplifting coastal Kaikoura range and the sea. Both bridges are located on SH1; Kekerengu approximately 60km northeast of Kaikoura and Tirohanga Bridge a further 6km north.

Prior to the Kaikoura earthquake a series of detailed seismic assessments were undertaken for a number of bridges as part of the NZTA program for assessing and, where economically justifiable, retrofitting bridges on national strategic routes. Tirohanga and Kekerengu Bridges were assessed as part of this scheme by Beca Ltd (Beca) during 2015 – 2016 (Carding & Stewart 2016, Stewart 2016). The objective of these assessments was to obtain a best estimate of the expected seismic performance of the existing bridges and, where appropriate, implement retrofitting measures. In agreement with NZTA, the performance requirement adopted for the seismic assessment was acceptable levels of damage under a 500 year return period event or damage control limit state (DCLS), and no collapse under an event with a 1,500 year return period. Under the DCLS earthquake, the NZTA Bridge Manual (BM) stipulates that the structure shall be usable by emergency traffic although damage may have occurred, and some temporary repairs may be required to enable use.

At 12:02 a.m. on 14th November 2016, an M_w 7.8 earthquake occurred in the northeast of the South Island of New Zealand causing fault rupture, ground shaking, liquefaction, and landslides. It ruptured the surface in a north eastward direction over a distance of approximately 180km crossing at least twelve faults and initiating up to 10m of vertical and 11m of horizontal movement. This event uplifted the land relative to the sea by 1 – 2m (refer to 2017 NZSEE special bulletin 2017 for details).

Following the earthquake, there was a discussion between KiwiRail and NZTA about how to get road and rail infrastructure back up and running and the question of whether to abandon the coastal road and instead develop an alternate road, was raised. During these discussions, it quickly became clear that repairing and reopening the existing coastal route was the most viable option due to its shorter distance, lower cost of upgrading and higher reliability during similar events. Hence, the New Zealand government announced the formation of the North Canterbury Transport Infrastructure Recovery (NCTIR) alliance between NZTA, KiwiRail, and several major contractors. The purpose was to repair the road and rail networks to reconnect communities with a more resilient network (refer to 2017 NZSEE special bulletin 2017 for details). Following this, NCTIR inspected the affected road alignments, embankments and bridges including our subject bridges i.e. Tirohanga Bridge and Kekerengu Bridge, and proposed rebuild and repair strategies for each structure.

In general, damage to some bridges occurred in the earthquake with some bypassed, but most were able to be opened quickly with restrictions imposed. Observations following the quake suggested that most of the structural damage sustained by the bridges was due to inertial loading, with the damage resulting from geotechnical issues believed to be secondary. Many bridges suffered minor to moderate structural damage due to the development of plastic hinges at the piers. Settlement at bridge abutments was widespread and in some cases prevented access over the bridges until fill or asphaltic concrete was placed to form ramps onto the bridge decks (refer to 2017 NZSEE special bulletin 2017 for details).

This paper presents the impact of the Kaikoura earthquake on the Tirohanga and Kekerengu Bridges and compares the observed performance with performance predicted by the seismic assessment, providing lessons learned for young geotechnical engineers to consider for seismic assessments of existing structures.

2 PRE-EARTHQUAKE ASSESSMENT

2.1 TIROHANGA STREAM BRIDGE

The GNS QMAP indicated that the site was underlain by alluvium, comprising river gravel and sand including modern river beds. This was indicated to be dominantly gravel with subordinate sand and silt. There was no ground investigation undertaken at this site prior to the earthquake, however observations made during the site walkover of the eastern abutment indicated near surface highly plastic firm silt and silty sand.

In the absence of any intrusive ground investigation data, but based on the geological map and inferred sloping bedrock topography, it was surmised that the site subclass would likely be D (deep or soft soil) but that a sensitivity check be undertaken considering it as Class C and even possibly Class E in accordance with NZS1170.5. In addition, both drained (coarse-grained) and undrained (fine-grained) properties for the abutment soils were proposed based on the site observation of surface silty material at the eastern abutment and dominant gravel given on the geological map. The intention was to estimate representative rather than moderately conservative geotechnical parameters. Without site specific data the following were applied:-

- Coarse grained soils: $\phi' = 30^0$, $K_p = 5$ and horizontal modulus of subgrade reaction = 5.z (MN/m2/m)
- Fine grained soils: $C_u = 25 + 10.z$ (kPa), $P_p = \gamma.z + 2\ C_u$ and horizontal modulus of subgrade reaction = 15 (MN/m2/m).

where z is the depth from ground level. The horizontal modulus of subgrade reaction for coarse grained soil was derived based on guidance provided in CIRIAR103. The undrained shear strength was proposed based on observation of consistency of soil on site.

