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INNOVATIVE SEISMIC DESIGN OF PEKA PEKA TO ŌTAKI EXPRESSWAY BRIDGES

JAMIL KHAN[1], LAURA CHEN[2]

[1] Technical Director - Structural Engineering, Beca Ltd, Wellington, New Zealand.
[2] Associate Structural Engineer, Beca Ltd, Wellington, New Zealand.

SUMMARY

The Peka Peka to Ōtaki (PP2Ō) Expressway is a 12 km section of the new Kapiti Expressway, north of Kapiti, New Zealand. Several innovative seismic design approaches were adopted for the bridges where sliding of the abutments, and the superstructure at the pier, were allowed underground shaking to give more economical designs and minimise damage at DCLS. This paper explains the modelling techniques in the non-linear pushover analysis undertaken for the structures. The paper also demonstrates how different types of bridge structures can be designed economically with an acceptable level of damage through improved understanding of their non-linear soil response and structural behaviour.

SEISMIC DESIGN PHILOSOPHY

The seismic loads for this project were assessed following the provisions of the Bridge Manual in conjunction with the site specific '*Seismic Hazard Spectra for the Pekapeka-Otaki Expressway*' report undertaken for NZ Transport Agency by GNS Science (GNS Science Consultancy Report 2015/34, 2015). Portions of this spectra are less than 70% of the equivalent design spectra that is produced using NZS 1170.5:2004. The design spectra used is the larger of the site specific spectra, and 70% of the NZS 1170.5:2004 spectra in accordance with section 5.2.3 from the Bridge Manual which states that the "Adopted spectra shall be within ±30% of the design spectrum determined for the specific site from NZS 1170.5", as shown in Figure 1.

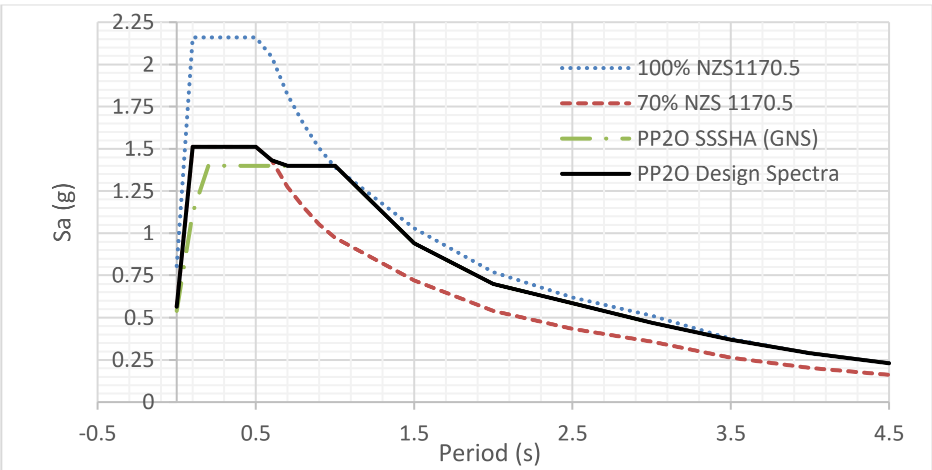

Figure 1: 1/2500 Year Design Response Spectra

The following parameters were adopted in the seismic design of the bridges:

- Design Life of 100 years
- Importance Level 3
- Site Subsoil Class 'D' – Deep or Soft Soil
- Structural Performance Factor; Sp = 0.7 (if "locked in"), 1.0 otherwise
- Seismic Hazard Factor, Z = 0.40 (from NZS 1170.5:2004, however maximum of 70% of the NZS 1170.5:2004 value and site-specific study spectra is used)
- Return period:
  - Minor Earthquake (SLS), $R_{SLS} = R_U/4$ (approx. 100-year return period)
  - Ultimate Limit State (ULS), R_U =1.8 (2500-year return period)
  - Major Earthquake (ME), $R_{ME} = 1.5 \times R_U = 2.7$.

The seismic design of the bridge is required to satisfy the Bridge Manual criteria shown in Figure 2.

	Earthquake severity		
	Minor earthquake (as 5.1.2(b)) Return period factor = $R_u/4$	Design level earthquake (as 5.1.2(a)) Return period factor = R_u (ULS event)	Major earthquake (as 5.1.2(c)) Return period factor = $1.5R_u$
Post-earthquake function - immediate	No disruption to traffic	Usable by emergency traffic	Usable by emergency traffic after temporary repair
Post-earthquake function – after reinstatement	Minimal reinstatement necessary to cater for all design-level actions	Feasible to reinstate to cater for all design-level actions, including repeat design-level earthquake*	Capable of permanent repair, but possibly with reduced load capacity
Acceptable damage	Damage minor	Damage possible; temporary repair may be required	Damage may be extensive; collapse prevented

Figure 2: Seismic Design Performance Criteria from Bridge Manual

The bridges were required to be designed to ensure that collapse does not occur under the Major Earthquake (ME). The requirements are not explicitly defined in the Bridge Manual or NZS 3101:2006. However, NZS 1170.5:2004 commentary Section C2 notes that for most structural configurations, it is assumed that a safety margin of at least 1.5 to 1.8 will be available. Therefore, providing the bridge superstructure is detailed adequately for the ULS demands, it will achieve the requirement of no collapse under the ME. Components critical to the lateral load path (shear keys), are designed for loads generated from the ME acceleration ($1.5R_u$).

