Innovation in Infrastructure

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Greater Wellington Rail Limited
Kaiwharawhara Pedestrian Over Bridge
Structural Assessment and Repair Options Report

July 2013
Prepared by Spiire

Quality Assurance Statement		
Task	**Responsibility**	**Signature**
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Prepared by	Tom Arthur	
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Approved for issue by	Sid Wade	

Issue Date	Revision No.	Author	Checked	Approved
1 August 2013	1			

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Executive Summary

Spiire New Zealand Ltd has been engaged by Greater Wellington Rail Ltd (GWRL) 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 inspected the bridge on 13 June 2013 and observed extensive corrosion to the steel I-beams and supporting piers. Subsequently 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.

Total bridge replacement including the provision of new ramps for accessible requirements was considered but this option is impractical due to insufficient platform widths and cost.

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.

Rough Order Budget Cost Estimates for the remedial options are:

Bridge Upgrading (replace beams, deck and piers)	**$550,000.00**
Total Bridge Replacement (includes new ramps)	**$2,470,000.00**

1. Existing Bridge

1.1 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 shallow 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.

1.2 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 work associated with the inspection.

The following investigative work was undertaken on site:

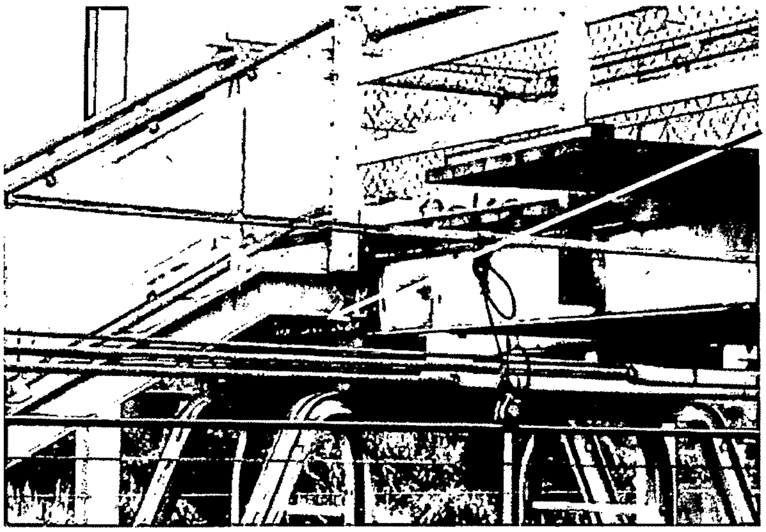
- Sections of timber decking were removed above piers to better assess the extent of corrosion of the spans
- Areas of asphalt and concrete were chipped away to expose the bottoms of some of the pier rail-irons where they extend into the concrete pad foundations
- Exploratory holes were drilled in timber 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.

1.3 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.



Daylight through the beam

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.

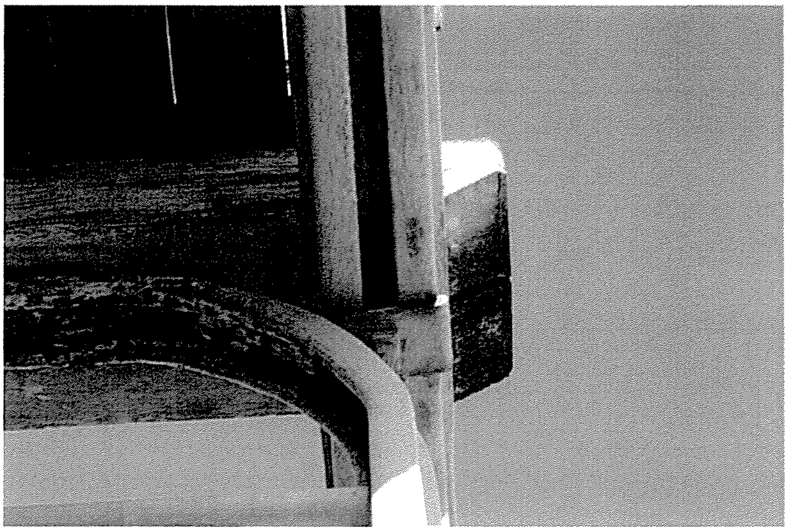


Photo Three: Timber corbel with split along bolt line.

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.

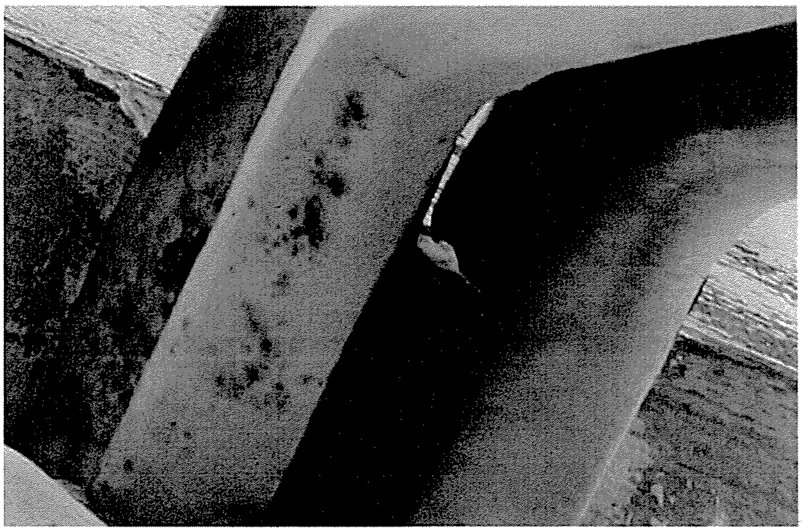


Photo Four: Extensive corrosion of rail section (between members). Surface corrosion and loose rust evident.

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

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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.

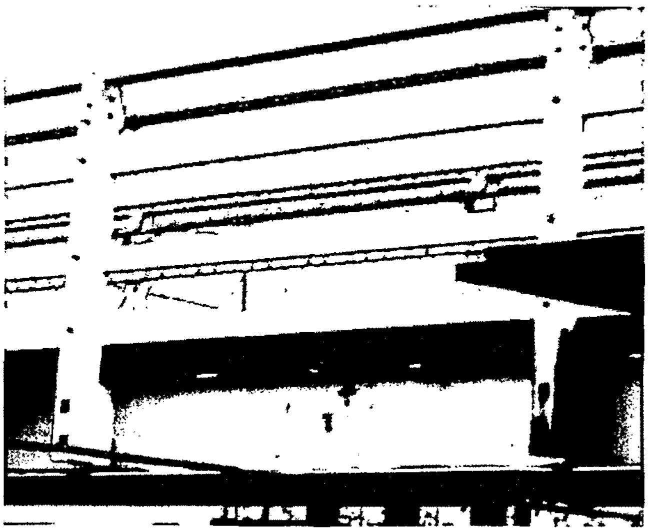

Photo Eight: Comparison of old and new balustrade supports

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

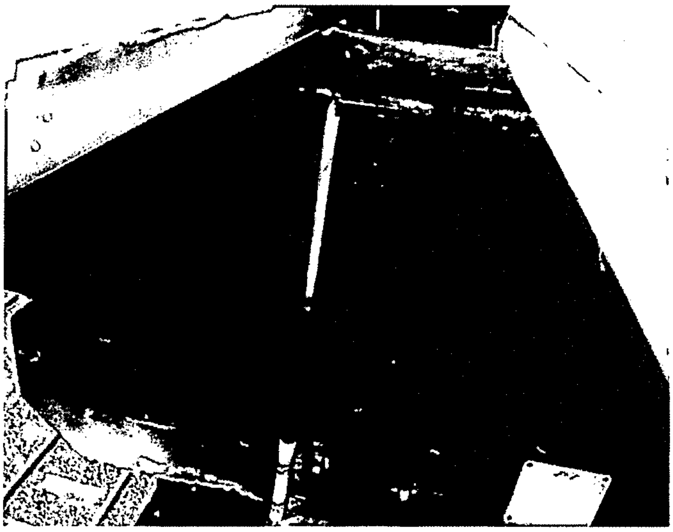

Photo Nine: Corrosion to the tops of the top flange of the I-beams under the timber packers supporting the deck.

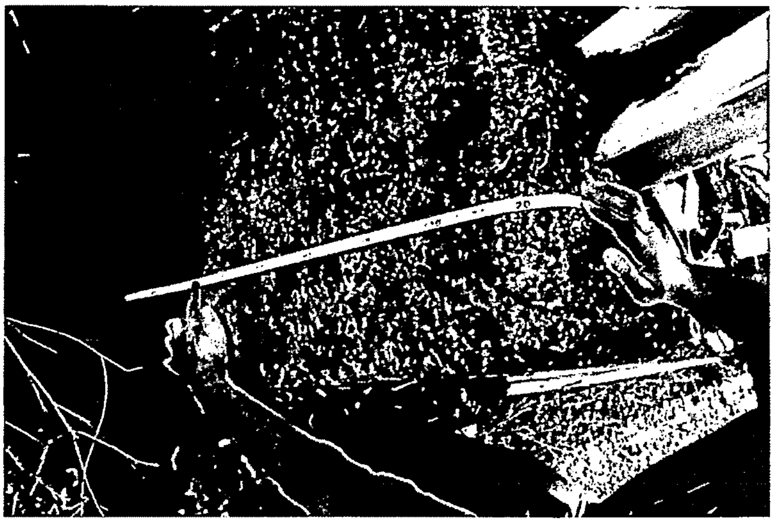

Photo Ten: Bottom of concrete pad foundation adjacent to the west boundary fence is 600mm below ground level, founded on solid ground.

Some steel splice plates have been attached relatively recently to the to the pier legs. These do not address the problem of extensive corrosion of the rail-iron piers.

Photo Eleven: East side pier. Extensive corrosion to the pier leg connecting bolts and the circular hollow section prop between rail-iron legs. Note plates added recently at joint.

1.4 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
- NZTA Bridge Manual, 3rd Edition: 2013
- 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. We do not recommend that material testing be undertaken to confirm the yield stress of the material. This is because the analysed stress in some of the bridge members being significantly higher than the yield stress, and because of the extensive rusting and significant loss of section in some parts of the structure.

Table 1: Member Utilisation Summary

Bridge Member Description	% of Current Code Strength & mode of Failure	Comments
Harbour side Pier, 53 lb/yard Rail-iron leg	58%, Compression.	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.
Central Pier, 53 lb/yard Rail-iron leg	84%, Compression.	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.
Hutt Road Pier, 53 lb/yard Rail-iron leg	22%, Compression.	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.
Main Support I-Beam	120%, Bending	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

2. Wellington City Council Requirements

A building consent is required for upgrading work to the bridge because the asset is not owned by a Network Utility Operator. This could possibly trigger the need to provide an accessible bridge. If so, this would require the provision of ramps. However the ramps are unlikely to be able to comply with minimum platform width requirements.

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 building code requirements.

Before a decision is made on the future of the bridge Spiire recommends presenting a submission to Council detailing the options for upgrading or renewal of the bridge with a view to obtaining Council's approval in principle.

3. 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.

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4. Conclusion and Recommendations

4.1 Discussion of Options Considered

The following options have been considered:

Option	
Temporary propping	Discounted due to: • The poor condition of the structure to be propped • Extent of other repair work that would need to be undertaken
Repair of Existing Structural elements	Discounted due to: • The condition of existing structure is so poor the we do not believe it to be practical or economic to repair
Replace Steel piers and bridge decks	Considered to be practical due to: • Cost effective • Existing steps can be utilised • Practical solutions
Total Replacement Provide new piers and bridge decks with accessible ramps	Considered to be unpracticable due to: • The non-compliance of the ramps • Cost

We provide a detailed breakdown of the last two options

4.2 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 rest of the existing structure, the extent of corrosion and loss of section of some of the bridge components we do not deem it practical or economic 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. Re-use or extending the existing foundations would be considered in the design process.
- 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 rough order budget cost estimate for the above is $550,000.00.

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

4.3 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 vehicle 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. The ramps are therefore non-compliant.

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 rough order budget cost estimate for the above is $2,470,000.00.

(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

5. Disclaimer

This Report has been prepared by Spiire New Zealand (Spiire) for the Greater Wellington Rail Limited in accordance with the brief and limited discussion with the client prior to inspection of the bridge. The Report and assessment therein are based on a visual and non-intrusive walk-around inspection of the bridge, the primary purpose of which is to assess the condition.

