Resistance From Bridge Abutment Passive Soil Pressure in Earthquakes

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ABSTRACT:
This paper summarises the results from recent experimental and analytical research on the force resistance available from passive soil pressures on wall type and bridge abutment structures translated against the backfill in earthquakes, and shows how the results can be applied in bridge seismic assessment and design.

1 INTRODUCTION
In current New Zealand bridge design practice there is a preference for isolated or semi-isolated superstructures using elastomeric or sliding bearings at the abutments, particularly for longer bridges. To some degree this approach may have arisen because of conservative estimates of the passive earth pressure resistance available when rigidly connected or monolithic type abutments are used, and uncertainty about the reliability of passive resistance under cyclic loading.

Recent assessments of the earthquake performance of State Highway and Territorial Authority bridges in New Zealand (Chapman et al, 2005; Wood et al, 2007; Wood, 2009) have shown that passive pressures at the abutments can provide significant resistance to longitudinal displacement leading to reductions in plastic deformation demands on the piers and their foundations. On small bridges this resistance may often be larger that the resistance available from the other components in the substructure. Longer over-crossing bridges often have high abutment walls. Because the passive resistance force is proportional to the square of the wall height the abutments of these longer bridges may also provide a large part of the total resistance. In addition to providing passive resistance, the abutment structure can introduce frictional damping when the forces reverse and the abutment slides outwards from the backfill.

Over the past 25 years, more than 10 experimental and theoretical studies of soil-structure interaction at bridge abutments and bridge pile caps have been carried out, and the collective results from this work provide a reliable base of information on passive pressure force-displacement relationships and how these are modified by cyclic loading. Given this information, there is merit in considering a wider application of monolithic abutments in the design of short to medium length bridges. With appropriate structural detailing and the use of dense granular backfill material, abutments can be designed to provide soil friction damping and economical resistance to superstructure inertia forces in both principal loading directions. Eliminating costly deck joints is a further advantage of using monolithic connections to the substructure.

2 EXPERIMENTAL RESEARCH
Experimental research related to monolithic bridge abutments was initiated in 1985 under a United States – New Zealand cooperative research project. Cyclic loading of model walls was carried out at the New Zealand Ministry of Works and Development Central Laboratories (now Opus Central Laboratories), (Thurston, 1986a, 1986b, 1987), and dynamic small-scale model centrifuge testing and prototype field studies were carried out by Earth Technology Corporation, California (Hushman et al, 1986; Crouse et al, 1987; Wood and Elms, 1990).

Cyclic lateral load testing of two large-scale piled bridge abutments was reported by Maroney and Chai (1994) and Maroney et al (1994). Gadre and Dobry (1998) carried out cyclic lateral load testing
on centrifuge models of bridge pile caps and abutments and Duncan and Mokwa (2001) carried out static load tests on an anchor block with its top surface at ground level. Cyclic load testing of a bridge pile cap backfilled with a number of different soils was carried out Rollins and Cole (2006). A piled bridge abutment structure was used by Heiner et al (2008) to cyclically load both mechanically stabilised (MSE) and un-stabilised approach fills. Wilson and Elgamal (2009) carried out static tests followed by dynamic loading of the backfill on a large scale model of a bridge abutment wall.

Details of the structure type, wall dimensions, and backfill soils used in relevant experimental research projects are given in Table 1.

