Kaiwharawhara Pedestrian Over Bridge Structural Assessment

July 2013
Prepared by Spiire
## Quality Assurance Statement

<table>
<thead>
<tr>
<th>Task</th>
<th>Responsibility</th>
<th>Signature</th>
</tr>
</thead>
<tbody>
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<td>Rob Bryant</td>
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</tr>
</tbody>
</table>

## Issue Date Table

<table>
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<tr>
<th>Issue Date</th>
<th>Revision No.</th>
<th>Author</th>
<th>Checked</th>
<th>Approved</th>
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</thead>
</table>

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Appendix 4.2  Bridge Replacement – Budget Cost Estimate
1. Executive Summary

Spiire New Zealand Ltd has been engaged by Greater Wellington Rail Ltd to complete a structural assessment of the pedestrian over bridge at Kaiwharawhara station. The bridge is located at Westminster Street, Kaiwharawhara, Wellington.

The bridge comprises steel I-beams with timber decking and balustrades supported by rail-iron piers. The bridge was constructed circa the middle of the 20th century and the stairs were replaced in 2005.

Spiire engineers have inspected the bridge and observed extensive corrosion to the steel I-beams and supporting piers. Following the discovery of extensive corrosion during the first site visit on June 13th 2013 the station was closed to the public due to concerns about the structural integrity of the bridge.

Spiire have completed a structural analysis of the bridge, based on compliance with current design practices and standards. It was found that the bridge rail-iron piers are overstressed. The analysis has not made allowance for the reduction in strength due to corrosion. In some areas there has been a significant loss of section.

The steel I-beams and rail irons forming the piers require replacement due to the extent of corrosion. It is not considered practical to repair these members.

It is therefore recommended that prior to re-opening the station the bridge spans and supporting rail-iron piers be replaced. The existing stairs, having been recently replaced are in good condition and can be reused.

We have prepared Budget Cost Estimates for the following:

- Bridge Upgrading
- Bridge Replacement

2. Existing Bridge

2.1 Description of Existing Structure

The bridge comprises two spans formed with steel I-beams with timber decking and balustrades supported by three rail-iron piers. The piers are supported on concrete pad foundations.

The bridge appears to have been constructed approximately in the middle of the 20th century. We have viewed drawing 45847 in Appendix 1 (undated) which we believe is a drawing of the subject Kaiwharawhara bridge. It appears in drawing 45847 that the rail-iron piers are older than the I-beam spans. The stairs were replaced in 2005. The timber deck and handrails have been replaced recently.

2.2 Bridge Inspection

The bridge was inspected on June 13th 2013 by Spiire engineers, Rob Bryant and Tom Arthur.

The bridge was inspected more closely on July 5th 2013 in conjunction with staff from Service Resources Ltd who undertook the physical works and reinstatement associated with the inspection.
The following investigative work was undertaken on site:

- Sections of timber decking were removed above piers to better assess the extent of corrosion of the spans.
- Areas of asphalt and concrete were chipped away to expose the bottoms of some of the pier rail-irons where they extend into the concrete pad foundations.
- Exploratory holes were drilled in timber corbels and also into timber packers bolted to the tops of the steel I-beams.
- A hole was excavated down beside one of the pier foundations adjacent to the west side boundary fence to confirm the depth of the foundation pad.

2.3 Condition of Bridge

Extensive corrosion was noted on the steel I-beam members. This was particularly evident on the web of the beam over the pier on the harbour side of the bridge and to a lesser extent over the central pier. Photographs one and two show extensive corrosion below the connection between the stairs and bridge.

Photo one: Close up of corrosion on beam web over the pier on the harbour side of the bridge.

Photo two: Location of corrosion to steel I-beam web.
The 2005 replacement stairs are in good condition.

Large timber corbels sit on the rail-iron piers. These were observed to be split along the centre where bolts attach the piers to the I-beams. The splits are typically 5-10mm wide and will weaken the connection between the piers and steel I-beams. 10mm diameter exploratory holes were drilled into the corbel members. The condition of the timber was found to be good with no evidence of degradation. Similar observations were made on holes drilled in the timber packers bolted to the top flange of the main I-beams.

Photo Three: Timber corbel with split along bolt line.
Extensive corrosion was observed on the rail-irons. Significant loss of section has been observed at the base of the legs and also at the top of the piers.

Photo Four: Extensive corrosion of rail section (between members). Surface corrosion and loose rust evident.

Photographs 5, 6 and 7 show extensive corrosion of the rail-irons below ground level on the east side and central piers.

Photo Five: Extensive corrosion of rail and loss of section at base.
Photo Six: Extensive corrosion of rail section at base

Photo Seven: Extensive corrosion of rail section at base
The bridge balustrade looks to have been repaired around the same time as the stairs. Some of the connections between the bridge superstructure and balustrade posts have deteriorated. On the left handside of the photograph 8 a replaced balustrade post can be observed. On the right hand side an original post is seen, the timber blocking fixed to the web of the I-beam has split and half has come away. Note the corrosion behind where the timber blocking used to be.