WAITOHU STREAM BRIDGE

Waitohu Stream Bridge is a three-span expressway bridge carrying the expressway over the Waitohu Stream and a local road. The Bridge is 94.21m long with a 24.0m wide carriageway. Each span consists of twelve simply supported, 1225mm deep precast prestressed Super-T concrete girders which are supported by laminated elastomeric bearings at the piers and abutments, and tied together by a cast in-situ deck slab and backwall.

The bridge has a straight horizontal alignment and a constant vertical grade, with a small vertical curve near the centre of the bridge. The ground is mainly comprised of clean river gravels overlying interbedded beach alluvium.

The bridge piers consist of reinforced concrete tapered crossheads with up stands, each supported by two 1600mm diameter columns on 2100mm diameter piles.

The bridge abutments consist of a reinforced concrete bank seat beam that is supported on a Mechanically Stabilised Earth (MSE) (steel strap) vertical faced wall.

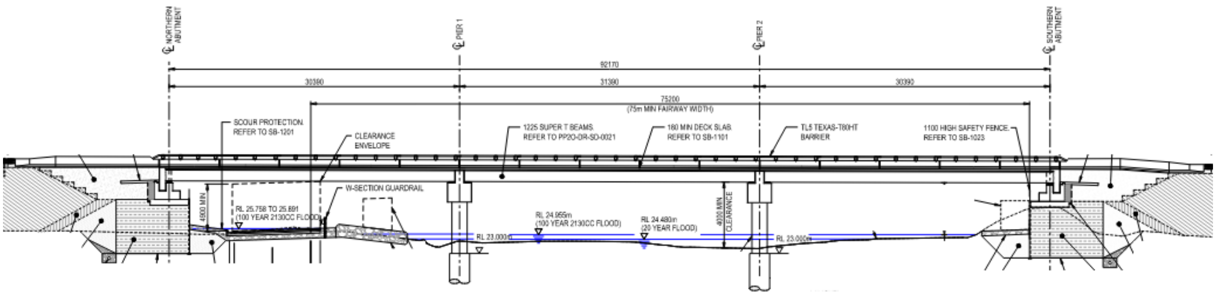

Figure 3: Longitudinal Section of the Bridge

Longitudinal Seismic Action Resisting System

In the longitudinal direction (in the direction of the expressway), the bridge superstructure is considered to be “locked-in”, and therefore a structural ductility factor of $\mu = 1$ was adopted for both ULS and SLS limit states.

Resistance to loads, such as ULS longitudinal seismic and thermal expansion, is provided by the passive pressures generated by the retained soil acting on the rear face of the abutment back walls. The bridge abutments are detailed as “semi integral” to accommodate the movements required to mobilise the necessary passive pressure, without transferring significant load into the abutment beam. This also allows for movement of the embankment in the bridge longitudinal direction.

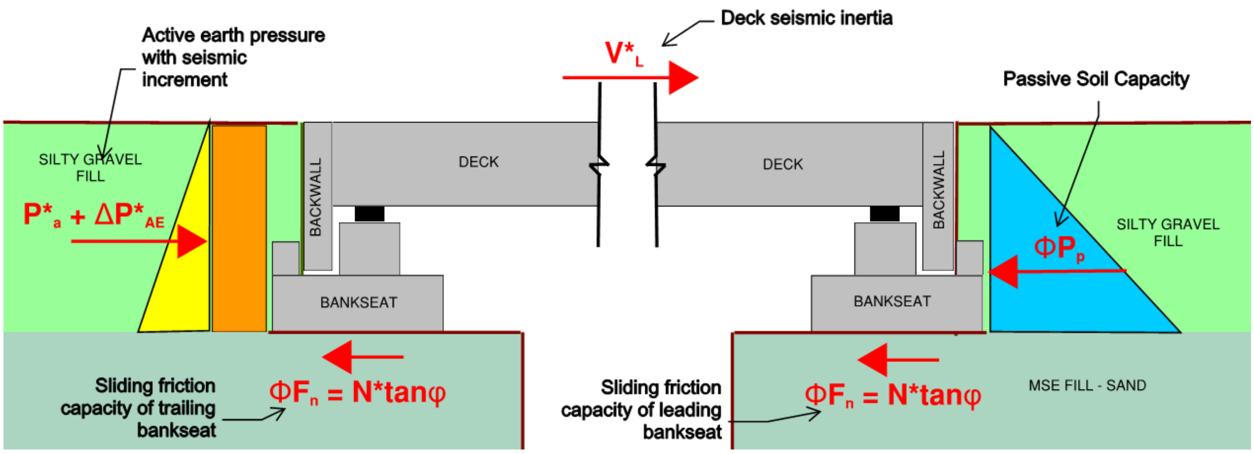

Figure 4: Longitudinal Seismic Loading (piers excluded for clarity)

Seismic inertia (V^*_L) combines with active soil pressure (P^*_a) and the seismic increment of active earth pressure (ΔP^*_{AE}) at the trailing abutment, as shown in Figure 4. This is initially resisted by the longitudinal friction capacity of both bankseats, with the back wall engaging the bankseats through the longitudinal shear key at the leading abutment, and the bankseat upstand at the trailing abutment. Once the friction capacity is exceeded, the backwall and bankseat will slide until the passive soil pressures (ΦP_P) at the leading abutment is engaged as the passive soil compresses, which “locks-in” the bridge structure. In a ME event, the passive soil resistance is exceeded, and the additional load is resisted by cantilever action at the piers.