Table 1. Experimental research projects relevant to bridge abutments

<table>
<thead>
<tr>
<th>Reference</th>
<th>Structure Type</th>
<th>Wall Height m</th>
<th>Wall Width/Height</th>
<th>Soil Type</th>
<th>Backfill Strength Parameters</th>
<th>Test Type and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thurston, (1986a, 1986b, 1987)</td>
<td>Model wall using steel plate</td>
<td>1.0</td>
<td>2.4</td>
<td>Dense sand</td>
<td>$\phi = 40^\circ$</td>
<td>Static cyclic testing. Both wall translation and wall rotation displacements. Sand was gravelly well-graded and moist.</td>
</tr>
<tr>
<td>Maroney and Chai (1994)</td>
<td>Full-scale bridge abutments on piles</td>
<td>1.7</td>
<td>1.8</td>
<td>Firm clayey silt</td>
<td>$c = 96$ kPa $\phi = 0^\circ$</td>
<td>Static cyclic testing to failure. Larger abutment was used as reaction frame. A theoretical estimate was made of stiffening effect of piles.</td>
</tr>
<tr>
<td>Gadre and Dobry (1998)</td>
<td>Bridge abutment</td>
<td>1.52</td>
<td>3.8</td>
<td>Dense sand</td>
<td>$c = 0$ kPa $\phi = 39^\circ$</td>
<td>Centrifuge models, static cyclic loading. Dry Nevada sand.</td>
</tr>
<tr>
<td></td>
<td>Bridge pile cap</td>
<td>0.84</td>
<td>1.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Duncan and Mokwa (2001)</td>
<td>Anchor block – top surface at ground level</td>
<td>1.1</td>
<td>1.8</td>
<td>Dense gravel/silt</td>
<td>$c = 48$ kPa $\phi = 35^\circ$</td>
<td>Static monotonic loading to failure. Block was 0.9 m long in direction of loading and base and end friction would have increased stiffness during initial loading.</td>
</tr>
<tr>
<td>Rollins and Cole (2006)</td>
<td>Bridge pile cap</td>
<td>1.1</td>
<td>4.6</td>
<td>Sand</td>
<td>$c = 0$ kPa $\phi = 39^\circ$</td>
<td>Static cyclic testing. Multiple cycles to failure. No contact with soil on base or ends. Stiffening effect of piles estimated by tests without backfill to give passive wall pressure component.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Silty sand</td>
<td>$c = 27$ kPa $\phi = 27^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Fine gravel</td>
<td>$c = 3.8$ kPa $\phi = 34^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Coarse gravel</td>
<td>$c = 7.2$ kPa $\phi = 40^\circ$</td>
<td></td>
</tr>
<tr>
<td>Heiner et al (2008)</td>
<td>Bridge pile cap</td>
<td>1.7</td>
<td>2.0</td>
<td>Dense sand</td>
<td>$\phi = 39^\circ$</td>
<td>Static cyclic loading. Stiffening of piles estimated by tests without backfill. MSE stabilized fill also tested.</td>
</tr>
<tr>
<td>Wilson and Elgamal (2009)</td>
<td>Bridge abutment</td>
<td>1.7</td>
<td>1.7</td>
<td>Dense silty sand</td>
<td>$\phi = 42^\circ$</td>
<td>Static monotonic loading followed by shaking table input to backfill.</td>
</tr>
</tbody>
</table>
2.1 Force-Displacement Curves

Passive resistance force-displacement curves from the horizontal translation tests on the wall structures listed in Table 1 are shown in Figure 1 (excluding the Crouse et al walls). Except for the Duncan and Mokwa curves, which were plotted directly from monotonic loading results, the curves are cyclic loading backbone curves derived from the peak force response at each displacement increment.

To illustrate more clearly the general trends and the differences between the curves, the experimental results have been smoothed to a limited degree. To further simplify the comparison, corrections have been applied to remove three-dimensional effects significant for width/height (W/H) ratios less than 5.0 (see Figure 6 below). With the exception of the Duncan and Mokwa results, the curves are for the passive resistance only with the stiffening effect of any piles, and friction on the underside and ends of the walls or caps, eliminated by numerical or experimental corrections. Base and end friction resistance are included in the Duncan and Mokwa curves but these effects are unlikely to be significant near the maximum loads when the block tended to lift up with slip likely on the surfaces not loaded by passive pressure. However, in the initial loading stages of the test in the sandy silt/clay natural soil the cohesion on the base and ends would have contributed to the resistance.

The axes in Figure 1 are dimensionless with the force \( P_d = P/0.5\gamma H^2 W \), where \( P \) is the total force on the wall and \( \gamma \) the unit weight of the soil. Plotting the force in this form results in a convenient relationship between \( P_d \) and the ultimate passive pressure coefficient, \( K_p \). (For a cohesionless soil and smooth wall the ultimate total force \( P_u = 0.5K_p \gamma H^2 W \) giving the ultimate dimensionless force \( P_{du} = K_p \).) The wall displacement, \( u \), has been made dimensionless by divided by the wall height.

