NZ TRANSPORT AGENCY
WAKA KOTAHI

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Old Mangere Numerical Capacity Review

January 2016

NZ TRANSPORT AGENCY
WAKA KOTAHI

Auckland Motorways

REPORT CONTROL SHEET

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Contents

1 Executive Summary

The results of the analysis have shown that the structure can carry the day to day (~200 people daily) traffic but the factor of safety are not ideal. The most venerable members are the edge girders and the girders most likely to get loaded during a crowd event. Under day to day load the girders have a factor of safety of 1.24 and under a crowd event 1.08. Both these factors of safety are not desirable and further action needs to implements to ensure the safety usage of the structure.

The results have shown that a ductile failure is more likely to occur opposed to a sudden shear failure which gives reassurance that there will be warning signs and a notice period if the structure starts to fail. Due to the unknown and erratic nature of a crowd event this needs to be managed for the foreseeable future. The management can entail a number of scenarios such as:

- Prevent access to the edge girders completely
- Have a lookout stationed on the bridge permanently to alert the authorities when a crowd is forming, so action can be taken to break up the gathering.
- Install a real time monitoring with stringent trigger levels to alert the authorities of a failure and have sufficient time to safely evacuate the structure.

The assessment has shown that the structure can accommodate 3kN/m2 (required minimum load) but this needs to be verified with a load test. This will give greater reassurance of the structure capacity and behaviours.

2 Introduction

2.1 Purpose of Report

The Old Mangere Bridge has been in a deteriorated condition for a number of years and has been restricted to pedestrian use since 1985. The bridge underwent some strengthening works in 2005-2011 to provide a capacity of 2.25KN/m2 and its condition has been subject to regular monitoring inspections by the AMA in its role as bridge consultant.

Due to a noticeable visual change in condition of Old Mangere bridge over the past number of years the management strategy for this structure is being reviewed. In December 2015 a full tactile principal inspection was carried out and has catalogued the main defects on the structure.

As at 19 January there is no numerical assessment/capacity based on the current condition factors and the current load carrying capacity is not reliably know (While a previous load assessment and strengthening works were carried out several years ago, the structure has recently deteriorated significantly)

The report will document the new numerical assessment being carried out.

3 Inspection for Assessment

3.1 Principal Inspection

A Special Principal Inspection was carried out on the structure on the 17th December 2015. The structure was inspected from topside and from a boat for the underside. The inspection report documents all the defects observed during the site visit. Reference should be made to the inspection report for all the condition related issues. The Principal Inspection will be used as the basis for the numerical assessment.

4 Assessment Methodology

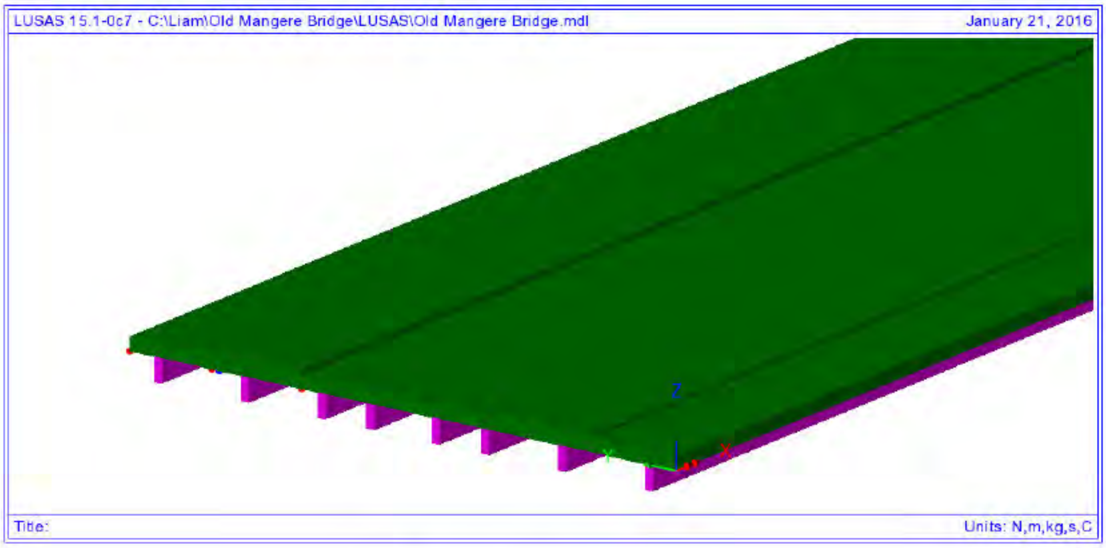
4.1 Modelling Assumptions

In order to take account of the severe conditional issues on the structure a number of conservative assumptions have been assumed:

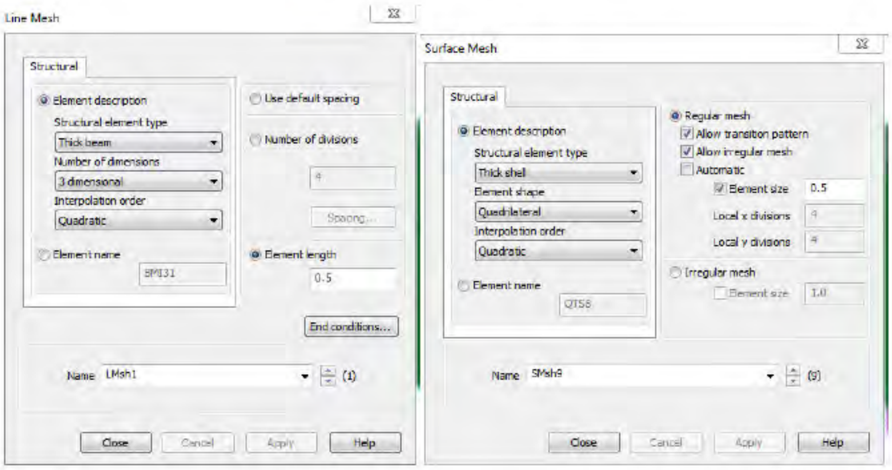
- The main RC beams are 1.219mmx305mm. For the analysis it will be assumed that the beams are 900mmx225mm. Due to extensive loss of section noted during the inspection
- When working out the capacity of the main beams the depth of the section will include the original slab thus the section will be 1200x225 (900mm+300mm slab)
- When calculating the shear capacity of the main beams some shear capacity from the top slab will be included. It is assumed all the shear links in the beam section are 100% deteriorated and will not contribute towards the shear capacity
- The beams have been modelled as simply supported.
- The retrofitted external post-tensioned (PT) system is being ignored initially. If the structure fails on this basis the PT system benefits will be evaluated.
- A weak young's Modulus (E) will be assumed with an E value of $14x10^9$N/m2 being adopted
- 40N concrete will be used for the 2005 strengthened concrete strips.
- A 50mm joint over the supports has been created in the shells representing the original slab joints. This ensures there is not continuous action in the original slab. This is a conservative assumption.
- The retrofitted 2005 RC slabs have been modelled as continuous over the supports
- Only 4 spans have been modelled as representative for the entire structure. To model all 17 spans would be excessive and time consuming.
- 380mm displacement/settlement based on the cloud survey will be applied to the structure.

4.2 Structural Modelling and Verification

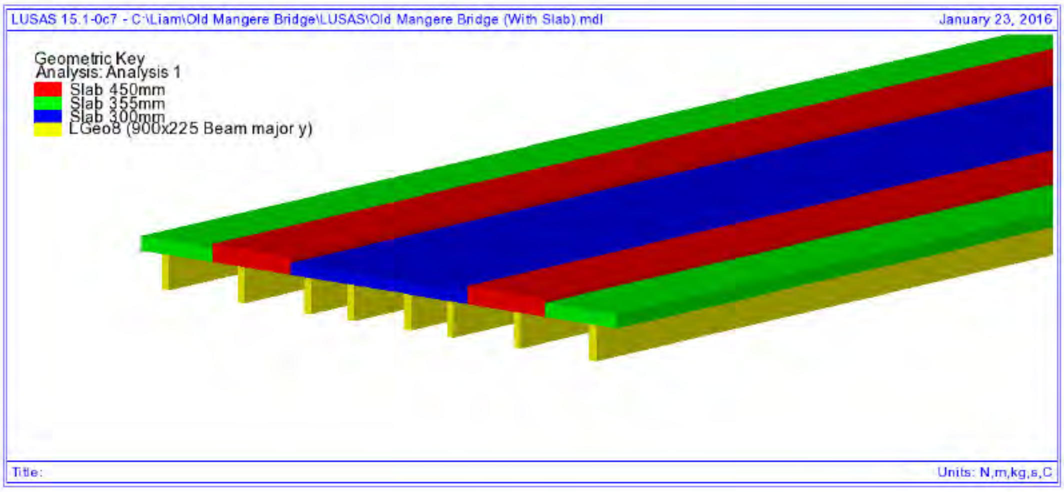
The assessment of Old Mangere was carried out using LUSAS Bridge Pro Version 15.1. Two models were created. The first modelled the structure before the 2005 RC slab and the second model will look at the behavior of the structure after the 2005 slab. A fleshed out view of the 2005 slab model is illustrated in tile 1 below. The main original top slab and the strengthened 2005 Reinforced Concrete (RC) slab were modelled as 3D thick quadratic shell elements. The RC beams underneath were modelled ad 3D thick beam line elements.



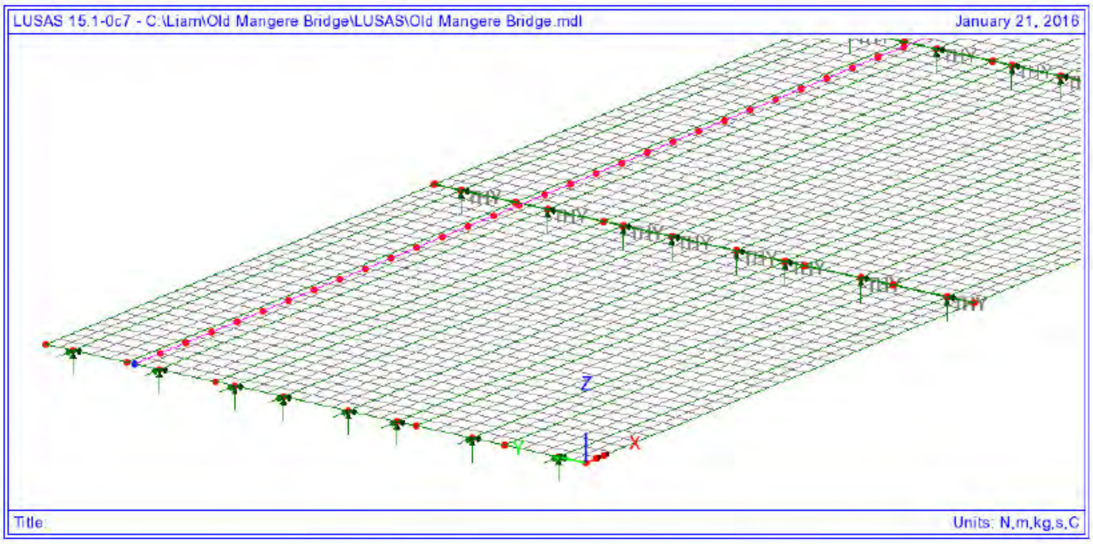
Tile 1 – LUSAS Model "Fleshed"



Tile 2 – View showing the mesh properties

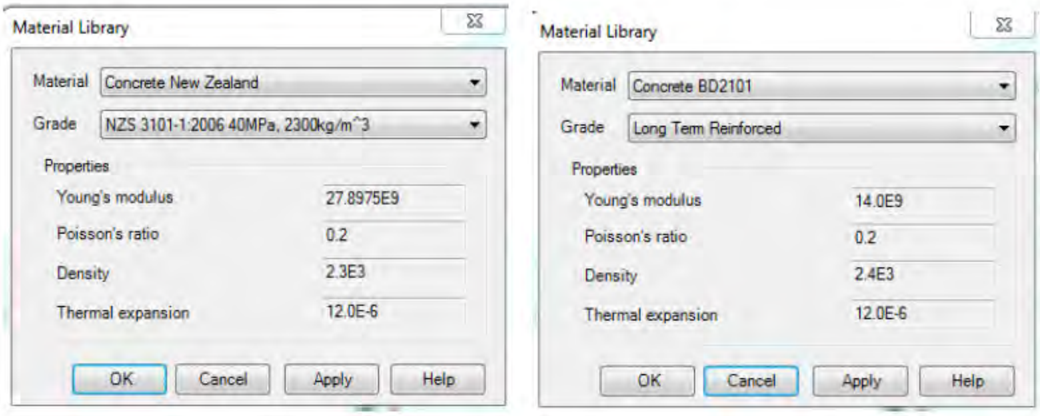


Tile 3 – View showing Geometric Properties

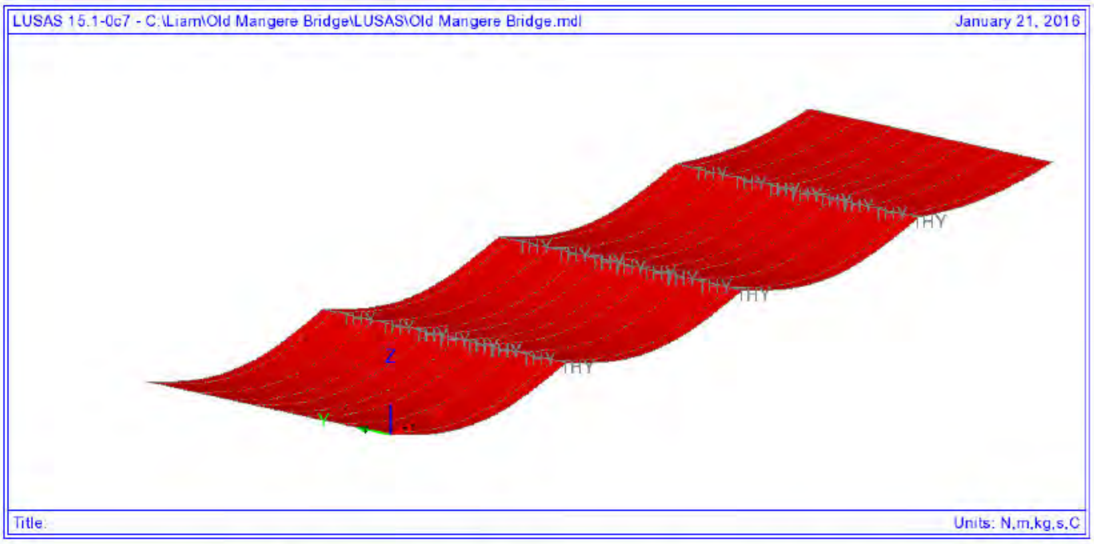


Tile 4 – View showing Support Conditions.

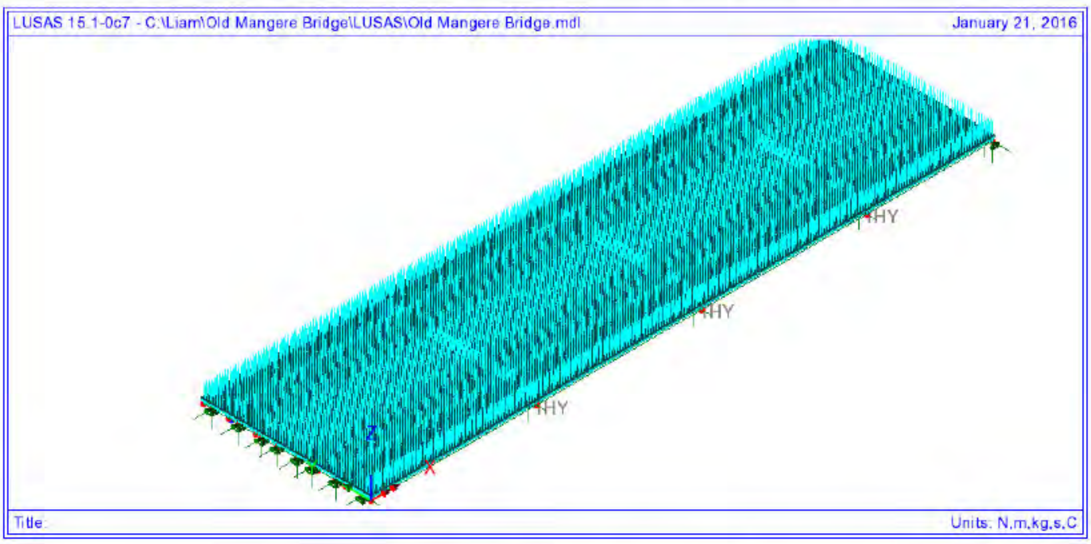
In order to ensure that the main RC beams act as simply supports end releases were introduced.