The active Kekerengu fault was located only 100m south of the site and it was emphasized that the surface expression of fault rupture could affect the bridge. As the alluvium material could contain horizons of sand, and because of the high hazard factor[1], in the pre-earthquake assessment the liquefaction risk was considered to be medium.

Tirohanga Bridge comprises two spans of cast in-situ reinforced concrete T-beams. The central pier and the abutments each comprise a cast in situ reinforced concrete wall supported by octagonal precast concrete driven piles, 9.75m and 11m long, respectively. Figure 1 shows the abutments and also the wall pier. It was noted that the bridge had been structurally strengthened in the past with tension bars along the deck beams to increase the vertical load-carrying capacity.

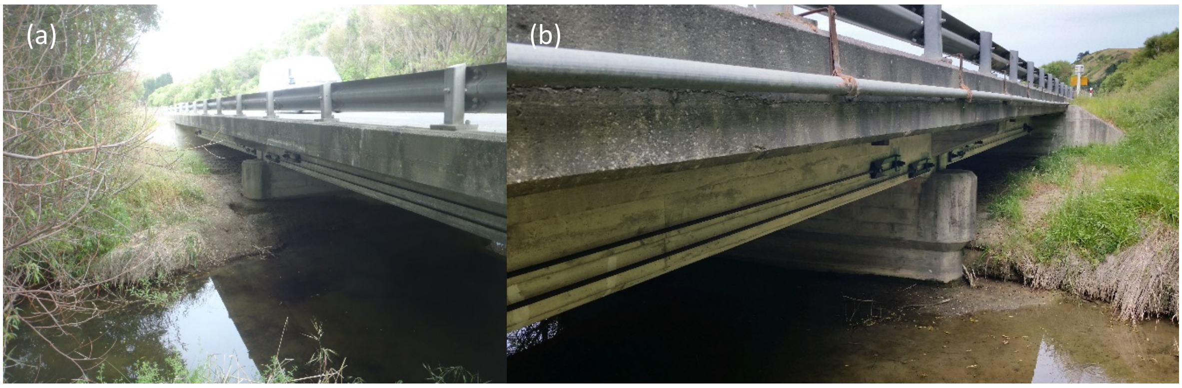

Figure 1. a) Tirohanga Bridge deck and pier, looking south, b) pier and abutment pre earthquake, looking north

[1] NZS 1170.5 Standard has produced a hazard map for New Zealand which gives an appropriate indication of the seismicity of the subject sites; the hazard factor ranges from 0.13 to 0.62 across the country and for the Tirohanga and Kekerengu bridges area, it is approximately within the range of 0.4 – 0.45.

The seismic assessment identified that damage to the piles and abutment walls under the DCLS earthquake would likely comprise cracking and spalling at the top of the piles, with some deformation and settlement at the abutments. Shear failure of the piles under transverse loading under a seismic event with a return period of 1,300 years was predicted. It was surmised that this shear failure could result in loss of support, leading to local collapse or settlement of the bridge. However, as the bridge was shown to have acceptable levels of damage, and the structural components of the bridge to have adequate capacity under DCLS loading, further seismic strengthening was not recommended.

2.2 KEKERENGU RIVER BRIDGE

The GNS QMAP indicated that the site is underlain by alluvium comprising river gravel and sand including modern river beds. Approximately 100m inland the ground rises with the QMAP indicating the slopes to be underlain by Waimea Formation silty mudstone. As with Tirohanga, no intrusive ground investigation was undertaken, however during a site walkover it was observed that the northern abutment was founded on mudstone and the southern on alluvial soil.

In the absence of any intrusive ground investigation data, but based on the geological map and site observations, the subsoil class was considered likely to be Class C (shallow soil) at the southern abutment and possibly Class B at the northern abutment, with a recommendation that a sensitivity check be undertaken for both. Drained (coarse-grained) and undrained rock mass properties were applied to the south and north abutments respectively, with the backfill to the north abutment assumed to be coarse-grained. Without site specific data the following parameters were applied:-

- Coarse grained soils: $\phi' = 30\text{-}34^{o}$, $K_p = 5\text{-}6$ and horizontal modulus of subgrade reaction = 10.z ($MN/m^2/m$)
- Rock mass properties: $K_p = 10$ and horizontal modulus of subgrade reaction = 90 ($MN/m^2/m$)

The horizontal stiffness of the rock abutment side was estimated to be between 3 and 9 times larger, and the strength 2 times larger, than the south abutment.