Photo Eight: Comparison of old and new balustrade supports

In photograph nine there are areas of significant corrosion of the top surface of the top flange of the beams and also extensive surface corrosion of the beams.

Photo Nine: Corrosion to the tops of the top flange of the I-beams under the timber packers supporting the deck.
2.4 Bridge Analysis

The structure has been assessed against the requirements outlined in Kiwi Rail Structures Code Supplement: Railway Bridge Design Brief, issue 6 (2008). This code makes reference to the following documents.

- AS/NZS1170:2001
- NZS3404:1997

Due to the irregular nature of the corrosion, the bridge has been analysed ignoring the reduction in section due to corrosion. Despite this, it was found that the strength of the rail-iron pier legs falls well short of current code requirements. The amount of loss of section due to corrosion of the rail-iron legs is in the order of 10% to 20% of the gross rail area in places, particularly the east side pier.
We have taken the yield stress of all structural steel, including the rails, to be 225 MPa. Because the analysed stress in some of the bridge members is significantly higher than the yield stress, and because of the extensive rusting and significant loss of section in some parts of the structure, we do not recommend that material testing be undertaken to confirm the yield stress of the material.

Table 1: Member Utilisation Summary

<table>
<thead>
<tr>
<th>Bridge Member Description</th>
<th>% of Current Code Strength &amp; mode of Failure</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Harbour side Pier, 53 lb/yard Rail-iron leg</td>
<td>58%, Compression.</td>
<td>No restraint is provided to the member major axis over 4.6m length of member. Critical load case is seismic load applied in the transverse direction. Analysis ignores loss of section from corrosion.</td>
</tr>
<tr>
<td>Central Pier, 53 lb/yard Rail-iron leg</td>
<td>84%, Compression.</td>
<td>Legs in central pier orientated such that no restraint is provided to the minor axis over 4.6m length of member. Critical load case is seismic load applied in the transverse direction. Analysis ignores loss of section from corrosion.</td>
</tr>
<tr>
<td>Hutt Road Pier, 53 lb/yard Rail-iron leg</td>
<td>22%, Compression.</td>
<td>This is the only pier to have lateral bracing in the longitudinal direction. Consequently, due to its stiffness relative to the other piers this pier attracts a disproportionate amount of load. Critical load case is seismic load in the longitudinal direction. This ignores loss of section from corrosion.</td>
</tr>
<tr>
<td>Main Support I-Beam</td>
<td>120%, Bending</td>
<td>Member satisfactory in bending. Lateral restraint assumed from deck fixed to compression flange at 2.4m (8ft) centres. Critical load case is uls, Dead + Live load. Beam Deflection noted as 17mm, G + 0.3Q</td>
</tr>
</tbody>
</table>

3. Wellington City Council Requirements

Because the asset is not owned by a Network Utility Operator a building consent is required for upgrading work to the bridge which could possibly trigger the need to provide an accessible bridge. If so, this would require the provision of ramps.

It is possible that Council could grant dispensation for a non-complying structure incorporating stairs.

If it is decided to upgrade the existing structure using stairs only in lieu of a complying structure with ramps then a submission would need to be put to Council setting out what is proposed to be constructed and putting forward a case for providing a structure that is compliant "as nearly as is reasonably practicable" to present day requirements. The existing stairs comply with present day requirements.

Before a decision is made on the future of the bridge we are able to present a submission to Council detailing the options for upgrading or renewal of the bridge with a view to obtaining Council’s approval in principle.

4. Health and Safety

The bridge was inspected on 13 June 2013. Because of the extensive corrosion discovered in the span at the seaward end of the bridge we recommended that the bridge be closed pending the completion of our detailed investigations on the grounds of safety.

Following our detailed inspection and structural analysis we see no reason to change our recommendation for the closing of the bridge in its present condition.
5. **Recommendations**

5.1 **Repair of Existing Structure**

The main bridge I-beam spans are severely corroded and require replacement. The rail iron piers are also in very poor condition with significant loss of section evident and are in need of replacement. We consider that the only parts of the bridge able to be incorporated into an upgraded structure are the three relatively new sets of stairs. These are constructed of galvanised steel channel stringers with galvanised folded plate treads and risers.

Because of the extremely poor condition of the existing structure, the extent of corrosion and loss of section of some of the bridge components we do not deem it practical to repair the existing bridge structure.

We recommend replacement of the existing bridge spans and piers incorporating:

- Reinforced concrete or galvanised structural steel piers with new reinforced concrete foundations
- Concrete deck with either steel or concrete supporting beams
- Galvanised steel balustrade.

We have prepared a budget cost estimate to replace all but the stairs:

**Our budget cost estimate for the above is** [value]

(Refer to Appendix 4.1 for a breakdown of costs).