![Figure 1. Experimental passive force-displacement curves for translation of wall structures listed in Table 1.](image)
Values of the ultimate passive dimensionless force shown in Figure 1 range from 10 (Thurston, dense sand, and Duncan and Mokwa dense gravel) to 25 (Rollins and Cole, coarse gravel). With the exception of Rollins and Cole coarse gravel and silty sand results the ultimate forces were within a rather narrow band of 10 to 16. These values are much higher than the values of 3 to 4.6 traditionally derived from Rankine theory for design applications. The ultimate dimensionless displacements (u/H), measured at the point where there was significant decrease in passive force, ranged from 0.03 to 0.1. Initial stiffness values measured at u/H = 0.001 range from 1,000 to 5,400 (dimensionless). The very stiff initial response of the Duncan and Mokwa anchor block in the natural soil test was probably caused by friction and cohesion on the base and end walls.

### 2.2 Cyclic Load Effects

No consistent test loading procedure was used in the cyclic load tests. Generally the tests were conducted by loading to a specified displacement then reducing the displacement to zero and reapplying the load to the same displacement with two to eight repeat cycles before the displacement increment was increased. There was no consistent rate of loading and the hold times at the peak displacement of each cycle varied. As a consequence of the different loading procedures, comparisons between the force-displacement curves can only be made in general terms.

Rollins and Cole (2006) reported that for a given deflection greater than u/H = 0.005 there was typically a 10 - 20% reduction in the peak load between the first and last of up to seven cycles with a large part of the degradation occurring in the first repeat cycle. (This evaluation was from results that included the effects of the piles.) Figure 2 shows the backbone and reloading curves for the wall passive pressure force in the dense gravel tests. As the peak displacement increases, the reloading curves depart significantly from the shape of the backbone curve. Most of this change is related to gaps that develop between the soil and wall. Very little passive force develops until the gap closes and then the passive force develops very rapidly until the stiffness decreases (concave down part of the reloading curve) as the displacement approaches the previous peak value. For u/H ratios greater than 0.02, the observed gap widths for all four soil types tested were typically about 50% of the peak displacements at the initiation of the first cycle at a particular displacement level. This percentage was about 30% at u/H = 0.01.

A cyclic force-displacement plot from the Thurston (1987) tests on dense sand are shown in Figure 3. The displacement cycles included movement away from the soil and only two cycles were repeated at each displacement increment. This loading sequence is more typical of that expected in an earthquake than the sequence applied in the Rollins and Cole tests.

The Thurston hysteresis loops show less departure from the backbone curve than the results of Rollins and Cole. This can be attributed to the gaps tending to fill as the wall moves away from the soil and the greatly reduced number of loading cycles. Following cycles to u/H = 0.008 the displacement at zero load was about 40% of the peak displacement.

![Figure 2. Rollins and Cole (2006) tests on coarse gravel. Backbone and continuous cycle force-displacement curves.](image1.png)

![Figure 3. Thurston (1987) tests on dense sand. Force-displacement hysteresis curves from complete cyclic load test.](image2.png)
3 ANALYTICAL METHODS

Maximum passive pressures can be computed using the well-known Rankine, Coulomb and Log-Spiral earth pressure theories. A summary of the basis and limitations of these theories is given by Duncan and Mokwa (2001). The main limitation for the present application is that none of these theories provide information of the relationship between passive resistance and wall displacement. The Log-Spiral method assumes a curved failure surface and is considered to give the most reliable estimate of the maximum passive resistance. Because of its complexity it is not widely used but charts of Log-Spiral pressure coefficients are available for simple wall geometries where the soil cohesion is zero. The Coulomb method assumes plane failure surfaces and is applicable to any wall geometry and to soils with cohesion but overestimates the passive resistance when the wall friction and backfill slope are significant. Rankine theory is generally only applicable to smooth walls and cohesionless backfills with a horizontal surface. It may grossly underestimate the passive resistance.

Typical values of $K_p$ computed by Sokolovski (1956) and by Caquot and Kerisel (1948) using Log-Spiral theory are shown in Figure 4. Curves are shown for the wall friction angle, $\delta$, equal to the soil friction angle, $\phi$, and for $\delta = 0.5 \phi$. For dense cohesionless soils the Log-Spiral method gives $K_p$ values of 8 to 18.