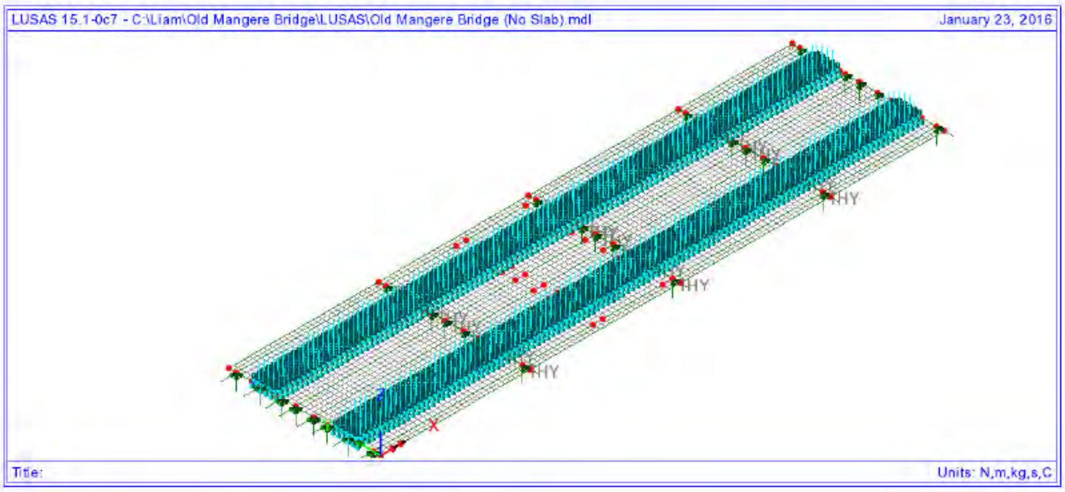
Tile 5 – View showing Material Proterties



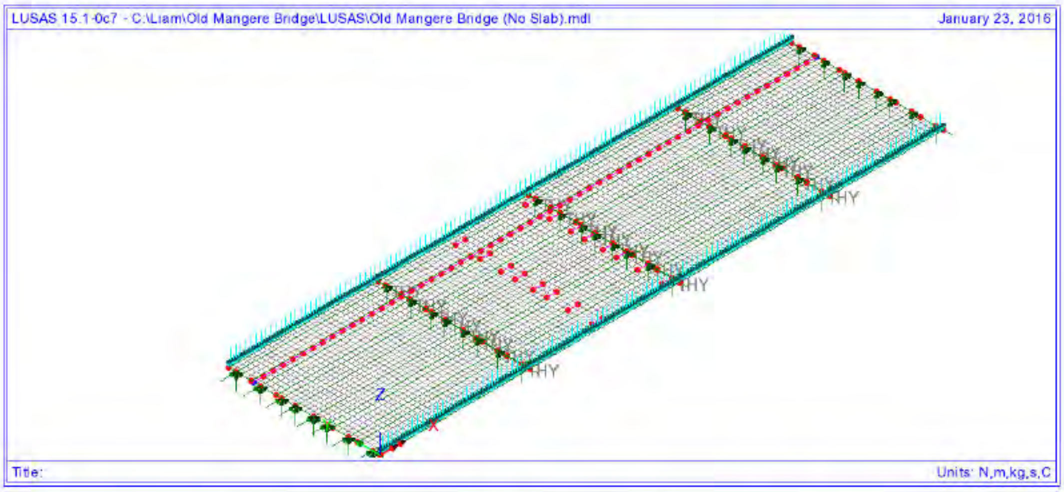
Tile 5 – View showing deformed mesh under Self Weight



Tile 5 – View showing application of self weight



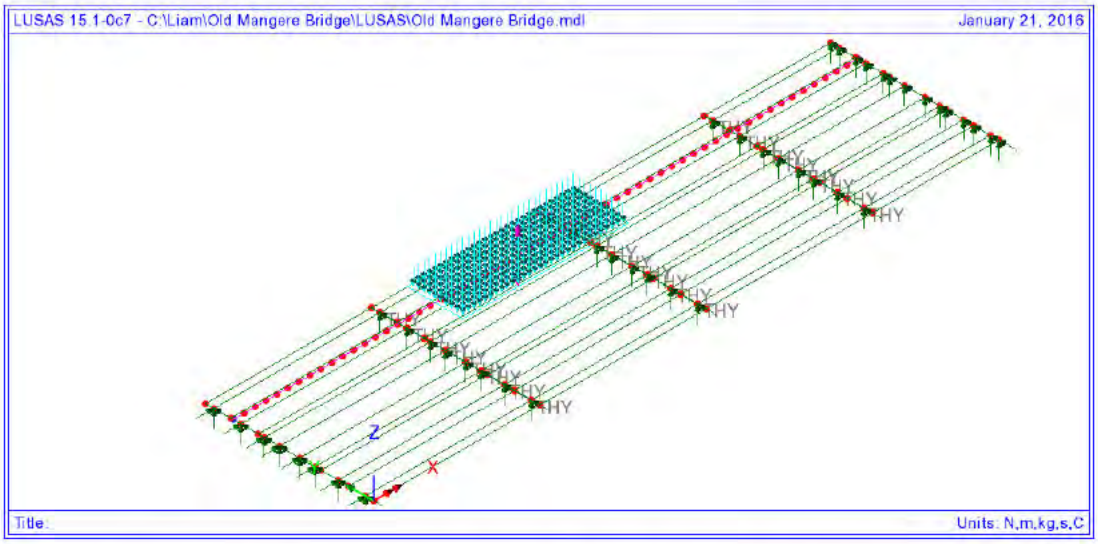
Tile 5 – View showing application of wet concrete from 2005 RC slabs



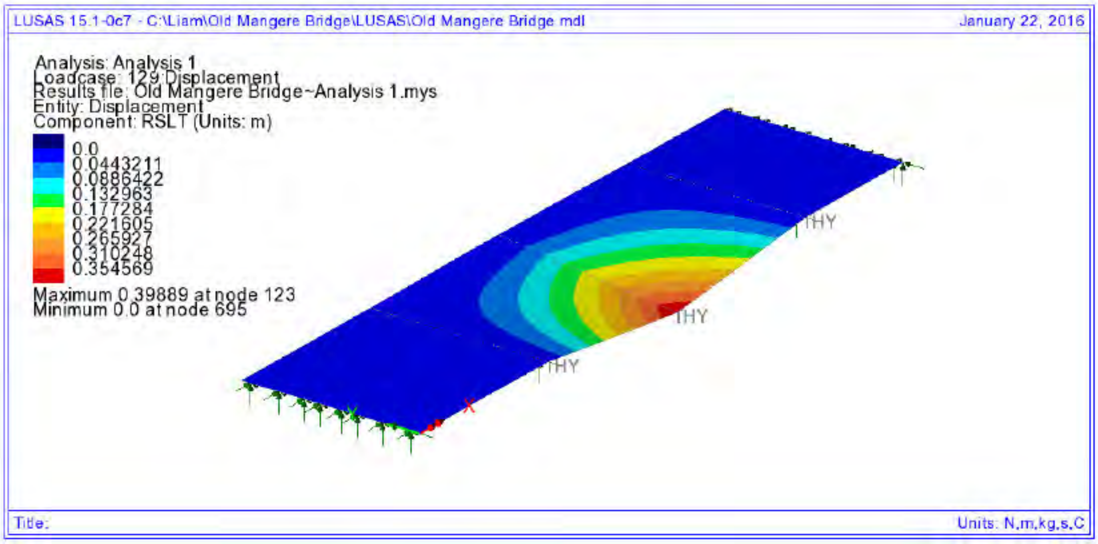
Tile 5 – View showing application of Parapet loads

For the application of live loading 2no. scenarios will be looked at:

1. 3kN/m2 applied as a UDL across the entire deck.
2. 3kN/m2 applied as a 15mx5m patch along the edge of the structure. This load is envisioned to represent a crowd gathering at the edge to observe something in the water. This patch load will be moved across the structure as a moving load at 1 m intervals to see how the loading is redistributed around the structure.



Tile 5 – View showing application of patch live load. It will be applied as a moving load.



Tile 5 – View showing application of 380mm settlement

5 Assessment Results

The results for the main beams are displayed in the table below. The most critical beams are the edge girders (MGE1) with a factor of safety of 1.1 in bending and 1.08 in shear. If the crowd loading is removed the factor of safety increase to 1.24 for bending and 2.67 for shear. When calculating the capacity no factors of safety were applied to the loads or the materials so the factors below are a pure global factors.

The loads from the settlement/displacement of the structure has been ignored as they were giving relieving effects.

The capacity of the sections are based on the British Standard BD44/95 the assessment of concrete structures. The loads applied were in accordance with NZTA Bridges Manual. As can be seen in the table below that a 1.1 gamma factor has been applied to take account of modelling discrepancies and additional self-weight such as concrete repairs and plating etc.

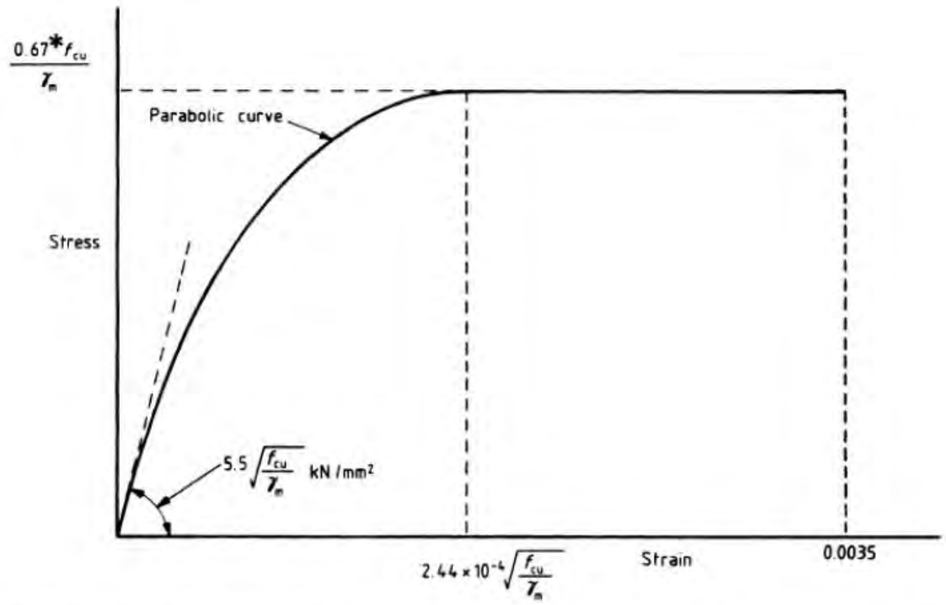
(BCMI reference)					R_A^*	S_D^*	$S_{D\text{-Wet Conc}}^*$	$S_{D\text{-Displacement}}$	$S_{Parapet}$	S_{3kPa}	$S_{3kPa\text{-Patch}}$	$RF = \frac{R_A^*}{S_{LL}^*}$	
Major Element		Minor Element											
Code	No.	Code	No.	Load Effect	Assessment. Resistance	Dead Load Effects	SDL (Wet Conc)	Displacement	Parapet SW	3kPa Pedestrian	3kPa Patch (LC88)	Structure Reserve Factor	Structure Reserve Factors with addition 1.1 factored applied
DK	6	MGE	1	Bending moment (Original)	363	-209.78	-49.11	18.94	-6.58	-20.54	-31.64	1.22	1.11
DK	6	MGE	1	Shear force (Original)	197	-55.48	-6.87	2.30	-4.60	-89.64	-63.14	1.18	1.08
DK	6	MGI	1	Bending moment (Original)	363	-174.30	-48.03	3.78	-3.25			1.62	1.48
DK	6	MGI	1	Shear force (Original)	197	-94.62	-35.20	48.80	-38.13			1.17	1.07
DK	6	MGI	1	Bending moment (RC Slab)	138					-32.04	-38.92	3.74	3.40
DK	6	MGI	1	Shear force (RC Slab)	330					-102.50	-44.81	3.22	2.93
DK	6	MGI	2	Bending moment (Original)	363	-167.98	-42.78	1.85	-1.17	-22.76	-13.47	1.55	1.41
DK	6	MGI	2	Shear force (Original)	197	-83.87	-22.70	29.13	0.00	-5.10	-8.28	1.72	1.58
DK	6	MGI	3	Bending moment (Original)	363	-165.27	-39.75	0.99	-0.57	-20.93	-17.68	1.60	1.46
DK	6	MGI	3	Shear force (Original)	197	-78.78	-13.80	11.60	0.00	-15.13	-13.90	1.83	1.66

Notes:
Displacement/settlement has been ignored as it generaly was giving relieving effects
No factors of safety were applied to the loads or materials so the values above are total factors
A 1.1 Gamma f3 factor should be considered for unknowns

Table 1 – Results table for main beams

6 Potential Results from Load Testing

This section will include a number of screen shots of the structure to illustrate what type of loading and deflections can be expected during a possible load test. The values will be shown in Von Mises Stress contours. Based on a typical stress strain curve the maximum compressive strength for a 14N cubic concrete would be 7.8N/mm2, tensile stress should generally be limited to 2 N/mm2.



*0.67 takes account of the ratio between the characteristic cube strength and the bending strength in a flexural member.

NOTE 1. f_{cu} is in N/mm².

NOTE 2. The equation for the parabolic curve between $\epsilon = 0$ and $2.44 \times 10^{-4}\sqrt{f_{cu}/\gamma_m}$ may be taken as:

$$f = \left(5500\sqrt{\frac{f_{cu}}{\gamma_m}}\right)\epsilon - \left(\frac{5500^2}{2.68}\right)\epsilon^2$$

where

f is the stress;

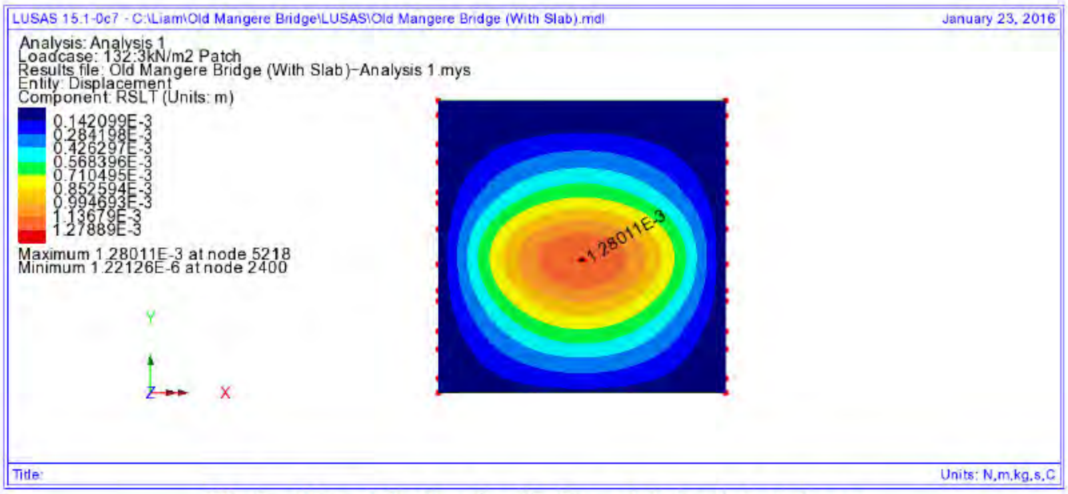
ϵ is the strain.

Figure 1 — Short term design stress-strain curve for normal weight concrete

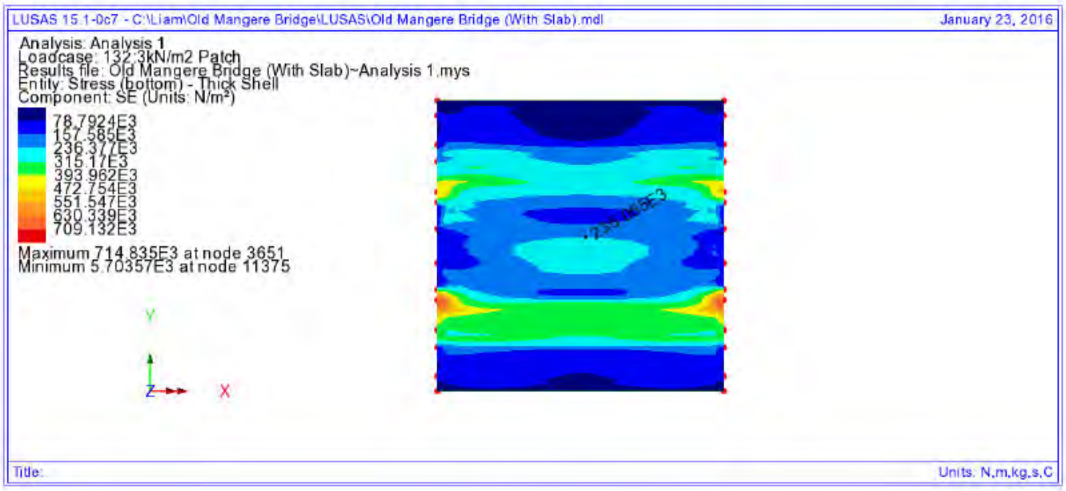
Tile 5 – View showing typical stress/strain curve

One possible load test would be to apply a 3kN/m2 patch load around the structure to represent a crowd load. In the model a 15m x 5m patch load was used. If the patch load is applied at mid span deflections of approximately 1.2mm would be expected. The maximum tensile stress on the bottom of the slab would be 0.2N/mm2 on the original structure and stress close to 0.7N/mm2 would be generated in the new RC slab. These values would be within allows strain/strain limits.

The modelling has shown that the 2005 RC slab attracts a large portion of the live loads and any subsequent loading since 2005. This is to be expected as the new RC slabs act as spines along the bridge stiffening it up and will naturally attract more load. As the structure deteriorates these slabs will play a more significant role in the capacity.



Tile 5 – Max deflection for 15x5m patch load at mid span



Tile 5 – View showing the stress under a 15x5m patch load at mid span

7 Conclusion

The results from the analysis of the deck have shown that the edge girders are the most venerable with a factor of safety of 1.08 in shear and 1.1 in bending. If the crowd load is removed from the structure these factors go to 2.67 for shear and 1.24 for bending.

The next most venerable beam is intermediate girder 2 (MGI2) with a bending capacity of 1.41 for bending and 1.56 in shear. If the crowd loading is removed these factors of safety increase to 1.56 for bending and 1.68 for shear.

The results have shown that the 2005 retrofitted RC slabs give significant capacity and attract a large portion of the load. The two slabs act as spines and it is to be expected that they will attach live load as it is applied. The slabs are key structural members that are vital to the capacity of the structure for loading applied after 2005. The results have shown that the slabs have a factor of safety of 2.93 which is deem sufficient.

In an ideal scenario factors of safety of 1.5 and above would be achieved to have some reassurance in the capacity. As the results have shown the edge girders do not achieve this in a crowd event. When the crowd loading is removed from the edge girders the factor of safety for bending only achieves 1.24 but shear increases to 2.67. This gives some reassurance that a sudden failure is unlikely for the structure under its own self weight and any failure will be ductile and progressive in nature.

The structure will inherently have more capacity than the results shown as conservative assumptions have been made to achieve these results. This inherent strength cannot be verified due to the significant deterioration of the structure, but equally will give some reassurance that there is capacity above the values shown. The capacities shown are for the structure at this point in time. As the structure deteriorates and is still subjected to load this will effect and reduce the structural behaviour of the bridge. This needs to be considers in the recommendations and strategy going forward.

8 Recommendations

The following recommendations need to be implemented going forward to ensure the safe usage of the bridge.

- The structure needs to be load tested to verify the results of this assessment. This will give further reassurance that the structure can carry the specified loading. The modelling has shown that the structure should be able to support a Universal Dead Load of 3kN/m2.

- The results have shown that the edge girders can supports a 3kN/m2 load but the factors of safety are not ideal and are below 1.2. Due to the deterioration of the structure and the potential for unknowns this is not deemed a safe factor of safety. The day to day load of people fishing is not a concern as it will have little to no impact on the capacity. The numbers have shown that sudden failure is unlikely under self-weight and any failure will be ductile.

- Measures need to installed that prevent crowd loading on the edge girder, or a method to safely monitor a potential failure with sufficient notice period to alert the authorities and evacuate the structure. This could be achieved with a real time monitoring system with stringent trigger levels and a robust contingency/management strategy when the alarms are triggered.

- The results of the analysis have shown that the centre of the deck has sufficient capacity and factors of safety. The reality is any failure in this area will be redistributed around the deck. Any potential failure will be in bending which will be ductile in behaviour and not sudden. But due to the ongoing deterioration of the structure this area should also have real time monitoring installed to ensure the future deterioration can be tracked and measures implement to ensure the safe usage of the structure

- The structure is to remain on 3 monthly tactile inspection from both top and under side. The deterioration is to be monitored and recorded in inspection reports. If the deterioration of the structure accelerates the frequency of the inspection is to be reviewed.