Similar to Tirohanga Bridge, Kekerengu Bridge is located in an area of high seismicity with the Clarence Fault within 20km of the bridge and the Kekerengu Fault within one kilometre.

The 76m long Kekerengu Bridge was constructed circa 1940. It is a five span, simply supported structure, the cast-in-situ reinforced concrete deck supported off four T-beams which were cast integrally with the abutments and semi integrally with the four piers. The northern abutment is a reinforced concrete wall founded directly on the mudstone rock and the southern abutment is a framed structure comprising a deep capping beam with four short columns and a spread footing founded below the river bed level. The piers are founded on spread footings. The southern abutment is fronted by a sloping wire mattress-faced revetment. The superstructure is monolithic with the abutments. Figure 2 provides a general view of the bridge, including the conditions at both abutments.

Figure 2: Kekerengu River Bridge, bridge overview looking south (left); north abutment on rock (middle); south abutment on soil (right) prior to earthquake

The bridge was assessed in both the transverse and longitudinal directions using a nonlinear static push-over analysis. The performance of the bridge was dependent on the stiffness and capacity of the soil, therefore sensitivity checks were carried out considering double and half the estimated soil stiffness and capacity. In the longitudinal direction, the seismic load was considered to be resisted by the out of plane bending of the walls and the lateral capacity of the soil surrounding the embedded portion of the walls. Transverse seismic loads were considered to be resisted by the in-plane bending capacity, shear capacity and overturning capacity of the pier and abutment walls which transfer the loads to spread footings and then to the ground through vertical bearing. Lateral transverse resistance is provided by the sliding capacity of the walls and partly by the passive resistance of the soil against the sides of the embedded walls. Comparing the results of the analyses for the expected soil properties in each of the longitudinal directions, it was concluded that the performance of the structure is similar in both directions.

In the longitudinal direction plastic hinges were assessed to form in the pier-deck connections at the top of the pier cap beams and at the top of the northern abutment. The southern abutment columns were estimated to start to yield at a seismic load level equivalent to an event with a return period of 200 years. The plastic rotation capacity of the northern abutment wall was assessed to be exceeded at a return period in excess of 1,500 years. In the transverse direction the bridge exhibited satisfactory behaviour and met the seismic performance requirements up to a return period in excess of 2,500 years.

In general, the assessment was found to be quite sensitive to the selection of appropriate site subsoil class. If the results of any subsequent geotechnical investigation concluded it was a Class D (deep or soft soil) site a strengthening scheme would be needed to be developed. However, based on the best estimate of the ground conditions it was concluded that the bridge complies with the damage control seismic performance requirements under the 500 year return period design earthquake, and the probability of collapse was low as the curvature ductility demand was acceptable and hence the structure would be usable after temporary repairs have been carried out.

3 SEISMIC RESPONSE OF THE TWO BRIDGE SITES

The subject sites experienced a Modified Mercalli VII i.e. very strong shaking level during the Kaikoura earthquake (NZSEE special bulletin 2017). Two strong motion stations recorded the ground shaking near this area. These were Kekerengu Valley Road (KEKS) station and Ward Fire Station (WDFS) located on site subsoil Class B (rock) and Class D (deep or soft soil), respectively (Figure 3). Reviewing the accelerograms recorded by these two stations provides valuable insights on the ground shaking experienced in this area. Figure 3 shows the locations of the two bridges in relation to nearby strong motion stations and the trace of fault rupture during the Kaikoura earthquake. The two bridges are approximately 6km apart and similar a distance from the KEKS station. The WDFS station is some 20km north east of the Tirohanga Bridge.

Although the epicentre of the earthquake was 100km south west of the sites, it appeared that there was dominant energy release near the subject sites around 60-70s after rupture initiation (refer to 2017 NZSEE special bulletin 2017 for details). Severe damage was induced by surface rupture across SH1, in particular near to the Tirohanga site where the road was closed.