To compare the Log-Spiral theoretical pressure coefficients with the test results shown in Figure 1 it is necessary to consider the effects of friction at the wall-soil interface. In most of the tests the walls were restrained from vertical movement so significant friction would have been mobilised on the interface with the walls tending to lift as they are forced against the backfill. The tests measured the horizontal component of the passive force whereas the Log-Spiral theory assumes that the pressure force on the wall is inclined at the wall-soil interface friction angle to the horizontal. That is:

$$P_d \text{ (measured)} = K_{pl} \cos \delta$$

Where $\delta$ = the wall-soil interface friction angle and $K_{pl}$ is the equivalent passive pressure coefficient from the Log-Spiral theory.

The horizontal passive resistance from the test results (maximum $P_d$ in Figure 1) and the Log-Spiral $K_{pl} \cos \delta$ equivalent horizontal pressure coefficient are compared in Table 2. (Log-Spiral solutions were unavailable for the clayey silt in the Maroney et al tests and the silty sand in the Wilson and Elgamal tests.) The theoretical predictions were within 25% of the measured ultimate passive forces demonstrating the validity of the Log-Spiral theory for predicting ultimate passive pressures on abutment structures.

Shamsabadi et al (2007) used limit-equilibrium methods based on Log-Spiral failure surfaces coupled with modified hyperbolic soil stress-strain behaviour to estimate abutment nonlinear force-displacement relationships as a function of the backfill soil shear strength parameters. Analytical results from this approach were verified by comparing them with some of the tests discussed above. The method requires numerical computer based evaluation and is not directly suitable for application to design. From these detailed numerical analyses involving hyperbolic soil stress-strain behaviour, Duncan and Mokwa (2001) and Shamsabadi et al (2007) developed simplified relationships to model the variation of passive resistance with wall displacement. The Duncan and Mokwa relationship is more easily applied than the Shamsabadi et al expression and was presented in the form:

$$P = \frac{1}{K_{max}} + \frac{R_f}{u} \frac{u}{P_{ub}}$$

(1)
Table 2. Comparison between measured passive resistance and Log-Spiral theory

<table>
<thead>
<tr>
<th>Test Reference</th>
<th>Soil Description</th>
<th>Soil Friction Degrees</th>
<th>Wall Friction Degrees</th>
<th>Soil Cohesion kPa</th>
<th>Test Ultimate Pd</th>
<th>Log-Spiral Kpl cos δ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thurston</td>
<td>Dense sand</td>
<td>40</td>
<td>20</td>
<td>0</td>
<td>9.2</td>
<td>10</td>
</tr>
<tr>
<td>Thurston</td>
<td>Loose sand</td>
<td>34</td>
<td>17</td>
<td>0</td>
<td>6.5</td>
<td>6.5</td>
</tr>
<tr>
<td>Gadre &amp; Dobry</td>
<td>Dense sand</td>
<td>39</td>
<td>39</td>
<td>0</td>
<td>11</td>
<td>13</td>
</tr>
<tr>
<td>Duncan &amp; Mokwa</td>
<td>Sandy silt/clay</td>
<td>35</td>
<td>4</td>
<td>48</td>
<td>19</td>
<td>22</td>
</tr>
<tr>
<td>Duncan &amp; Mokwa</td>
<td>Dense gravel</td>
<td>50</td>
<td>6</td>
<td>0</td>
<td>12</td>
<td>10</td>
</tr>
<tr>
<td>Rollins &amp; Cole</td>
<td>Sand</td>
<td>39</td>
<td>30</td>
<td>0</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td>Rollins &amp; Cole</td>
<td>Silty sand</td>
<td>27</td>
<td>21</td>
<td>27</td>
<td>19</td>
<td>21</td>
</tr>
<tr>
<td>Rollins &amp; Cole</td>
<td>Fine gravel</td>
<td>34</td>
<td>26</td>
<td>3.8</td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>Rollins &amp; Cole</td>
<td>Coarse gravel</td>
<td>40</td>
<td>30</td>
<td>7.2</td>
<td>22</td>
<td>21</td>
</tr>
<tr>
<td>Heiner et al</td>
<td>Dense sand</td>
<td>39</td>
<td>30</td>
<td>0</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td>Fang et al (1994)</td>
<td>Loose sand</td>
<td>31</td>
<td>19</td>
<td>0</td>
<td>5.5</td>
<td>5.7</td>
</tr>
</tbody>
</table>

Where \( P \) = horizontal passive resistance per unit width of wall; \( P_{ult} \) = ultimate horizontal passive resistance per unit width; \( u \) = wall displacement; \( K_{max} \) = initial slope of the force-displacement curve; \( R_f \) = failure ratio = \( P_{ult} / \text{hyperbolic asymptote as } y \rightarrow \infty \). From comparisons with test results \( R_f \approx 0.9 \).