5.2 **Replacement Structure**

We have considered a replacement structure incorporating a fully complying ramp while re-using the existing stairs includes:

- Reinforced concrete or galvanised structural steel piers with new reinforced concrete foundations for the span and ramp supports
- Precast reinforced concrete deck with either steel or concrete supporting beams for both the spans across the tracks and for the ramp spans
- Galvanised steel balustrade
- Reinforced concrete impact wall as protection to the bridge supports along the west boundary

In addition we note the following:

- Approximately eight lighting poles and two traction support poles will require relocating, working around or incorporating into a design for ramps on the two platforms
- A ramp along the west boundary will reduce the width for vehicles access along the maintenance track beside the railway track
- Ramps landing on the platforms require to be a minimum of 1.5 metres clear width for a wheelchair and a pram to pass. With a structure width of say 1.8 metres, and a platform width of 4.3 metres overall, this leaves only 1.25 metres either side of the ramp to the edges of the platform. There will be over 25 metres of narrow platform and it is a sub-standard width for passengers to walk on the platform and pass others.

We have prepared a budget cost estimate for a replacement bridge structure. This incorporates ramps complying with requirements for disabled while also re-using the existing stairs.

**Our budget cost estimate for the above is** [value]

(Refer to Appendix 4.2 for a breakdown of costs).

We consider this not to be a practical option for the following reasons:

- The cost is significant
- The station platforms are too narrow for the required width of ramps
Appendix 1
Existing Bridge Drawings

- Bridge before the stairs were replaced, numbered 45847
- Bridge with replacement stairs, in 5 sheets, numbered 120079
KAIWARRA
PROPOSED FOOT OVERBRIDGE.
SCALE 4 & 2' TO 1'
Appendix 2
Design Features Report and Structural Analysis Summary Calculations
GREATER WELLINGTON RAIL LTD

Structural Assessment of
Kaiwharawhara Pedestrian Over Bridge

At
Westminster Street, Wellington

For
Greater Wellington Rail Ltd

Design Features Report
July 2013
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Appendix A Structural Analysis Summary Calculations
1. **GENERAL**

1.1 **Objective**

The Design Features Report (DFR) is a detailed document defining the design criteria used in analysing the structure and recording key outcomes. It outlines design loading, structural modelling assumptions, material properties and design standards.

1.2 **Scope**

Spiire have been engaged by Greater Wellington Rail Ltd to complete a structural assessment of the pedestrian over bridge at Kaiwharawhara station in Wellington.

During the first inspection on 13th June 2013, Spiire engineers observed corrosion to the main horizontal UB sections and it was recommended that the pedestrian over bridge be closed pending further analysis and inspection of the bridge.

Spiire are to assess the extent of corrosion to the bridge, analyse the structure to determine adherence to current design standards and to provide an estimate on the cost of repairs / structural upgrades necessary.

1.3 **Means of Compliance**

The structure has been assessed against the requirements outlined in Kiwi Rail Structures Code Supplement: Railway bridge design brief, issue 6 (2008). This document makes reference to the following documents.

- AS/NZS1170:2001
- NZS3404:1997

1.4 **Alternative Solutions**

Remedial works and replacement options are summarised.

2. **THE STRUCTURE**

2.1 **General**

The over bridge at Kaiwharawhara carries pedestrian traffic from the car park on Westminster Street to the two station platforms. The structure is comprised of two spans of around 11m over railway lines. The bridge was constructed from 14" x 5.5" Universal beams simply supported on piers formed using railway rails.

The location of the structure is Westminster Street, Kaiwharawhara, Wellington.

The original three flights of stairs were replaced in 2005. Significant corrosion to the webs of the universal beams has occurred where the original stairs were connected.

It is not known when the structure was constructed. Some of the rails used for legs from the bridge plinths date from 1870 though it is thought the bridge was constructed later than this.

2.2 **Gravity structure**

The bridge is supported by 3 piers formed using bent railway lines. The supporting rail-irons date from the 1870's, due to the extensive corrosion observed on these members it is assumed that they are mild steel. 2 No pairs of steel UB sections span between the piers with a timber deck and balustrade above.
2.3 Lateral Load Resisting structure

The structure has raking legs providing stability parallel to the direction of the railway line below. The lateral stability perpendicular to the railway line is providing by diagonal bracing members provided on the foundations at the Westminster Street end of the bridge.

3. SOIL CONDITIONS

3.1 Description of Site Soil Conditions

The concrete pad foundations have not been checked as part of this analysis. We confirm that there are no signs of significant settlement of the bridge supporting piers.

4. DESIGN LOADS

4.1 General

For the purposes of consideration of loading, this structure Importance Level 2 in accordance with AS/NZS 1170.0:2002.

4.2 Imposed Loads

4.2.1 Vertical loads

The table below summarizes all vertical loads including both superimposed dead and live loads. It is thought that the bridge would originally have been designed for an imposed load of 100 lb / sq ft. This approximates to 4.79 kPa. This is slightly below the imposed load used for this analysis.

Table 1: Imposed Gravity Loads

<table>
<thead>
<tr>
<th>Level / Area</th>
<th>Use</th>
<th>Live Load</th>
<th>Dead Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck</td>
<td>Pedestrian Loads</td>
<td>5.0 kPa</td>
<td>0.6 kPa</td>
</tr>
</tbody>
</table>

4.2.2 Barriers and Handrails

The following loads apply for all barriers and handrails. Note, the balustrade itself was not within the scope of this project. Instead the bridge has been checked for the worst case horizontal loading due to wind acting on the balustrade.