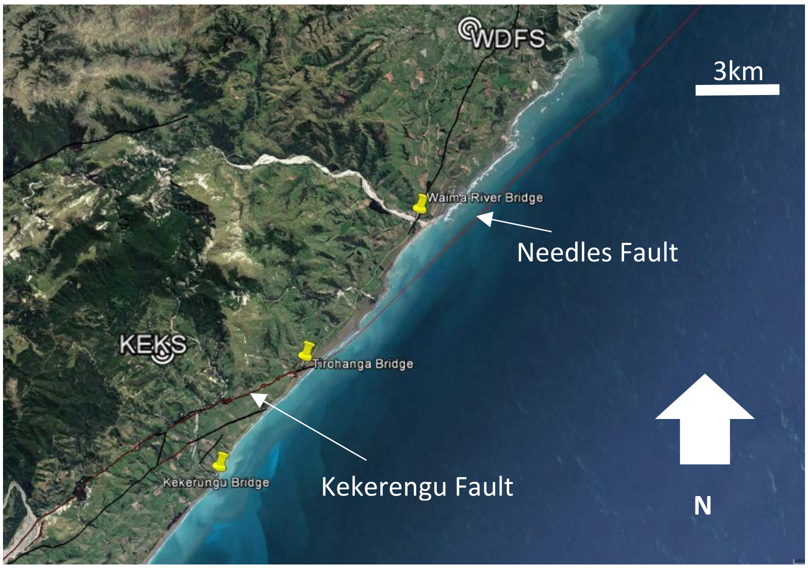

Figure 3: Aerial map of subject sites, black line is the active fault trace from GNS Science NZ active fault map, the red line are the segments that ruptured during Kaikoura earthquake, the yellow line shows the current SH1 road alignment (base map from Google Earth)

The acceleration response spectra of the two strong motion stations, were calculated by GNS Science using the recorded accelerograms (Figure 4). It can be seen that at the KEKS and WDFS stations very large short period accelerations (including peak ground acceleration) occurred, far exceeding the design spectra. The design spectra are calculated based on NZS 1170.5:2004. The design spectra presented in Figure 4 are evaluated assuming a 1/500 year annual probability of exceedance to be consistent with the performance requirements adopted for the seismic assessment undertaken prior to the earthquake (refer to Section 2). Another interesting point regarding the seismic demand imposed by the Kaikoura earthquake was the high vertical accelerations, which were approximately 0.40g recorded by nearby strong motions stations, and more than 1g in other areas.

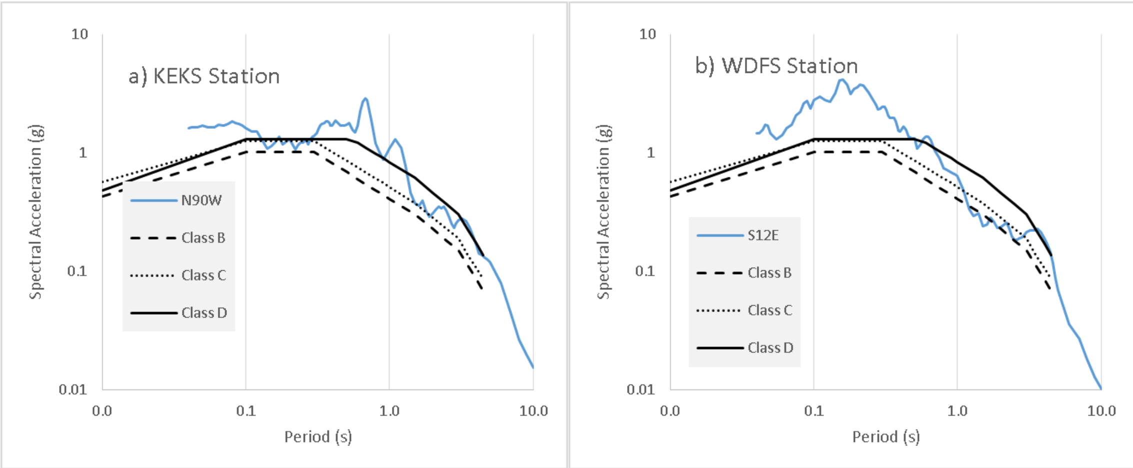

Figure 4: Horizontal 5% damped pseudo-acceleration response spectra observed in: a) KEKS station; and b) WDFS station. The NZS 1170.5:2004 design spectra are drawn for a return period of 1 in 500 years

Although, the recorded accelerograms at the KEKS and WFDS provide insights into the response of the ground during shaking, they may not be representative based on their distance away from our subject sites. Local site effects at the strong motion stations can be significantly different from the bridge sites, hence the local ground motion at the two sites was compared with the simulated ground motion by SeisFinder (Seisfinder 2018). SeisFinder, developed under QuakeCore, is a web application which adopts computationally-intensive earthquake resilience calculations to generate estimated local ground motion simulations.