From finite element plane strain analyses of a rigid wall translated against an elastic uniform soil Wood (1985) showed that:

\[
P = 0.5 \, E \, u
\]  

(2)

Where \( E \) = Young’s modulus of elastic soil

From equation (2), the initial stiffness can be written as \( K_{max} = 0.5 \, E_i \), where \( E_i \) is the initial tangent Young’s modulus for a nonlinear soil. Using this expression for initial stiffness and converting to dimensionless form gives:

\[
P_u = \frac{u_d}{\frac{\gamma H}{E_i} + 0.9 \, \frac{u_d}{P_{du}}}
\]  

(3)

Where \( u_d = u/H \);

\( P_{du} \) = dimensionless ultimate horizontal passive force and the other terms are as defined previously.

A comparison between the hyperbolic relationship of equation (3) and the backbone curve from the dense sand cyclic load tests carried out by Thurston (1987) is given in Figure 5. The hyperbolic curve parameters were \( P_{du} = 9.6 \) and \( E_i = 50 \, \text{MPa} \). This value of \( E_i \) is equivalent to an initial dimensionless wall stiffness \((u/H = 0.001)\) of 2,200.

The empirical hyperbolic force-displacement curve is a moderately good fit to the experimental backbone curve and would be satisfactory for assessment of bridge performance or in design applications for this type of backfill soil.

Figure 5. Thurston (1987) dense sand test backbone curve and fitted hyperbolic force-displacement curve.
Passive pressure theories and the hyperbolic force-displacement relationship are based on plane strain assumptions and therefore only apply to walls that are long in relation to their height. There is little published information on the increase in wall passive resistance and stiffness per unit length of wall expected for walls that are short in the direction normal to the load. Duncan and Mokwa (2001) corrected their experimental results to compare them with theoretical solutions using an empirical relationship developed from anchor plate tests by Ovensen (1964) and Brinch Hansen (1966). The experimental results shown in Figure 1 were corrected to give equivalent plane strain values using the Das and Seeley (1975) sand test box results for shallow vertical anchors. Removing geometric effects allowed a more direct comparison of the influence of the soil parameters. The correction factor applied is plotted against the wall W/H ratio in Figure 6.

Shamsabadi et al (2007) corrected their theoretical predictions for comparison with the Rollins and Cole (2006) test results (W/H = 4.6) using factors of 1.2 for silty sand and 1.4 for the other soils tested. These factors were based on the surface cracking observed in the tests which showed widening of the failure surface in front of the pilecap. Research based on elastic and nonlinear finite element analysis (FEA) would provide a first step in obtaining more reliable information on three-dimensional passive pressure effects.

4 NUMERICAL ANALYSES

There is very limited published numerical analysis research on the passive resistance of abutment structures. Wood (1985) carried out a plane strain nonlinear FEA of a simple rigid wall structure translated against a backfill modelled as a uniform nonlinear dense cohesive soil using the DIRTMOD software. A number of different constraint conditions were applied to the wall but in the results discussed here the wall was restrained against rotation and vertical displacement. Variable modulus elements were used in which the shear modulus is a function of the mean confining stress, the shear strain amplitude and the stress history. The volumetric behaviour was assumed to be elastic. Details of the model are shown in Figure 7.

Martin and Yan (1995) carried out a similar plane strain nonlinear analysis on a rigid wall with the vertical and bottom soil boundaries at distances of 10 H and 5 H respectively from the wall (H = wall height) to minimise boundary effects. They used the FLAC software and modelled the soil as a linear elastic-perfectly plastic material using the Mohr-Coulomb failure criterion. Analyses were carried out for a range of soil cohesion and friction angles but force displacement results were only presented for a sand with $\phi = 30^\circ$, $\delta = 15^\circ$ and $c = 0$. These properties are representative of a loose to very loose sand.