APPENDIX A –

Calculations

(BCMI reference)					R_A*	S_D*	$S_{D\ Wet\ Conc}$*	$S_{D\ Displacement*}$	$S_{Parapet*}$	S_{3kPa*}	$S_{3kPa\ Patch*}$	$RF = \frac{R_A*}{S_{All}*}$	
Major Element		Minor Element											
Code	No.	Code	No.	Load Effect	Assessment. Resistance	Dead Load Effects	SDL (Wet Conc)	Displacement	Parapet SW	3kPa Pedestrian	3kPa Patch (LC88)	Structure Reserve Factor	Structure Reserve Factors with addition 1.1 factored applied
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DK	6	MGI	3	Shear force (Original)*	197	-78.78	-13.80	11.60	0.00	-15.13	-13.90	1.83	1.66

Notes:
* Shear capacity from original slab included
Displacement/settlement has been ignored as it generaly was giving relieving effects
No factors of safety were applied to the loads or materials so the values above are total factors
A 1.1 Gamma f3 factor should be considered for unknowns

Calculations

NZ TRANSPORT AGENCY WAKA KOTAHI — Auckland Motorways

Project			Part of structure/scheme and status			Job ref
Old Mangere			Original Slab			
Office of Issue	**Telephone No**	**Division**	**Calculations by**	**Checked by**	**Date**	**Calc Sheet no**
AMA			[redacted] s9(2)(a)	BECA	23/01/2016	of

Code ref	Calculations	Remarks/ output	Checked by Inits & date
BD44/95 cl. 5.3.2	**Moment of Resistance of a Singly-Reinforced Slab to BD44/95**		
	Inputs		
	Width of slab — **b** — 1500 mm		
	Total depth of beam — **D** — 300 mm		
	Cover to tension bars — **c** — 30 mm (Assumed)		
	Reinforcement diameter — **Ø** — 10 mm (Assumed)		
	Reinforcement spacing — **s** — 300 mm (Assumed)		
	Steel yield stress — f_y — 210 N/mm^2		
	Concrete cube strength — f_{cu} — 14 N/mm^2		
	Partial safety factor for concrete — γ_{mc} — 1		
	Partial safety factor for steel — γ_{ms} — 1		
	~~i.e. a 300mm deep slab with 10mm bars at 300mm spacing and 30mm cover.~~		
	~~Calculation~~		
	~~Effective depth — d = D - c - (Ø / 2)~~		
	~~= 300 - 30 - (10 / 2)~~		
	~~= 265 mm~~		
	~~Number of bars, — n = b / s~~		
	~~= 1500 / 300~~		
	~~= 5.000~~		
	~~Area of reinforcement — A_s = n x π x (Ø / 2)^2~~		
	~~= 5 x π x (10 / 2) ^ 2~~		
	~~= 392.7 mm^2~~		
~~Equation 5~~	~~Lever arm, z — (1 - (0.84 x (f_y / g_ms) x As) / ((f_cu / g_mc) x b x d)) x d~~		
	~~= (1 - (0.84 x (210 / 1) x 392.7) / ((14 / 1) x 1500 x 265)) x 265~~		
	~~= 261.7 mm~~		
	~~But not greater than 0.95d~~		
	~~= 251.75 mm~~		
	~~Therefore, — z = 251.8 mm~~		
~~5.3.2.3~~	~~Moment of resistance is given by the lower of equations 1 and 2:~~		
~~Equation 1~~	~~M_u = (f_y / g_{ms}) x A_s x z~~		
	~~= (210 / 1) x 392.7 x 251.75 / 10^6~~		
	~~= 20.76 kNm~~		
~~Equation 2~~	~~M_u = (0.225 x f_{cu} / g_{mc}) x b x d^2~~		
	~~= (0.225 x 14 / 1) x 1500 x 265 ^ 2 / 10^6~~		
	~~= 331.81 kNm~~		
	~~Therefore, the ultimate moment of resistance of the section is 20.76 kNm~~		

Calculations

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AMA			s9(2)(a)	BECA	23/01/2016	of

Code ref	Calculations				Remarks/ output	Checked by Inits & date
BD44/95						
cl. 5.3.3	**Shear Resistance of a Slab to BD44/95**					
	Inputs					
	Width of beam	**b**	1500 mm	(from bending calcs)		
	Effective depth of section	**d**	265 mm	(from bending calcs)		
	Area of bending reinforcement	A_s	392.7 mm^2	(from bending calcs)		
	Concrete cube strength	f_{cu}	14 N/mm^2	(from bending calcs)		
	Partial safety factor for concrete	γ_{mc}	1			
	Partial safety factor for steel	γ_{ms}	1			
	Distance from support	a_v	300 mm			
	Tension reinforcement anchored?	**Y/N**	N	(for shear enhancement)		
	Calculation					
5.3.3.2	In no case should v exceed 0.92 $\sqrt{(fcu / \gamma_{mc})}$ or 7 / $\sqrt{\gamma_{mc}}$ N/mm^2, whichever is lesser.					
	0.92 $\sqrt{(fcu / \gamma_{mc})}$	= 0.92 x (14 / 1) ^ 0.5 =	3.44 N/mm^2			
	7 / $\sqrt{\gamma_{mc}}$	= 7 / (1) ^ 0.5 =	7 N/mm^2			
	Therefore, limit on v	v_{lim} =	3.44 N/mm^2			
Table 9	Depth factor	$\xi_s = (500 / d)^{0.25}$		(modified by IAN 4/96)		
		= (500 / 265) ^ 0.25				
		=	1.172			
	Depth factor cannot be less than 0.7					
	Therefore,	ξ_s =	1.172			
Table 8	Ultimate shear stress in concrete,	$v_c = \frac{0.24}{\gamma_{mv}}\left(\frac{100A_s}{b_w d}\right)^{1/3}(f_{cu})^{1/3}$				
		where γ_{mv} = 1.25 , f_{cu} is not greater than 40 and				
		$\left(\frac{100A_s}{b_w d}\right)$	should not be taken less than 0.15 or greater than 3.0.			
	$\left(\frac{100A_s}{b_w d}\right)$	= ((100 x 392.7) / (1500 x 265)) =	0.099			
		> 0.15, < 3.0, therefore, use	0.15			
	Therefore,	v_c = (0.24 / 1.25) x 0.15 ^ (1/3) x 14 ^ (1/3)				
		=	0.246 N/mm^2			
	Therefore,	$\xi_s \times v_c$ =	1.172 x 0.246			
		=	0.288 N/mm^2			

Calculations

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AMA			[redacted]	BECA	23/01/2016	of

Code ref	Calculations			Remarks/ output	Checked by Inits & date
cl. 5.3.3.3	Section close enough to support for enhanced shear strength?		Yes		
	$3d/a_v =$	$3 \times 265 / 300 =$	2.65		
	$\xi_s v_c \times 3d/a_v =$	$0.288 \times 2.65 =$	0.763 N/mm^2		
		But is limited to $v_{lim} =$	3.44 N/mm^2		
	Therefore the shear strength $\xi_s v_c$ of the concrete =		0.288 N/mm^2		
	The ultimate shear resistance of the section is given by				
	$V_u = \xi_s v_c b_w d$				
	$\xi_s v_c \times b_w \times d =$	$0.288 \times 1500 \times 265 =$	114.5 kN		
	Therefore the shear capacity of the section is =		**114.5 kN**		

Calculations

NZ TRANSPORT AGENCY WAKA KOTAHI — Auckland Motorways

Project			Part of structure/scheme and status			Job ref
Old Mangere			New Slab			
Office of Issue	**Telephone No**	**Division**	**Calculations by**	**Checked by**	**Date**	**Calc Sheet no**
AMA			[redacted]	BECA	23/01/2016	of

Code ref	Calculations	Remarks/ output	Checked by Inits & date
BD44/95			
cl. 5.3.2	**Moment of Resistance of a Singly-Reinforced Slab to BD44/95**		
	Inputs		
	Width of slab — **b** 2500 mm		
	Total depth of beam — **D** 225 mm		
	Cover to tension bars — **c** 40 mm		
	Reinforcement diameter — **Ø** 25 mm		
	Reinforcement spacing — **s** 400 mm		
	Steel yield stress — f_y 275 N/mm^2		
	Concrete cube strength — f_{cu} 40 N/mm^2		
	Partial safety factor for concrete — γ_{mc} 1		
	Partial safety factor for steel — γ_{ms} 1		
	i.e. a 225mm deep slab with 25mm bars at 400mm spacing and 40mm cover.		
	Calculation		
	Effective depth $\mathbf{d} = D - c - (Ø / 2)$		
	$= 225 - 40 - (25 / 2)$		
	$=$ 172.5 mm		
	Number of bars, $\mathbf{n} = b / s$		
	$= 2500 / 400$		
	$=$ 6.250		
	Area of reinforcement $\mathbf{A_s} = n \times \pi \times (Ø / 2)^2$		
	$= 6.25 \times \pi \times (25 / 2)$ ^ 2		
	$=$ 3068.0 mm^2		
Equation 5	Lever arm, **z** $= (1- (0.84 \times (f_y / \gamma_{ms}) \times As) / ((f_{cu} / \gamma_{mc}) \times b \times d)) \times d$		
	$= (1- (0.84 \times (275 / 1) \times 3068) / ((40 / 1) \times 2500 \times 172.5)) \times 172.5$		
	$=$ 165.4 mm		
	But not greater than 0.95d		
	$=$ 163.875 mm		
	Therefore, **z** $=$ 163.9 mm		
5.3.2.3	Moment of resistance is given by the lower of equations 1 and 2:		
Equation 1	$\mathbf{M_u} = (f_y / \gamma_{ms}) \times A_s \times z$		
	$= (275 / 1) \times 3068 \times 163.875 / 10$^6		
	$=$ 138.26 kNm		
Equation 2	$\mathbf{M_u} = (0.225 \times f_{cu} / \gamma_{mc}) \times b \times d^2$		
	$= (0.225 \times 40 / 1) \times 2500 \times 172.5$ ^ 2 / 10^6		
	$=$ 669.52 kNm		
	Therefore, the ultimate moment of resistance of the section is 138.26 kNm		

Calculations

NZ TRANSPORT AGENCY WAKA KOTAHI — Auckland Motorways

Project			Part of structure/scheme and status			Job ref
Old Mangere			New Slab			
Office of Issue	Telephone No	Division	Calculations by	Checked by	Date	Calc Sheet no
AMA			s9(2)(a) [redacted]	BECA	23/01/2016	of

Code ref	Calculations	Remarks/ output	Checked by Inits & date
BD44/95			
cl. 5.3.3	**Shear Resistance of a Slab to BD44/95**		
	Inputs		
	Width of beam — **b** — 2500 mm (from bending calcs)		
	Effective depth of section — **d** — 172.5 mm (from bending calcs)		
	Area of bending reinforcement — A_s — 3068.0 mm^2 (from bending calcs)		
	Concrete cube strength — f_{cu} — 40 N/mm^2 (from bending calcs)		
	Partial safety factor for concrete — γ_{mc} — 1		
	Partial safety factor for steel — γ_{ms} — 1		
	Distance from support — a_v — 300 mm		
	Tension reinforcement anchored? — **Y/N** — N (for shear enhancement)		
	Calculation		
5.3.3.2	In no case should v exceed 0.92 $\sqrt{(fcu / \gamma_{mc})}$ or 7 / $\sqrt{\gamma_{mc}}$ N/mm^2, whichever is lesser.		
	0.92 $\sqrt{(fcu / \gamma_{mc})}$ = 0.92 x (40 / 1) ^ 0.5 = 5.82 N/mm^2		
	7 / $\sqrt{\gamma_{mc}}$ = 7 / (1) ^ 0.5 = 7 N/mm^2		
	Therefore, limit on v — v_{lim} = 5.82 N/mm^2		
Table 9	Depth factor — $\xi_s = (500 / d)^{0.25}$ (modified by IAN 4/96)		
	= (500 / 172.5) ^ 0.25		
	= 1.305		
	Depth factor cannot be less than 0.7		
	Therefore, ξ_s = 1.305		
Table 8	Ultimate shear stress in concrete, $v_c = \frac{0.24}{\gamma_{mv}}\left(\frac{100A_s}{b_w d}\right)^{1/3}(f_{cu})^{1/3}$		
	where γ_{mv} = 1.25 , f_{cu} is not greater than 40 and		
	$\left(\frac{100A_s}{b_w d}\right)$ should not be taken less than 0.15 or greater than 3.0.		
	$\left(\frac{100A_s}{b_w d}\right)$ = ((100 x 3068) / (2500 x 172.5)) = 0.711		
	> 0.15, < 3.0, therefore, use 0.711		
	Therefore, v_c = (0.24 / 1.25) x 0.711 ^ (1/3) x 40 ^ (1/3)		
	= 0.586 N/mm^2		
	Therefore, $\xi_s \times v_c$ = 1.305 x 0.586		
	= 0.765 N/mm^2		

Calculations

NZ TRANSPORT AGENCY — Auckland Motorways

Project			Part of structure/scheme and status			Job ref
Old Mangere			New Slab			
Office of Issue	Telephone No	Division	Calculations by	Checked by	Date	Calc Sheet no
AMA			[redacted]	BECA	23/01/2016	of

Code ref	Calculations	Remarks/ output	Checked by Inits & date
cl. 5.3.3.3	Section close enough to support for enhanced shear strength? Yes		
	$3d/a_v =$ 3 x 172.5 / 300 = 1.725		
	$\xi_s v_c \times 3d/a_v =$ 0.765 x 1.725 = 1.32 N/mm^2		
	But is limited to $v_{lim} =$ 5.82 N/mm^2		
	Therefore the shear strength $\xi_s v_c$ of the concrete = 0.765 N/mm^2		
	The ultimate shear resistance of the section is given by		
	$V_u = \xi_s v_c b_w d$		
	$\xi_s v_c \times b_w \times d =$ 0.765 x 2500 x 172.5 = 329.9 kN		
	Therefore the shear capacity of the section is = 329.9 kN		

Calculations

NZ TRANSPORT AGENCY WAKA KOTAHI — Auckland Motorways

Project			Part of structure/scheme and status			Job ref
Old Mangere Bridge			Main Girders			
Office of Issue	Telephone No	Division	Calculations by	Checked by	Date	Calc Sheet no
AMA			[redacted]	BECA	23/01/2016	of

Code ref	Calculations	Remarks/ output	Checked by Inits & date
BD44/95			
cl. 5.3.2	**Moment of Resistance of a Singly-Reinforced Beam to BD44/95**		
	Inputs		
	Width of beam — **b** — 225 mm		
	Total depth of beam — **D** — 1200 mm (Includes top slab)		
	Cover to tension bars — **c** — 0 mm (including shear link diameter)		
	Reinforcement diameter — **Ø** — 20 mm (Originally bars were 32mm)		
	Number of bars — **n** — 5 (Assume 1 bars complete lost)		
	Steel yield stress — f_y — 210 N/mm^2		
	Concrete cube strength — f_{cu} — 14 N/mm^2		
	Partial safety factor for concrete — γ_{mc} — 1		
	Partial safety factor for steel — γ_{ms} — 1		
	i.e. a 1200mm deep x 225mm wide beam with 5No 20mm bars at 0mm cover.		
	Calculation		
	Effective depth — **d** = D - c - (Ø / 2)		
	= 1200 - 0 - (20 / 2)		
	= 1190 mm		
	Area of reinforcement — A_s = n x π x (Ø / 2)^2		
	= 5 x π x (20 / 2) ^ 2		
	= 1570.8 mm^2		
Equation 5	Lever arm, **z** = (1- (0.84 x (f_y / γ_{ms}) x As) / ((f_{cu} / γ_{mc}) x b x d)) x d		
	= (1- (0.84 x (210 / 1) x 1570.8) / ((14 / 1) x 225 x 1190)) x 1190		
	= 1102.0 mm		
	But not greater than 0.95d		
	= 1130.5 mm		
	Therefore, **z** = 1102.0 mm		
5.3.2.3	Moment of resistance is given by the lower of equations 1 and 2:		
Equation 1	M_u = (f_y / γ_{ms}) x A_s x z		
	= (210 / 1) x 1570.8 x 1102 / 10^6		
	= 363.51 kNm		
Equation 2	M_u = (0.225 x f_{cu} / γ_{mc}) x b x d^2		
	= (0.225 x 14 / 1) x 225 x 1190 ^ 2 / 10^6		
	= 1003.66 kNm		
	Therefore, the ultimate moment of resistance of the section is 363.51 kNm		

Calculations

NZ TRANSPORT AGENCY WAKA KOTAHI — Auckland Motorways

Project			Part of structure/scheme and status			Job ref
Old Mangere Bridge			Main Girders			
Office of Issue	Telephone No	Division	Calculations by	Checked by	Date	Calc Sheet no
AMA			[redacted]	BECA	23/01/2016	of

Code ref	Calculations	Remarks/ output	Checked by Inits & date
BD44/95			
cl. 5.3.3	**Shear Resistance of a Beam to BD44/95**		

Inputs

Width of beam	b	225 mm	(from bending calcs)
Effective depth of section	d	1190 mm	(from bending calcs)
Area of bending reinforcement	A_s	1570.8 mm^2	(from bending calcs)
Shear link diameter	Ø	0 mm	
Number of legs of links	n	0	
Shear link spacing	s_v	0 mm	
Shear reinforcement yield stress	f_{yv}	210 N/mm^2	
Concrete cube strength	f_{cu}	14 N/mm^2	(from bending calcs)
Partial safety factor for concrete	γ_{mc}	1	
Partial safety factor for steel	γ_{ms}	1	
Applied shear force	V	181.9 kN	
Coexistent Moment	M	0 kNm	
Distance from support	a_v	1520 mm	
Tension reinforcement anchored?	Y/N	Y	(for shear enhancement)

Calculation

Shear stress in section $v = V / (b \times d)$
= 181.9 / (225 x 1190) x 1000
= 0.679 N/mm^2

5.3.3.2 — In no case should v exceed 0.92 $\sqrt{(fcu / \gamma_{mc})}$ or 7 / $\sqrt{\gamma_{mc}}$ N/mm^2, whichever is lesser.

0.92 $\sqrt{(fcu / \gamma_{mc})}$ = 0.92 x (14 / 1) ^ 0.5 = 3.44 N/mm^2
7 / $\sqrt{\gamma_{mc}}$ = 7 / (1) ^ 0.5 = 7 N/mm^2

Therefore, limit on v v_{lim} = 3.44 N/mm^2
0.679N/mm2 < 3.44N/mm2 therefore shear stress does not exceed vlim.