Our report is therefore limited to observable condition and does not include a full quantitive assessment which may involve further testing and/or destructive inspection. As Spiire has not carried out a full quantitive assessment it provides preliminary comments only as to the bridges degree of compliance with the New Zealand Building Act or any other relevant codes or standards

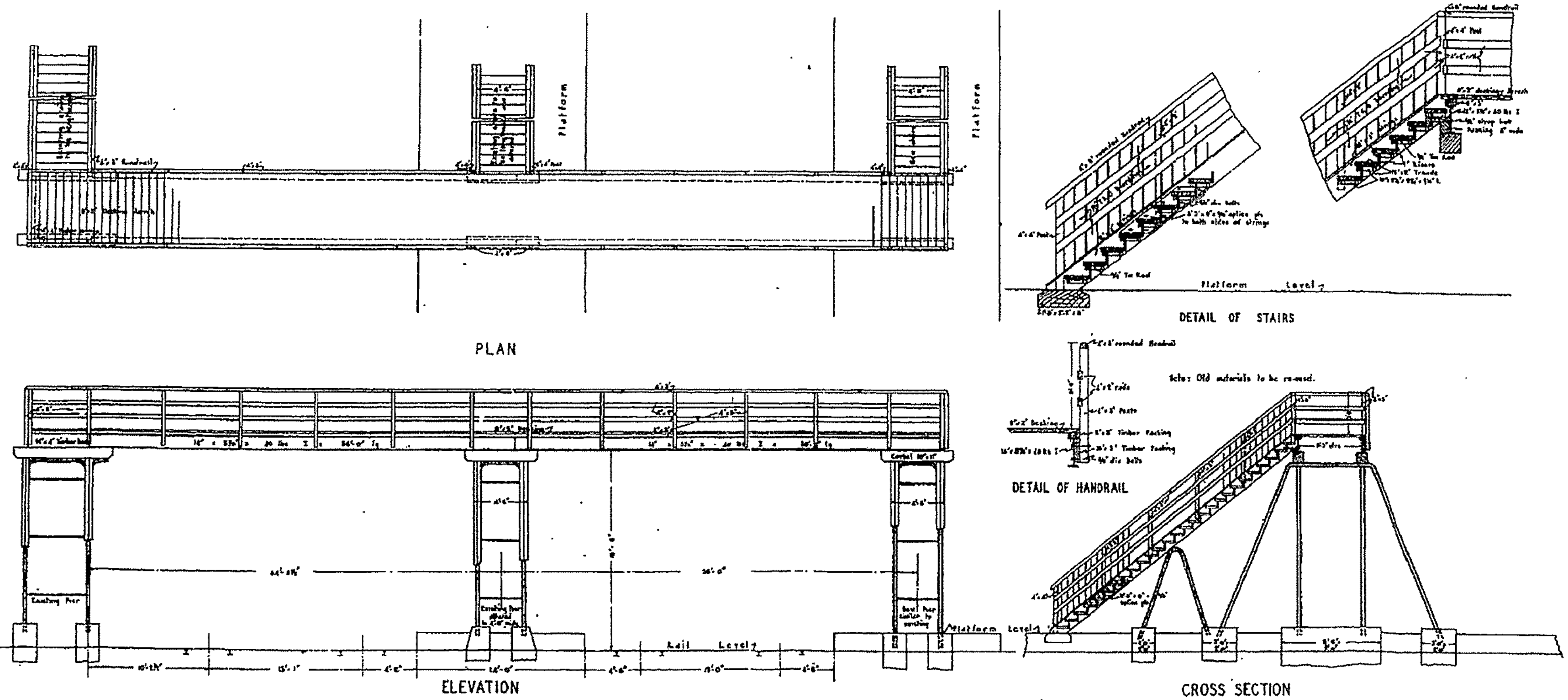
Given the limited scope of Spiire's inspection work, no liability is accepted by Spiire or any of its employees and they disclaim and exclude to the fullest extent all possible legal liability (whether arising in contract, tort, under statute, or otherwise) to the Client and any third party in connection with its inspection and assessment of the bridge and for the information contained in or omitted from this Report. In accepting delivery and/or using this Report, the recipient agrees that he/she accepts the Report on the basis set out herein.

This disclaimer shall apply notwithstanding that the Report may be made available to other persons for an application for permission or approval to fulfil a legal requirement.

Appendix 1

Existing Bridge Drawings

- Bridge before the stairs were replaced, numbered 45847
- Bridge with replacement stairs, in 5 sheets, numbered 120079



KAIWARRA

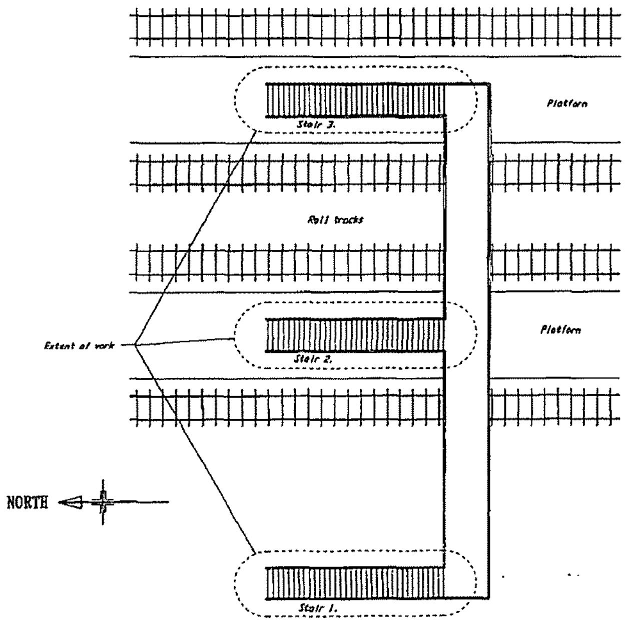
PROPOSED FOOT OVERBRIDGE.

SCALE 4' & 2' TO 1"

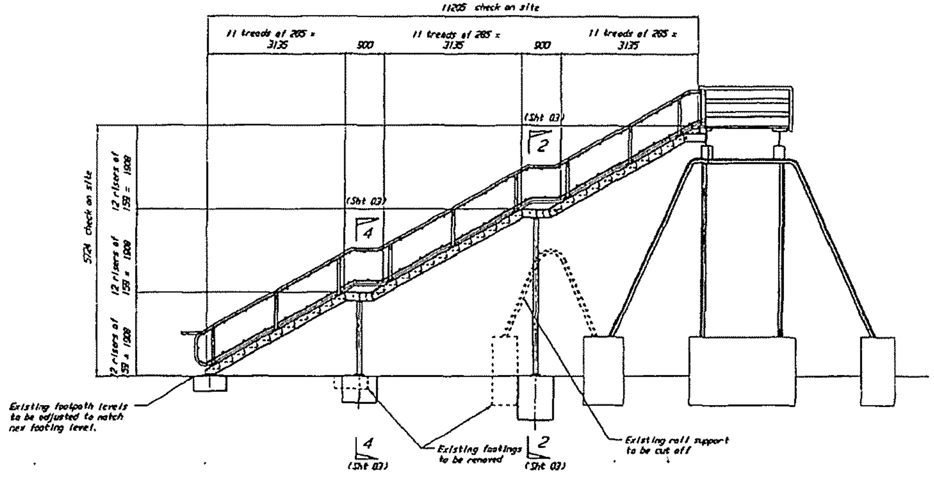
INCHES

METRIC

45847

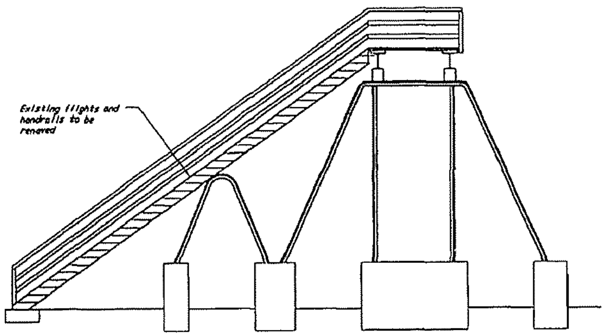


PLAN - EXISTING



ELEVATION STAIR 1 - PROPOSED

For proposed elevations of stairs 2 and 3 see sheets 4 & 5.



ELEVATION - EXISTING

NOTE:

1. The Contractor shall check all dimensions on site prior to commencing work. Any variations found shall be reported to the Engineer.
2. Particular care is needed when working in the vicinity of overhead electrical wires.

ISSUE	BY	DATE	REASON
2	C.B	25/07/05	Redesigned
1	C.B	10/01/05	Handrail fixing changed. For Tender.
C	C.B	20/10/04	Landing added
B	C.B	14/06/04	For Tender
A	C.B	01/06/04	For approval.

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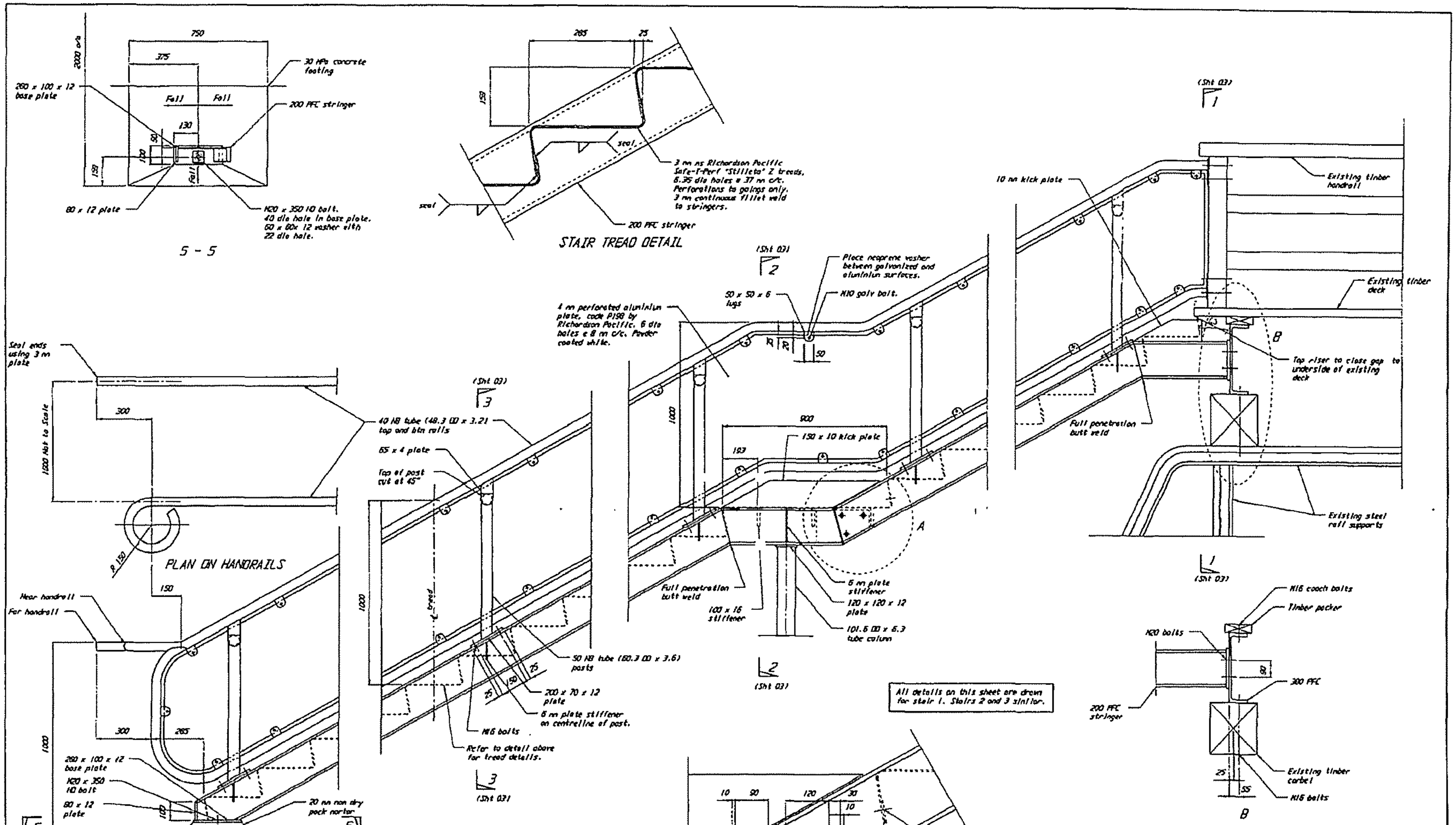
TRACK AND STRUCTURES ENGINEERING
PO Box 593
WELLINGTON

DESIGNED:	O. S.	07/2005
DRAWN:	C. J. B.	06/2004
CHECKED:	O. S.	07/2005
APPROVED:		

BRIDGE 5 - WAIRARAPA LINE
KAIWHARAWHARA STATION FOOTBRIDGE
STAIRWAY REPLACEMENT
PLAN and ELEVATION STAIR 1

SCALES: 1:50
ISSUE: 2
SHEET N°: 1 of 5 sheets
CE N°: 120 079



NOTES:

1. Excavate to "Good Ground" as per NZS 3604. If necessary backfill with compacted hardfill to comply with "Good Ground".
2. Concrete 30 MPa at 28 days.
3. Reinforcing steel grade 300 min.
4. Structural steel grade 300 min.
5. Bolts: Grade 8.8 hot dip galvanized.
6. Welds: 6 mm fillet all round the joint except where shown otherwise.
7. Finish: Hot dip galvanise all steelwork.