The Wood and Martin and Yan FEA force-displacement curves are compared in Figure 8 with tests...
results on sand materials of similar properties to those used in the numerical studies. The sand in the Thurston test had a $\phi = 34^\circ$ and this would account for the stiffer response than shown by the Martin and Yan analysis. The Wood numerical analysis curve agreed reasonably well with the Rollins and Cole curve from the test on dense sand.

A hyperbolic curve fit to the Rollins and Cole dense sand results is also shown in Figure 8. The fitted curve was constructed using $P_{du} = 15$ and $E_i = 90$ MPa (dimensionless initial stiffness $= 3,600$ at $u/H = 0.001$). Good agreement between the fitted and test curve confirms the validity of the hyperbolic assumption.

Few of the published test results include information on the pressure distribution or the depth of the centre of pressure on the test wall. Pressure distributions at $u/H > 0.01$ in Thurston’s tests on dense and loose sand were approximately linear and of triangular shape giving a centre of pressure at $0.67H$ below the top of the wall. These results were generally consistent with the pressures computed in the Wood (1985) nonlinear FEA study shown in Figure 9.

The centre of pressure of the wall pressure distributions shown in Figure 9 drops from $0.61H$ below the top of the wall for the elastic soil, to $0.64H$ below the top for the nonlinear soil at $u/H = 0.015$. At $u/H < 0.015$ the depth of the centre of pressure reaches a maximum of $0.67H$.

5 APPLICATION TO BRIDGE SEISMIC ASSESSMENT

The influence of soil passive abutment pressures on the longitudinal earthquake response of a small bridge carrying a main arterial traffic route across the Waiwhetu Stream in Lower Hutt is illustrated by applying generalised results from the testing and analytical research discussed above. This three-span bridge has a prestressed precast concrete superstructure with a main span formed from single hollow core 610 mm square section beams and shorter approach spans at either end formed with solid 300 mm wide by 270 mm deep beam units. The two piers were formed by extending nine vertical 460 mm square section prestressed concrete piles to a 915 mm deep by 760 mm wide capping beam which forms the seating for the superstructure precast beams. The abutments are 915 mm deep by 1,220 mm wide spread footings. Spans of 3.2, 14.5, 3.2 m give a total bridge length of 20.9 m. The overall width of the superstructure and abutments is 15.5 m (Wood, 2009).

Details of the piers and abutments are shown in Figures 10 and 11. Figure 12 shows a schematic layout of the superstructure and abutments identifying the longitudinal earthquake forces acting on
A longitudinal force-displacement relationship for the piers was computed using a conventional nonlinear FEA with the soil surrounding the piles modelled as nonlinear Winkler springs. The force-displacement relationship for passive pressure on the abutments was derived using equation (2) and the test results in Figure 1 for dense cohesionless soils.

The abutment was assumed to be translated against the backfill by the superstructure inertia force without significant rotation. (It is proposed to strengthen the connection of the beams to the abutment so that this assumption is likely to be approximately realised.) A simple elastic FEA analysis of the abutment showed that for translation against the backfill the combined vertical wall and base footing were about 9% stiffer than the simple vertical walls studied by Wood (1985).

Table 3 summarises the parameters used to derive the abutment force-displacement curve and lists the magnitude of the other longitudinal direction forces shown in Figure 12.