Table 9 — Depth factor $\xi_s = (500 / d)^{0.25}$ (modified by IAN 4/96)
= (500 / 1190) ^ 0.25
= 0.805

Depth factor cannot be less than 0.7
Therefore, ξ_s = 0.805

Table 8 — Ultimate shear stress in concrete,

$$v_c = \frac{0.24}{\gamma_{mv}}\left(\frac{100\,A_s}{b_w d}\right)^{1/3}(f_{cu})^{1/3}$$

where γ_{mv} = 1.25 , f_{cu} is not greater than 40 and

$$\left(\frac{100A_s}{b_w d}\right)^{1/3}$$ should not be taken less than 0.15 or greater than 3.0.

$$\left(\frac{100A_s}{b_w d}\right)^{1/3}$$ = ((100 x 1570.8) / (225 x 1190)) ^ (1/3) = 0.837
> 0.15, < 3.0, therefore, use 0.837

Project	Part of structure/scheme and status	Job ref
Old Mangere Bridge	Main Girders	

Office of Issue	Telephone No	Division	Calculations by	Checked by	Date	Calc Sheet no
AMA			[redacted]	BECA	23/01/2016	of

Code ref	Calculations	Remarks/ output	Checked by Inits & date
	Therefore, v_c = (0.24 / 1.25) x 0.837 x (14) ^ (1/3)		
	= 0.387 N/mm^2		
	Therefore, $\xi_s \times v_c$ = 0.805 x 0.387		
	= 0.312 N/mm^2		
cl. 5.3.3.3	Section close enough to support for enhanced shear strength? Yes		
	$3d/a_v$ = 3 x 1190 / 1520 = 2.349		
	$\xi_s v_c \times 3d/a_v$ = 0.312 x 2.349 = 0.733 N/mm^2		
	But is limited to v_{lim} = 3.44 N/mm^2		
	Therefore the shear strength $\xi_s v_c$ of the concrete for assessment = 0.733 N/mm^2		
	The ultimate shear resistance of the section is given by		
	$V_u = \xi_s v_c b_w d + (f_{yv}/\gamma_{ms}) A_{sv} (d/s_v)$		
	$\xi_s v_c \times b_w \times d$ = 0.733 x 225 x 1190 = 83.5 kN		
	A_{sv} = n x π x (Ø / 2)^2		
	= 0 x π x (0 / 2) ^ 2		
	= 0.0 mm^2		
	Therefore, $(f_{yv}/\gamma_{ms}) A_{sv} (d/s_v)$ = (210 / 1) x 0 x (1190 / 0)		
	= #DIV/0! kN		
cl. 5.3.3.2	For the links to be effective, the tensile capacity of the longitudinal reinforcement must		
	exceed $M/z + (V - \xi_s v_c b_w d)/2$ where z = 0.9d		
	= (0 / 1071) + (181900 - 83500) / 2 = 49200 N		
	Tensile capacity = $A_s f_y/\gamma_{ms}$ = 1570.8 x 210 / 1 = 329868 N		
	Therefore tensile capacity of bending reinforcement is adequate.		
	Also, for the links to be effective, A_{sv} (sin α + cos α) $(f_{yv} / \gamma_{ms}) > 0.2 b_w s_v$ and α > 30°		
	This sheet only valid if the links are vertical, i.e α = 90°		
	A_{sv} (sin α + cos α) (f_{yv} / γ_{ms}) =		
	0 (sin 90 + cos 90) (210 / 1) = 0 N		
	$0.2 b_w s_v$ = 0.2 x 225 x 0 = 0 N		
	Therefore the shear links are not effective.		
	Therefore the shear capacity = 83.5 kN		
	83.5kN < 181.9kN Therefore the section fails in shear.		

TRANSPORT AGENCY / Auckland Motorways	Project **Old Mangere Bridge**		Job ref.	
	Part of Structure **Slice Result**		Sheet No.	
	Drawing ref. s9(2)(a)	Date Jan-16	Check by	Date

Ref	Calculation	Output

SLICE RESULTS FROM LUSAS

	Project **Old Mangere Bridge**		Job ref.
	Part of Structure **Slice Result**		Sheet No.
	Drawing ref. [redacted]	Date Jan-16	Check by / Date
Ref	Calculation		Output

MGE1

Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
SW						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	14.472	-0.2225		-5.55E+04	5.08E+05	-2.10E+05	-14.2062	0.476052
Wet Concrete						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	14.472	-0.1275		-6.87E+03	1.33E+05	-4.91E+04	29.4436	6.87E-03
Displacement						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	14.472	-0.1275		2.30E+03	-2.76E+05	1.89E+04	-85.4988	-4.58E+04
Parapet Self Weight						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	14.472	-0.1275		-4.60E+03	1.85E+04	-6.58E+03	-20.0727	-0.121373
3kPa Pedestrian UDL						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	14.472	0.0545563		-9.96E+04	4.37E+04	-2.05E+04	6.5983	36.2965
3kPa Patch						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	14.472	0.0545563		-6.31E+04	7.12E+04	-3.16E+04	-41.9397	-4.49133

MGI1

Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
SW						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	12.034	-0.25		-9.46E+04	4.87E+05	-1.74E+05	-12.0339	0.844818
Wet Concrete						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	12.034	-0.25		-3.52E+04	1.32E+05	-4.60E+04	14.7661	0.353502
Displacement						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	12.034	-0.25		4.88E+04	-1.66E+05	3.78E+03	-132.308	-7.41E+04
Parapet Self Weight						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (+Z)	0.5	22.548	12.034	-0.25		-3.81E+04	8.97E+03	-3.25E+03	-24.732	-0.130875
3kPa Pedestrian UDL						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	12.034	0.118912	71.3406	-1.03E+05	4.92E+04	-3.20E+04	4.53249	54.8635
3kPa Patch						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	12.034	0.118912	-49.9459	-4.48E+04	4.82E+04	-3.69E+04	-32.4846	-11.0821

MGI2

Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
SW						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	9.902	-0.25		-8.39E+04	4.48E+05	-1.68E+05	-17.637	
Wet Concrete						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	9.902	-0.25		-2.27E+04	1.15E+05	-4.28E+04	-29.3808	
Displacement						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	9.902	-0.25		2.91E+04	-8.15E+04	1.85E+03	-142.443	
Parapet Self Weight						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	9.902	-0.25		0	3.15E+03	-1.17E+03	-13.4057	
3kPa Pedestrian UDL						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	9.902	-0.25		-5.10E+03	7.64E+04	-2.28E+04	-14.8793	
3kPa Patch						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	9.902	-0.25		-8.28E+03	4.63E+04	-1.35E+04	64.6999	

MGI3

Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
SW						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	8.531	-0.25		-7.88E+04	4.34E+05	-1.65E+05	-15.8958	
Wet Concrete						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	8.531	-0.25		-1.38E+04	1.02E+05	-3.98E+04	-21.1523	
Displacement						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	8.531	-0.25		1.16E+04	-3.24E+04	993.383	-141.472	
Parapet Self Weight						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	8.531	-0.25		0	1.59E+03	-569.628	-5.1544	
3kPa Pedestrian UDL						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	8.531	-0.25		-1.51E+04	6.22E+04	-2.09E+04	-18.4001	
3kPa Patch						Shear	Axial	Main Bending	Snd Bending	
Title	Dist	X	Y	Z	Px	Py	Pz	Mx	My	Mz
Slice 1 (-Z)	0.5	22.548	8.531	-0.25		-1.39E+04	5.56E+04	-1.77E+04	48.239	

APPENDIX B –

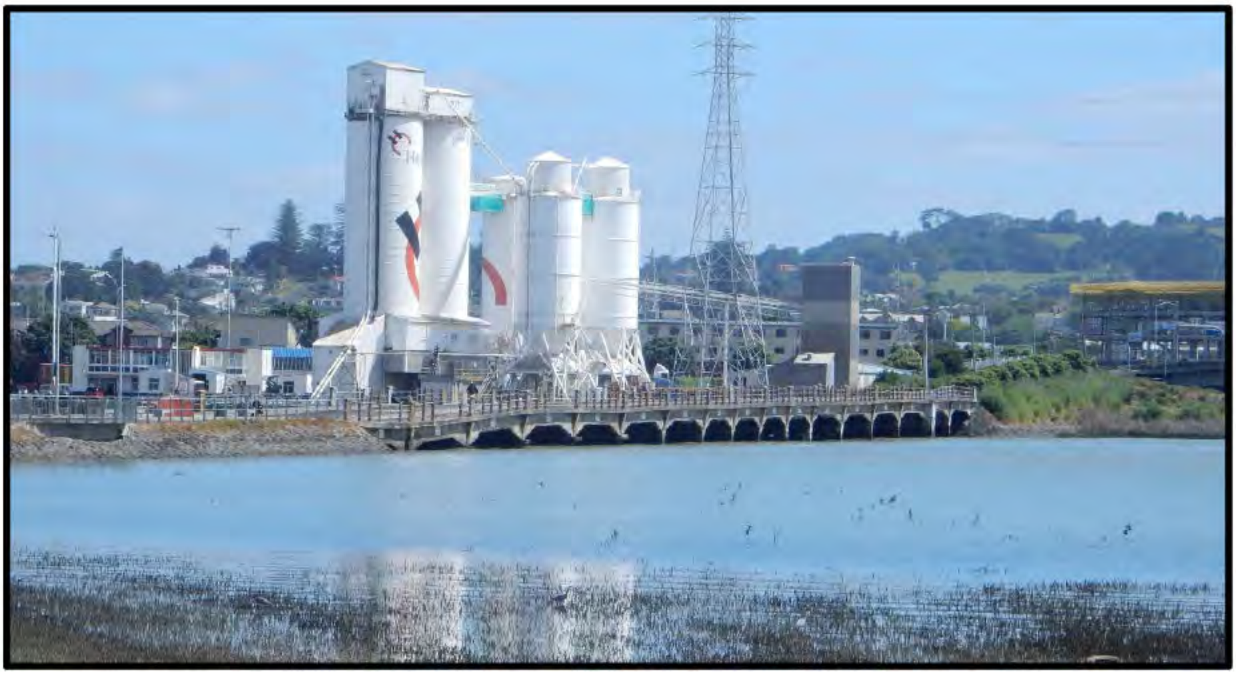
Principal Inspection

AUCKLAND MOTORWAY ALLIANCE

SPECIAL INSPECTION

Old Mangere Bridge

Date: December 2015



AMA – SPECIAL INSPECTION

Old Mangere Bridge

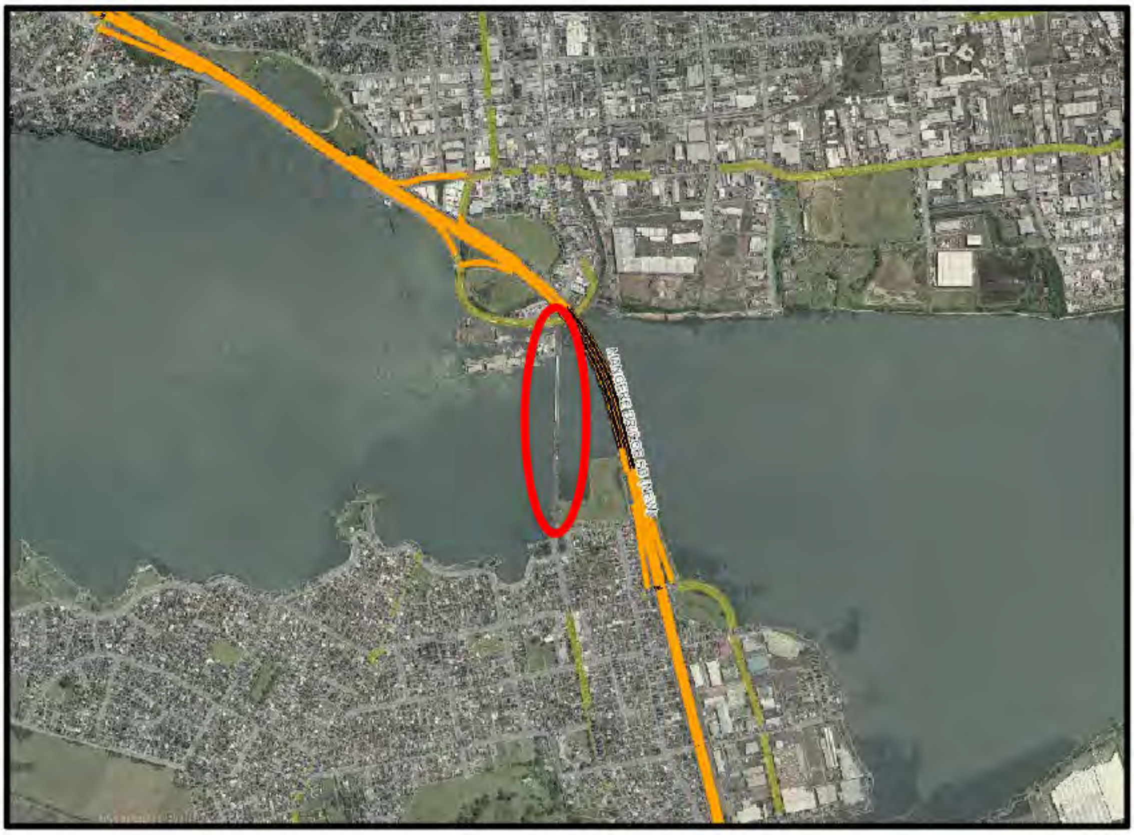
Report:-	
Prepared By:-	s9(2)(a) [redacted] Structures Manager
Reviewed By:-	s9(2)(a) [redacted] Structures Inspectors
Authorised By:-	s9(2)(a) [redacted] Structures Manager
Report Date:-	

Contents

1. Bridge Details

Bridge Name:	Old Mangere
BSN:	-
Highway:	SH20
RP:	020-0010/02.54-I
Date of inspection:	17/12/2015
Inspector:	s9(2)(a) [redacted]

2. Location Plan



3. Recommendations for maintenance/monitoring/inspections:-

HIDDEN PARTS NOT EXAMINED (EXCLUDING FOUNDATION)		COMPONENT					REASON			
ITEM	DESCRIPTION	LOCATION	EST COST	PRIORITY	PRIORITY WITHIN	QUANTITY	SEVERITY	RISK SCORE	WORKS CATEGORY	
1										
2										
3										
4										
5										
Signed		Name Liam Coleman					Date:			

Old Mangere Bridge
December 2015

RISK SEVERITY & PROBABILITY SCORES.

Effect or Defect	Severity Factor	Certain	Likely	Probable	Possible	Unlikely
		5	4	3	2	1
Catastrophic / unpredictable / undetectable failure, which causes injuries and/ or fatalities	5+	25+	20+	15+	10+	5+
Predictable / detectable failure, which may cause injuries / fatalities	5	25	20	15	10	5
Predictable / detectable failure, leading to road closure	4	20	16	12	8	4
Predictable/detectable deterioration, leading to reduced usage based on assessment results	3	15	12	9	6	3
Further deterioration in condition of structure, leading to higher severity impact in future years.	2	10	8	6	4	2
Further deterioration of structure, leading to increase in eventual repair, cost and public complaints	1	5	4	3	2	1
Nil or Negligible consequences	-1	-5	-4	-3	-2	-1

Priority		
	H	High; work should be done during the next financial year to ensure the safety of the public or safeguard structural integrity or avoid a high cost penalty.
	M	Medium; work should be done during the next financial year; postponement carries some cost penalty.
	L	Low; work should be done within the next two financial years.

[HIGHWAYS ENGLAND – BD63/07]

High priority – 'Must be done as soon as possible'.

This includes maintenance of any component:

- Which is unsafe to the public.
- Which has a medium to high risk of failure.
- Which is in very poor condition and requires prompt action.
- Which if not actioned would result in much higher future costs.

Medium priority – 'Preferable to do as soon as possible'.

This includes maintenance that is not a 'high' priority, but which should be done as soon as possible to ensure the structural integrity and safety of the structure is maintained to an appropriate standard.

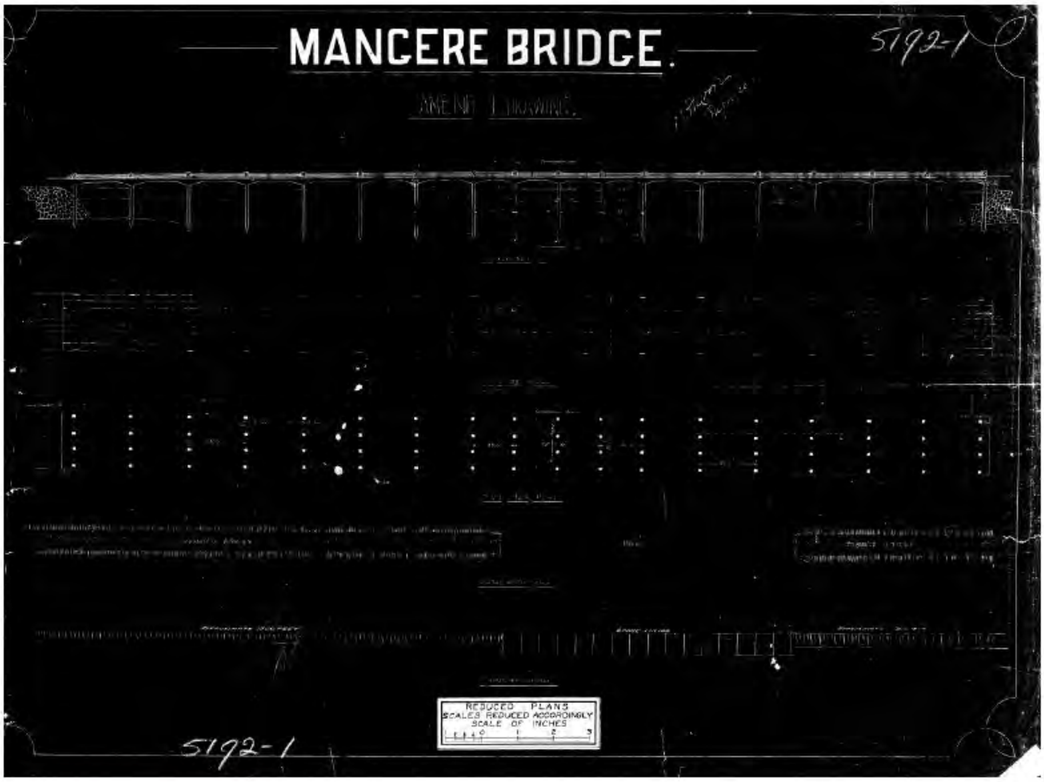
[NZTA – SM018]

4. Bridge Structural Details

The bridge was constructed circa 1914 and has 17 simply supported reinforced concrete spans which vary from 11.43m to 15.24m.

Each superstructure span comprises 8 precast beams with cast insitu deck slabs. All support piers comprise of driven piles supporting a cast insitu cap beam.

The structure was closed to vehicle live load in 1985 and only accommodates pedestrian and recreational users (the bridge is used extensively as a fishing platform and by recreational cyclists).



5. Bridge Deterioration

The general condition of the bridge has suffered widespread deterioration over the last 100 years. The combination of a severe marine environment and poor concrete quality, with lack of adequate concrete cover has resulted in widespread corrosion of reinforcement and spalling of concrete. Inspections over the last thirty years indicate that this deterioration has been gradual with the worst areas being the interior 6 main beams of each span, the pile cap beams and the cap beam/pile top joints.

In order to reinstate the structural capacities of these critical elements, repairs have been carried out over a number of years. These repairs include:
Installation of external prestressing tendons to the sides of all interior main beams.
Installation of 2 continuous reinforced concrete strips along the full length of the deck surface.
Installing steel bracing to support one pier cap.
Installing steel clamps to a number of broken and deteriorated concrete foundation piles.

Other repairs have been carried out when necessary including those in response to 3 known ship impacts.

6. Principal Inspection Report Details

Inspection Details:-

A structure in good condition has a General Inspection (GI) every two years and Principal (Detailed) Inspection (PI) every 6 years in accordance with the NZTA Specification S/6.

Due to the condition of Old Mangere the structure currently receives 4 inspections yearly. A walkover of the structure is carried out and a visual inspection from a boat. A within touching distance / physical inspection of the underside was carried out where access permitted.

Inspection Comments:-

Overall the structure is in a very poor condition with significant areas of delamination of the concrete and extensive loss of section to the tensile reinforcement. The structure has had a number of boat strikes in the past which has placed excess displacements and deflections on the structure, which has resulted in emergency repairs.

Any areas that are subjected to wetting and drying conditions are generally spongy when struck with a hammers. The majority of previous concrete durability repairs have failed or are brittle sounding when struck with a hammer.