ISSUE	BY	DATE	REASON
2	CJB	25/07/05	Redesigned. Redrawn.
1	CJB	10/01/05	Hand rail top fixing changed. For Tender.
C	CJB	29/10/04	Stair pitch changed
B	CJB	14/06/04	For tender
A	CJB	01/05/04	For approval.

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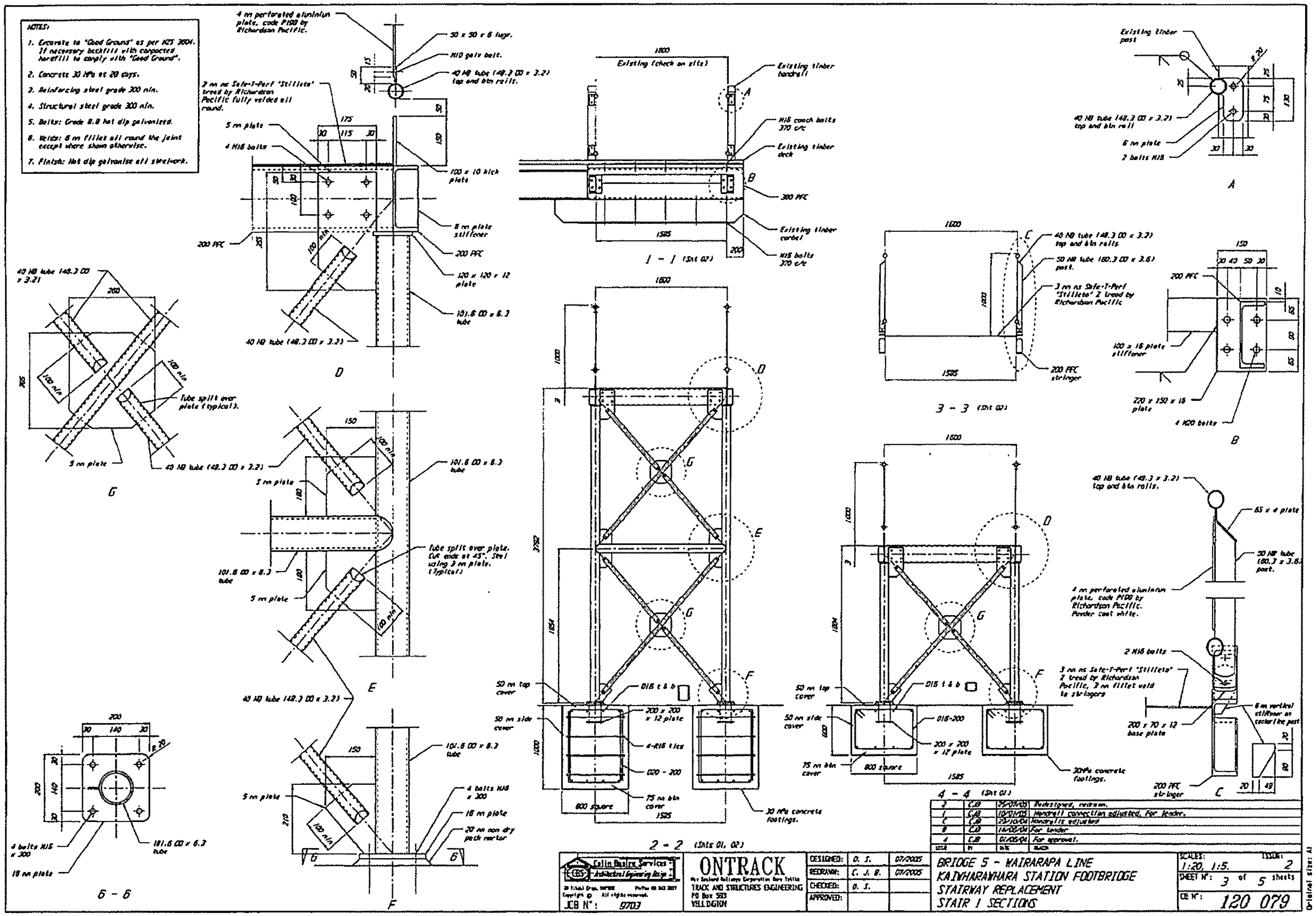
REDRAWN:	C. J. B.	07/2005
CHECKED:	O. S.	
APPROVED:		

BRIDGE 5 - WAIRARAPA LINE
KAIWHARAWHARA STATION FOOTBRIDGE
STAIRWAY REPLACEMENT
STAIR 1 DETAILS

SCALES: 1:10, 1:5 — ISSUE: 2

SHEET N°: 2 of 5 sheets

CE N°: 120 079


NOTES:
1. Excavate to "Good Ground" as per NZS 3604. If necessary backfill with compacted hardfill to comply with "Good Ground".
2. Concrete 30 MPa at 28 days.
3. Reinforcing steel grade 300 min.
4. Structural steel grade 300 min.
5. Bolts: Grade 8.8 hot dip galvanized.
6. Welds: 6 mm fillet all round the joint except where shown otherwise.
7. Finish: Hot dip galvanise all steelwork.
4 mm perforated aluminium plate, code P100 by Richardson Pacific.
3 mm ns Safe-T-Perf "Stiletto" tread by Richardson Pacific fully welded all round.
Existing (check on site)
Existing timber handrail
M16 coach bolts 370 c/c
Existing timber deck
300 PFC
Existing timber corbel
200 PFC
101.6 OD x 6.3 tube
40 NB tube (48.3 OD x 3.2)
Tube split over plate (typical).
30 MPa concrete footings.
D
E
F
G
1 - 1 (Sht 02)
2 - 2 (Shts 01, 02)
3 - 3 (Sht 02)
4 - 4 (Sht 01)
6 - 6
A
B
C
2	C.B	25/07/05	Redesigned, redrawn.
1	C.B	10/01/05	Handrail connection adjusted. For tender.
C	C.B	25/10/04	Handrails adjusted
B	C.B	14/05/04	For tender
A	C.B	01/05/04	For approval.
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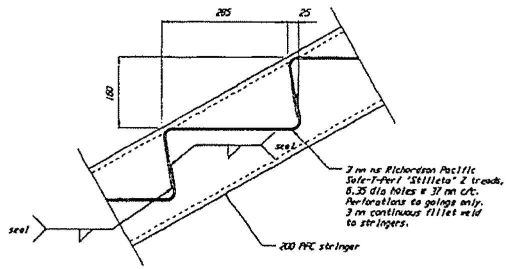
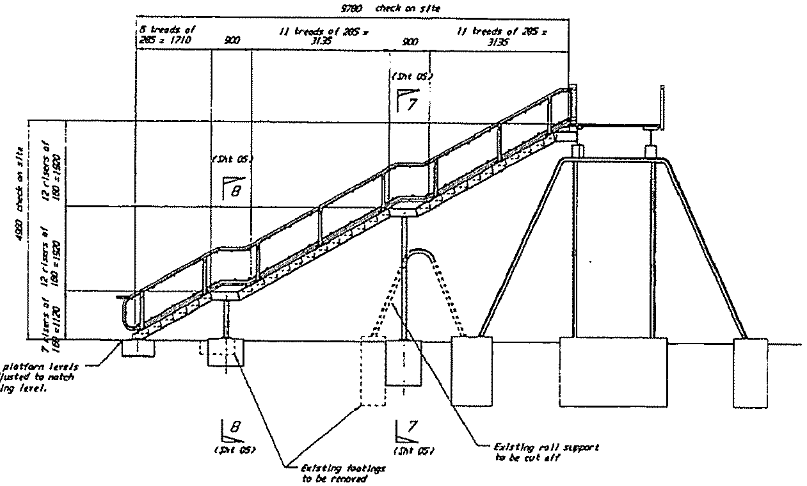
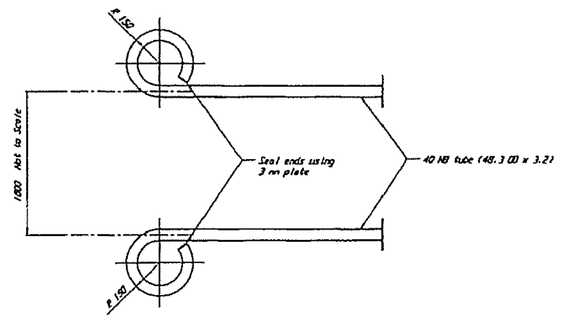
REDRAWN: C. J. B. 07/2005
CHECKED: D. S.
APPROVED:
BRIDGE 5 - WAIRARAPA LINE
KAIWHARAWHARA STATION FOOTBRIDGE
STAIRWAY REPLACEMENT
STAIR 1 SECTIONS
SCALES: 1:20, 1:5.
ISSUE: 2
SHEET N°: 3 of 5 sheets
CE N°: 120 079
Original size: A1



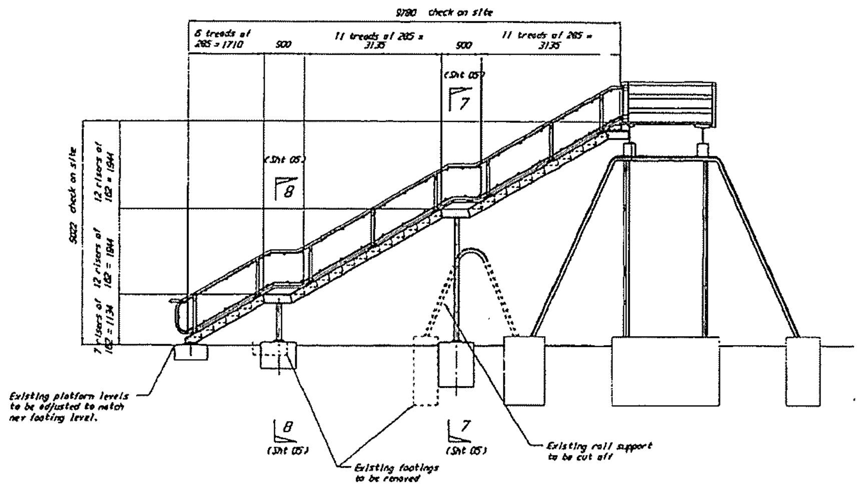
STAIR TREAD DETAIL - STAIR 2



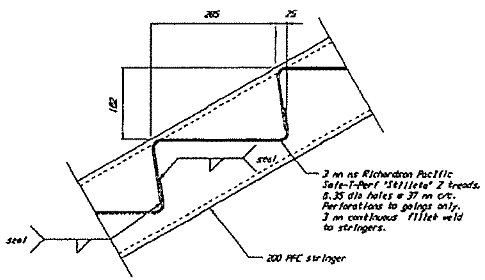
ELEVATION STAIR 2 - PROPOSED



PLAN ON HANDRAILS - STAIRS 2 and 3
Plan at lower end of handrails



ELEVATION STAIR 3 - PROPOSED



STAIR TREAD DETAIL - STAIR 3

Refer to sheet 01 for plan view.

NOTE:

1. The Contractor shall check all dimensions on site prior to commencing work. Any variations found shall be reported to the Engineer.
2. Particular care is needed when working in the vicinity of overhead electrical wires.