Figure 5 illustrates the spectral acceleration plots for the nearest SeisFinder simulation stations which are 0.5km and 1.3km respectively from Kekerengu and Tirohanga bridges. By inspection it can be seen that they are broadly similar to those recorded at the KEKS and WDFS stations and, due to the lack of any closer ground motion records, are considered representative of the seismic response of the ground at our sites, for this study.

The seismic assessment of the bridges prior to the Kaikoura earthquake determined that the natural periods of the two bridges depended largely on the stiffness of the soil as well as the extent of nonlinearity during the target seismic shaking and was evaluated to be within the range of 0.2s – 0.8s. However, for the level of damage observed after the earthquake, it is surmised that the effective natural period of the two bridges were within the range of 0.2 – 0.4 seconds. This was the lower bound of expected natural period for the bridge structure and was determined based on limited observed nonlinearity within the superstructure and more importantly within the soil body, indicating the natural period did not increase during the shaking. Figure 5 illustrates that both bridges have experienced much larger levels of shaking, including corresponding peak ground accelerations (PGA), than the design PGAs corresponding to 1 in 500 and 1,500 years return periods.

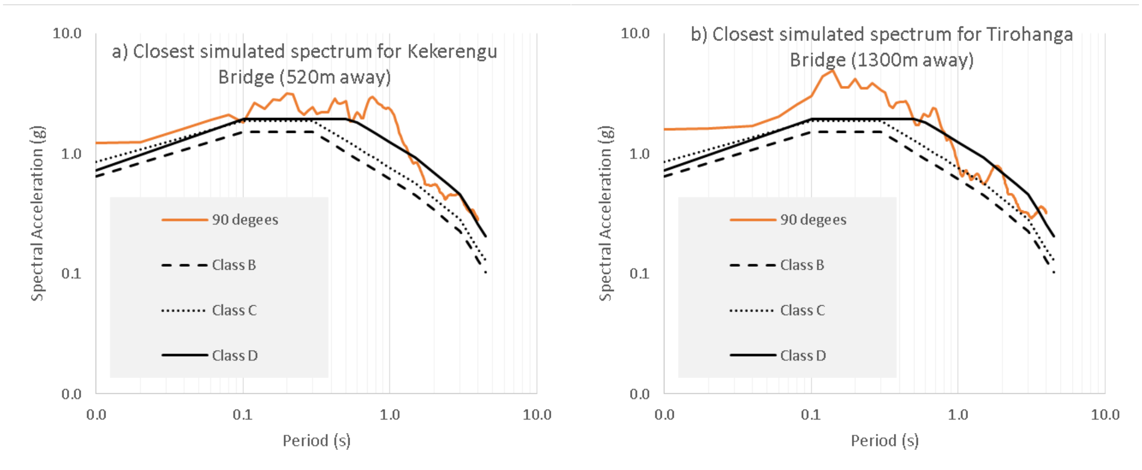

Figure 5. Horizontal 5% damped pseudo-acceleration response spectra simulated for a) Kekerengu Bridge and b) Tirohanga Bridge. The NZS 1170.5:2004 design spectra are drawn for a return period of 1,500 years

4 POST-EARTHQUAKE

4.1 TIROHANGA STREAM BRIDGE

4.1.1 Ground conditions

In 2017 NCTIR commissioned seven boreholes of up to 25m below ground level and 14 CPTs along the road alignment with one borehole and one CPT in the proximity of the southern abutment. The ground investigation identified that the ground comprises 6m of clayey silt over sandy gravel, with the clayey silt layer possibly being embankment fill used in the bridge approaches. The ground water table was found to be at similar elevation as the Tirohanga stream. The site class was evaluated to be the site Soil Class D. Under the ULS design criteria the site was considered prone to liquefaction, resulting in an estimated 25 – 100mm of free-field liquefaction-induced settlement.

4.1.2 POST-EARTHQUAKE INSPECTION

NCTIR’s inspection of the bridge identified that it sustained minor damage comprising settlement of approaches and minor displacement of the piled wing walls relative to the abutments. No surface expression of liquefaction was seen anywhere at the site. In addition, there was no slumping or slope failures evident on the sand dunes on the seaward side of the site and no evidence of lateral spreading of stream banks was found (pers. comm. Nick Tunnicliffe 2018).

Most notably, around 180m south west of the bridge site, where the Kekerengu fault crosses SH1, severe road damage was observed. The fault rupture caused 2.5m of vertical and 5m of horizontal ground movement (Figure 6). The lowering of the ground level on the northern side of the fault allowed water to pond on the ground surface (Figure 7a, below). As such, flooding during future significant rainfall events was identified as presenting a major threat to the structure, which contributed to the proposal to construct a new bridge to replace the existing one.