<u>Transverse Seismic Action Resisting System</u>

In the transverse direction, a different structural ductility factor was adopted depending on the level of shaking. μ = 1 where the frictional resistance of the abutment bankseat on the MSE wall has not been exceeded, so an elastic ‘locked-in’ approach was adopted until this resistance was overcome, in which case a μ = 3 was used for the design of the potential plastic hinge zones of the columns and piles. The design of all other elements is based on the resulting actions arising from the overstrength member capacities of the columns and piles.

The earthquake inertial loading on the superstructure is initially transferred through the transverse shear keys at the abutments into the abutment bankseat, which then transfers the load through friction into the underlying Mechanically Stabilised Earth (MSE) wall and into the ground.

Once the frictional resistance at the bank seats is overcome (during seismic shaking beyond the intensity of the SLS2 event), the deck will slide until the load is transferred through the transverse shear keys at the piers into the pier crossheads, columns and piles in bending and their ductility is taken into account.

Two cases were therefore considered for the modelling and design of the piers:

- Case 1 was modelled with springs representing the sliding friction capacity at the abutments. This results in a shorter period of the structure, with a proportion of the load still resisted at the abutments.

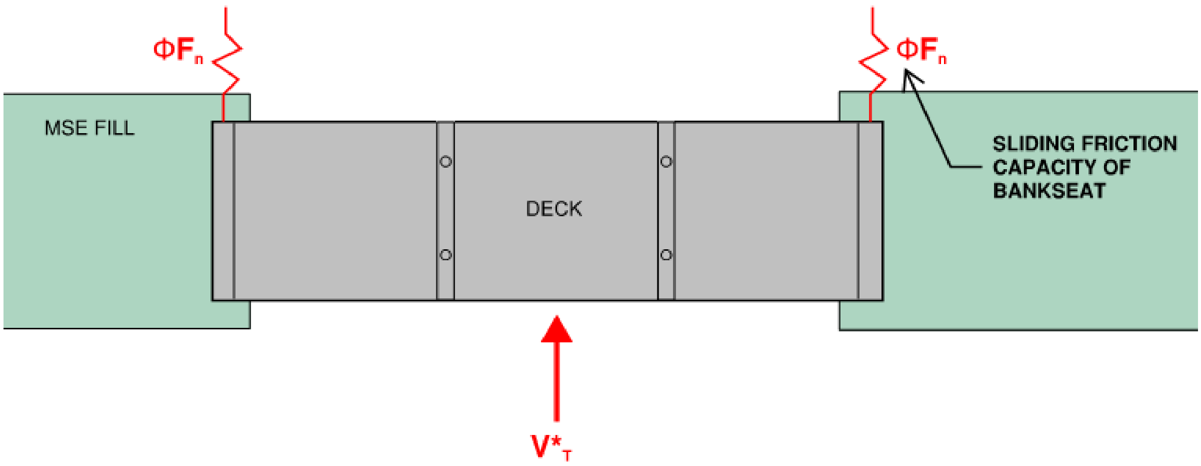

Figure 5: Case 1 – Transverse Seismic Loading with Friction Resistance at Abutments

- Case 2 ignored any frictional resistance at the abutments, and the piers resisted the full seismic load of the bridge. In this case, the abutments were modelled as rollers and the structure had a longer period.

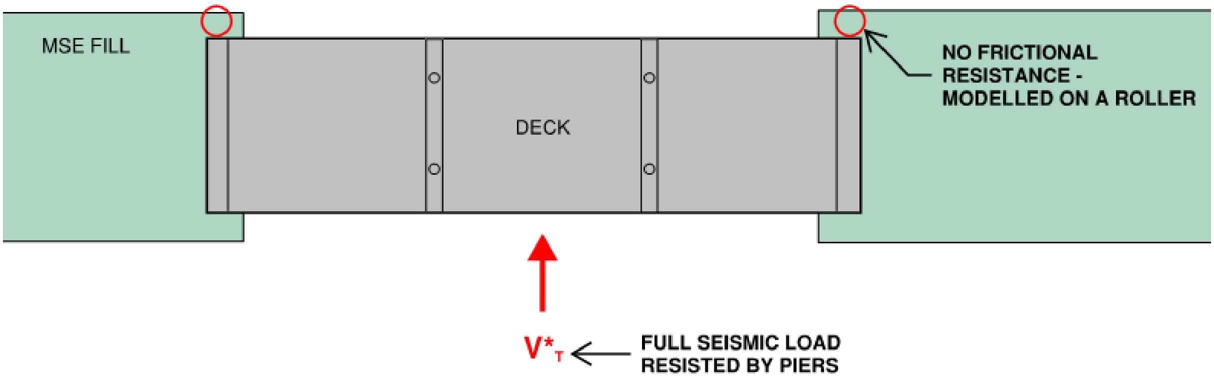

Figure 6: Case 2 – Transverse Seismic Loading with No Friction Resistance at Abutments

Plastic hinges occurred in the pile and the top of the column, with the base of the columns outside of the plastic hinge zones. The plastic hinge lengths were estimated based on the method given in Section 4.2.8 of *Displacement-Based Seismic Design of Structures, Priestley, et al (2007)* which was considered more applicable to the design of bridge piers than the procedure given in NZS 3101:2006. The ductile detailing extends one pile or column diameter, either side of the plastic hinge regions.