Table 2 : Barrier and Handrail loads

<table>
<thead>
<tr>
<th>Level / Area</th>
<th>Top Edge</th>
<th>Infill</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horizontal kN/m</td>
<td>Vertical kN/m</td>
</tr>
<tr>
<td>Ballustrade</td>
<td>0.75</td>
<td>0.72</td>
</tr>
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</table>

4.3 Wind Loads

As per Kiwi Rail Structures Code Supplement, cl 5.7 a wind load of 2.2 kPa has been applied to the projected windward area of the bridge. The windward side of the bridge is considered to be ‘open’, a factor of 0.50 has been applied to the leeward area of the balustrade (50% shielding).

No shielding has been applied to the plinth members.
4.4 Seismic Loads

4.4.1 Site Parameters
Site subsoil class: D
Proximity to fault, D = 0 km. Site is directly adjacent to the Wellington fault line.

4.4.2 Analysis Methodology
The seismic analysis has been completed in accordance with AS/NZS 1170.5:2002, using the equivalent static analysis method.

Design Spectra are in accordance with AS/NZS 1170.5:2002 for site subsoil class D.

For the purposes of the analysis, the project x and z directions are considered to be the project longitudinal (perpendicular to train line) and transverse directions respectively.

4.4.3 Seismic Load Coefficient
The seismic load coefficient has been determined in accordance with AS/NZS 1170.5:2002. Section 3, based on the following assumptions.
Zone factor, Z = 0.40
Period, T = 0.4s for both directions

\( C(T) = 3.0 \)
\( N(T,D) = 1.0 \) (for both ULS & SLS)

The structure has been assumed to be nominally ductile. \( \mu = 1.25 \)

Ultimate Limit State

\( R_u = 1.00 \)
\( S_p = 1.00 \)

Elastic site spectra for horizontal load, \( C(T) = 1.20 \)

Horizontal design coefficient, \( C_d(T) = 1.05 \)

Serviceability Limit State

\( R_s = 0.25 \)
\( S_p = 0.70 \)

Elastic site spectra for horizontal load, \( C(T) = 0.30 \)

Horizontal design coefficient, \( C_d(T) = 0.184 \)

4.4.4 Seismic Weight Assumptions
The seismic weight has been distributed as per guidance in the bridge manual, cl 5.3.2. The full mass of the bridge superstructure plus half the mass of the piers has been considered to act at level of the bridge deck.

Due to stairs having limited bracing for lateral load resistance, it has been assumed that half the mass of the stairs will contribute to the seismic weight of the bridge.

The seismic weight of the structure has been calculated including the imposed loads multiplied by 0.30. This is based on AS/NZS 1170.5:2002, cl 4.2(1).

5. SERVICEABILITY CRITERIA

5.1 Seismic Deflections
Not checked
5.2 Wind Deflections
Not checked

5.3 Gravity Deflections
Bridge beam deflection calculation under G + 0.3Q gave a mid-span deflection of 17mm.
This is within acceptable limits for a pedestrian bridge.

6. SOFTWARE
The following computer applications were used for the design:

Table 6: Software used in design

<table>
<thead>
<tr>
<th>Analysis type</th>
<th>Software used</th>
<th>Archive files</th>
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<tbody>
<tr>
<td>3D frame analysis</td>
<td>MICROSTRAN, V9.0</td>
<td></td>
</tr>
<tr>
<td>General spreadsheet design</td>
<td>EXCEL 2010</td>
<td></td>
</tr>
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7. DRAWING AND SPECIFICATION NOTES
The purpose of this section is to ensure that the design requirements are included in the drawings or the specification.

7.1 Floors

7.1.1 Design Loads
Refer to Section 2 DESIGN LOADS and section 5.3 Gravity Deflections.

7.2 Foundations
The foundations are standard pad footings.

7.3 Material Properties (Typical)

7.3.1 Concrete Strengths
Foundations: Unknown MPa

7.3.2 Reinforcing Steel
Foundation Reinforcing bars: Unknown

7.3.3 Structural Steel
Rolled Steel Sections and rail-irons: \( f_y = 225 \text{ MPa} \) & \( f_u = 432 \text{ MPa} \) assumed
Appendix A
Structural Analysis Summary Calculations
following a view to site & measurement of the
rail sections forming the legs / piers of the
footbridge, it is thought that the members are
similar to 'so NZR'.

calculative values required by microstrain software
have hence not been given in fig 13.1.

\[ f_{eq} = \sqrt{\frac{483 \times 10^6}{3361}} = 37.91 \]

\[ \gamma = \sqrt{\frac{99.5 \times 10^6}{3361}} = 17.21 \]

section weight per m = \[3361 \times 10^-10 \times 7850 \]
= 26.36 kg/m

circular hollow section members are located
between the legs of the piers, they are 48.3 m.
It has been assumed that they are 7.3 m thick.
Due to extensive cracking observed at Kasiri there
the members are inclined \(\gamma = 23.5\) MPa
\(f_{eq} = 482\) MPa.
**Fig. 1.3.1 Properties and Dimensions of N.Z.R. Rail Sections.**