Overall the structure is in a very poor condition and it is very hard to give a full comprehensive catalogue of defects. From a boat inspection there are limitations on how close you can get to some of the elements. Some of the more significant areas of concern were, but are not limited to just these areas are:

1. **Intermediate Support 5-8** - The crosshead beam has a crack the full depth which was instigated from a previous bridge strike. The area was subsequently clamped. The clamps have broken and are loose when struck with a hammer. The area is generally spongy when struck with a hammer. If this section were to fail MGE2 and MGI6 would no longer have effective support under their own dead weight.

2. **Generally on spans 3-9** the longitudinal tensile reinforcement is floating in mid-air and the shear stirrups have completely corroded. The composite behaviour of reinforced concrete is no longer functional and in reality the bars are probably doing very little in the capacity of the structure. At these locations the top slab and the supplementary strengthening slabs are given the structure capacity.

3. **Intermediate Support 17-20** - The crosshead pier under MGE2 has extensive cracking and is showing initial signs of delamination and cracking. Supplementary piles have been introduced in this area to support the cross head beams. If the concrete in this area was to fully crush MGE2 would be floating in mid-air and questionable if it could hold its own self weight in this situation.

4. **Intermediate support 21-24** - Intermediate support pile 24 is no longer plum following a previous boat strike. There is cracking and delamination of the pile and crosshead at this location.

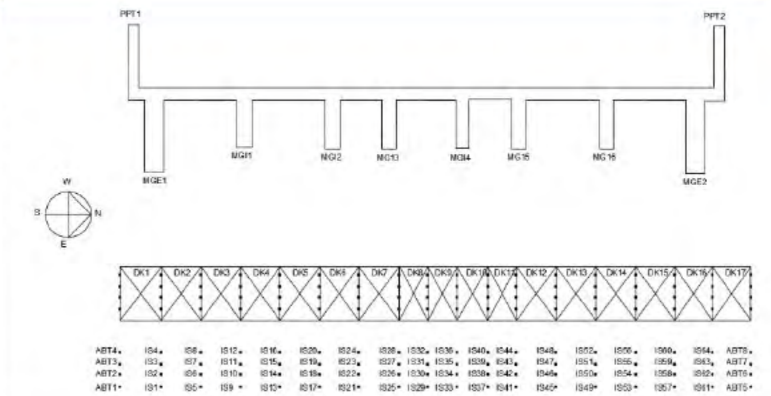
5. **Intermediate support 41-44 (Span11/12), 45-48 (12/13) & 49-52(13/14)** - These 3 intermediate piers are the areas of most concern. There is extensive delamination, cracking and loss of section to the reinforcement. Some of the corner reinforcement on the piles have plastically deformed and buckled. As a result the pile will have little to no tensile capacity and completely relying on the crush strength in the concrete. A number of the piles have extensive cracking which could be a bi product of the deterioration, lack of tensile capacity and initial signs of failure

6. **Span 17 & Abutment** – There is extensive, delamination and loss of section to the beams. The abutment is pilled and surrounded with a concrete stone mix. There is a vertical crack to the stone concrete surround which will expose the piles behind to the elements. The condition of the piles in both abutments is unknown and are hidden. On the bridge beams there appear to be shear cracking to the ends of the beams. Due to access it is not known if the cracks are a result of delamination or shear induced. The webs of some of the beams appear to show crushing delimitation probably as a result of the external post-tension system.

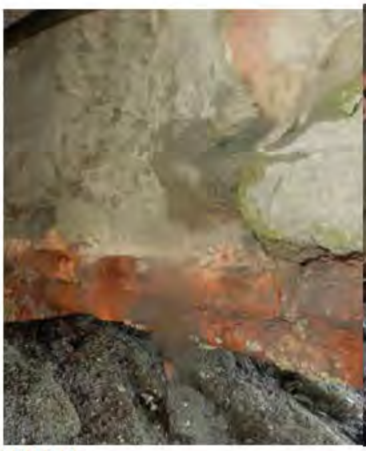
7. **The longitudinal strengthening slabs** added in 2005 has transverse cracking over the support on IS17/20 (span 5/6) and at the point of contra-flexure on span 2. The two main cracks noted are the full width of the deck. The cracks at present are less than 0.3mm in width.

7. Span 1

Defect confirmed at this inspection [D1]

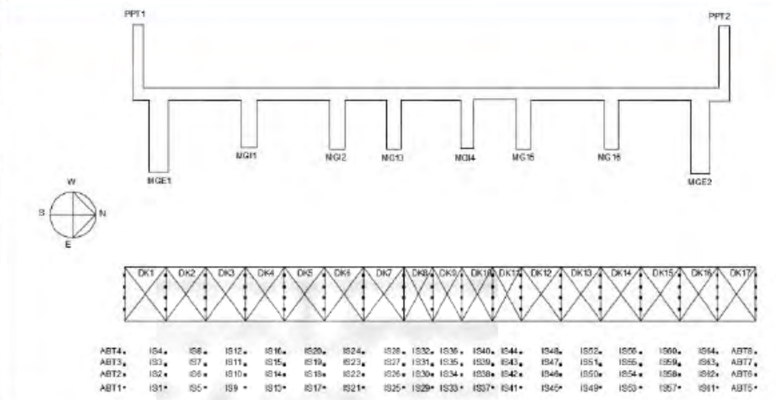
Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

Comments: Heavy delamination of concrete and reinforcement throughout the span. The pictures below show the level of loss of section to the main tensile reinforcement following a strike with a hand held hammer. The main reinforcement has loss in excess of 50% and is no longer bonded to the concrete beams. Generally all the concrete sounded spongey/brittle when struck with a hammer.



BEFORE AFTER BEFORE AFTER

8. Span 2

Defect confirmed at this inspection [D2]	
Component: Deck & Beams	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

Comments: All the beams are showing signs of delamination and loss of section to the reinforcement. Previous repairs have failed and are hiding the true deterioration of the structure. Black low oxygen corrosion was noted where the concrete broke off the beam. Generally all the concrete sounded spongey/brittle when struck with a hammer.

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9. Intermediate Support 5-8

Defect confirmed at this inspection [D3]

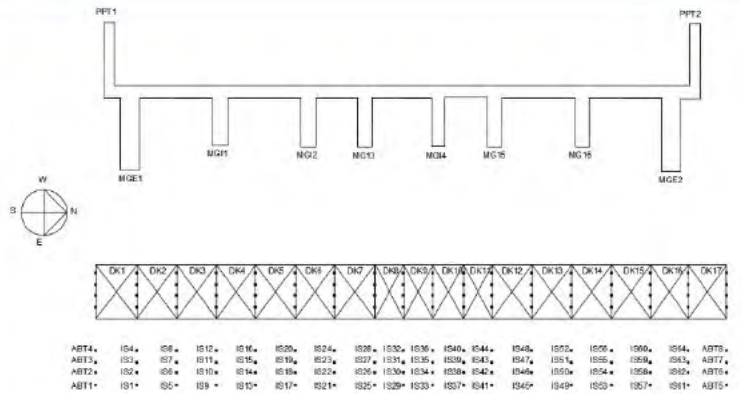
Component: IS5-IS8 Piers & Cross Head Beam

Defect Type: Spalling, Delamination and displacement

Extent: D

Severity: 5

Priority: HIGH

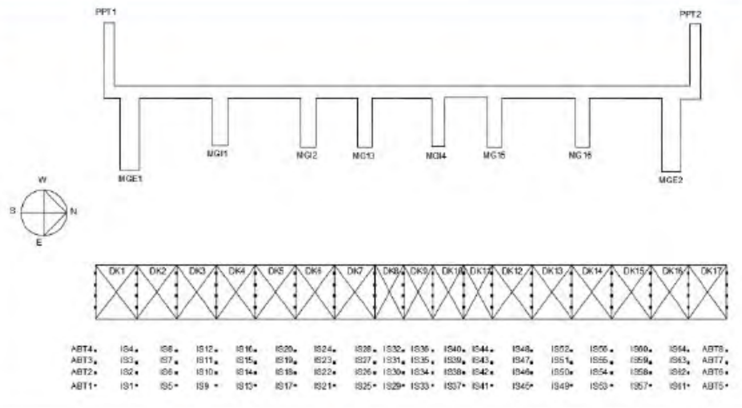


Risk Severity: 5+

Risk Score: 25+

Comments: The crosshead beam in-between MGI5 & MGI6 has a crack the full depth. The area was previously clamped but the clamps are loose and no longer functional. The cross head beam above MGE2 is crushed with a number of cracks. Generally all the concrete sounded spongey when struck with a hammer

10. Span 3

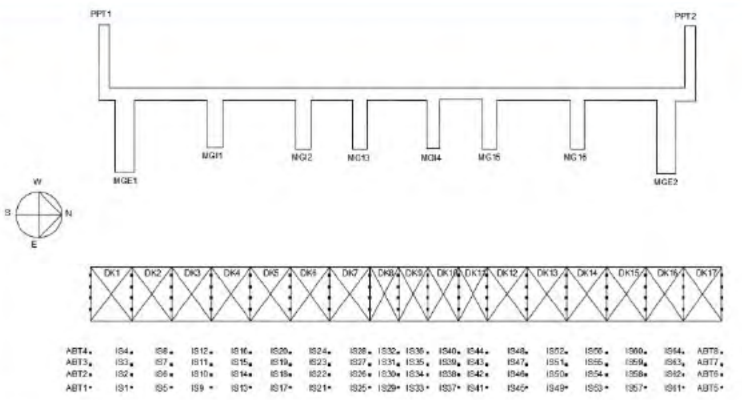
Defect confirmed at this inspection [D4]	
Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The majority of the beams bottom reinforcement are floating in mid-air with 100% section loss to the shear links






11. Span 4

Defect confirmed at this inspection [D5]	
Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	

Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The majority of the beams bottom reinforcement are floating in mid-air with 100% section loss to the shear links




12. Span 5

Defect confirmed at this inspection [D6]

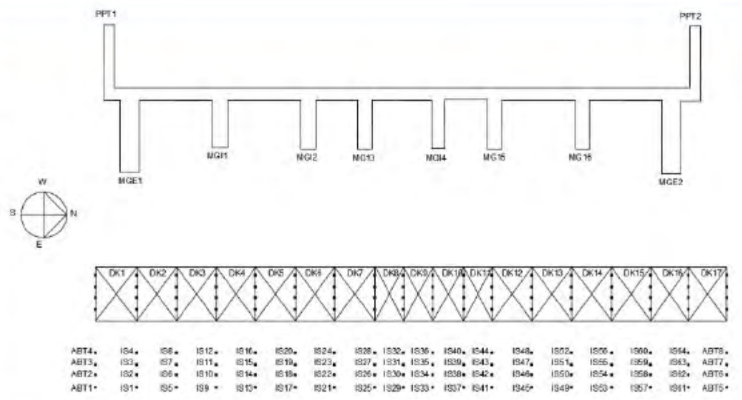
Component: All Elements

Defect Type: Spalling & Delamination

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The majority of the beams bottom reinforcement are floating in mid-air with 100% section loss to the shear. On MGE1 a number of the tensile reinforcement bars have broken. The concrete at the ends of the beams are delaminating and crushing.

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13. Intermediate Support 17-20

Defect confirmed at this inspection [D7]

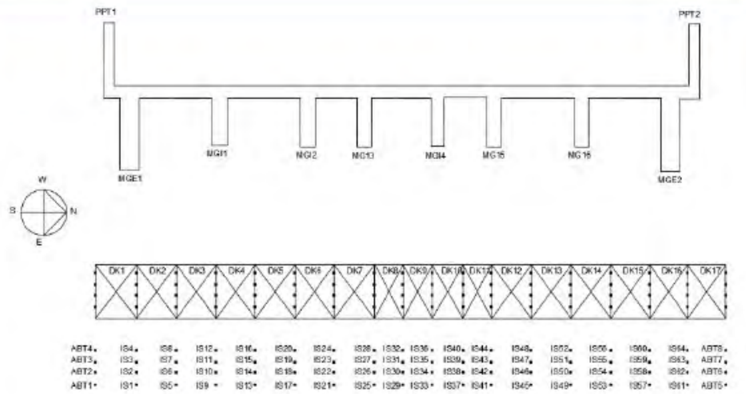
Component: IS20 Piers & Cross Head Beam

Defect Type: Spalling, Delamination and cracking

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+

Risk Score: 20+

Comments: There is a full depth crack running through the cross head beam under MGE2. It appears there is a crack along the cross head and the top of the pile. A supplementary support pier has been added at this location to strengthen the area. The newer support pile sounds spongey when struck with a hammer but is not showing signs of distress. With the damage to the cross head beam it would be questionable the benefit of the new pier under current load arrangements. The transverse post tensioning will have less of an impact due to the cracked concrete.





14. Span 6

Defect confirmed at this inspection [D8]

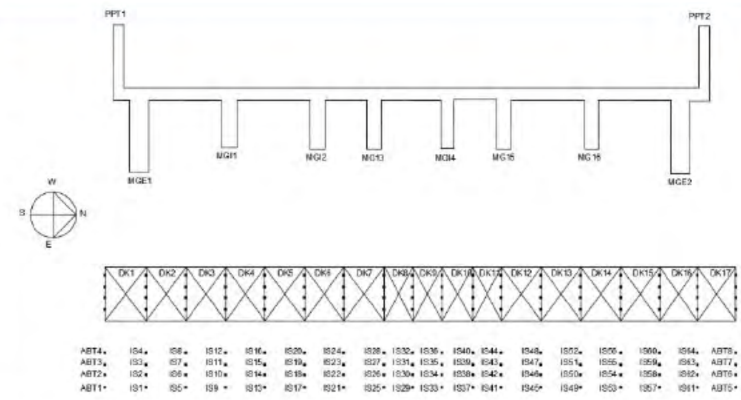
Component: All Elements

Defect Type: Spalling & Delamination

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+

Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The majority of the beams bottom reinforcement are floating in mid-air with 100% section loss to the shear links.

15. Intermediate Support 21-24

Defect confirmed at this inspection [D9]

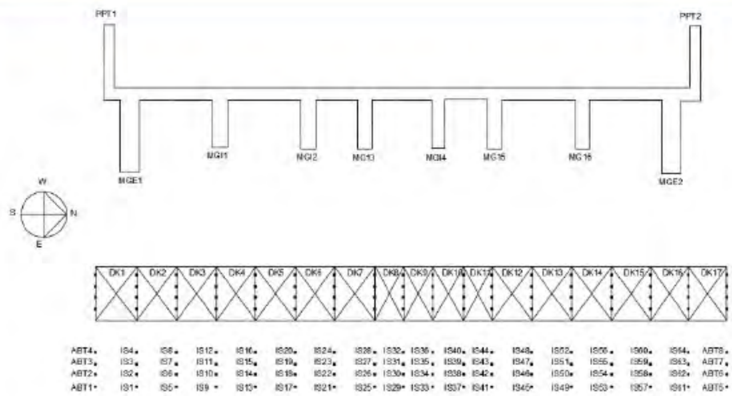
Component: IS21 & IS24 Piers & Cross Head Beam

Defect Type: Spalling, Delamination and cracking

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+	Risk Score: 20+

Comments: IS24 is no longer vertical/plum, initially due to a boat impact. There is a large crack the full depth of the cross head. Areas broke loose when struck with a hammer revelling corroded reinforcement. Cracking was also noted on IS21, it is assumed the reinforcement behind is corroded with loss of section.

BEFORE

AFTER

BEFORE

AFTER

16. Span 7

Defect confirmed at this inspection [D10]

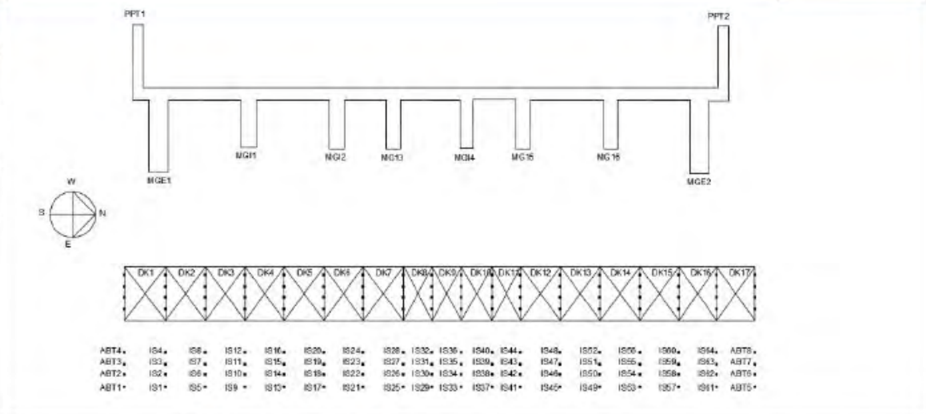
Component: All Elements

Defect Type: Spalling & Delamination

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+

Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The majority of the beams bottom reinforcement are floating in mid-air with 100% section loss to the shear links. The reinforcement in MG1 has completely deteriorated with 100% section loss to a number of bars

17. Span 8

Defect confirmed at this inspection [D11]

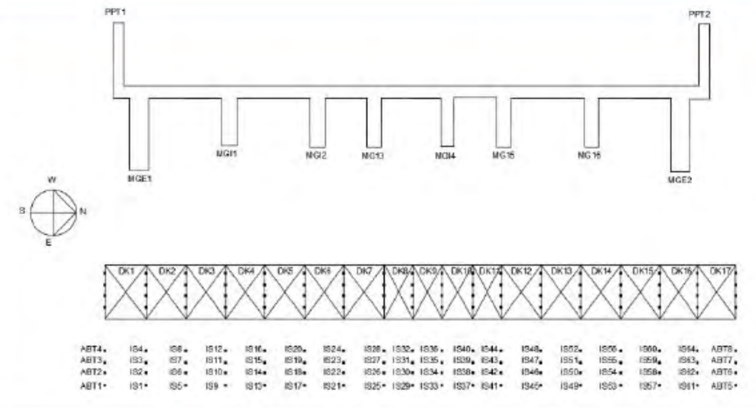
Component: All Elements

Defect Type: Spalling, Delamination & cracking

Extent: D

Severity: 5

Priority: HIGH



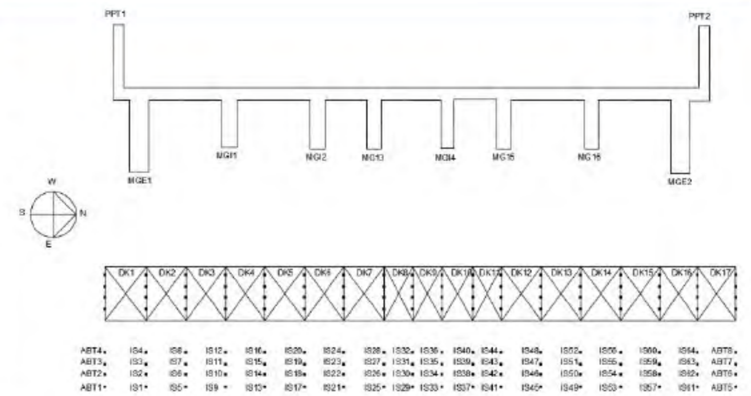
Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement.