ISSUE	BY	DATE	REASON
2	CJB	02/08/05	Totally redrawn - For tender

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PO Box 593
WELLINGTON

DESIGNED:	D. S.	08/2005
DRAWN:	C. J. B.	08/2005
CHECKED:	D. S.	
APPROVED:		

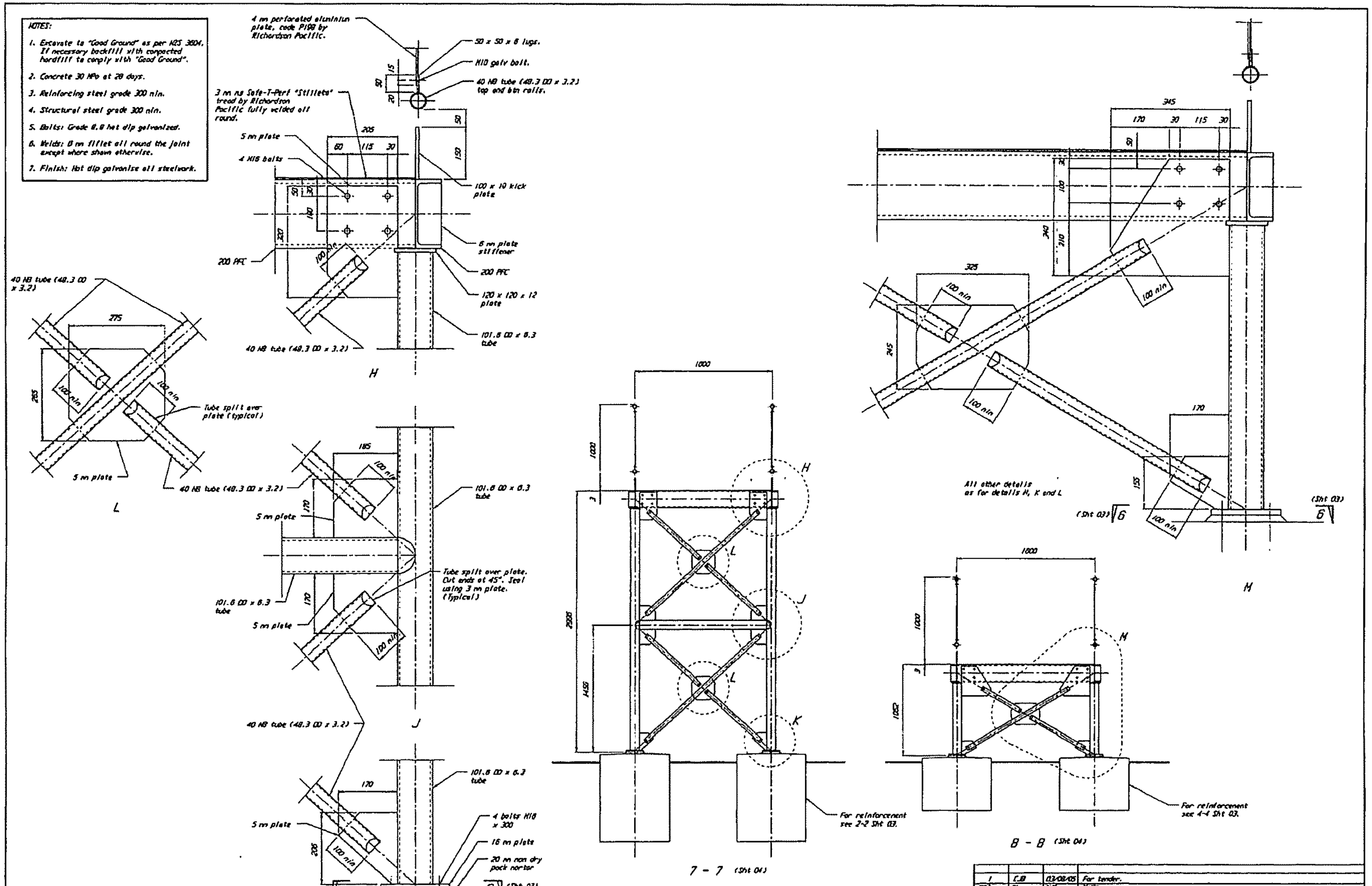
BRIDGE 5 - WAIRARAPA LINE
KAIWHARAWHARA STATION FOOTBRIDGE
STAIRWAY REPLACEMENT
ELEVATIONS STAIRS 2 and 3

SCALES: 1:50, 1:10, 1:5
ISSUE: 2
SHEET Nº: 4 of 5 sheets
CE Nº: 120 079

Original size: A1

NOTES:

1. Excavate to "Good Ground" as per NZS 3604. If necessary backfill with compacted hardfill to comply with "Good Ground".
2. Concrete 30 MPa at 28 days.
3. Reinforcing steel grade 300 min.
4. Structural steel grade 300 min.
5. Bolts: Grade 8.8 hot dip galvanized.
6. Welds: 8 mm fillet all round the joint except where shown otherwise.
7. Finish: Hot dip galvanise all steelwork.



1	C.B	03/08/05	For tender.
ISSUE	BY	DATE	REASON

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REDRAWN:	C. J. B.	08/2005
CHECKED:	O. S.	
APPROVED:		

BRIDGE 5 - WAIRARAPA LINE
KAIWHARAWHARA STATION FOOTBRIDGE
STAIRWAY REPLACEMENT
STAIRS 2 and 3 SECTIONS

SCALES: 1:20, 1:5. ISSUE: 1
SHEET N°: 5 of 5 sheets
CE N°: 120 079

Original size: A1

Appendix 2

Design Features Report and Structural Analysis Summary Calculations

Innovation in Infrastructure

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

Task	Responsibility	Signature
Project Manager	Rob Bryant	
Prepared by	Tom Arthur	
Reviewed by	Bevan White	
Approved for issue by	Sid Wade	

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Appendix A Structural Analysis Summary Calculations

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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 has 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
- NZTA Bridge Manual, 3rd Edition: 2013
- 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 four railway tracks. The bridge was constructed from 14" x 5.5" Universal beams simply supported on timber transom beams and 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 Load Resisting System

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.

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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 (for this station having a capacity of less than 250 people) 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

Level / Area	Use	Live Load	Dead Load
Bridge Deck	Pedestrian Loads	5.0 kPa	0.6 kPa

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

Level / Area	Top Edge			Infill	
	Horizontal kN/m	Vertical kN/m	Inwards, outwards, or downwards kN	Horizontal kPa	Any direction kN
Ballustrade	0.75	0.72	0.60	2.2 (wind)	0.5

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.

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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

$C_h(T)$ = 3.0

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

The structure has been assumed to be nominally ductile. μ = 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).

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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 defection 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

Analysis type	Software used	Archive files
3D frame analysis	MICROSTRAN, V9.0	
General spreadsheet design	EXCEL 2010	

7. Design Notes

7.1 Superstructure

7.1.1 Design Loads

Refer to Section 4 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

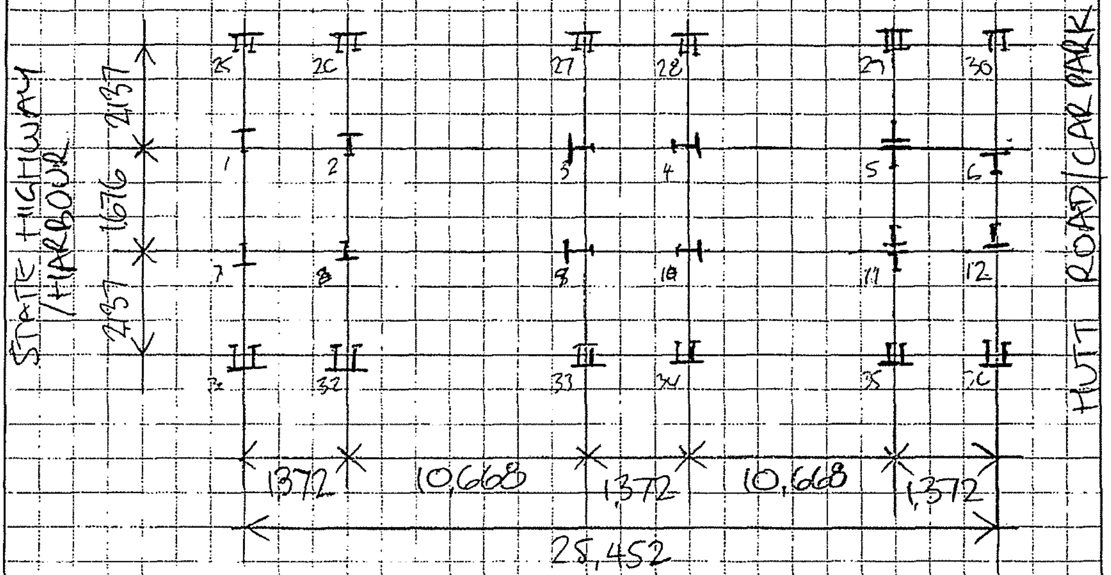
Rolled Steel Sections and rail-irons: fy = 225 MPa & fu = 432 MPa assumed

Appendix A

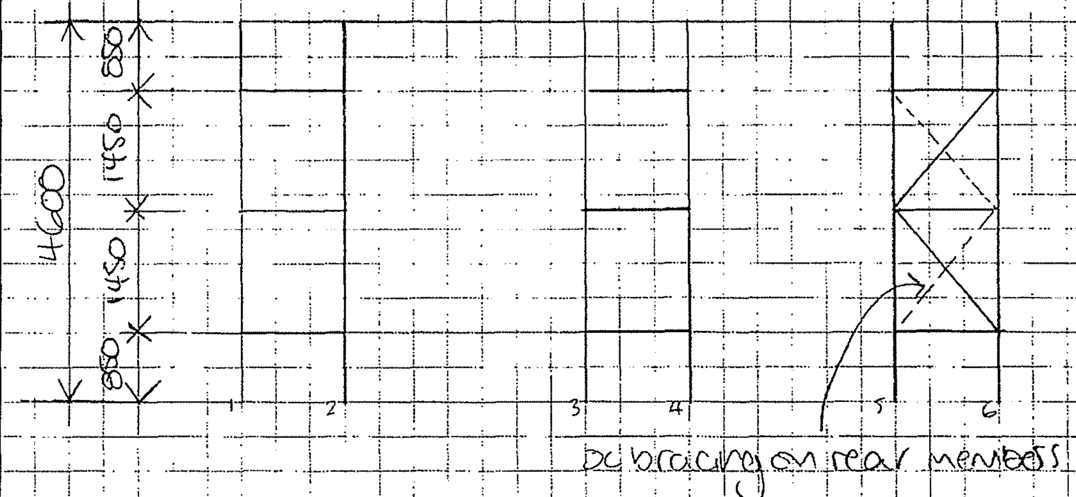
Structural Analysis Summary Calculations

spiire Innovation in Infrastructure	Title: KAIWHARAWHARA FOOT BRIDGE	Job No. 706880
		Page No. 01.
	Description: RATIONALISED BRIDGE GEOMETRY FOR MODEL.	Date: 02/07/13
		Author: TA
		Reviewer:
	SUMMARY CALCULATIONS	Revision:

PLAN. SHOWING SET OUT DIMENSIONS & MEMBER ORIENTATION.



ELEVATION.



Following a visit to site & measurement of the rail sections forming the legs / plinths of the footbridge, it is thought that the members are similar to '50 NZR'

Calculate values required by Microstran software that have not been given in fig 1.3.1.

$$r_x = \sqrt{\frac{483 \times 10^4}{3361}} = 37.91$$

$$r_y = \sqrt{\frac{99.5 \times 10^4}{3361}} = 17.21$$

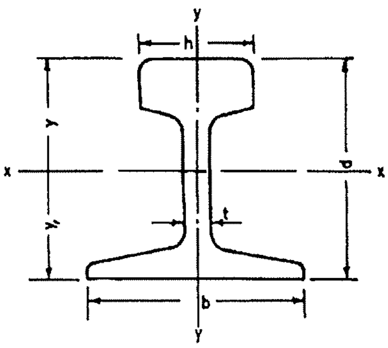
Section weight per m $= 3361 \times 10^{-6} \times 7850$

$= 26.38$ kg/m

Circular hollow section members are located between the legs of the plinths. They are 48.3 Ø, it has been assumed that they are 2.3mm thick.

Due to extensive corrosion observed, it is assumed that the members are mild steel. $f_y = 225$ MPa

$f_u = 432$ MPa



53 N.Z.R.
PROPERTIES USED

SECTION	AREA	d	b	h	t	y	y'	I_{xx}	Z_{xx}	$Z_{x'x}$	I_{yy}	Z_{yy}
	mm^2	mm	mm	mm	mm	mm	mm	cm^4	cm^3	cm^3	cm^4	cm^3
40lbs/yd	2574.2	85.7	82.6	47.6	12.7	44.1	41.3	240	54.1	57.4	58.7	14.3
53 N.Z.R.	3361.3	104.8	95.3	54.0	11.9	52.8	51.8	483	91.3	93.1	99.5	20.8
55 B.S.	3471.0	104.8	104.8	54.8	11.1	51.8	52.8	508	97.7	96.0	125.6	23.9
56 N.Z.R.	3580.6	103.2	101.6	57.2	12.7	50.4	52.8	524	104.2	99.3	119.4	23.4
70 B.S.	4419.4	117.5	117.5	60.3	13.1	60.3	57.4	820	136.7	142.9	214.8	36.4
70 R.B.S.	4438.7	123.8	117.5	60.3	12.7	64.7	59.1	923	142.9	155.9	216.4	36.9
72 N.Z.R.	4529.0	123.8	117.5	60.3	12.7	64.8	59.5	941	145.8	159.0	220.6	37.6
75 A.S.C.E.	4729.0	122.2	122.2	62.7	13.7	63.5	58.4	953	149.1	163.9	248.5	40.8
85 R.B.S.	5387.1	138.1	131.8	65.1	13.9	71.9	66.3	1385	190.3	205.8	283.0	42.9
90 RA-A	5690.3	142.9	130.2	65.1	14.3	78.2	64.5	1611	208.1	249.1	338.7	52.1
91 N.Z.R.	5722.6	142.9	131.8	65.1	14.3	78.5	64.3	1582	201.5	245.8	347.6	52.7
100 B.S.	6329.0	146.1	146.1	69.9	14.7	75.2	70.8	1844	245.3	260.5	422.4	57.9
100 R.B.S.	6329.0	152.4	146.1	69.9	14.3	79.5	72.8	2001	251.9	274.5	420.8	57.7
50 NZR	6400	153.0	132.0	66.0	15.0	81.5	71.5	1975	241	276	333.5	50.6

RAIL SECTIONS PROPERTIES AND DIMENSIONS

Fig. 1.3.1 Properties and Dimensions of N.Z.R. Rail Sections.