9/73
<table>
<thead>
<tr>
<th>Ref. No.</th>
<th>Size (D x B)</th>
<th>Approximate Mass/ft</th>
<th>Metric Equivalent (D x B)</th>
<th>Mass/m</th>
<th>Thickness</th>
<th>Area</th>
<th>Nom. of Inert.</th>
<th>Rad. of Gyr.</th>
<th>Sec. Mod.</th>
</tr>
</thead>
<tbody>
<tr>
<td>NBSB 1</td>
<td>3 x 1.50</td>
<td>4</td>
<td>76 x 38</td>
<td>6</td>
<td>0.16</td>
<td>1.18</td>
<td>1.66</td>
<td>0.13</td>
<td>1.19</td>
</tr>
<tr>
<td>NBSB 2</td>
<td>4 x 1.75</td>
<td>5</td>
<td>102 x 44</td>
<td>7</td>
<td>0.17</td>
<td>1.14</td>
<td>3.66</td>
<td>0.19</td>
<td>1.58</td>
</tr>
<tr>
<td>NBSH 1</td>
<td>4 x 3</td>
<td>10</td>
<td>102 x 76</td>
<td>15</td>
<td>0.24</td>
<td>2.94</td>
<td>7.79</td>
<td>1.33</td>
<td>1.63</td>
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<tr>
<td>NBSB 3</td>
<td>4.5 x 2</td>
<td>7</td>
<td>114 x 51</td>
<td>10</td>
<td>0.19</td>
<td>2.06</td>
<td>6.65</td>
<td>0.38</td>
<td>1.80</td>
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<tr>
<td>NBSB 4</td>
<td>5 x 2.50</td>
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<td>127 x 64</td>
<td>13</td>
<td>0.20</td>
<td>2.65</td>
<td>10.91</td>
<td>0.79</td>
<td>2.03</td>
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<tr>
<td>NBSHB 2</td>
<td>5 x 4.50</td>
<td>20</td>
<td>127 x 114</td>
<td>30</td>
<td>0.29</td>
<td>5.88</td>
<td>25.0</td>
<td>6.59</td>
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<td>NBSB 5</td>
<td>6 x 3</td>
<td>12</td>
<td>152 x 76</td>
<td>18</td>
<td>0.23</td>
<td>3.53</td>
<td>21.0</td>
<td>1.46</td>
<td>2.44</td>
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<tr>
<td>NBSB 6</td>
<td>6 x 5</td>
<td>25</td>
<td>152 x 127</td>
<td>37</td>
<td>0.33</td>
<td>7.35</td>
<td>45.2</td>
<td>9.88</td>
<td>2.48</td>
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<tr>
<td>NBSB 7</td>
<td>6 x 7.5</td>
<td>15</td>
<td>178 x 89</td>
<td>22</td>
<td>0.25</td>
<td>4.42</td>
<td>35.9</td>
<td>2.41</td>
<td>2.85</td>
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<tr>
<td>NBSB 8</td>
<td>7 x 4</td>
<td>18</td>
<td>203 x 102</td>
<td>37</td>
<td>0.28</td>
<td>5.30</td>
<td>55.9</td>
<td>3.51</td>
<td>3.24</td>
</tr>
<tr>
<td>NBSB 9</td>
<td>8 x 6</td>
<td>25</td>
<td>203 x 152</td>
<td>52</td>
<td>0.35</td>
<td>10.30</td>
<td>115.1</td>
<td>19.54</td>
<td>3.34</td>
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<tr>
<td>NBSB 10</td>
<td>8 x 9</td>
<td>9</td>
<td>229 x 102</td>
<td>31</td>
<td>0.30</td>
<td>6.18</td>
<td>81.1</td>
<td>4.15</td>
<td>3.62</td>
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<tr>
<td>NBSB 11</td>
<td>8 x 11</td>
<td>11</td>
<td>229 x 187</td>
<td>74</td>
<td>0.40</td>
<td>14.71</td>
<td>208.1</td>
<td>40.17</td>
<td>3.76</td>
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<tr>
<td>NBSB 12</td>
<td>9 x 4.50</td>
<td>25</td>
<td>254 x 114</td>
<td>37</td>
<td>0.30</td>
<td>7.35</td>
<td>122.3</td>
<td>6.49</td>
<td>4.08</td>
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<tr>
<td>NBSB 13</td>
<td>10 x 5</td>
<td>20</td>
<td>254 x 152</td>
<td>60</td>
<td>0.36</td>
<td>11.77</td>
<td>204.8</td>
<td>21.76</td>
<td>4.17</td>
</tr>
<tr>
<td>NBSB 14</td>
<td>10 x 8</td>
<td>40</td>
<td>254 x 203</td>
<td>82</td>
<td>0.40</td>
<td>16.18</td>
<td>288.7</td>
<td>54.74</td>
<td>4.22</td>
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<tr>
<td>NBSB 15</td>
<td>10 x 12</td>
<td>30</td>
<td>305 x 127</td>
<td>45</td>
<td>0.33</td>
<td>8.83</td>
<td>206.9</td>
<td>8.77</td>
<td>4.84</td>
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<tr>
<td>NBSB 16</td>
<td>12 x 8</td>
<td>65</td>
<td>305 x 203</td>
<td>97</td>
<td>0.43</td>
<td>19.12</td>
<td>487.8</td>
<td>65.28</td>
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<tr>
<td>NBSB 17</td>
<td>13 x 8.5</td>
<td>35</td>
<td>330 x 127</td>
<td>52</td>
<td>0.35</td>
<td>10.30</td>
<td>283.5</td>
<td>10.80</td>
<td>5.25</td>
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<tr>
<td>NBSB 18</td>
<td>14 x 5.50</td>
<td>40</td>
<td>356 x 140</td>
<td>60</td>
<td>0.37</td>
<td>11.77</td>
<td>377.1</td>
<td>14.80</td>
<td>6.66</td>
</tr>
</tbody>
</table>