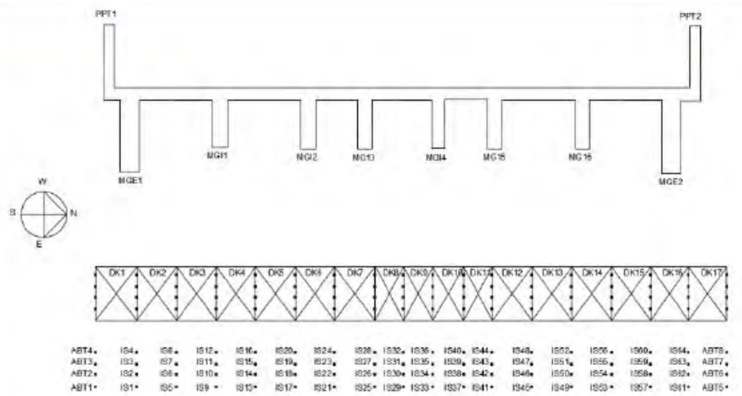





18. Span 9

Defect confirmed at this inspection [D12]	
Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+
Comments: All elements on the deck have areas of delamination and loss of section to reinforcement.	

19. Intermediate Support 29-32

Defect confirmed at this inspection [D13]	
Component: IS29-IS32 Piers & Cross Head Beam	
Defect Type: Spalling, Delamination and cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+
Comments: There are full depth cracks on the cross head beams under MGE1. There are stepped cracks to the soffit of all the cross beams as a result of delamination and rust jacking.	

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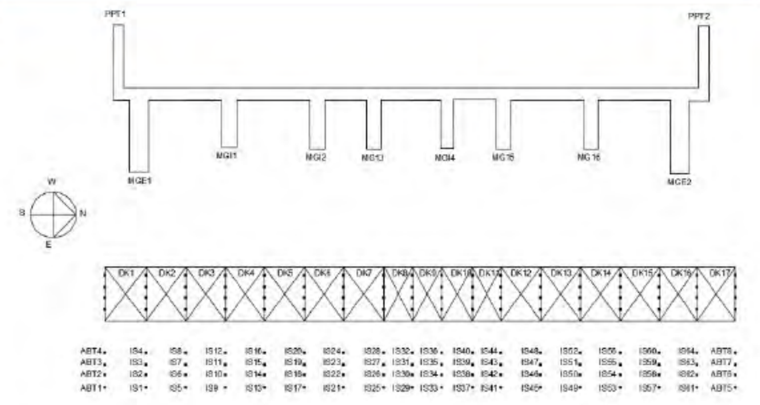
Defect confirmed at this inspection [D14]	
Component: IS29-IS32 Piers & Cross Head Beam	
Defect Type: Spalling, Delamination and cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

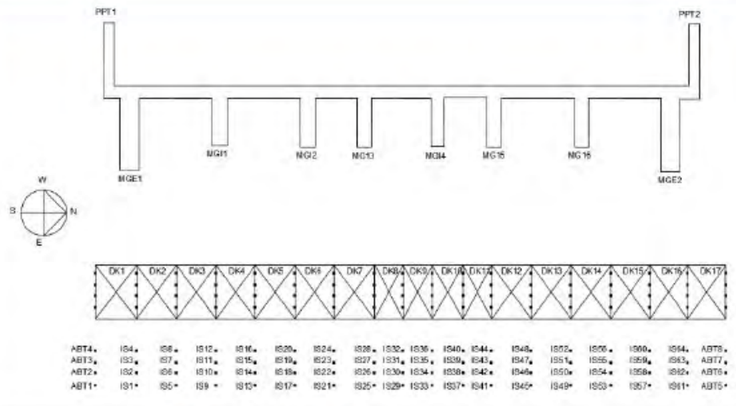
Comments: IS31 was spongey when struck and areas delaminated. There is also transverse cracking to the pile.

BEFORE

AFTER

20. Span 10

Defect confirmed at this inspection [D15]

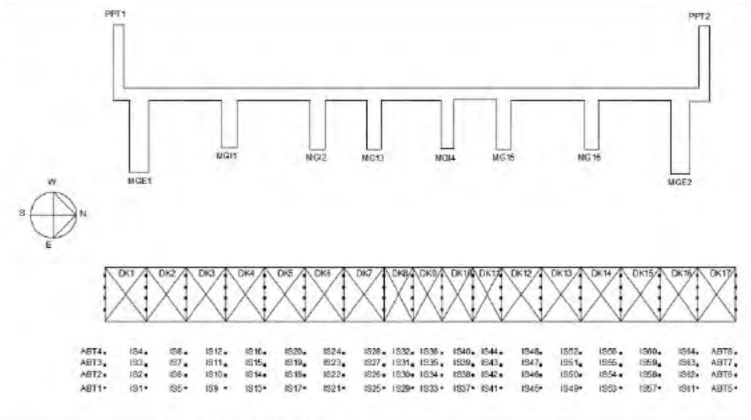
Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	

Risk Severity: 5+	Risk Score: 20+

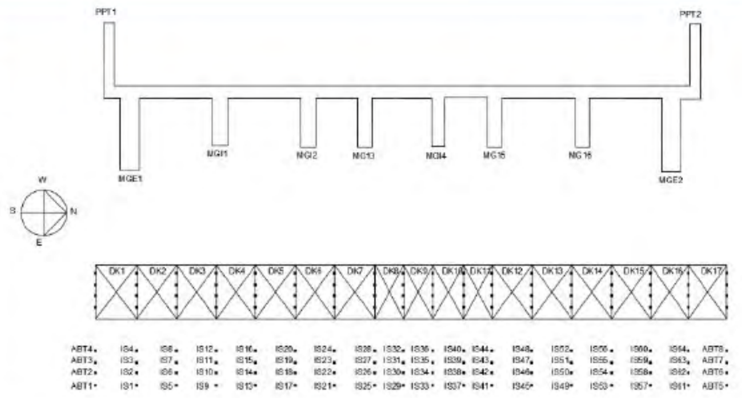
Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The soffit of the deck is a little better in this span compared to the previous spans as it is above the tidal range.




21. Intermediate Support 29-32

Defect confirmed at this inspection [D16]	
Component: IS37-IS40 Piers & Cross Head Beam	
Defect Type: Spalling, Delamination and cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+
Comments: There transverse cracks on IS38 & IS40. The piles sound and feel spongey when struck with a hammer and areas delaminated when hit.	

22. Span 11

Defect confirmed at this inspection [D17]	
Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The soffit of the deck is a little better in this span compared to the initial spans as it is above the tidal range.

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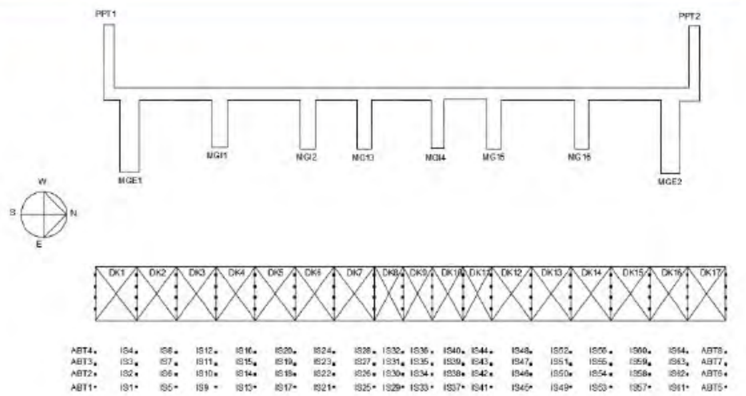
23. Intermediate Support 41-44

Defect confirmed at this inspection [D18]	
Component: IS41-IS44 Piers & Cross Head Beam	
Defect Type: Spalling, Delamination and cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+
Comments: There transverse cracks on IS38 & IS40. The piles sound and feel spongey when struck with a hammer and areas delaminated when hit.	

BEFORE

AFTER

24. Intermediate Support 41-44

Defect confirmed at this inspection [D19]

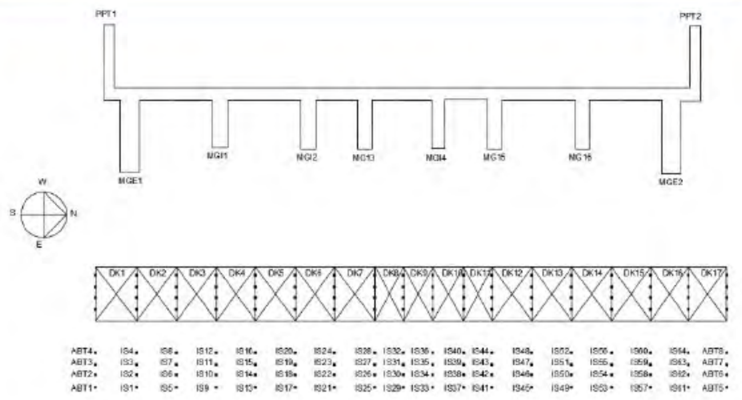
Component: IS45-IS48 Piers & Cross Head Beam

Defect Type: Spalling, Delamination and cracking

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+	Risk Score: 20+

Comments: There is extensive cracking and delamination of the cross head beams and the tops of the piles. The concreted delaminated when struck and generally is spongey when hit.

BEFORE

AFTER

25. Span 12

Defect confirmed at this inspection [D20]

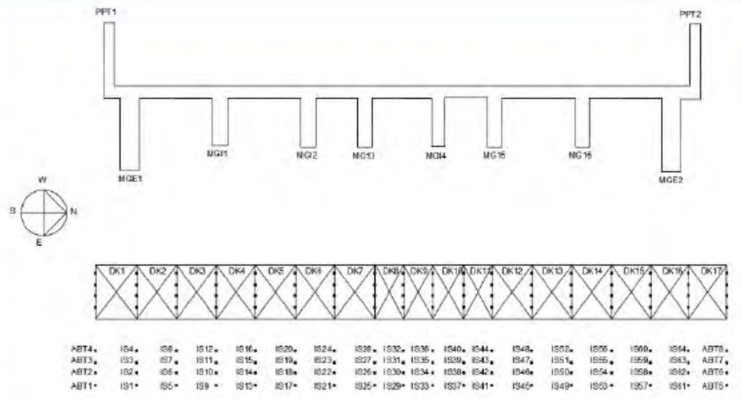
Component: All Elements

Defect Type: Spalling & Delamination

Extent: D

Severity: 5

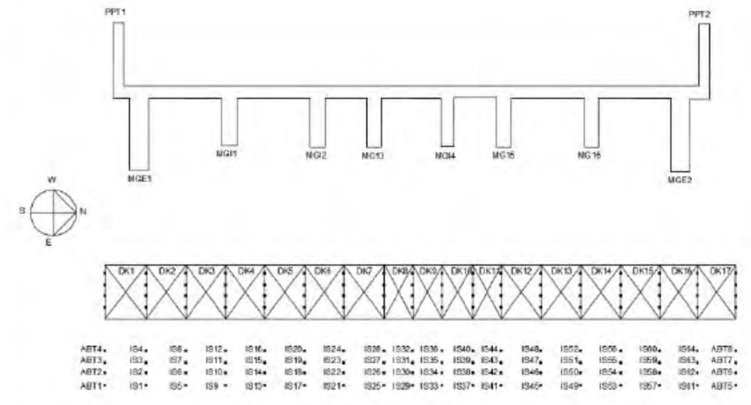
Priority: HIGH



Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The soffit of the deck is a little better in this span compared to the initial spans as it is above the tidal range.

26. Intermediate Support 45-48

Defect confirmed at this inspection [D21]	
Component: IS45-IS48 Piers & Cross Head Beam	
Defect Type: Spalling, Delamination and cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

Comments: There is extensive cracking and delamination of the cross head beams and the tops of the piles. IS46 & IS47 main reinforcement has buckled in the corners there is extensive crush and cracking of the concrete at this location. The cross head is heavily delaminated at this location. The concrete generally is spongey when struck.

BEFORE AFTER

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Defect confirmed at this inspection [D22]

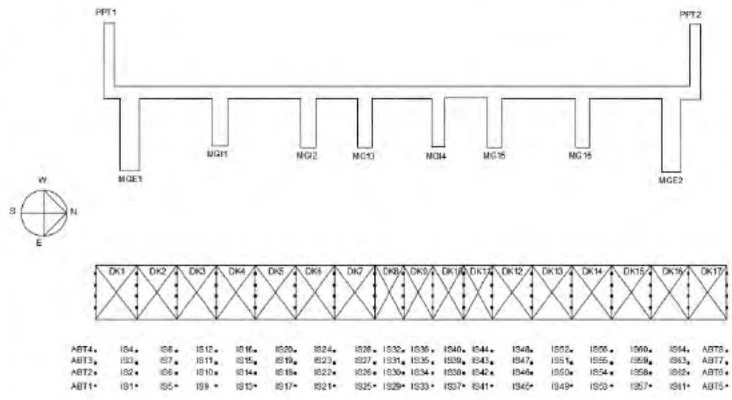
Component: IS45-IS48 Piers & Cross Head Beam

Defect Type: Spalling, Delamination and cracking

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+

Risk Score: 25+

Comments: There is extensive cracking and delamination of the cross head beams and the tops of the piles. IS46 & IS47 main reinforcement has buckled in the corners there is extensive crush and cracking of the concrete at this location. The cross head is heavily delaminated at this location. The concrete generally is spongey when struck. The horizontal strut to pier connection has cacking and very spongy when struck.

BEFORE

AFTER

Old Mangere Bridge
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Defect confirmed at this inspection [D23]

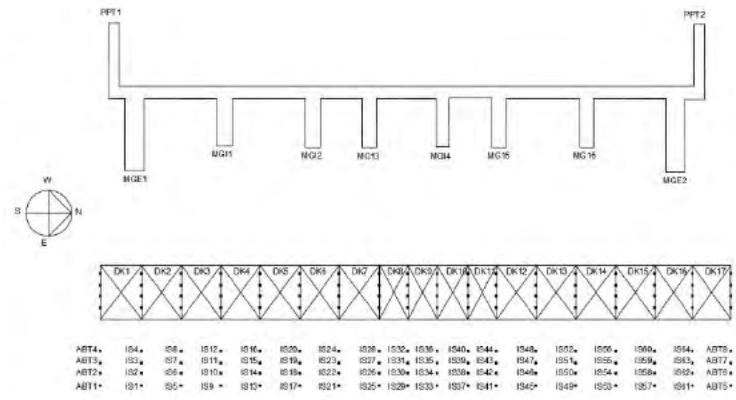
Component: IS45-IS48 Piers & Cross Head Beam

Defect Type: Spalling, Delamination and cracking

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+

Risk Score: 25+

Comments: There is extensive cracking and delamination of the cross head beams and the tops of the piles. IS46 & IS47 main reinforcement has buckled in the corners there is extensive crush and cracking of the concrete at this location. The cross head is heavily delaminated at this location. The concreted generally is spongey when struck

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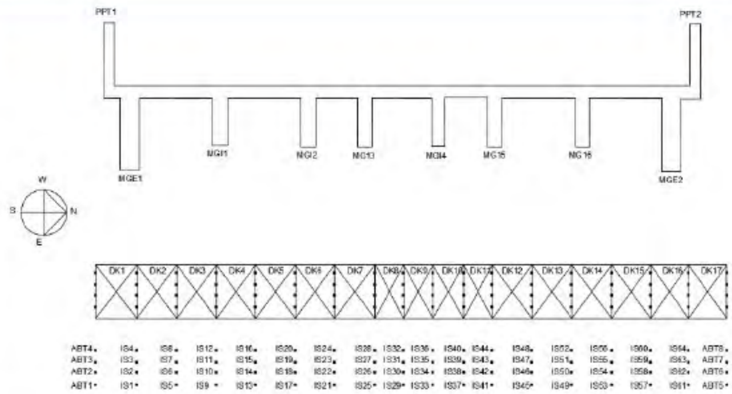
27. Span 13

Defect confirmed at this inspection [D24]	
Component: All Elements Defect Type: Spalling & Delamination Extent: D Severity: 5 Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The soffit of the deck is a little better in this span compared to the initial spans as it is above the tidal range.

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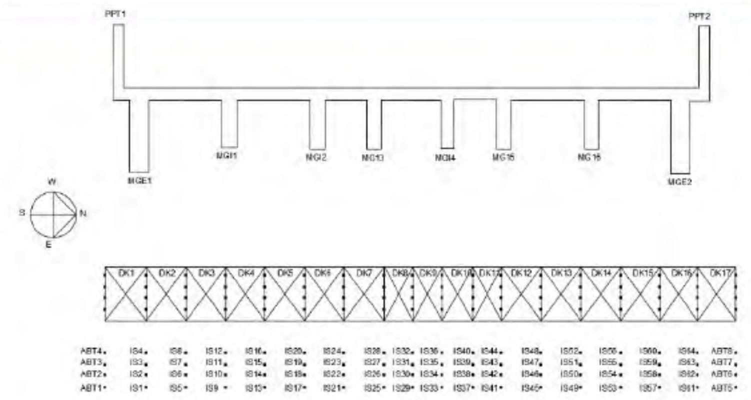
28. Intermediate Support 49-52

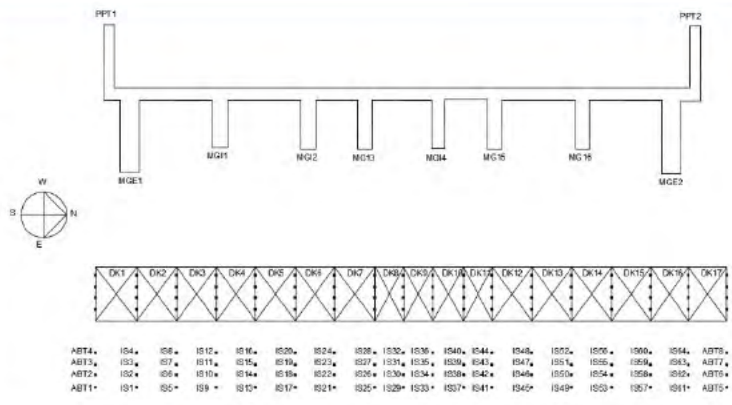
Defect confirmed at this inspection [D25]

Component: IS49-IS52 Piers & Cross Head Beam	
Defect Type: Spalling, Delamination and cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+

Comments: There is extensive cracking and delamination of the cross head beams and the top of pile 51. The corner reinforcing is buckled and there are transverse cracks across the pile. The concreted generally is spongey when struck.

29. Span 14

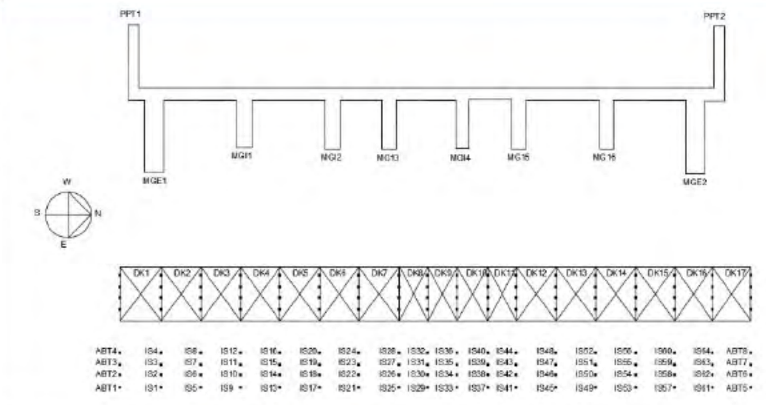
Defect confirmed at this inspection [D26]

Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	

Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The soffit of the deck is a little better in this span compared to the initial spans as it is above the tidal range.