9/73

EXTRACT FROM HISTORICAL STEELWORK HANDBOOK.

TABLE NO. 3.8 PROPERTIES OF BEAMS TO BRITISH STANDARD 4 1921

IMPERIAL UNITS See separate page for notes

Ref No.	Size D x B	Approximate Mass/ft	Metric Equivalent D x B	Metric Equivalent Mass/m	Thickness Web	Thickness Flange	Area	Mom. of Inert. X - X	Mom. of Inert. Y - Y	Rad. of Gyr. X - X	Rad. of Gyr. Y - Y	Sec. Mod. X - X	Sec. Mod. Y - Y
	ins	lbs	mm	kg	ins		ins2	ins4		ins		ins3	
NBSB 1	3 x1.50	4	76x38	6	0.16	0.25	1.18	1.66	0.13	1.19	0.33	1.11	0.17
NBSB 2	4 x1.75	5	102x44	7	0.17	0.24	1.47	3.66	0.19	1.58	0.36	1.83	0.21
NBSHB 1	4 x3	10	102x76	15	0.24	0.35	2.94	7.79	1.33	1.63	0.67	3.89	0.88
NBSB 3	4.50x2	7	114x51	10	0.19	0.32	2.06	6.65	0.38	1.80	0.43	2.96	0.38
NBSB 4	5 x2.50	9	127x64	13	0.20	0.35	2.65	10.9	0.79	2.03	0.55	4.36	0.63
NBSHB 2	5 x4.50	20	127x114	30	0.29	0.51	5.88	25.0	6.59	2.06	1.06	10.0	2.93
NBSB 5	6 x3	12	152x76	18	0.23	0.38	3.53	21.0	1.46	2.44	0.64	7.00	0.97
NBSHB 3	6 x5	25	152x127	37	0.33	0.56	7.35	45.2	9.88	2.48	1.16	15.1	3.95
NBSB 6	7 x3.50	15	178x89	22	0.25	0.40	4.42	35.9	2.41	2.85	0.74	10.3	1.38
NBSB 7	8 x4	18	203x102	37	0.28	0.40	5.30	55.9	3.51	3.24	0.81	13.9	1.75
NBSHB 4	8 x6	35	203x152	52	0.35	0.65	10.30	115.1	19.54	3.34	1.38	28.8	6.51
NBSB 8	9 x4	21	229x102	31	0.30	0.46	6.18	81.1	4.15	3.62	0.82	18.0	2.07
NBSHB 5	9 x7	50	229x178	74	0.40	0.83	14.71	208.1	40.17	3.76	1.65	46.25	11.48
NBSB 9	10 x4.50	25	254x114	37	0.30	0.51	7.35	122.3	6.49	4.08	0.94	24.47	2.88
NBSHB 6	10 x6	40	254x152	60	0.36	0.71	11.77	204.8	21.76	4.17	1.36	40.96	7.25
NBSHB 7	10 x8	55	254x203	82	0.40	0.78	16.18	288.7	54.74	4.22	1.84	57.74	13.69
NBSB 10	12 x5	30	305x127	45	0.33	0.51	8.83	206.9	8.77	4.84	1.00	34.49	3.51
NBSHB 8	12 x8	65	305x203	97	0.43	0.90	19.12	487.8	65.38	5.05	1.85	81.30	16.30
NBSB 11	13 x5	35	330x127	52	0.35	0.60	10.30	283.5	10.80	5.25	1.03	43.62	4.33
NBSB 12	14 x5.50	40	356x140	60	0.37	0.63	11.77	377.1	14.80	5.66	1.12	53.87	5.38

59.65 kg/m

9.40mm 16mm

7548mm²

156,960,870mm⁴

6,160225mm⁴

143.7mm

28.45mm

80271mm³

88162mm³

The bridge will be analysed for loads generated in accordance with the KiwiRail structures code supplement, CSW/0201, issue 6. (code clause)

Load Case One.

Normal everyday use. It is assumed that during crowd loading will not coincide with high wind load.

Dead load of bridge decking.

2 No plys of timber decking = 2x 0.23 = 0.46 kN/m^2
Round up to 0.60 kN/m^2 for blocking, corbels etc.

Live load

Full live load due to crowd loading = 5 kN/m^2 (5.5)
It is thought the bridge would have been designed for 100 lb/sqft. = 100x 0.0479
= 4.79 kN/m^2
Adopt 5 kN/m^2 based on footpath load for paths made of concrete.

Wind load

A wind load of 2.2 kN/m^2 will be applied to the projected windward area of the bridge. (5.7)

Calculate projected windward area, based on details in drawing 45847 & photographs.

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

Projected area of bridge (per m run)

3N° 100mm horizontal timbers	=	3 x 0.10	= 0.30 m²/m
100mm posts @ 1,422mm c/c	=	0.10 / 1.422	= 0.07 m²/m
14' beam	=	14 x 0.0254	= 0.356 m²/m
Packing & decking on beam			= 0.150 m²/m
Wire mesh assuming 0.1m²/m² solidity ratio for mesh 1.20m tall	=	0.10 x 1.20	= 0.12 m²/m

TOTAL PROJECTED AREA = 0.996 m²/m

Wind load acting on UB members

Windward member = 0.996 x 2.20 = 2.19 kN/m

Leeward member = 0.996 x 2.20 x 0.50 = 1.10 kN/m

Dead load of handrails.

Assume 0.6 kN/m² & 1.20m tall handrail.

Dead load = 0.60 x 1.20
= 0.72 kN/m

It is assumed that the stairs are largely self supporting. Only load from the section of stairs fixed to the 14x5.5 UB's will be considered

Stairs span between PFC stringers. Based on width of stairs being 1600mm.

Deadload along stringer. (assume weights as bridge)

$= 0.72 + 1.60/2 \times 0.60$

$= 1.20$ kN/m

Live load along stringer

$= 1.60/2 \times 5$

$= 4$ kN/m

Wind load acting on sides of stair wells, based on 2.2kN/m^2 acting on projected windward area.

Handrail, 48.3mm ø	$= 0.0483 \times 2 = 0.0966$ m^2/m
200 deep PFC stringer	$= 0.20$ m^2/m
900mm tall sheeting, 0.80m^2/m^2	$= 0.80 \times 0.90 = 0.72$ m^2/m
TOTAL AREA	$= 1.016$ m^2/m

Calculate additional area due to inclination of stairs.

Stair flight rises 1908mm over 3585mm horizontal distance.

Length of stairs $= (3585^2 + 1908^2)^{1/2} = 4,061$ mm

Stairs projected area continued.

$4061 / 3585 = 1.133$.

Projected windward area.

$= 1.016 \times 1.133 = 1.151 N^2/M$

Calculate point loads at stringer connections to 14x5½UB beams, based on half of 3585 load going to the bridge

Dead load $= 1.20 \times 3.585/2 = 2.151$ KN

Live load $= 4 \times 3.585/2 = 7.17$ KN.

Wind load (windward face)

$= 1.151 \times 2.2 \times 3.585/2 = 4.539$ KN

Wind load (leeward face)

$= 1.151 \times 2.2 \times 3.585/2 \times 0.50 = 2.27$ 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

Wind load $= (5.022 \times 1.60)/2 \times 2.20/2 = 4.419$ KN

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Title: KAIWHARAWHARA FOOTBRIDGE
Description: WEIGHT OF STAIRS.
Job No. 706880
Page No. 06A.
Date: 02/07/13
Author: TA
Reviewer:
Revision:

Calculate weight of stairs. Weight to be estimated per linear metre of stairwell

STAIR COMPONENT	WEIGHT CALC	WEIGHT (kN/m)
3mm TREADS	$1.676 \times 0.003 \times 1.5 \times 7850 \times 9.8 \times 10^{-3}$	0.580
200 PFC STRINGERS	$2 \times 22.9 \times 9.81 \times 10^{-3}$	0.499
BALLUSTRADE	$2 \times [0.004 \times 27 + 2.61 \times 9.81 \times 10^{-3} \times 2]$ (mesh; handrail)	0.318
TOTAL WEIGHT		1.397

Vertical height of stairs = 4.960m
Horizontal length of stair = 9.70m.

$$\text{Weight of stairs} = (4.96^2 + 9.70^2)^{1/2} \times 1.397$$
$$= 10.89 \text{ kN}$$

Allow additional 10% for fittings and assume bridge will resist half 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. **NZS 1170.5.**

Hazard factor, Z — c3.1.4/t3.3

Z = 0.40 site in Wellington

Spectral shape factor, $C_h(T)$ = 3.0 — t3.1

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

Return period factor, R — t3.5

for ultimate limit state, probability of earthquake being exceeded in 1 year = 1/500 $R_U = 1.0$

for serviceability limit state, probability of earthquake being exceeded in 1 year = 1/25 $R_S = 0.25$

Near fault factor N(T,D)

for ultimate limit state annual probability of exceedance < 1/250

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 0.4s — c3.7

Elastic site spectra for horizontal loading, C(T) — c3.1

$C(T) = C_h(T) \cdot Z \cdot R \cdot N(T,D)$

$= 3.0 \times 0.40 \times 1.0 \times 1.0$

$= 1.20$ (ULS)

Horizontal design action coefficients, $C_d(T_1)$ — NZS1170.5.

$C_d(T) = \frac{C(T) S_p}{k_\mu} \geq \left(\frac{Z}{20} + 0.02\right) R_u$ — c5.2.1.1

Structural performance factor, $S_p = 1.0$ (ULS) — c4.4.1

$k_\mu = \frac{(\mu - 1) T_1}{0.7} + 1$ soil class D, $T_1 = 0.40$

Structural ductility factor, $\mu = 1.25$ based on elements of the structure being brittle. — c2.2

$\therefore k_\mu = 1.143$

$C_d(T) = \frac{C(T_1) \times 1.0}{1.143} = C(T) \div 1.05$ <u>ULS</u>

Structural performance factor SLS $S_p = 0.70$. — c5.2.1.2

$C(T) = 3.0 \times 0.40 \times 0.25 \times 1.0$ ($R_s = 0.25$)

$= 0.30$ (SLS)

$C_d(T) = \frac{0.30 \times 0.70}{1.143}$

$= 0.184$ (SLS)

Seismic weight, W_i — c4.2

$W_i = G_i + \Sigma \psi_E Q_i$

$\psi_E = 0.30$

conservatively include live load for calculation of seismic weight.

Dead load calculated as per bridge manual Cl 5.3.2

NZS 1170.5.

BRIDGE COMPONENT	WEIGHT CALC	WEIGHT (kN)
DECK	0.60 × 25.42 × 1.676	25.56
HANDRAIL	(25.42 + 1.676) × 2 × 0.72	39.01
LEGS	460 × 24 × 26.38 × 2 × 9.81 × 10^{-3} / 2	57.14
STAIRS	10.89 × 3 / 2	16.34
BEAMS	25.42 × 2 × 60 × 9.81 × 10^{-3}	29.92
TOTAL WEIGHT, G_i		167.97 kN

ψ_E = 0.30 applied to live load

$Q_i = 5 \times 25.42 \times 1.676$

$= 213.02$ kN.

$W_i = 167.97 + 0.30 \times 213.02$ e4.2(1)

$= 231.88$ kN.

Horizontal seismic shear, V C6.2.1.2

$V = C_d(T) W_t$

$= 1.05 \times 231.88$

$= 243.4$ kN (ULS).

Equivalent static horizontal force, for a single storey structure such as this bridge the equivalent static force at the the deck level is equal to the seismic shear. C6.2.1.3

Assume seismic shear will be spread equally over deck of structure as weight relatively evenly distributed over bridge.

Apply load as UDL along bridge UB members.

Seismic load = 243.4 / 2x25.452

= 4.78 kN/m

Load will be applied in both principal directions.