**Notes:**
- Moments of Inertia: X-X, Y-Y
- Radii of Gyration: X, Y
- Section Modulus: X, Y

**Table No. 3.8: Properties of Beams to British Standard 4, 1921**

**IMPERIAL UNITS**

See separate page for notes.
The bridge will be analysed for loads generated in accordance with the KiwiRail Structures code supplement, CSW16201, issue 6.

**Load Case One:**

Normal everyday use. It is assumed that during normal loco running with no concern with high wind loads.

Drain load of bridge decking.

2 No. pugs of timber decking = 2 x 0.23 = 0.46 kNm²

Round up to 0.60 kNm² for blocking, rails etc.

**Live load:**

Full live load due to crowd (decking) = 0.5 kNm²

It is thought the bridge would have been designed for:

100 lb/sqft = 100 x 0.0479 = 4.79 kNm²

Adopt 5 kNm² based on footpath (load for rails made of concrete).

**Wind load:**

A wind load of 2.2 kNm² will be applied to the projected windward area of the bridge.
calculate projected windward area, based on
designs in drawing 45847 & photographs

It is assumed that the full windward area of
the windward face is exposed at 50% of the
leeward face is exposed.

Projected area of bridge (per m run)

\[3 \times 0.10 = 0.30 \text{ m}^2\]

\[0.10 \times 1.42 = 0.142 \text{ m}^2\]

\[14 \times 0.0254 = 0.356 \text{ m}^2\]

Projecting & Declaring on Beam

Wire mesh assuming 0.1 m^2/m^2 density ratio

\[0.10 \times 1.20 = 0.12 \text{ m}^2\]

TOTAL PROJECTED AREA = 0.996 m^2/m

Wind load acting on GB members:

Windward member \[= 0.996 \times 2.20 = 2.19 \text{ kN/m}\]

Leeward member \[= 0.996 \times 1.20 \times 0.50 = 1.10 \text{ kN/m}\]

Dead load of handrails

Assume 0.6 kN/m^2 = 1.20 m 100 mm handrail

\[0.6 \times 1.20 = 0.72 \text{ kN/m}\]
It is assumed that the stairs are entirely self-supporting. Only load from the section of stairs flanked by the 14 x 55 ub's will be considered.

Stairs span between pcc stringers, based on width of stairs being 1600mm.

Dead load along stringer (assume weight as bridge)
\[ = 0.72 \times 1.60 \times 0.60 \]
\[ = 1.20 \text{ kN/m} \]

Live load along stringer
\[ = 1.60 \times 5 \]
\[ = 8.0 \text{kN/m} \]

Wind load acting on sides of stair well, based on 2.24 m/s²
acting on projected outward area.

Handrail 1.483 m/kg
\[ = 0.483 \times 2 = 0.966 \text{ kN/m} \]

200 deep pcc stringer
\[ = 0.20 \text{ kN/m} \]

900mm tail sheeting 0.80 kN/m²
\[ = 0.80 \times 0.90 = 0.72 \text{ kN/m} \]

Total area
\[ = 1.016 \text{ m}^2 / \text{m} \]

calculate additional area due to inclination of stairs.

Stair flight rises 1908mm over 3525mm nominal distance.

Length of stairs
\[ = \left( 3525^2 - 1908^2 \right)^{1/2} = 4.061 \text{ m} \]
Staircase projected area continued:

Projected windward area:

\[ A = 1.016 \times 1.133 = 1.151 \text{ m}^2 \]

Calculate point loads at stringer connections to 14 x 50.45 beams, based on half of 3585 load going to the bridge:

Dead load = \( 1.20 \times 3.585 \times \frac{1}{2} = 2.151 \text{ kN} \)

Live load = \( 4 \times 3.585 \times \frac{1}{2} = 7.17 \text{ kN} \)

Wind load (windward face):

\[ W = 1.151 \times 2.2 \times 3.585 \times \frac{1}{2} = 4.839 \text{ kN} \]

Wind load (leeward face):

\[ W = 1.151 \times 2.2 \times 3.585 \times 0.50 = 2.27 \text{ kN} \]

Calculate load on bridge due to wind acting on stairs, wind acting to bridge span.