**Table 3. Bridge abutment analysis assumptions and forces**

<table>
<thead>
<tr>
<th>Item</th>
<th>Value</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil unit weight</td>
<td>19 kN/m³</td>
<td>Dense sandy gravel backfill.</td>
</tr>
<tr>
<td>Soil friction angle</td>
<td>38°</td>
<td>Typical for compacted gravels.</td>
</tr>
<tr>
<td>Wall and footing friction</td>
<td>29°</td>
<td>Typical for soil-concrete interface.</td>
</tr>
<tr>
<td>Abutment initial stiffness</td>
<td>3,000</td>
<td>Dimensionless value at u/H = 0.001- from Figure 1.</td>
</tr>
<tr>
<td>Abutment initial $E_i$</td>
<td>64 MPa</td>
<td>Calculated from initial stiffness.</td>
</tr>
<tr>
<td>Abutment vertical reaction</td>
<td>450 kN</td>
<td>Span reaction plus abutment weight.</td>
</tr>
<tr>
<td>Abutment frictional resistance</td>
<td>2</td>
<td>Dimensionless. (Divided by: 0.5 $\gamma H^2 W$).</td>
</tr>
<tr>
<td>Abutment ultimate passive resistance</td>
<td>12</td>
<td>Dimensionless. From Figure 1.</td>
</tr>
<tr>
<td>Abutment ultimate passive resistance</td>
<td>1,500 kN</td>
<td>In force units for comparison with other forces.</td>
</tr>
<tr>
<td>Abutment frictional + passive resistance</td>
<td>14</td>
<td>Frictional from abutment at active pressure end.</td>
</tr>
<tr>
<td>Active EQ pressure force</td>
<td>100 kN</td>
<td>From Mononobe-Okabe at PGA = 0.52 g (1000 yr).</td>
</tr>
<tr>
<td>Static at-rest pressure force</td>
<td>50 kN</td>
<td>Small - can be neglected.</td>
</tr>
<tr>
<td>Dynamic weight of superstructure</td>
<td>4,900 kN</td>
<td>Includes abutments and pier pile caps.</td>
</tr>
</tbody>
</table>
The static gravity pressure forces acting on low abutment walls are small and can often be neglected. Likewise, the earthquake-induced pressure at the active pressure end of the bridge is small but it is a function of the ground acceleration and should be evaluated to assess its magnitude. Even with the short end spans of the Waiwhetu Stream bridge, the frictional resistance from the footings of the abutments is substantial. The pile cap tests of Gadre and Dobry (1998) indicated that this would add to the passive resistance but it is not clear whether using the total sum of these forces is generally valid. For the present assessment, the frictional resistance at the base of the abutments was neglected at the passive pressure end but considered to be fully effective at the active pressure abutment.

Figure 13 compares force-displacement curves for the resistance from only the two piers and the total resistance from the piers and abutments. The forces have been put in dimensionless form by dividing by the dynamic weight of the superstructure (see Table 3). The resistance forces in this form are equal to the acceleration response of the bridge dynamic mass in g units of acceleration.

The change in slope in the pier resistance curve at a displacement of about 95 mm is caused by plastic hinges forming in the pier piles at a depth of about 4 m below the tops of the piles.

Also shown in Figure 13 is the 1,000 year return period inelastic demand spectrum computed from the NZS 1170.5 (2004) site category Class D subsoil elastic spectrum. The elastic spectrum was reduced to the inelastic spectrum by assuming a displacement ductility factor of 3, appropriate when only the piers provide the resistance, and using the NZSEE (2006) reduction method which is based on an equivalent hysteretic damping calculation.

The intersection points of the force-displacement curves with the demand spectrum gives an indication of the peak longitudinal displacement response of the bridge in a 1000 year return period earthquake. The inelastic demand spectrum is only valid for the pier resistance acting alone. For the combined pier and abutment resistance case there is substantial damping from nonlinear soil passive resistance and sliding friction in the large displacement response during the design level earthquake but less from yielding in the piles. Overall the effective damping may be a little greater than for the pier system acting alone. Further research is required to quantify the amount of damping available from soil-structure interaction at the abutments.

Figure 13 shows the very significant influence of the abutment soil-structure interaction on the longitudinal earthquake response of the Waiwhetu Stream bridge. Abutment resistance reduced longitudinal displacements in the design level earthquake by a factor of about 2 (assuming similar damping for both cases considered) and eliminated the need to consider strengthening the piles which have limited ductility. It was more cost effective to strengthen the connections to the abutments to take full account of the resistance that they provide rather than to jacket the piles.

6 CONCLUSIONS

(a) Information available from experimental and analytical research projects can be used to reliably estimate the passive soil resistance available at bridge abutments and this resistance should be included in assessment and design analyses.

(b) The abutment resistance can be substantial on some bridges reducing ductility demands on other components in the substructure.

(c) Further research into the dynamic response of typical abutment structures is required to provide simple design rules for estimating the damping available from soil-structure interaction.
7 REFERENCES