For serviceability limit state (SLS) Cd(T) = 0.184

∴ Horizontal seismic shear / equivalent static horizontal load

= 0.184 x 231.88

= 42.67 kN (SLS)

Load shared equally along UB members as previous

Seismic load = 42.67 / 2x25.45

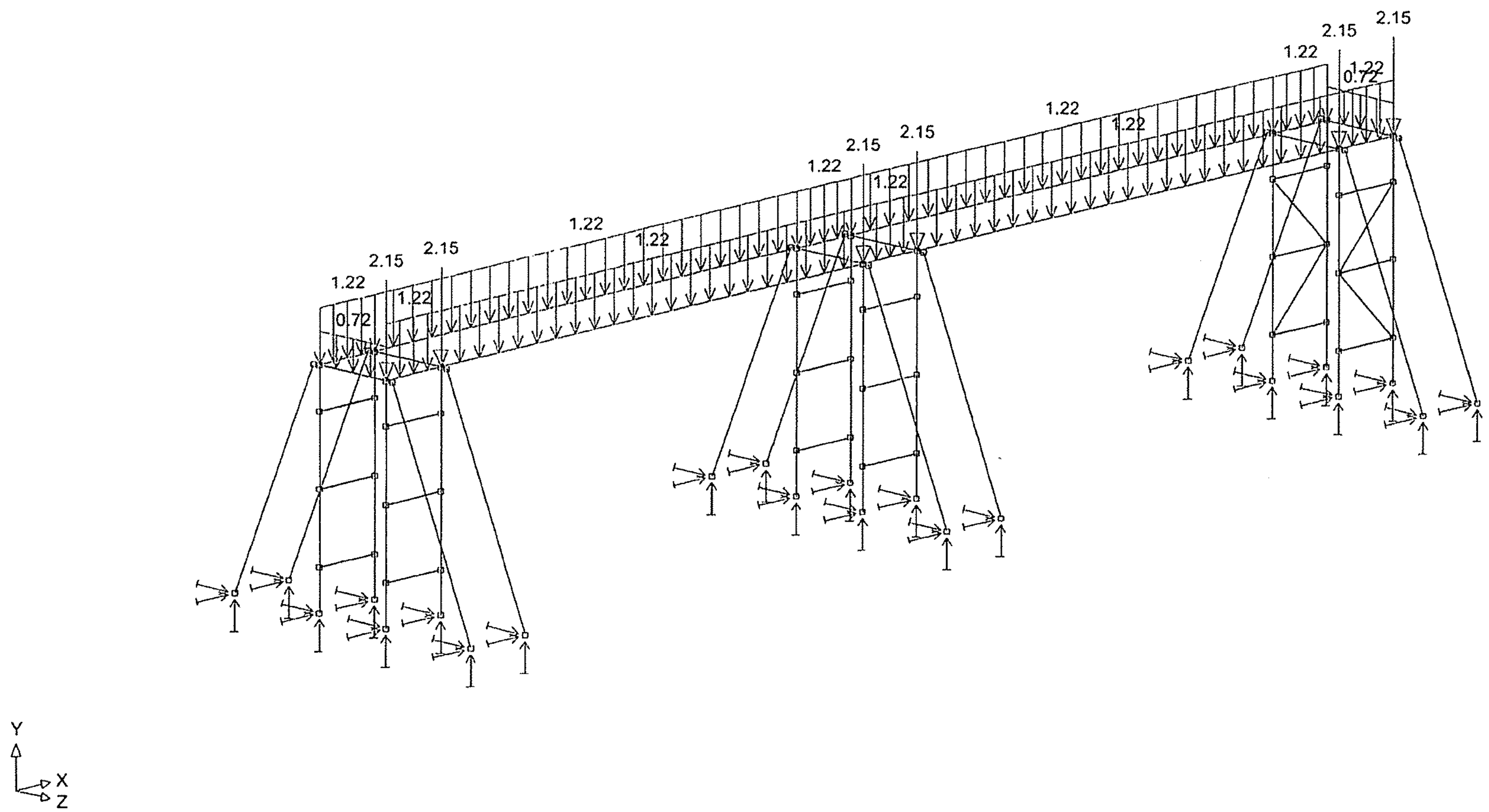
= 0.838 kN/m

spiire

Spiire New Zealand Ltd
Job: Kaiwharawhara Footbridge Model
Kaiwharawhara Footbridge

09/07/2013
04:25:36 p.m.

Load Cases:
1 P Dead Load (gy=-9.81)



theta: 225 phi: 14

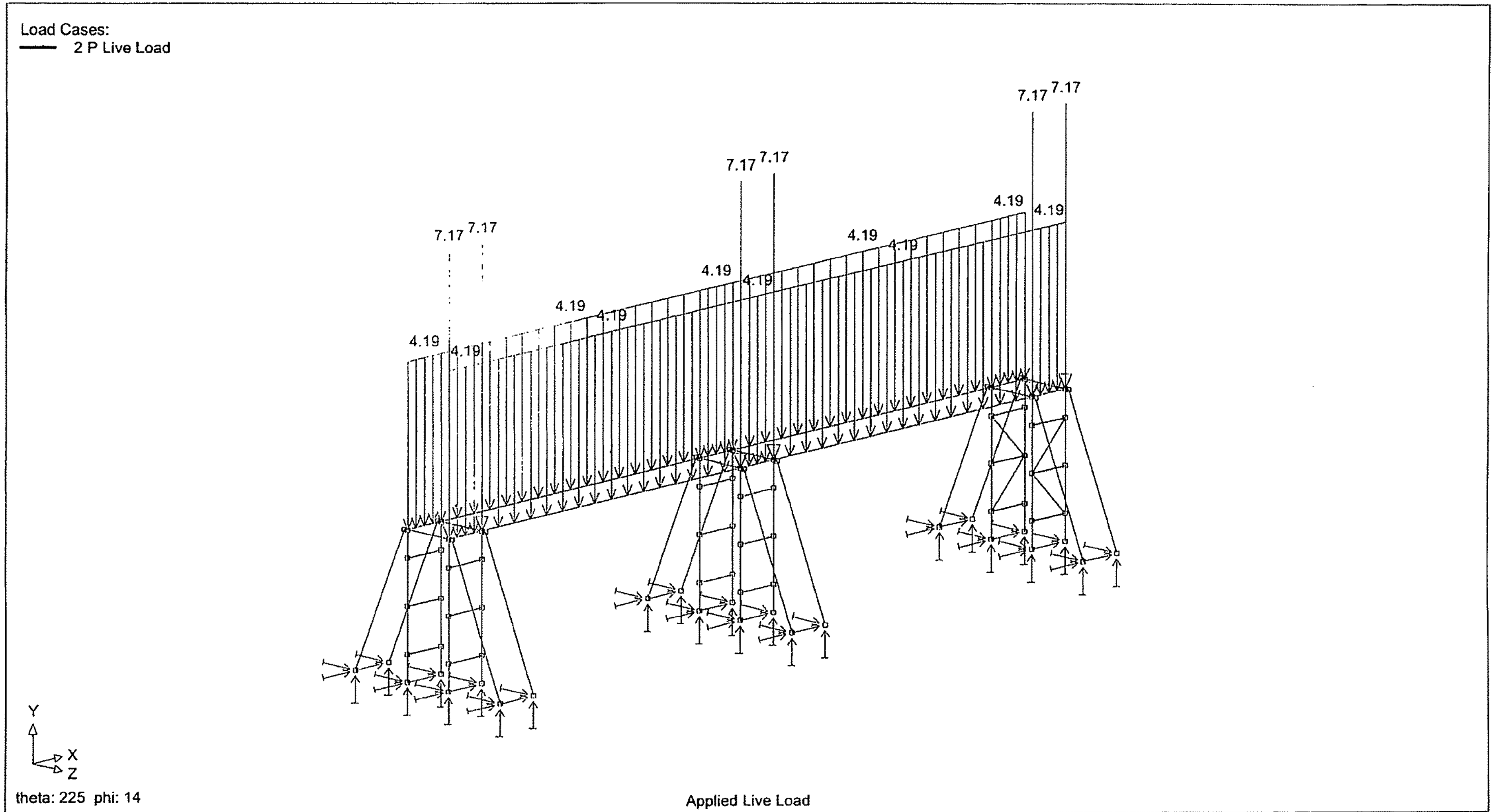
Applied Dead Load

Microstran V9.01.130412 {521501}WGDAT1\Jobs\706880 GW Rail Ltd - Structural Assessment Ped Overbridge Kaiwharawhara\Working\Structural\Microstran Bridge Model\Kaiwharawhara Footbridge Model.msw

spiire

Spiire New Zealand Ltd
Job: Kaiwharawhara Footbridge Model
Kaiwharawhara Footbridge

09/07/2013
04:25:59 p.m.



Applied Live Load

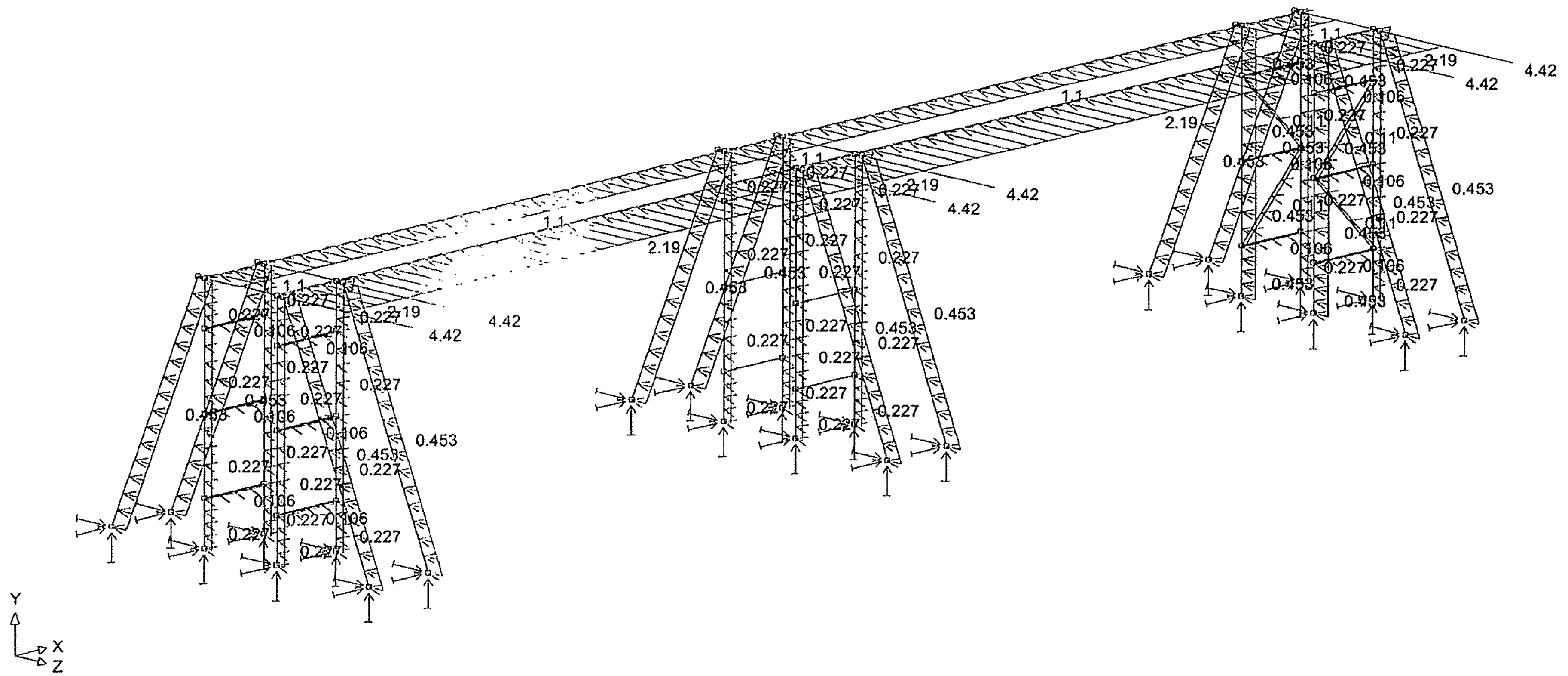
Microstran V9.01.130412 {521501}WGDAT1\Jobs\706880 GW Rail Ltd - Structural Assessment Ped Overbridge Kaiwharawhara\Working\Structural\Microstran Bridge Model\Kaiwharawhara Footbridge Model.msw

spiire

Spiire New Zealand Ltd
Job: Kaiwharawhara Footbridge Model
Kaiwharawhara Footbridge

09/07/2013
04:26:31 p.m.