Stairs are 'solid', conservatively assume that bridge resists half the total horizontal load on the stairs. Load shared equally between 2 stringers:

\[ \text{Total load} = \left( \frac{0.022 \times 1.60}{2} \right) \times 2 = 4.419 \text{ kN} \]
CALCULATE WEIGHT OF STAIRS. WEIGHS TO BE ESTIMATED PER INLONGMENT OF STAIRCASE.

<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>WEIGHT CALC</th>
<th>WEIGHT (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3mm TREADS</td>
<td>1.676 x 0.003 x 1.5 x 780 x 9.81 x 10^{-3}</td>
<td>0.580</td>
</tr>
<tr>
<td>250 PFC STRINGERS</td>
<td>2 x 22.9 x 9.81 x 10^{-3}</td>
<td>0.489</td>
</tr>
<tr>
<td>BALLUSTAGE</td>
<td>2 x [0.004 x 27 + 2.6 x 9.81 x 10^{-3} x 2]</td>
<td>0.318</td>
</tr>
<tr>
<td>TOTAL WEIGHT</td>
<td>1.397</td>
<td></td>
</tr>
</tbody>
</table>

Vertical length of stair = 4.96 m.
Horizontal length of stair = 9.70 m.

Weight of Stairs = \((4.96^2 + 9.70^2)^{1/2} \times 1.397\) = 10.89 KN.

Allow additional 10% for fitting and assume bridge will resist unit weight of stairs in seismic load condition.
Earthquake design loads will be calculated in accordance with the NZTA bridge manual 3rd edition. This is based on guidance in NZS 1170.5

Hazard factor, Z

$Z = 0.40$ site in Wellington

Spectral shape factor, $C_u(t) = 3.0$

based on soft soil assumed as bridge built on reclaimed land. Return period = 0.40 seconds

Return period factor, $R$

For ultimate limit state, probability of earthquake being exceeded in 1 year = $0.500, R_u = 1.0$

For serviceability limit state, probability of earthquake being exceeded in 1 year = $0.25, R_s = 0.25$

Near fault factor $N(t, D)$

For ultimate limit state annual probability of exceedance = $0.50$

Bridge located in close proximity to fault line $D < 2km$

$N(t, D) = N_{max}(t)$

$N_{max}(t) = 1.0$ based on period of 0.45.

Elastic site spectra for horizontal loading, $C(t)$

$c(t) = C_u(t) R N(t, D)$

$= 3.0 \times 1.0 \times 1.0 \times 1.0$

$= 1.20$ (m/s)
Horizontal design action coefficients, \( C_a(T) \)

\[
C_a(T) = \frac{C(T)S_p}{K_\mu} \geq (2.20 - 0.02)K_u
\]

Structural performance factor, \( S_p = 1.0 \) (ULS)

\[
K_\mu = \frac{(\mu - 1)T + 1}{0.7}
\]

Soil class D, \( T = 0.40 \)

Structural gravity load \( C_a(T) = 1.0 \times 1.0 = C(T) = 1.05 \) (ULS)

Structural performance factor, \( S_L, S_p = 0.70 \)

\[
C(T) = 3.0 \times 0.40 \times 0.25 \times 1.0 \quad (K_u = 0.25)
\]

\[
= 0.30 \quad (S_L)
\]

\[
C_a(T) = \frac{0.30 \times 0.70}{1.143}
\]

\[
= 0.184 \quad (S_L)
\]

Seismic weight, \( w_i \)

\[
w_i = Q_i + \psi_i Q_i
\]

\[
\psi_i = 0.30
\]

Conservatively include live load for calculating seismic weight.

Design load calculated as per bridge manual (15.3.2)
<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>WEIGHT (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DECK</td>
<td>0.60 x 25.42 x 1.676</td>
</tr>
<tr>
<td>HANDRAIL</td>
<td>(25.42 + 1.676) x 2 x 0.72</td>
</tr>
<tr>
<td>LEGS</td>
<td>4.60 x 24 x 26.38 x 2 x 9.81 x 10^-3 * 1/2</td>
</tr>
<tr>
<td>STANDS</td>
<td>0.89 x 3.12</td>
</tr>
<tr>
<td>BEAMS</td>
<td>25.42 x 2 x 60 x 9.81 x 10^-3</td>
</tr>
<tr>
<td>TOTAL WEIGHT</td>
<td>167.917 kN</td>
</tr>
</tbody>
</table>

4 \( W = 0.30 \) applied to live load  
\[ W = 5 \times 25.42 \times 1.676 \]  
\[ = 213.02 \text{ kN} \]  

\[ W' = 167.917 + 0.30 \times 213.02 \]  
\[ = 231.80 \text{ kN} \]  

Horizontal seismic shear \( V \)  
\[ V = c_4.2(1)W' \]  
\[ = 105 \times 231.80 \]  
\[ = 243.4 \text{ kN} \] (L5).  