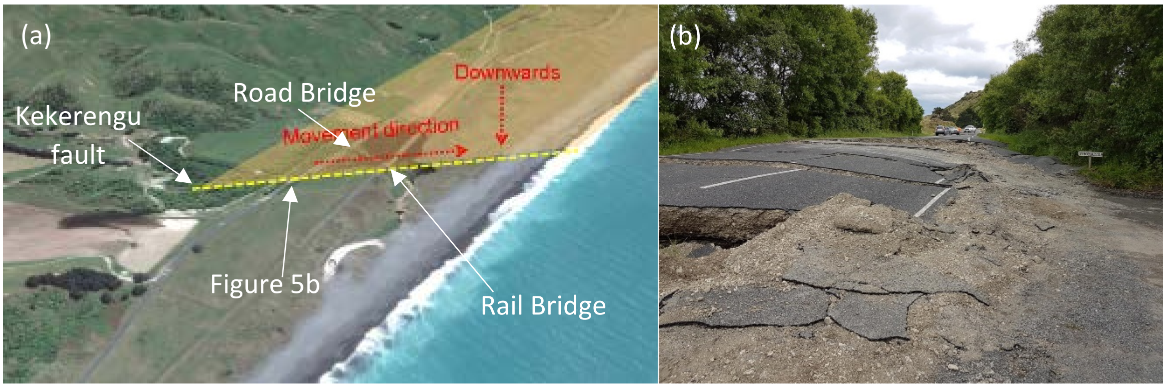

Figure 6: a) Strike-slip fault causing, ~2.5m downwards and 5m horizontal eastwards b) Road damage 185m away from bridge site (Photos courtesy of NCTIR)

Another interesting observation on site was the performance of a KiwiRail Bridge located approximately 100m to the east of the road. Although no slumping or slope failures were observed on the elevated approach embankments, which are located on similar ground conditions to the road bridge, the rail bridge sustained severe damage to the extent that girders unseated from an abutment (Figure 7b). The damage to the rail bridge could be attributed to the proximity to Kekerengu fault trace and also high vertical accelerations.

Figure 7: a) Flooding of the bridge site after earthquake, b) Rail Bridge unseated girder, c) new Tirohanga road and Rail bridges (photos courtesy of NCTIR and GEER)

The new road bridge is shown in Figure 7c and is constructed approximately on the same alignment, but at a significantly higher elevation to reduce the potential for flooding. The higher elevation of the proposed alignment could result in consolidation of the clayey silt layer identified in the ground investigation.

4.2 KEKERENGU RIVER BRIDGE

4.2.1 Ground conditions

Due to the comparatively good performance of the bridge and the requirement for only minor repairs, no subsequent investigation was carried out.

4.2.2 POST-EARTHQUAKE INSPECTION

The bridge was inspected during the initial response by NCTIR (MacDonald 2018). According to the inspection report the bridge structure withstood the earthquakes well and demonstrated considerable robustness, although the bridge approaches were damaged due to minor settlement and horizontal movement. Minor subsidence in the approach embankment, surface undulations at abutments, and relatively minor approach and pavement damage in the form of longitudinal and transverse cracks was observed. The approaches were reconstructed through the placement of fill and resealing, which quickly made this road bridge usable again. Structurally, the post-earthquake inspection also concluded that there was only minor damage including some vertical and diagonal cracking to the pile cap and concrete beams at the southern abutment (Figure 8). The diagonal crack in the pile cap implies a vertical punching shear failure which had begun to manifest itself due to the earthquake shaking. No cracking was observed at the top of the northern abutment. The faces of the abutments and wing walls appeared to remain vertical. Hence, it was concluded that seismic vulnerabilities and severity of localised failures envisaged for the abutments and piers had not manifested themselves as suggested by the pre-earthquake assessment.