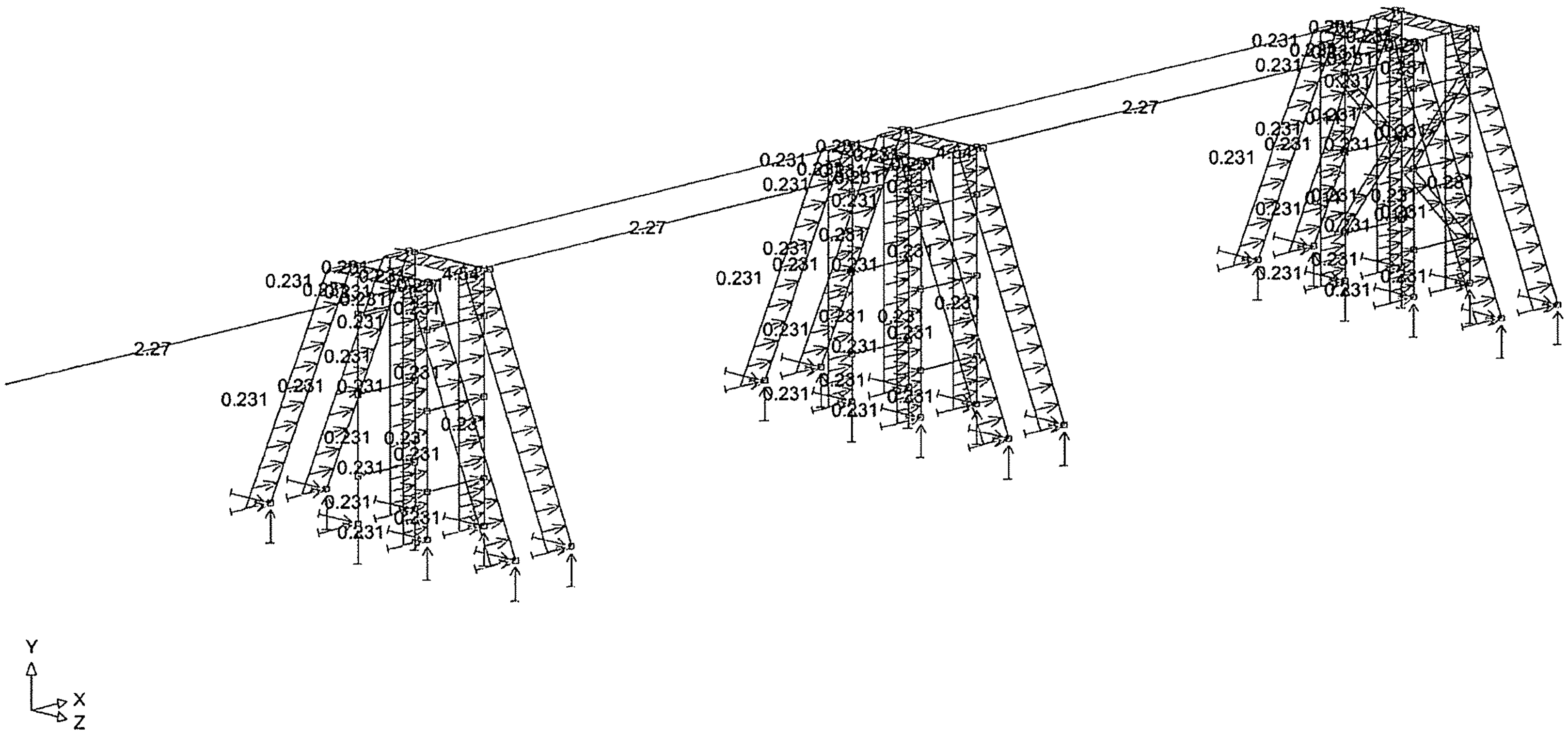
Load Cases:
3 P Wind Load, z (wind on face)



theta: 225 phi: 14

Microstran V9.01.130412 {521501}\WGDAT1\Jobs\706880 GW Rail Ltd - Structural Assessment Ped Overbridge Kaiwharawhara\Working\Structural\Microstran Bridge Model\Kaiwharawhara Footbridge Model.msw

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



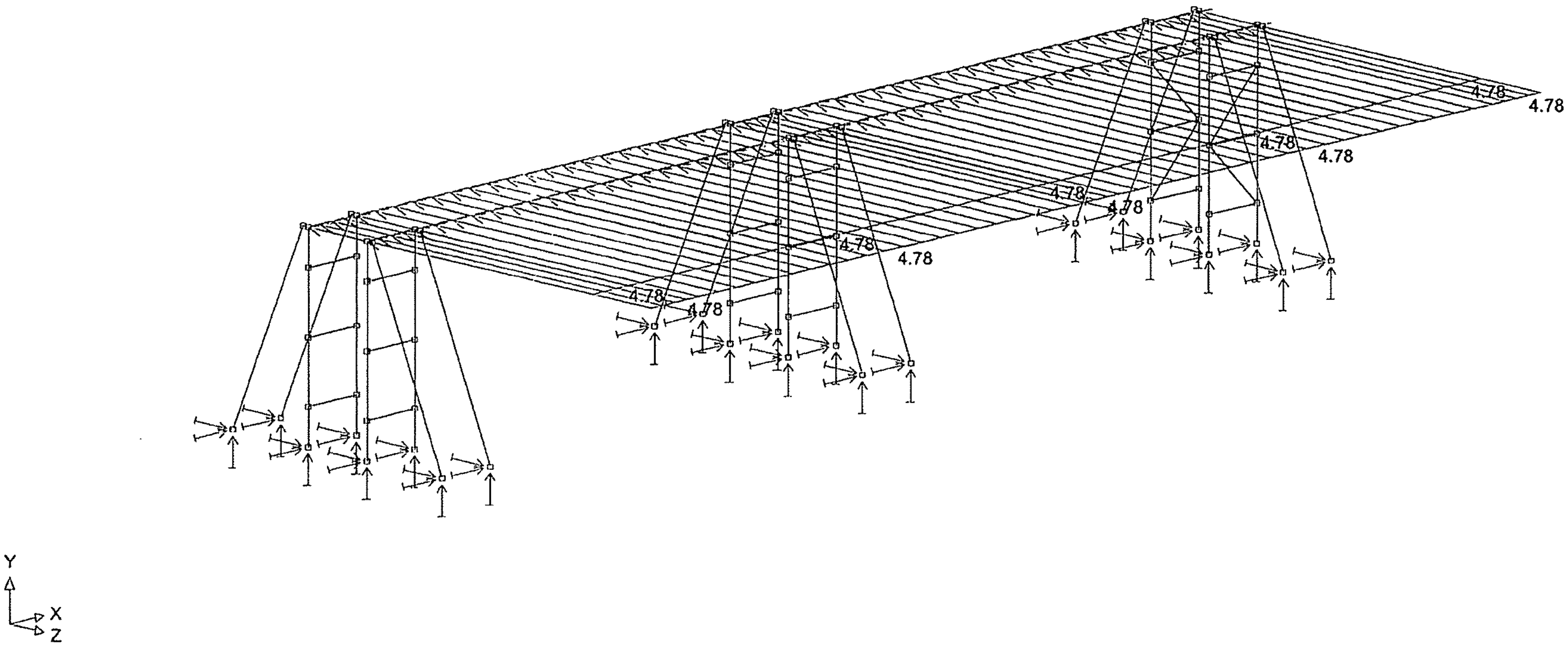
theta: 225 phi: 14

spiire

Spiire New Zealand Ltd
Job: Kaiwharawhara Footbridge Model
Kaiwharawhara Footbridge

09/07/2013
04:27:28 p.m.

Load Cases:
5 P ULS Seismic Load, z (load on face)



theta: 225 phi: 14

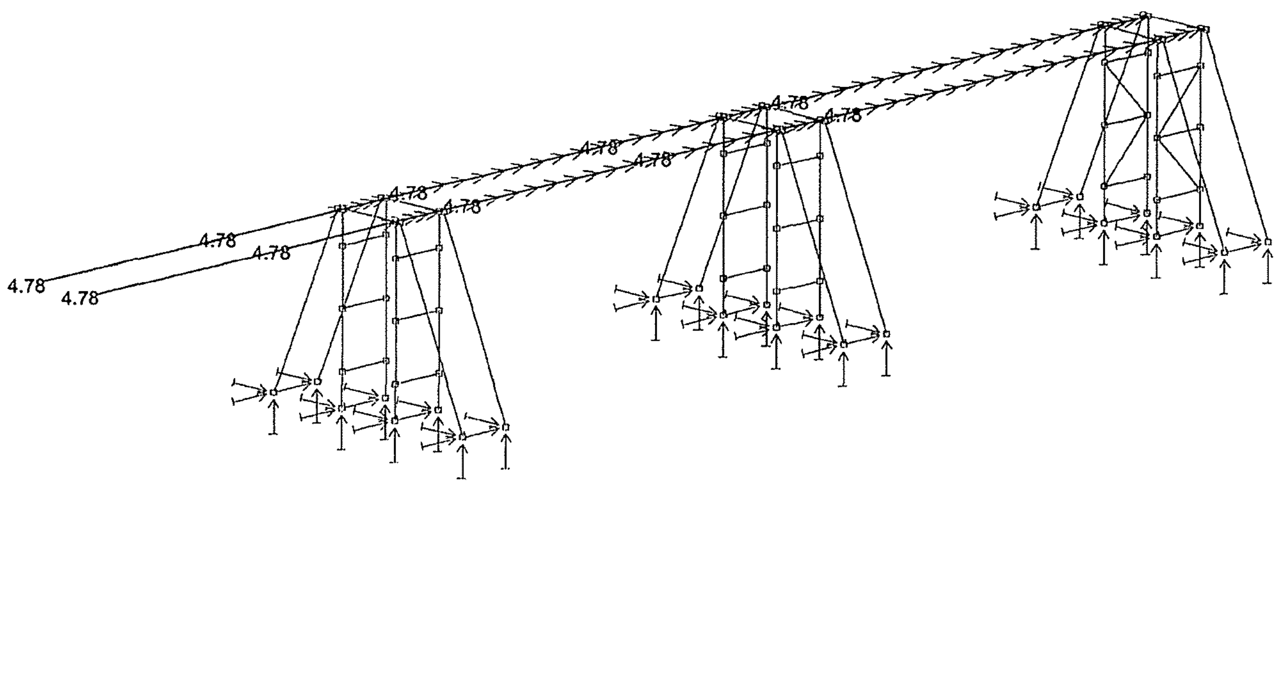
Microstran V9.01.130412 {521501}\WGDAT1\Jobs\706880 GW Rail Ltd - Structural Assessment Ped Overbridge Kaiwharawhara\Working\Structural\Microstran Bridge Model\Kaiwharawhara Footbridge Model.msw

spiire

Spiire New Zealand Ltd
Job: Kaiwharawhara Footbridge Model
Kaiwharawhara Footbridge

09/07/2013
04:27:40 p.m.

Load Cases:
6 P ULS Seismic Load, x (load on end)



theta: 225 phi: 14

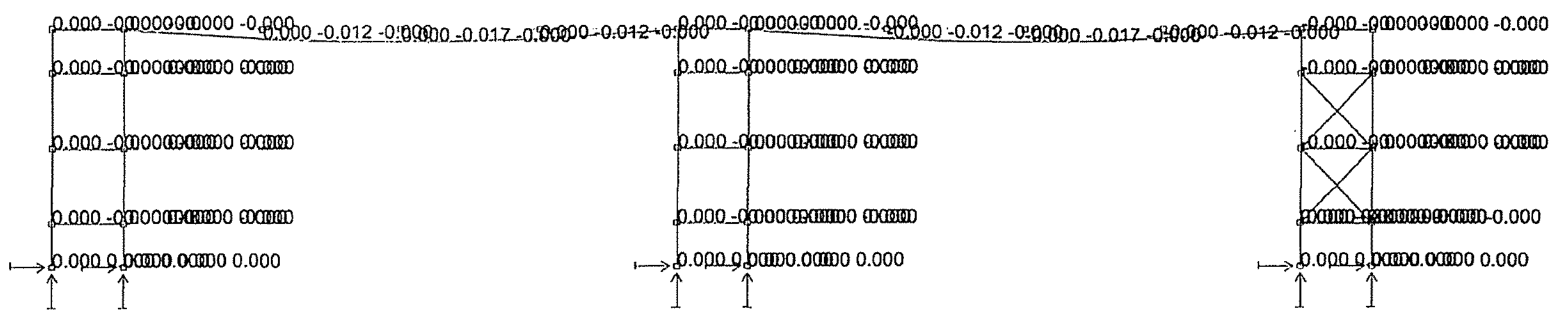
Microstran V9.01.130412 {521501}WGDAT1\Jobs\706880 GW Rail Ltd - Structural Assessment Ped Overbridge Kaiwharawhara\Working\Structural\Microstran Bridge Model\Kaiwharawhara Footbridge Model.msw

spiire

Spiire New Zealand Ltd
Job: Kaiwharawhara Footbridge Model
Kaiwharawhara Footbridge

12/07/2013
11:12:26 a.m.