30. Span 15

Defect confirmed at this inspection [D27]	
Component: All Elements	
Defect Type: Spalling & Delamination	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+
Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The soffit of the deck is a little better in this span compared to the initial spans as it is above the tidal range. The piers have been strapped up with supplementary steels following a boat strike a few years ago.	





31. Span 16

Defect confirmed at this inspection [D28]

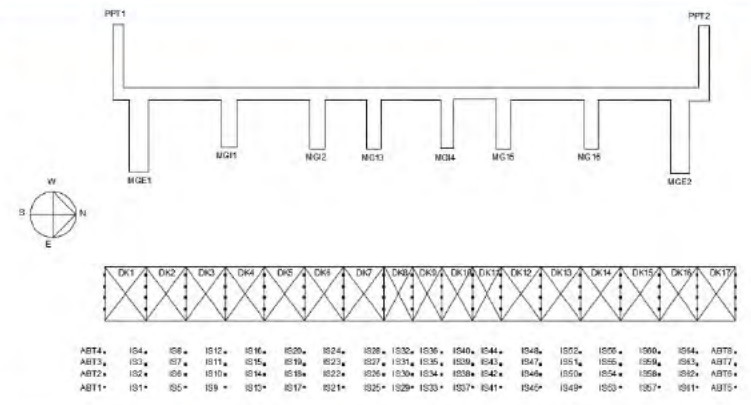
Component: All Elements

Defect Type: Spalling & Delamination

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The soffit of the deck is a little better in this span compared to the initial spans as it is above the tidal range. The piers have been strapped up with supplementary steels following a boat strike a few years ago.

Old Mangere Bridge
December 2015

32. Span 17

Defect confirmed at this inspection [D29]

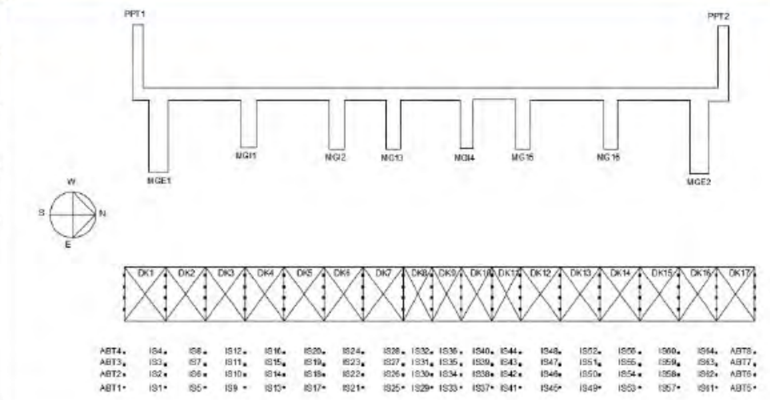
Component: All Elements

Defect Type: Spalling & Delamination

Extent: D

Severity: 5

Priority: HIGH



Risk Severity: 5+	Risk Score: 20+

Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. There are fracture to the abutment that surround the piles support at this location. The ends of the beams have heavily delaminated and loss of section to the reinforcement. The bed stones area are, missing, loose or heavily cracked.



Old Mangere Bridge
December 2015

Defect confirmed at this inspection [D30]

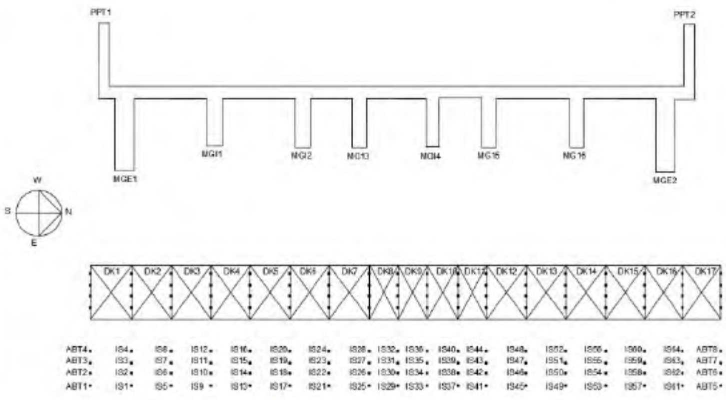
Component: All Elements

Defect Type: Spalling & Delamination

Extent: D

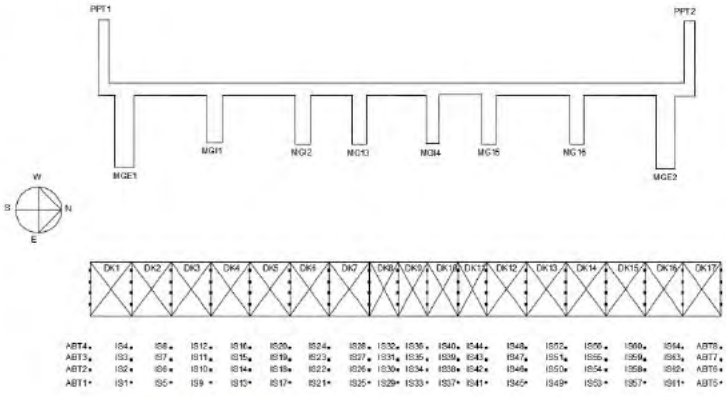
Severity: 5

Priority: HIGH



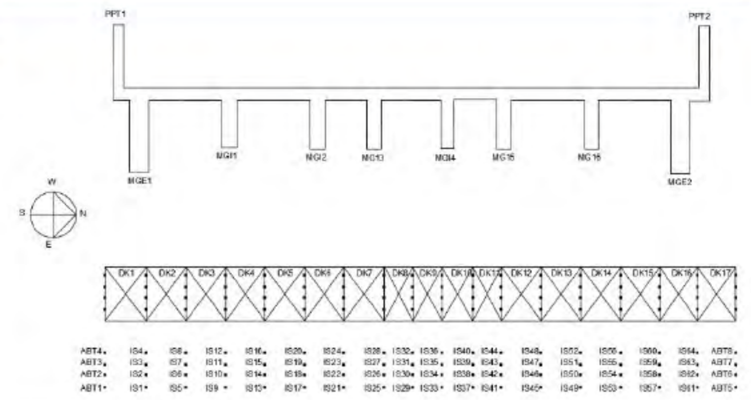
Risk Severity: 5+	Risk Score: 20+

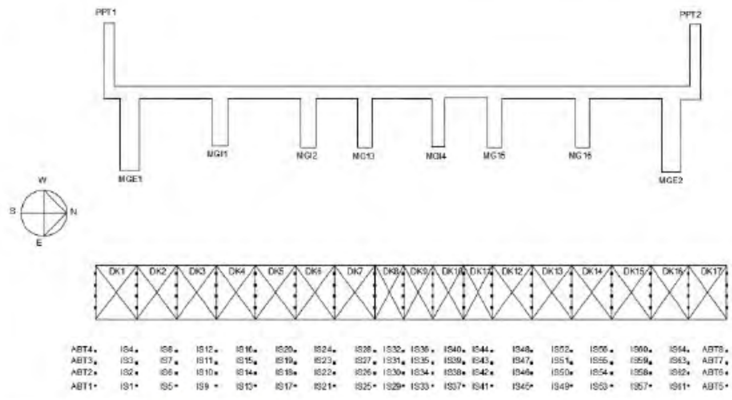
Comments: All elements on the deck have areas of delamination and loss of section to reinforcement. The ends of the beams have heavy delaminated and loss of section to the reinforcement. The previous post-tension system seems to be crushing the concrete and there appears to be shear cracks at the supports.

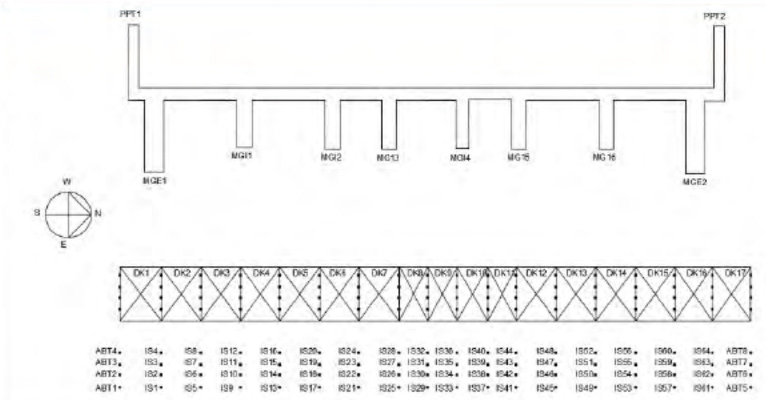
Defect confirmed at this inspection [D31]	
Component: MGI3	
Defect Type: Spalling & Delamination and buckling	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5+	Risk Score: 20+
Comments: MGI3 appears to be buckling over IS51-51 at the end of the beam.	

Old Mangere Bridge
December 2015

33. Top Side

Defect confirmed at this inspection [D32]	
Component: All Elements	
Defect Type: Cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5	Risk Score: 15
Comments: Cracking of previous repairs and the original footpath slabs	

Defect confirmed at this inspection [D33]	
Component: All Elements	
Defect Type: Cracking	
Extent: D	
Severity: 5	
Priority: HIGH	
Risk Severity: 5	Risk Score: 15
Comments: Cracking to strengthening slab which appears to go the full width of the bridge, in span 5/6 over the pier and span 2 at the point of contra-flexure	

Defect confirmed at this inspection [D34]	
Component: Parapet	
Defect Type: Leaning	
Extent:	
Severity:	
Priority: HIGH	
Risk Severity: 5	Risk Score: 15
Comments: Parapet leaning on the west side. AMATrac has been raised to replace with new handrail.	

34. General Views

1. General View looking North (Note settlement of south span)

2. General view looking North

3. General View looking East

4. General view looking West (picture taken from walkway on the mainline structure)

5. General view looking South

6.

7.

8.

9.
10.

Bridge Inspection Report

NZ TRANSPORT AGENCY WAKA KOTAHI	Bridge Name: Nelson Street Bridge	Highway: MIS	ID: NA	BCI (av): 44 / BCI (crit): 22
Bridge Type:		**Report Type :**	Principal Inspection	

Marking Code		Extent marking code	Severity code
0 = Not inspected 1 = Satisfactory 2 = Monitor next inspection R = Routine maintenance (provide comment) S = Structural maintenance (provide comment)	**Deck width** 10.553 **Total bridge length** 248 **Span** 1 of 17 **Span length** 15.24 GPR Survey REQ'D	A = No defect B = Not > 5% C = Moderate; 5 - 20% D = Wide; 20 - 50% E = > 50%	1 = as new 2 = early signs of defect 3 = moderate defect 4 = severe defect 5 = element failed

Ext = Extent ; **Sev** = Severity	Inspector : L. Coleman	Next Inspection Type : Feburauy 2016
	Date : 17/12/15	Date (mth/yr) :

Set	No	Description	Mark	Ext/Sev	Brief description of defect and comments
Superstructure Elements	1	Primary load carrying element	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	2	Secondary element(s) – Transverse beams	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	3	Secondary element(s) – Other (incl. deck)	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	4	Half/Hinge joints			
	5	Seismic linkages/Holding Down bolts			
	6	Parapet beam or cantilever	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	7	Cross bracing	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
Load-bearing Substructure	8	Foundations			
	9	Abutments	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	10	Head wall	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	11	Pier / column	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	12	Cross-head / capping beam	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	13	Bearings	S	E5	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	14	Bearing plinth / shelf	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
Durability Elements	15	Superstructure drainage	2	B2	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	16	Substructure drainage	2	B2	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	17	Movement / expansion joints			
	18	Painting : Superstructure elements			
	19	Painting : substructure elements			
	20	Painting : barriers/guardrails			
Safety Elements	21	Access / walkways / gantries			
	22	Guardrail / handrail / safety fences	S	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	23	Carriageway surfacing	2	C4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	24	Footway / verge / footbridge surfacing	2	C3	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
Waterway Elements	25	Invert / river bed			
	26	Aprons			
	27	River bed upstream			
	28	River bed downstream			
	29	Scour	2	B2	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	30	River banks	2	A1	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
Retaining Elements	31	Revetment / batter slope paving	2	B2	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	32	Wing walls	2	C3	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	33	Retaining walls	2	C3	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	34	Embankments	2	C3	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
Other	35	Approach rails / barriers / walls	2	A1	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	36	Approach adequacy	2	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	37	Signs			
	38	Lighting			
	39	Services			
	40	Appearance	2	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
Subsidence	41	Pavement depression	2	D4	All Structure in a poor condition and is to to be monitored and inspected 3 monthly
	42	Abutment backfill			
	43	Movement/Cavity present			

Proforma no...........................

APPENDIX C –

Original Drawings

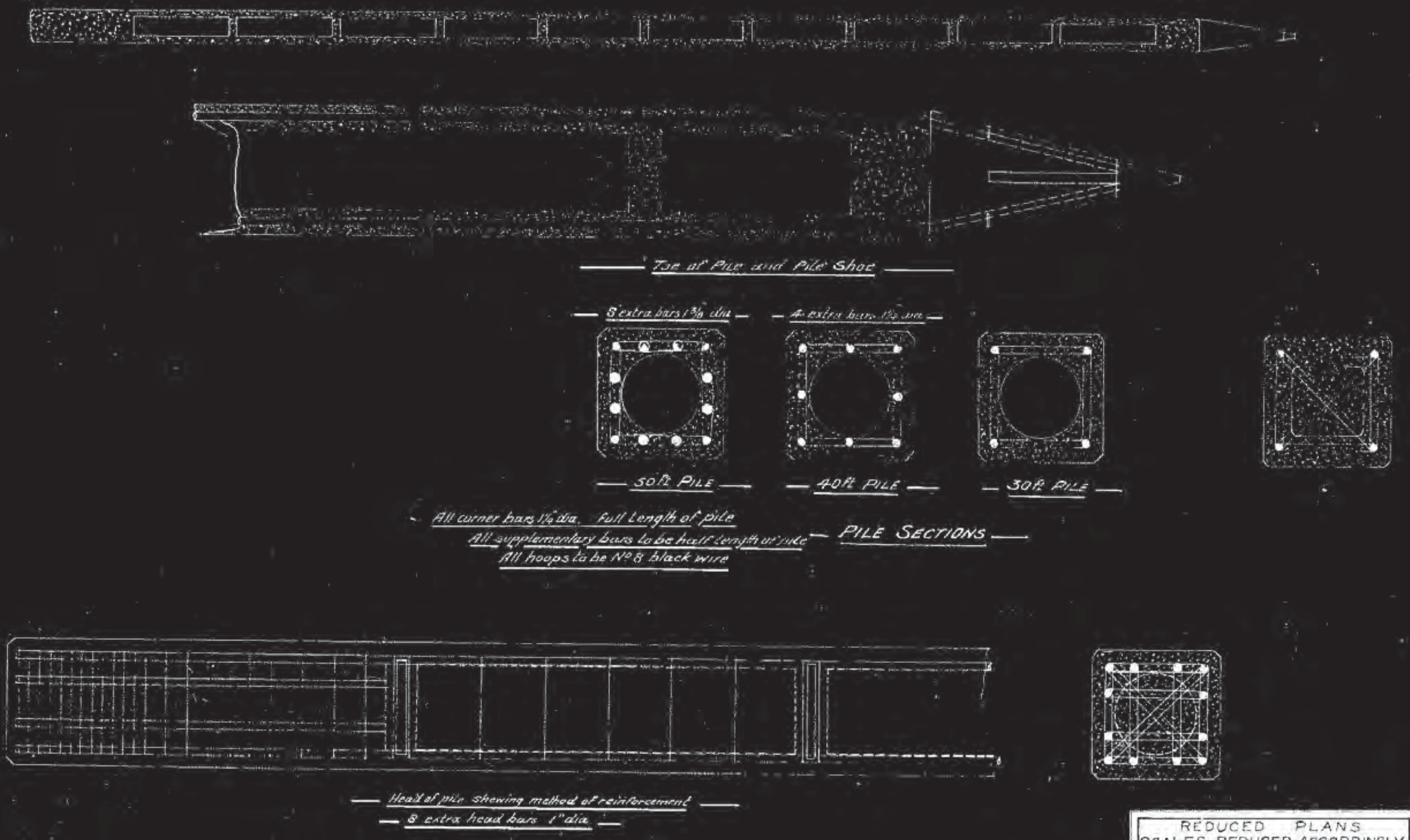
5192-4

MANGERE BRIDGE

PILE DETAIL PLAN

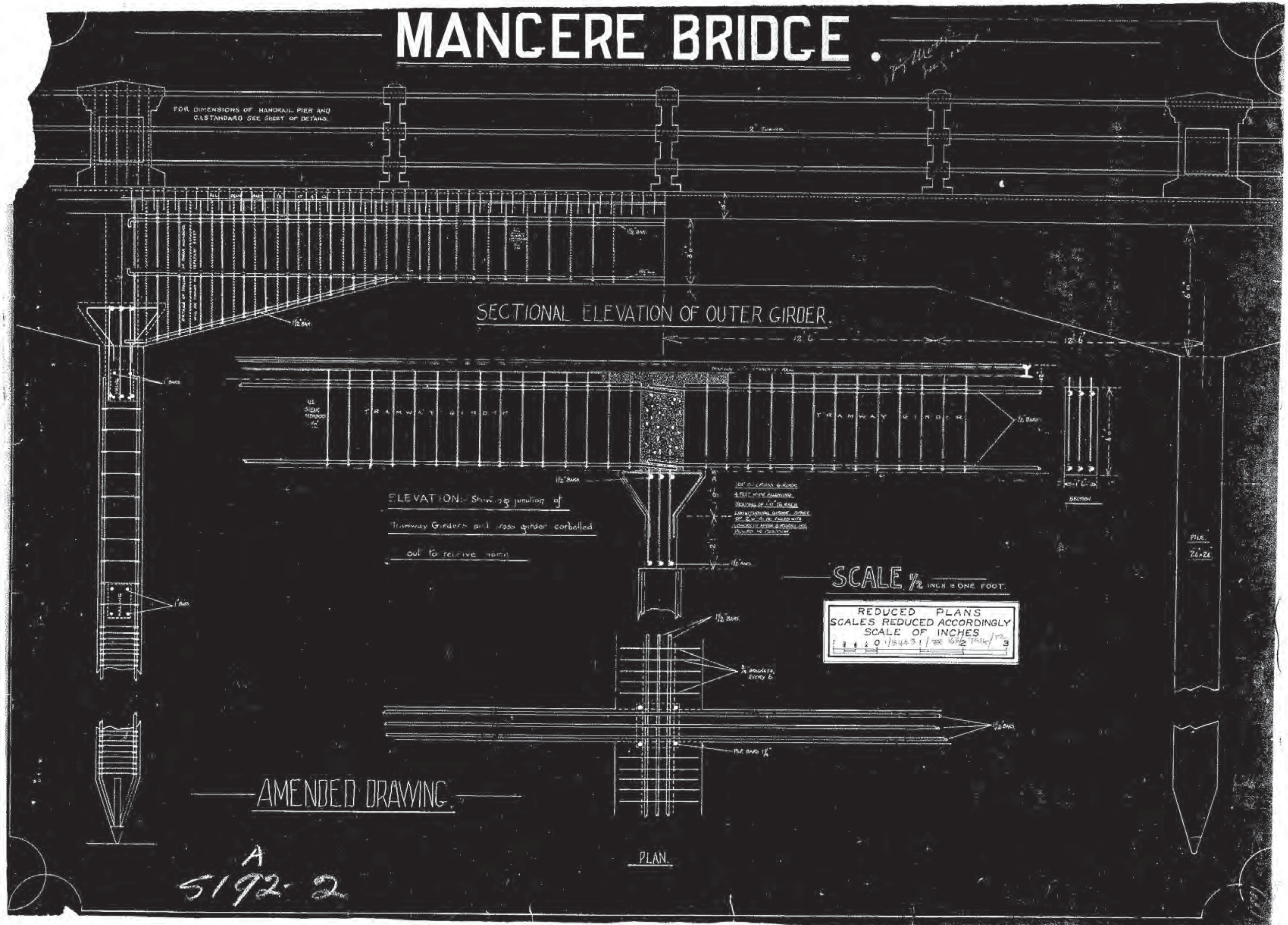
SCALE ¼" AND 1"