Plastic rotations in the piers are checked against the relevant material strain limits given in NZS 3101:2006, and the pile and column's overstrength capacities were used to determine the maximum overstrength shear force in the pier.

As the pier columns are supported on oversized pile shafts, additional detailing is provided at the top of the pile to prevent column anchorage failure, in accordance with the AASHTO bridge design code appendix, 'Precast Bent System for High Seismic Regions".

OTAKI RIVER BRIDGE

Otaki River Bridge is the largest structure of the project. It carries the new Expressway alignment over the Ōtaki River, its associated flood plains and local access roads. It accommodates two lanes of traffic in each direction, a shared path on the eastern side of the bridge and carries services over the river.

The bridge is 333m long with a 24.0m wide carriageway and 2.5m wide shared path. It has 10 spans and does not have a skew.

Each span consists of 11 simply supported, 1525mm deep precast prestressed Super-T concrete girders which are supported by laminated elastomeric bearings at the piers and guided (longitudinally) pot bearings at the abutments, and tied together by a cast in-situ deck slab. Deck joints are provided at each end of the bridge, using proprietary steel finger joints.

The bridge piers consist of reinforced concrete tapered crossheads with up stands, each supported by two 1600mm diameter columns on 2100mm diameter piles, founded on sandy gravel.

The abutments are reinforced concrete beams with back walls, supported by two 1800mm diameter bored concrete piles.

The ground is mainly comprised of clean river gravels overlying interbedded beach alluvium with a silt layer separating the two. At either side of the river banks, a moderately thick layer of soft silt/top soil is encountered.

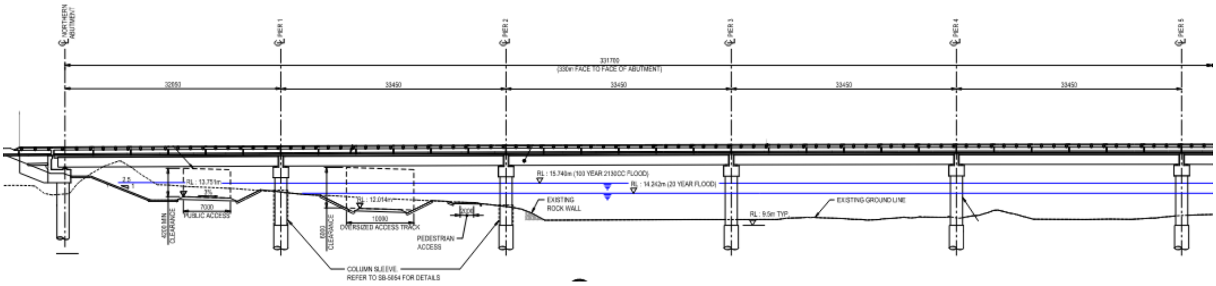

Figure 7: Longitudinal Section of the Bridge (only half shown for clarity)

Longitudinal Seismic Action Resisting System

Longitudinal seismic loads (in the direction of the expressway) are transferred to the substructure through the crosshead upstand wall between the beam ends, and are resisted by the columns and piles in bending (cantilever behaviour). A structural ductility factor of $\mu = 3$ for the design of the potential plastic hinge zones of the columns and piles was adopted. The design of all other elements is based on the resulting actions arising from the overstrength member capacities of the columns and piles. To facilitate the longitudinal inertia loads being resisted by the piers only, the abutment and deck are decoupled, and the abutment bearings are guided (longitudinally) pot bearings, allowing the deck to slide over the abutment.

Transverse Seismic Action Resisting System

Transversely, horizontal seismic demands are transferred from the superstructure to the substructure through shear keys at the piers and abutments, where loads are resisted by the piles in bending. A structural ductility factor of $\mu = 3$ for the design of the potential plastic hinge zones of the columns and piles was adopted. The design of all other elements is based on the resulting actions arising from the overstrength member capacities of the columns and piles.

At the piers, plastic hinges occur in the pile and the top of the column. The pile and column overstrength capacities are considered to determine the maximum overstrength shear force in the pier. The ductile detailing extends one pile or column diameter either side of the upper and lower plastic hinge regions, with these regions determined based on the method given in Section 4.2.8 of *Displacement-Based Seismic Design of Structures, Priestley et al (2007).*

The base of the columns are outside of the plastic hinge zones, with design moment demands taken from the seismic analysis, and shear demands derived from the overstrength shear capacity of the pile. The concrete contribution was included in the overstrength shear capacity of the column section.

As the pier columns are supported on oversized pile shafts, additional detailing is provided at the top of the pile to prevent column anchorage failure, in accordance with the AASHTO bridge design code appendix, ‘Precast Bent System for High Seismic Regions”.

Usability of Bridge in a Post-Earthquake Situation

In a Minor Earthquake, the bridge will not have more than minor damage to the primary structural members and the bridge capacity for live loads will be maintained. Damage to secondary and non-structural components will not significantly impede the operational functionality of the bridge. Damage to such components could be cleared and access restored within 24 hours for full traffic use. Repair of all damage should be accomplished within one month.

In a Design Level Earthquake, the bridge could be useable by emergency traffic within 3 days, although damage may have occurred to the knock-off blocks, expansion joints and bearings. Temporary repairs that may be required to allow the bridge to be available for emergency traffic with control traffic management are described below.