Load Cases:
—— 16 C 1.0 Dead + 0.3 Live (SLS)



theta: 270 phi: 0

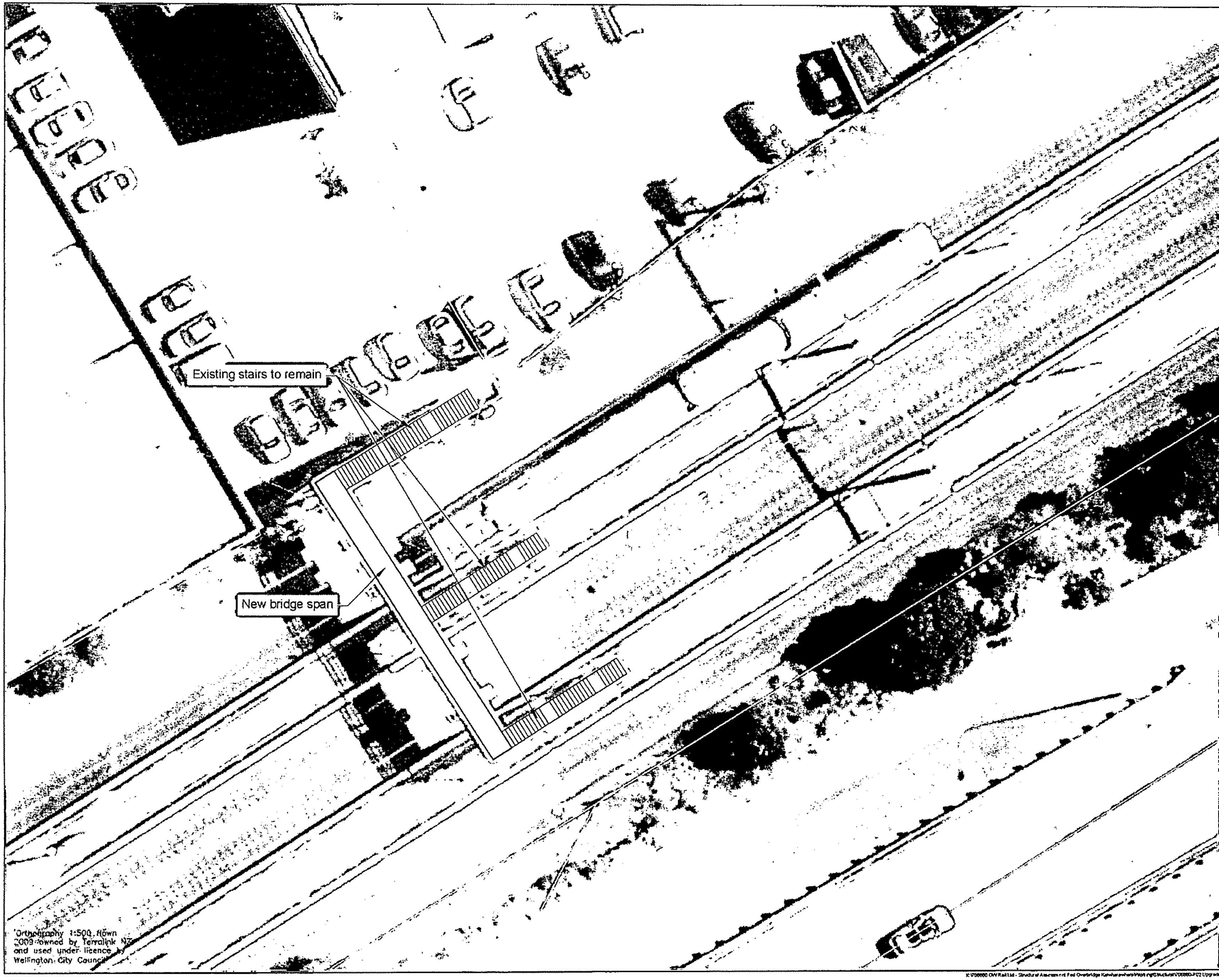
Main Beam Deflections, G + 0.3 Q (SLS)

Displaced Shape

Microstran V9.01.130412 {521501}WGDAT1\Jobs\706880 GW Rail Ltd - Structural Assessment Ped Overbridge Kaiwharawhara\Working\Structural\Microstran Bridge Model\Kaiwharawhara Footbridge Model.msw

Appendix 3.1

Concept Plan of Upgraded Bridge



Revision		App	Date
Surveyed			
Designed	RGB		07/13
Drawn	PCV		07/13
Reviewed	RGR		19/07/13
Approved	SAW		19/07/13

Client
Greater Wellington Rail Ltd

Project Title
KAIWHARAWHARA
RAILWAY STATION
PEDESTRIAN OVERBRIDGE

Sheet Title
Concept Plan of
Upgraded Bridge

spiire
Innovation in Infrastructure

Scale (A3 Original) 1:250

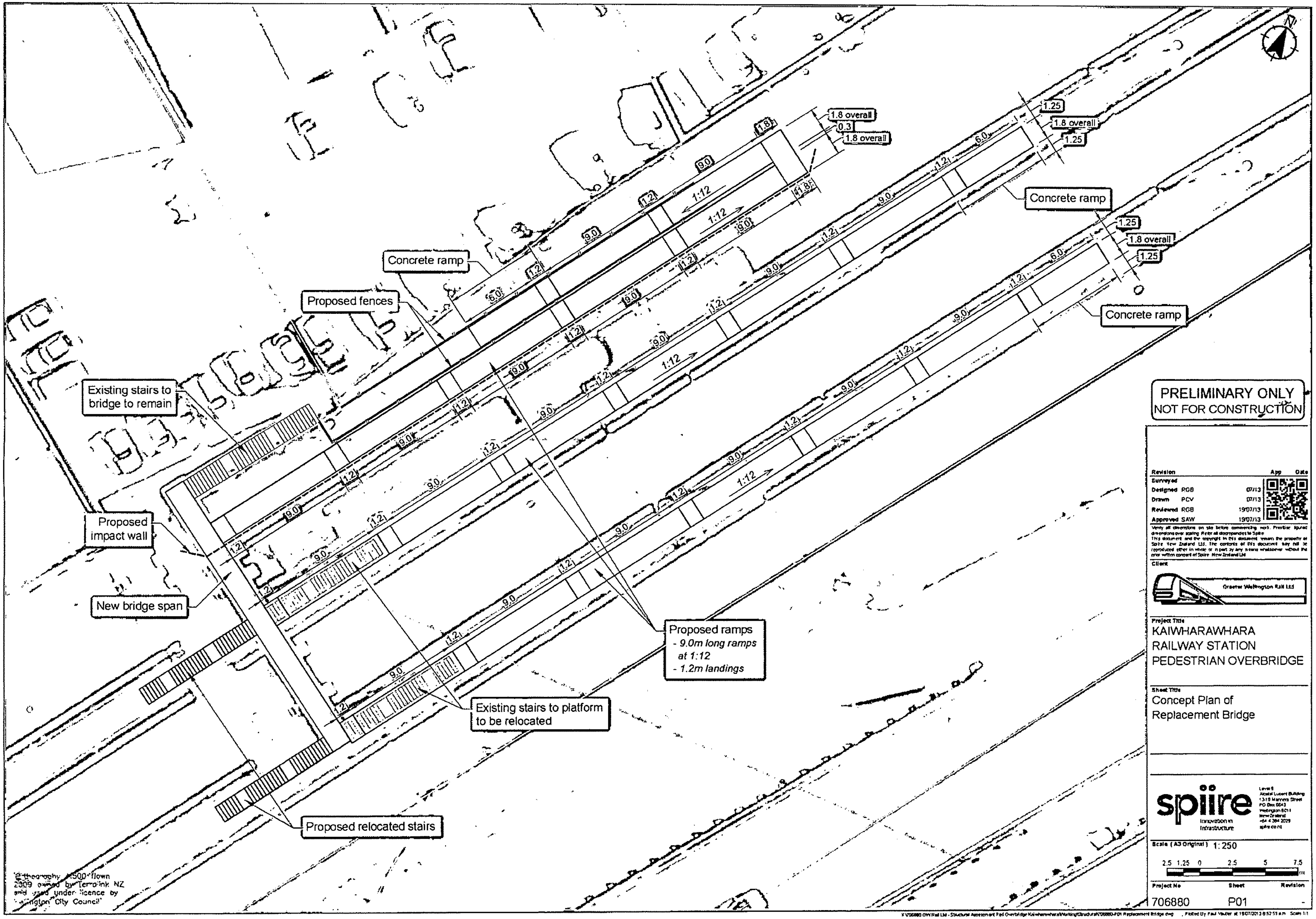
2.5 1.25 0 2.5 5 7.5 m

Project No	Sheet	Revision
706880	P02	-

Appendix 3.2

Concept Plan of Replacement Bridge

[redacted]



PRELIMINARY ONLY
NOT FOR CONSTRUCTION
Concrete ramp
Concrete ramp
Concrete ramp
Proposed fences
Existing stairs to bridge to remain
Proposed impact wall
New bridge span
Proposed ramps
- 9.0m long ramps at 1:12
- 1.2m landings
Existing stairs to platform to be relocated
Proposed relocated stairs
1.8 overall
0.3
1.8 overall
1.25
1.8 overall
1.25
1:12
Revision
Surveyed
Designed RGB 07/13
Drawn PCV 07/13
Reviewed RGB 19/07/13
Approved SAW 19/07/13
Client
Greater Wellington Rail Ltd
Project Title
KAIWHARAWHARA RAILWAY STATION PEDESTRIAN OVERBRIDGE
Sheet Title
Concept Plan of Replacement Bridge
spiire
Innovation in Infrastructure
Scale (A3 Original) 1:250
Project No 706880
Sheet P01
Revision -

Appendix 4.1

Bridge Upgrading – Rough Order Budget Cost Estimate

Kaiwharawhara Pedestrian Overbridge **15-Jul-13**

Job Number 706880

Bridge Upgrading - Rough Order Budget Cost Estimate

Main spans	$	36,500.00
Main span piers	$	45,600.00
Pier foundations	$	23,000.00
Crainage	$	10,000.00
Handrails	$	10,000.00
Fit existing stairs to new piers	$	10,000.00
Asphalt	$	5,000.00
Signage, markings	$	5,000.00
Demolition/Deconstruction	$	30,000.00
New lighting poles, etc. estimate	$	10,000.00
Alterations to traction overhead, estimate	$	20,000.00
KiwiRail, protection, permit, etc. estimate	$	20,000.00
Bridge hanger and protection, estimate	$	10,000.00
Consents	$	5,000.00
Margin 8%	$	19,208.00
Sub Total	$	259,308.00
Working in rail corridor 30%	$	77,792.40
Preliminary & General 12%	$	31,116.96
Sub Total	$	368,217.36
Contingency 20%	$	73,643.47
Physical Works Total	$	441,860.83
Professional Fees (Budget)	$	110,000.00
TOTAL BUDGET ESTIMATE	$	551,860.83
TOTAL BUDGET ESTIMATE (ROUNDED)		**$550,000.00**

Appendix 4.2

Bridge Replacement – Rough Order Budget Cost Estimate

Kaiwharawhara Pedestrian Overbridge **15-Jul-13**

Job Number 706880

Bridge Replacement - Rough Order Budget Cost Estimate

Bottom ramps	$	42,000.00
Ramp support piers	$	153,000.00
Ramp spans	$	335,800.00
Main spans	$	36,500.00
Main span piers	$	45,600.00
Pier foundations	$	23,000.00
Crainage	$	24,000.00
Ramp Handrails	$	92,000.00
Span handrails	$	10,000.00
Relocate stairs	$	64,000.00
Asphalt	$	20,000.00
Signage, markings	$	10,000.00
Fencing	$	8,000.00
Impact wall	$	51,000.00
Demolition/Deconstruction	$	30,000.00
New lighting poles, etc. estimate	$	30,000.00
Alterations to traction overhead, estimate	$	20,000.00
Relocate traction poles, estimate	$	60,000.00
Bridge hanger and protection, estimate	$	10,000.00
KiwiRail, protection, permit, etc. estimate	$	50,000.00
Consents	$	10,000.00
Margin 8%	$	89,992.00
Sub Total	$	1,214,892.00
Working in rail corridor 30%	$	364,467.60
Preliminary & General 12%	$	145,787.04
Sub Total	$	1,725,146.64
Contingency 20%	$	345,029.33
Physical Works Total	$	2,070,175.97
Professional Fees (Budget)	$	400,000.00
TOTAL BUDGET ESTIMATE	$	2,470,175.97
TOTAL BUDGET ESTIMATE (ROUNDED)	**$**	**2,470.000.00**