Equivalent static horizontal force for a single storey structure such as this bridge, the equivalent static force in line with the deck load is equal to the seismic shear.
Assume seismic load will be carried equally over blocking structure and weight relatively evenly distributed over length.

Apply load as unit along length of members.

Seismic load \( L = 2 \times 3.14 \times \frac{23.18}{2} = 4.78 \text{ kN/m} \)

Load will be applied in both horizontal directions.

For eccentricity (SLS):\( \text{Seismic Load} \times \text{SLS} \times \text{SLS} = 0.184 \).

Vertical seismic (SLS) equivalent static horizontal load \( L = 0.184 \times 23.18 = 42.67 \text{ kN} \) (SLS).

Load is shared equally along 16 members as members are

Seismic load \( L = 42.67 \times \frac{2}{2} = 0.838 \text{ kN/m} \)
Load Cases:
- 1 P Dead Load (g_y = -9.81)

Applied Dead Load

MicroStran V9.01.130412 (52150)\Jobs\706880 GW Rail Ltd - Structural Assessment Ped Overbridge Kaiwharawhara\Working\Structural\MicroStran Bridge Model\Kaiwharawhara Footbridge Model.msw
Load Cases:

2 P Live Load
Load Cases:

- 3 P Wind Load, z (wind on face)

Theta: 225 phi: 14
Load Cases:

- 4 P Wind Load, x (wind on end)
Load Cases:

- 6 P ULS Seismic Load, x (load on end)
Load Cases:
- 16 C 1.0 Dead + 0.3 Live (SLS)
Appendix 3.1
Concept Plan of Upgraded Bridge
Existing stairs to remain

New bridge span

Preliminary only
Not for construction

RAVPARAVARDA
RAILWAY STATION
PEDESTRIAN OVERBRIDGE

Concept Plan of
Upgraded Bridge
Appendix 3.2
Concept Plan of Replacement Bridge
Proposed fences

Concrete ramp

Existing stairs to bridge to remain

Proposed impact wall

New bridge span

Proposed relocated stairs

Proposed ramps
- 9 m long ramps at 1:12
- 1.2 m landings

Existing stairs to platform to be relocated

PRELIMINARY ONLY
NOT FOR CONSTRUCTION

KANWARAWHARA
RAILWAY STATION
PEDESTRIAN OVERBRIDGE

Concept Plan of Replacement Bridge
Appendix 4.1
Bridge Upgrading – Budget Cost Estimate
## Bridge Upgrading - Budget Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost</th>
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<tbody>
<tr>
<td>Main spans</td>
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<tr>
<td>Main span piers</td>
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<tr>
<td>Pier foundations</td>
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<tr>
<td>Drainage</td>
<td>$</td>
</tr>
<tr>
<td>Handrails</td>
<td>$</td>
</tr>
<tr>
<td>Fit existing stairs to new piers</td>
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</tr>
<tr>
<td>Asphalt</td>
<td>$</td>
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<tr>
<td>Signage, markings</td>
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<tr>
<td>Demolition/Deconstruction</td>
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</tr>
<tr>
<td>New lighting poles, etc. estimate</td>
<td>$</td>
</tr>
<tr>
<td>Alterations to traction overhead, estimate</td>
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</tr>
<tr>
<td>KiwiRail, protection, permit, etc. estimate</td>
<td>$</td>
</tr>
<tr>
<td>Bridge hanger and protection, estimate</td>
<td>$</td>
</tr>
<tr>
<td>Consents</td>
<td>$</td>
</tr>
<tr>
<td>Margin 8%</td>
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<tr>
<td>Sub Total</td>
<td>$</td>
</tr>
<tr>
<td>Working in rail corridor 30%</td>
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</tr>
<tr>
<td>Preliminary &amp; General 12%</td>
<td>$</td>
</tr>
<tr>
<td>Sub Total</td>
<td>$</td>
</tr>
<tr>
<td>Contingency 20%</td>
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<tr>
<td>Physical Works Total</td>
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<tr>
<td>Professional Fees (Budget)</td>
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<td><strong>TOTAL BUDGET ESTIMATE</strong></td>
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<tr>
<td><strong>TOTAL BUDGET ESTIMATE (ROUNDED)</strong></td>
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</table>
Appendix 4.2
Bridge Replacement – Budget Cost Estimate
### Bridge Replacement - Budget Cost Estimate

<table>
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<th>Item</th>
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<tr>
<td>Main spans</td>
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</tr>
<tr>
<td>Main span piers</td>
<td>$1111111111111111111</td>
</tr>
<tr>
<td>Pier foundations</td>
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</tr>
<tr>
<td>Drainage</td>
<td>$1111111111111111111</td>
</tr>
<tr>
<td>Ramp Handrails</td>
<td>$1111111111111111111</td>
</tr>
<tr>
<td>Span handrails</td>
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</tr>
<tr>
<td>Relocate stairs</td>
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<tr>
<td>Asphalt</td>
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<tr>
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