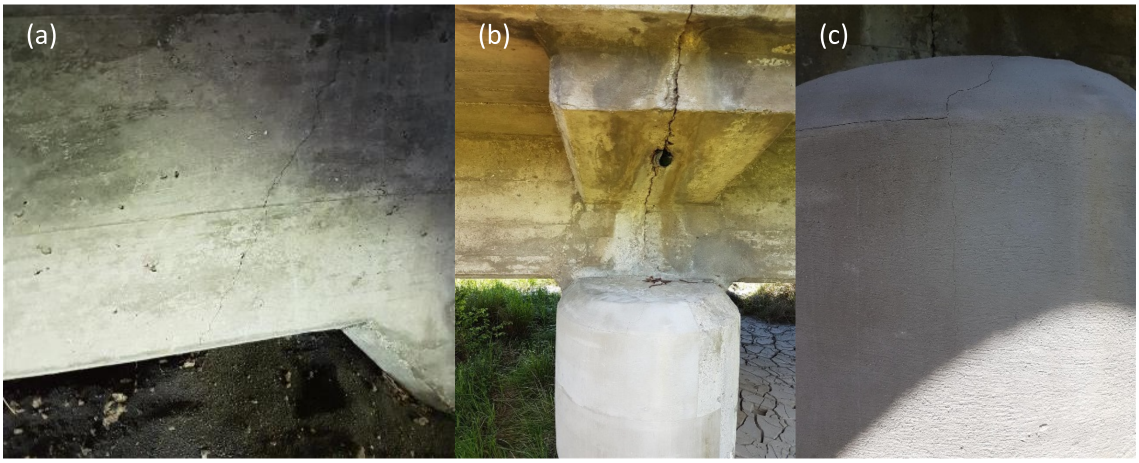

Figure 8: a) Minor cracking of capping beam at southern abutment, b) opening of pre-existing horizontal cracks of construction joints at the beam soffits at southern pier c) cracked concrete repairs at the top of pier (photos courtesy of NCITR)

Despite the high level of ground motion experienced, observational evidence shows that neither of the two bridges showed any signs of significant earthquake damage. However, contrary to these two bridges, many other bridges farther away from the epicentre sustained severe damage.

5 DISCUSSION

The Kaikoura earthquake caused severe damage to infrastructure, and particularly the road and rail transportation networks. Notably large segments of the main highway up the South Island, SH1 and the KiwiRail Main North Line were damaged and had to be closed between Picton and North Canterbury. Closure of SH1 increased travel time between Christchurch and Picton from around 3.5 hours to more than 8 hours. The vulnerability of the state highway network to different and cascading hazards presented a substantial risk throughout the emergency operation, and during the recovery phase. The earthquake highlighted the need for major strengthening and engineering structures along critical transport routes. Therefore, it is very important to learn from this event as well as from similar events experienced around the globe.

Despite severe damage to roads and some bridges, the majority of bridges performed well during the earthquake, including the subject bridges of this paper, which only sustained minor damage. The abutment wall at Tirohanga Bridge moved marginally and no ground damage was observed. However, the wider area encompassing the bridge site settled by about

2 – 3m due to ground displacement across the fault surface. Although a seismic assessment carried out prior to the Kaikoura earthquake indicated that the bridge would meet the performance requirements set out by the Bridge Manual, when subjected to higher levels of ground shaking the level of predicted damage did not materialise.

Meanwhile, closer to the earthquake epicentre, Stinking Stream Bridge on SH7, located 130km south west of Tirohanga site, sustained more severe damage. Although Stinking Stream Bridge was closer to the epicentre, it experienced lower levels of ground shaking than the Tirohanga site. The nearest strong motion station to this site recorded horizontal peak ground accelerations of 0.73g. But the interesting feature of this bridge was that not only was the geological setting similar to Tirohanga, but also the structural form was broadly comparable. However, on this site, due to liquefaction and lateral spreading effects, the approach embankment settled and displacements of the wing walls caused major cracking.

Like Tirohanga, Kekerengu Bridge also sustained much less damage than predicted, noting that greater damage was predicted than for Tirohanga Bridge. The inspection of the bridge site after the Kaikoura earthquake found only minor cracks in the concrete, most of which were deemed to be pre-existing. Although the bridge sustained minor damage, there were other bridges with similar structural configurations and geological settings. For example, farther from the epicentre, Flaxbourne River Bridge and Needles Creek Bridge located approximately 25km north east of Kekerengu Bridge on SH1 suffered moderate to severe damage. These two bridges suffered more damage, the deck of the Flaxbourne Bridge separated from its piers and abutments and plastic hinges were observed at both abutments. At Needles Creek Bridge liquefaction ejecta was observed with lateral spreading cracks near to the piers. Both abutment walls settled 150mm and plastic hinges developed at the top and bottom of the piers (GEER 2017). Considering that the level of shaking was similar to Kekerengu Bridge, it highlights there are many uncertain factors in ground behaviour which makes the assessment of existing bridges difficult.

Palermo et al. (2017) inspected many bridges after the Kaikoura event and they concluded that in general wall pier bridges showed much less damage than other forms of pier structure as this form of construction has higher structural redundancy. Both Kekerengu and Tirohanga Bridges have wall-shaped piers.