On the trailing edge, where the gap between the deck and back wall will open:

- Remove any protruding damaged elements of the expansion joint. Install 250UB37 beams at 1m spacings. Extend these beams 1m beyond the ends of the Super T beams and knock-off block and install holding down bolts to the Super T end for restraint. Provide timber blocking at the end of beams and infill sand to form approach ramps. Install 25mm thick steel plate over the UBs and approach ramps.

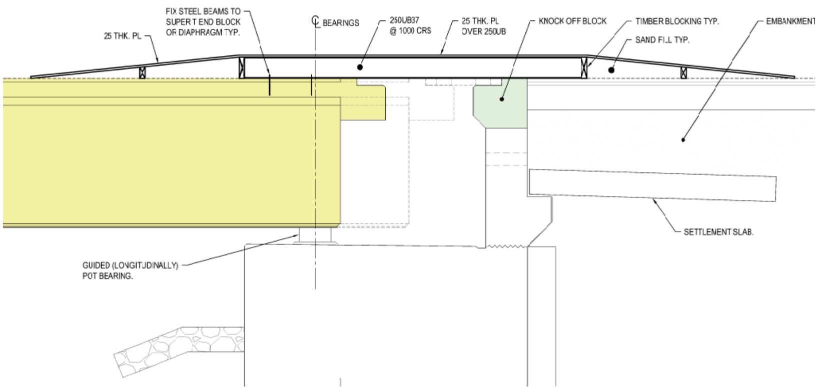

Figure 8: Temporary Repairs at the Trailing End of the Bridge Following a Design Level Earthquake

On the leading edge, where the gap between the deck and back wall will close:

- Remove any protruding damaged elements of the expansion joint, knock off block and approach pavement. Flatten the surface and install a 20mm thick steel plate over the damaged expansion joint. Fix the plate to Super T end blocks/end diaphragms.

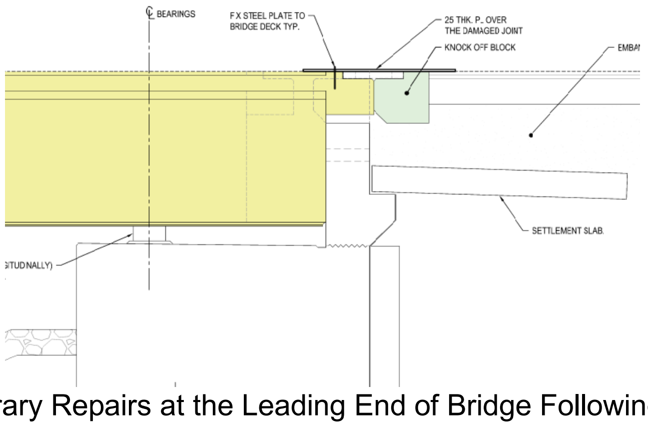

Figure 9: Temporary Repairs at the Leading End of Bridge Following a Design Level Earthquake

In a Major Earthquake, the bridge will not collapse although damage may be extensive. It should be useable by emergency traffic after temporary repairs (as described above) and should be capable of permanent repair, although a lower level of loading may be acceptable.

SCHOOL ROAD BRIDGE

School Road Bridge is a two-span steel composite bridge to carry the local road over the new expressway and rail track. The bridge superstructure consists of 1800mm deep steel I-girders which are tied together by a concrete deck slab and backwall.

The superstructure is supported on elastomeric bearings that sit on reinforced concrete bankseats at the abutments, and a bearing at the top of each of the columns at the pier.

The central pier consists of two diamond-shape columns with 1500mm diameter reinforced concrete-column core that are supported on a single 12.0m x 6m x 1.5m reinforced concrete spread footing.

Shear keys restrain movement in the transverse direction, while allowing some movement in the longitudinal direction before they fully engage. Concrete shear keys in the backwall provide transverse and longitudinal restraint to the superstructure, keying into the bankseat.

The deck is continuous over the pier, where it is supported on anti-uplift guide slide pot bearings.

The bridge semi-integral abutments consist of a reinforced concrete bankseat beam that is supported on an MSE (steel strap) vertical-faced wall on compacted hardfill rafts.

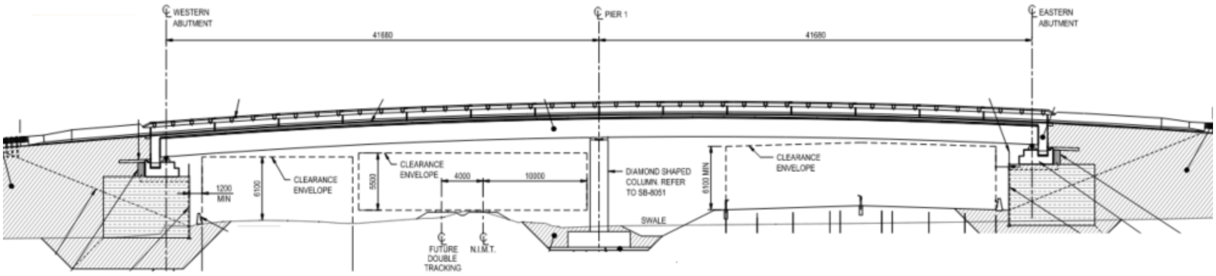

Figure 10: Longitudinal Section of School Road Bridge

Longitudinal Seismic Action Resisting System

In the longitudinal direction, the bridge superstructure is considered to be "locked in", with a structural ductility factor of μ = 1 adopted.