These comparisons shows that the uncertainty in the characterization of site, soil and structure properties makes the assessment of existing structures challenging, especially in the absence of specific ground investigation information and as-built drawings. There is the potential, either consciously or subconsciously, of combining conservatism at different levels of site classification, soil stiffness and strength assessments, which can lead to an over-conservative outcome for the assessment of existing structures, especially considering the cost and traffic disruption if significant retrofitting is undertaken.

6 SUMMARY

In summary there are a number of lessons learned from comparison of seismic assessment of the subject bridges with the observed performance after the Kaikoura earthquake:

- In general the seismic assessments carried out prior to the earthquake indicated none of the bridges required strengthening in order to meet the performance requirement set out by the Bridge Manual. This assessment was validated by the earthquake as the inspections showed minor damage within the structure and surrounding soil, even though the shaking experienced during the Kaikoura earthquake was larger than the targets adopted for the seismic assessment. Nevertheless, both Kekerengu and Tirohanga bridges performed much better than anticipated in terms of the extent of repairable damage.
- No obvious soil degradation, e.g. liquefaction, was observed at any of the two bridge sites, whereas other bridges, located in similar geological settings, suffered severe damage as the local soil deposits liquefied. Liquefaction susceptibility across a typical bridge site is extremely difficult to predict reliably, even with some site specific ground investigation information.
- Geotechnical engineering has to consider natural geological materials that are potentially variable both in terms of their properties and their spatial distribution. But combining conservativism in assessing strength and stiffness properties of soil, with soil class, can result in an overly conservative evaluation of soil performance. Conversely, it can also lead to under-estimation of structural loads.
- Whether consciously or subconsciously, there is likely to be an inherent conservatism in geotechnical assessment where there is a lack of any site specific ground investigation data, which will result in predicting more damage in the structure. Checking the sensitivity of the structure to stiffer and stronger soil properties could reduce the predicted damage quite considerably - matching that observed but the design should be based on the best estimate properties of the soil and the structure.
- In the seismic assessment of both bridges, a pushover analysis, which is a pseudo-static structural method was combined with soil stiffness properties derived from CIRIA R103. The latter is based on pile static testing appropriate for long term assessments and not transient seismic loads. Moreover, static methods ignore the

hysteretic damping inherent in dynamic soil response which reduces the estimated damage in the numerical model. In addition, the former ignores the additional damping gained as a result of yielding reinforcement and cracking concrete elements as would be captured in a time history analysis.

- "Local" effects, such as fault rupture, have potential to be more significant than seismic soil behaviour. Generally, the predicted performance of Tirohanga Bridge matched the observations better than Kekerengu Bridge, which sustained comparatively less damage. On the other hand Tirohanga Bridge had to be rebuilt because of the increased flood risk due to settlement of the entire site by 3m. This is an important factor to consider by asset owners - as well as undertaking accurate assessment evaluations, the holistic picture of the structure within a wide geological setting should not be ignored. In this case the subsequent flooding events resulted in rebuilding a bridge which was only damaged minimally.
- Although the predicted thickness, stiffness and strength of the soil strata, when combined with the pseudo-static assessment, were meant to evaluate the 'best estimate' of the seismic response of the Tirohanga and Kekerengu Bridges, the Kaikoura earthquake highlighted that the accuracy of this evaluation is overwhelmed by the potential variability in the inputs. Nevertheless, the prediction assessment does provide invaluable insights into the performance of the structure. Despite the shortcomings, the best outcome of such seismic assessment is enhancing one's engineering judgement. Therefore, it is imperative to have the wisdom of an experienced engineer fully involved in the modelling process, so the modelling skills of the developing engineer can be 'truthed' against the experienced practitioner.
- All bridges influenced by fault ruptures sustained some level of damage. Given a further and possibly stronger earthquake, it is even harder to reliably predict the effects of fault rupture when undertaking future seismic assessments. Therefore, definition of damage control limit state and collapse prevention limit state needs further research in this regard.

In summary, geotechnical engineers, compared to other fields of engineering, work with materials and conditions that are difficult to characterise. Unlike many other engineering disciplines, they cannot simply specify the types of materials or geometries with which they would like to work. The development of the geotechnical input models are strongly dependent on the available data, the experience of the individual engineer and precedent of the profession as a whole. Hence the study of case histories in general, and of failed or collapsed structures in major events in particular, provide valuable insight for future designs if the observed performance is documented systematically.

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