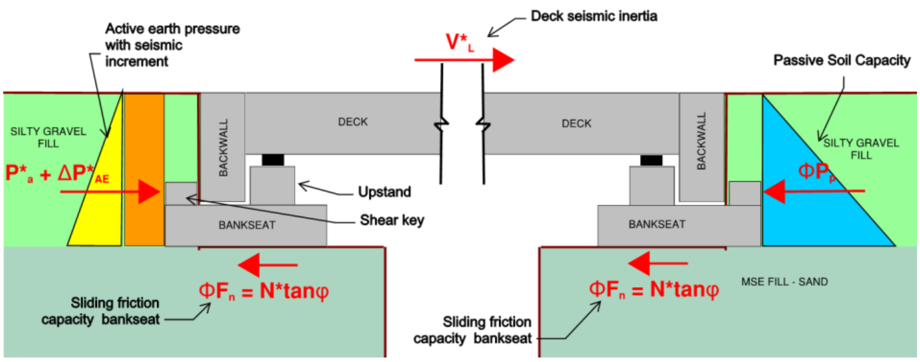

Figure 11: Longitudinal Seismic Loading

Seismic inertia (V^*_L) combines with active soil pressure (P^*_a) and the seismic increment of active earth pressure (ΔP^*_{AE}) at the trailing abutment, as shown in Figure 11. This is initially resisted by passive soil pressures (ΦP_P) at the leading abutment, however, as the passive soil compresses, the back wall at the abutments bear on the bankseat upstand and shear key, engaging the longitudinal friction capacity of both bank seats. These components combine to provide resistance to the ULS longitudinal demands.

The bridge abutments are detailed as “semi integral” to allow them to accommodate the day-to-day movements (thermal), but also to allow them to resist seismic inertial loads through friction when required.

Transverse Seismic Action Resisting System

Frictional resistance, between the underside of the abutment bankseat and the underlying Mechanically Stabilised Earth (MSE) wall, resists the horizontal demands generated by the earthquake shaking and other transverse loads. It is expected that the abutment bankseats will slide on the MSE wall under ULS and ME loading, with the demands exceeding the friction capacity between the abutment and the MSE wall.

In the transverse direction, as shown in Figure 12, the earthquake inertial loading on the superstructure is transferred through the transverse shear keys into the bankseat, which then transfers the load through friction into the MSE wall and into the ground. A small amount of load is resisted through the sliding friction of the pier bearings (μ=0.02). The pier bearings provide a nearly frictionless interface and are sized to accommodate the expected MCE relative movements. However, if this movement is exceeded (beyond MCE), then the columns will engage the ‘keeper ring’ which will mean that further movement is accommodated through column flexure and yielding. To add resilience, the columns have also been detailed for ductility, with their shared footing capacity designed for the overstrength column capacity.

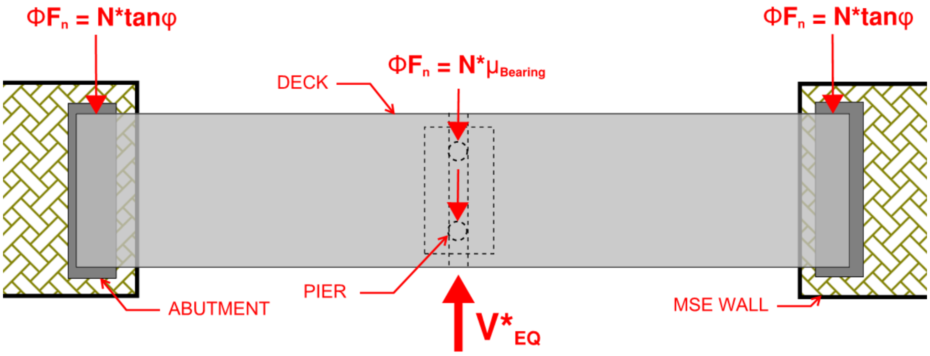

Figure 12: Transverse Load Resisting Arrangement

Beyond SLS loading, in the transverse direction, there is the potential for the bankseats to slide on the MSE wall, giving rise to the potential for residual displacement following a significant earthquake (>SLS). The dimension of the top surface of the MSE wall was determined to provide sufficient seating to the bankseat under the greater of two subsequent ULS earthquake events, or a ME event (R_{ME} = 1.5R_{ULS}). This ME event is associated with an equivalent earthquake return period of greater than 1/10,000 years. A similar philosophy is applied to the design of the sliding pier bearing and its keeper ring, so that the superstructure remains supported. Components critical to the lateral load path (shear keys, etc.) are capacity designed for loads generated from the upper bound abutment sliding capacity, or overstrength capacity of the columns.

Movements greater than allowed for at ME will result in loading and yield of the column, leaving the potential for enhanced P-delta effects on the column when the bridge returns to service. The columns are able to withstand the P-delta actions generated by full ULS HN loading, even when combined with residual column displacements of 300mm (in addition to the 500mm sliding capacity of the bearings) before their stability may start to be compromised.

A simple Newmark sliding block analysis for the transverse direction suggests that sliding of the abutments under ME shaking would be <550mm (225mm average) for friction equal to $\varphi_{crit}=32^o$. This is significantly less than what would be required for unseating of the bankseat to occur from the MSE wall, where edge distances of >1000mm are provided in the transverse direction.

Time History Analysis – Newmark Sliding Block

Newmark sliding block analysis models the bridge as a rigid block sitting on the ground and is used to determine the estimated displacement of the bridge under seismic shaking. An earthquake shaking record is applied to the model and when the frictional resistance is exceeded, the block moves relative to the ground, aggregating over the duration of shaking. This provides an estimation of the expected sliding of the bridge on the MSE walls for a given scaled earthquake record.

Analysis Method

The method of analysis involves numerical integration of the earthquake time history record at times when the ground acceleration exceeds the friction capacity of the block. This is shown diagrammatically in Figure 13. An initial integration over time determines the velocity of the block, with a subsequent integration of the block velocity determining the displacement of the block. With the reaction of the block being the key factor in the frictional resistance, vertical accelerations are of importance and the vertical component of the earthquake record is included in the analysis resulting in variation of the sliding capacity throughout the shaking record.

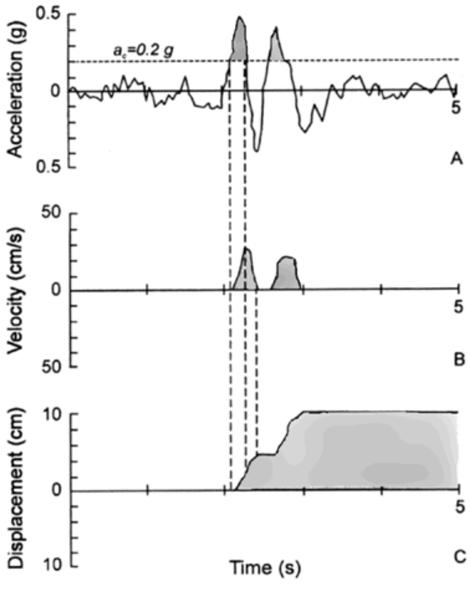

Figure 13: Newmark's Sliding Block Integration Method

Selection of Earthquake Records and Scaling

The suite of earthquake records was selected from those recommended in Table A2 from Oyarzo-Vera et al. (2012) '*Seismic Zonation and Default Suite of Ground-Motion Records for Time-History Analysis in the North Island of New Zealand'*. For the sliding block analyses, these were scaled to match the design peak ground acceleration for ULS (1.0R_u) and MCE (1.5R_u). A separate scaling was applied to the earthquake in each of its horizontal components and vertical component.

Estimated Displacements

Figure 14 shows the expected displacement of the 'block' under the MCE earthquake records from the Newmark's time-history sliding analysis. This is based on an interface friction value of 32° and assumes that 53% (average) of the participating bridge weight is resisted by the abutment reactions. From these 14 analyses (each earthquake component was used separately), the estimated maximum displacement at MCE is 512mm with an average maximum displacement of 157mm. At ULS, this reduced to 36mm and 19mm, respectively.

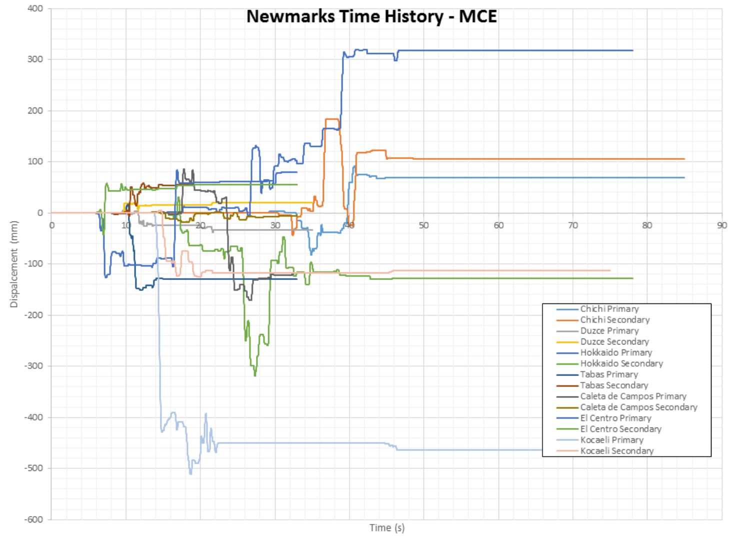

Figure 14: MCE Time Displacement History for Earthquake Suite
(32° interface friction, 53% reaction at abutments)

Time History Analysis – 3D Model in SAP2000

While Newmark's sliding block provides a reasonable estimate of displacements, it is unable to capture any resonance effects that may occur between the earthquake shaking and the bridge. To give greater confidence in the sliding block analysis, time history analysis of a simplified model of the bridge that accounts for mass distribution and stiffness of the structure is used.

Analysis Method

A schematic of the SAP2000 model used for this analysis is shown in Figure 15. The model consists of a spine element with the composite deck properties, which has rigid outriggers that connect the deck to the pier bearings, and the abutment bank seats. Lumped masses are distributed appropriately along the structure and at the abutments. The model is restrained in the longitudinal direction to account for the passive soil pressure behind each abutment. The analysis time step is set at 0.0005 seconds, with Raleigh damping of 5% used for a period range of 0.01 seconds to 1.0 seconds. Peak reaction values are reviewed to check that no unrealistic damping occurs under higher modal effects that could be caused by the rapid loss of stiffness associated with sliding.

The pier bearings are free-float pot bearings, which are considered to give a minimum friction of 2% of their axial loading. These are represented by the 'friction pendulum' link type. When determining the fundamental transverse period of the bridge, the link is given a very low stiffness (roller), to allow for sliding in the modal response. In the time-history analysis, this stiffness is increased to the equivalent column stiffness to allow for its deformation under lateral loading.

A 'friction pendulum' link is also used at each of the abutments to represent their sliding behaviour. Unlike at the piers, the abutments will not slide until shaking is severe. Because of this, their stiffness is included when determining the fundamental period of the structure. The stiffness of the links include allowance for the stiffness of the MSE wall below.

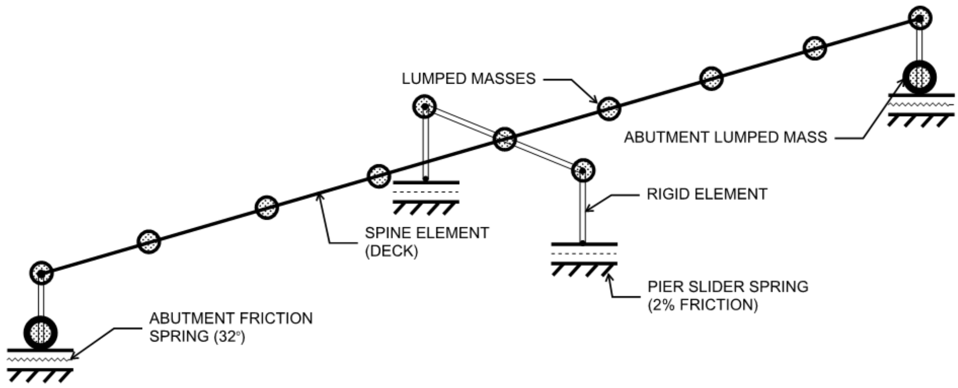

Figure 15: Schematic Seismic Model

Selection of Earthquake Records and Scaling

Initially, three earthquake records were selected from those recommended in Oyarzo-Vera et al. (2012) '*Seismic Zonation and Default Suite of Ground-Motion Records for Time-History Analysis in the North Island of New Zealand'*.

Earthquake records are scaled in accordance with Section 5.5 of NZS 1170.5:2004. The primary record of the two horizontal components is selected. This is determined from the record with the lower k_1 scaling factor over the $0.4T_1$ to $1.3T_1$ period range. This scale factor, combined with any family scaling factor, k_2, was used in the analysis of the horizontal and vertical components. At least one of the selected records includes forward directivity effects, as required in NZS 1170.5:2004. As the purpose of the analysis is to determine the expected displacement, and the main mechanism for transverse displacement is sliding, a structural performance factor (S_p) of 1.0 was used when scaling the earthquake records.

There was some concern that the scaling factors for these records were becoming large and that they may not be representative of realistic ground motions at MCE. Because of this, more records were sourced from the *'Representative Ground-Motion Ensembles for Several Major Earthquake Scenarios in New Zealand'* by Tarbali and Bradley (SESOC Bulletin, 2014). This paper presents several suites of earthquakes that have been determined to best represent the ground motion in Wellington City due to; the Wellington Fault, Wairarapa Fault, and Ohariu Fault, which are the same fault lines that dominate the PP2O Seismic Hazard. Three earthquake records were selected from each fault line scenario. These were selected as those with the lowest scaling factor to try and maintain the appropriateness of the record when scaled to the PP2O demand spectra.

CONCLUSION

Several innovative unique seismic design approaches were adopted for the bridges where sliding of the abutments, and the superstructure at the pier, were allowed under ground shaking to give more economical designs and minimise damages at DCLS.

- Waitohu Stream Bridge deck is designed to slide on MSE walls at the abutments. The longitudinal seismic forces are initially resisted by the friction capacity of both bankseats. Once the friction capacity is exceeded, the backwall and bankseat will slide, until the passive soil pressures at the leading abutment is engaged to "lock-in" the bridge. The transverse seismic forces are initially shared between the piers and abutment through friction between bankseats and MSE walls. Once the frictional resistance at the bankseats is overcome the piers will resist the additional load.

- Otaki River Bridge piers are designed to take full longitudinal seismic loads by decoupling the deck from the abutments. The abutments interact with piers to resist the transverse seismic loads. The piers are pushed to have the maximum demand around the same location for both longitudinal and transverse seismic loads so that the pier piles can be designed efficiently.
- School Road Bridge deck is designed to slide on pot bearings at the piers. The seismic design philosophy for this bridge revolves around the abutments being the primary source of lateral resistance. These provide resistance through passive pressure from the soil behind the abutment in the longitudinal direction, and friction between the bankseat and the MSE wall in both the transverse and longitudinal direction.

By allowing the structure to slide at ULS shaking and beyond, the design actions were able to be reduced giving an economic design solution, while still providing a robust structural system. Modelling of this can be undertaken as a rigid block, however this generally underestimates maximum displacements when compared to a 3D model, especially at lower shaking intensities when modal amplification can be the difference between sliding being initiated, or not. However, the rigid block model can still be useful for preliminary estimates of displacement.

At the time this paper was written, the construction of these bridges was nearly complete, with the project due to open in 2022.

ACKNOWLEDGEMENT

The authors would like to acknowledge the numerous and continuing contributions of the Waka Kotahi NZTA, other partners and suppliers in the development and implementation of the innovative approaches taken on this project.

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