R. F. Moore C.E.
18/9/11

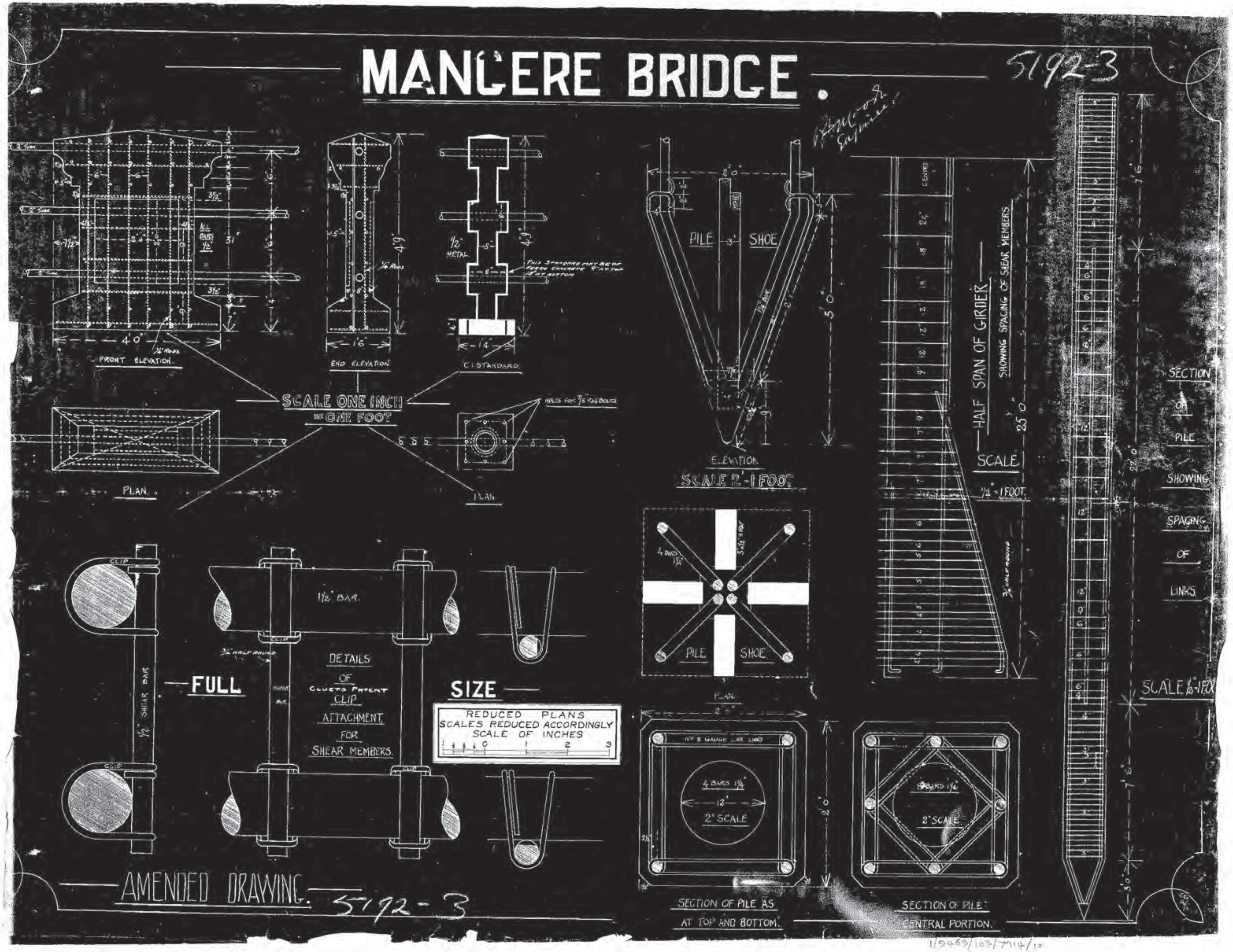


REDUCED PLANS
SCALES REDUCED ACCORDINGLY
SCALE OF INCHES
0 1 2 3

5192-4



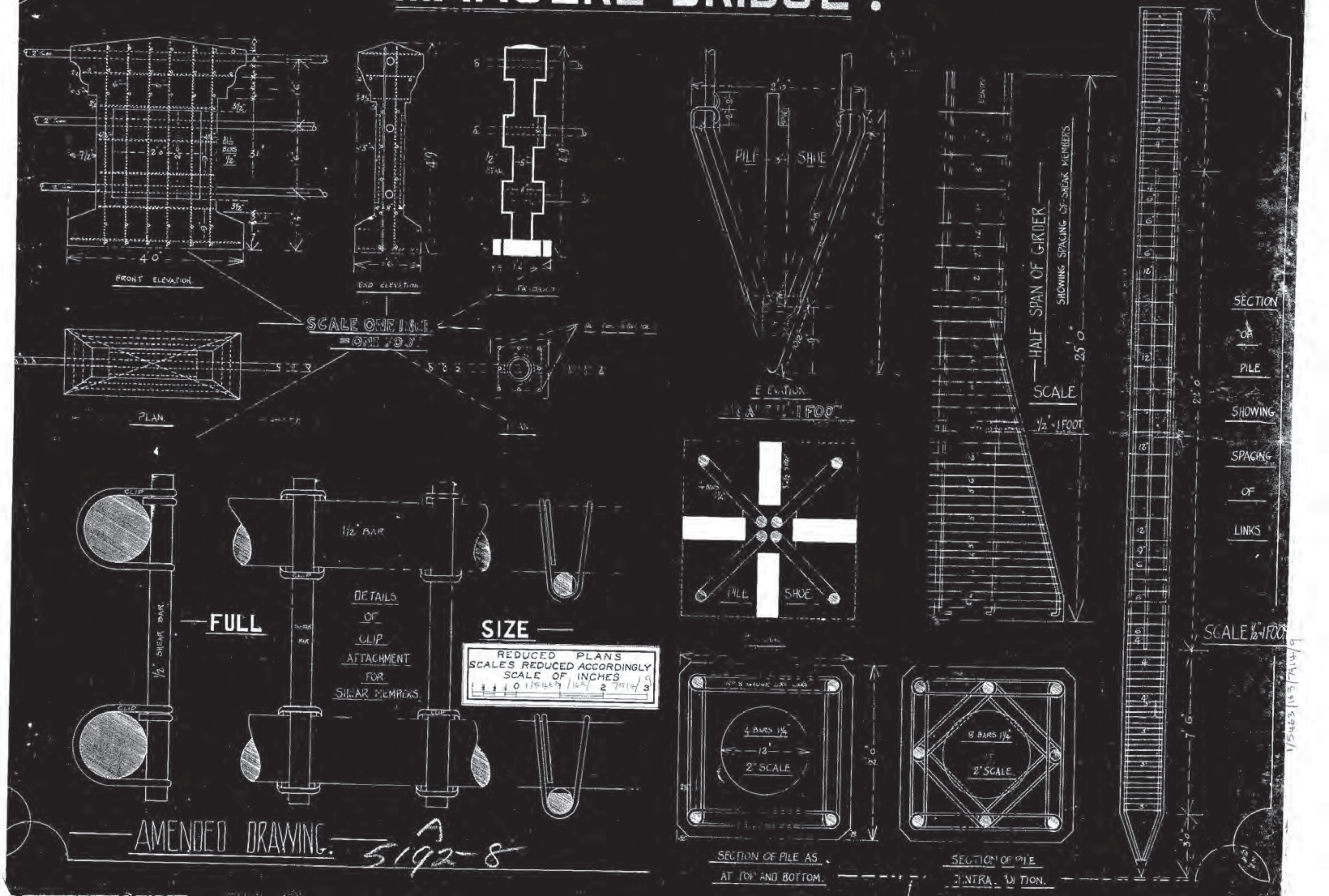
1/5463/163/7914/A2

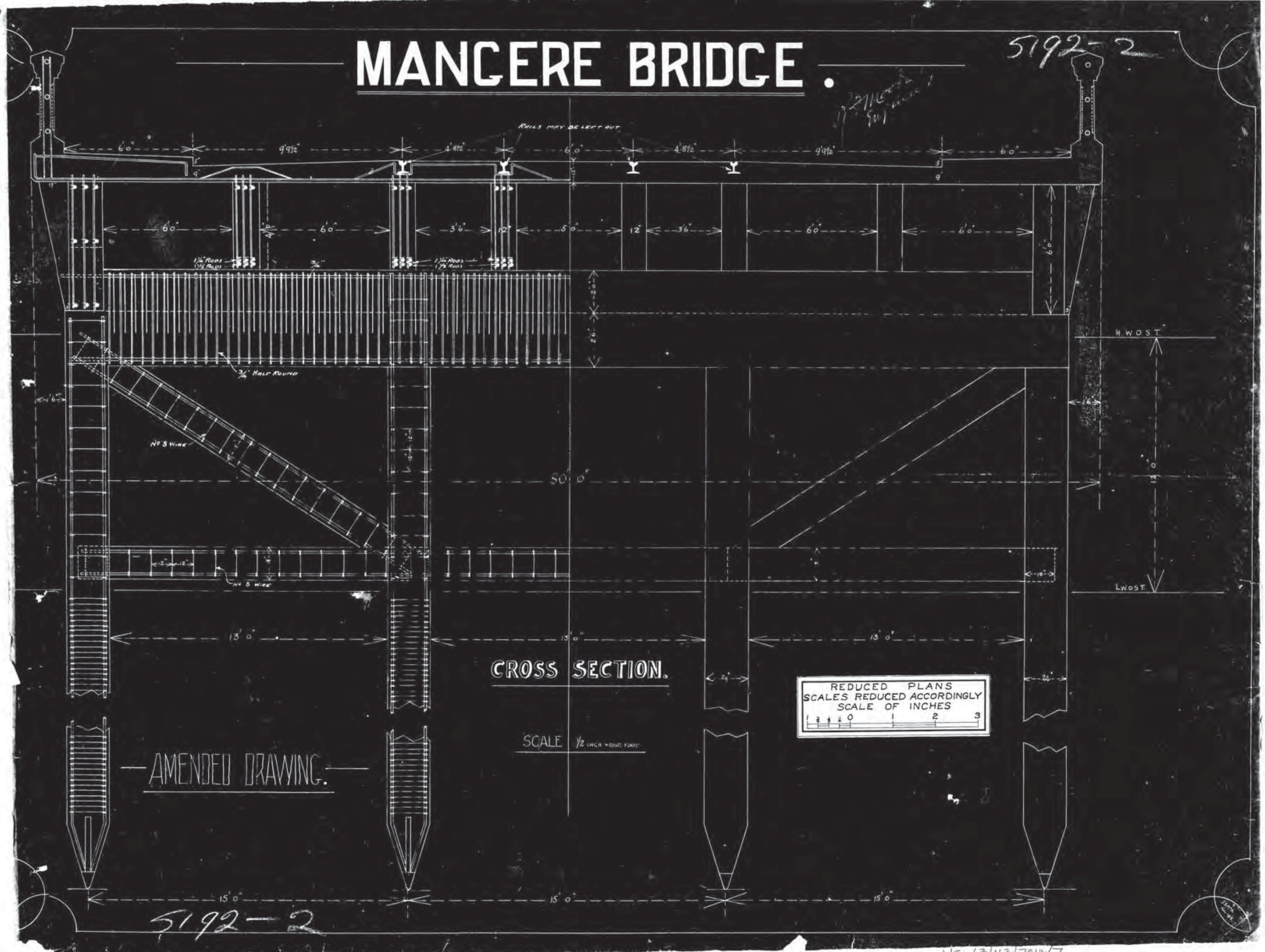


474

1/5463/163/7914/10

MANCERE BRIDGE.



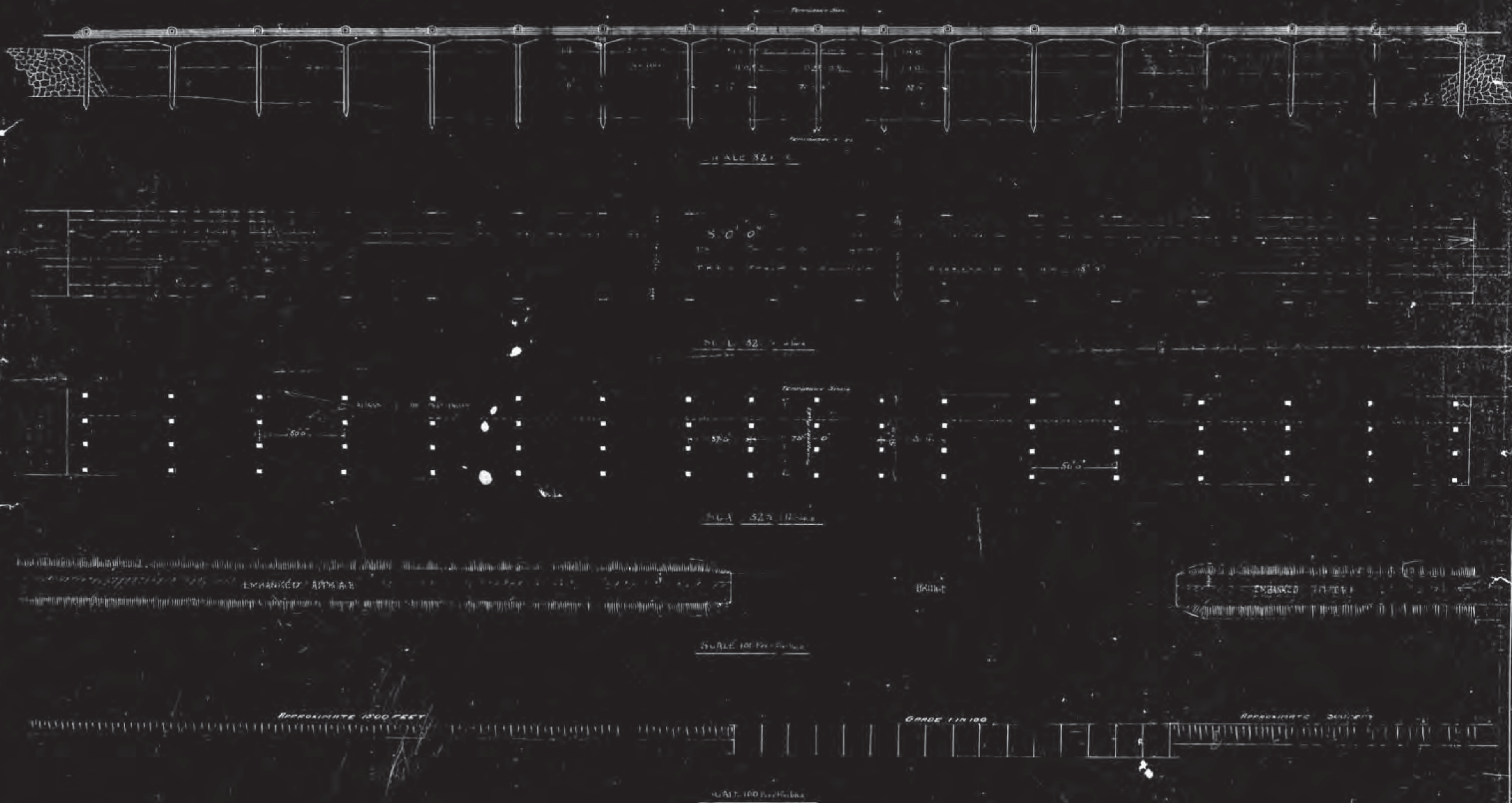


1/54 63/163/7914/7

MANCERE BRIDGE.

AMEND'D DRAWING.

5192-1



REDUCED PLANS
SCALES REDUCED ACCORDINGLY
SCALE OF INCHES

5192-1

474

1/5463/163/7914/4

— MANGERE BRIDGE —

— PLAN SHEWING LENGTH OF PILES & DEPTH DRIVEN —

— Scale 30 feet to an Inch. —

Note.
Weight of hammer 4 tons 15 cwt.
Final set ¼" to blow 6'-0" drop.

REDUCED PLANS
SCALES REDUCED ACCORDINGLY
SCALE OF INCHES
1 ¾ ½ ¼ 0 1 2 3

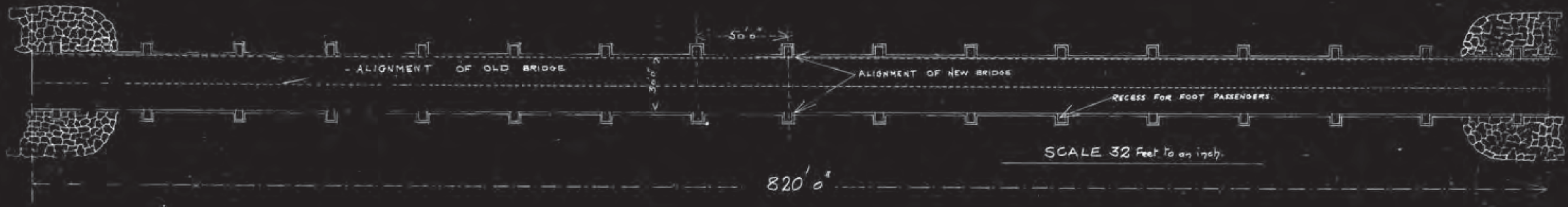
A
5192-11

1/5463/163/7914/2

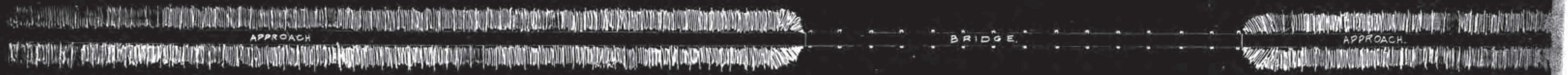
MANGERE BRIDGE

AMENDED PLAN SHOWING ALTERATION IN ALIGNMENTS OF OLD AND NEW BRIDGES

R.J. Moore C.E.
9th July 1912



SCALE 32 Feet to an inch.



SCALE 100 Feet to an inch.

REDUCED PLANS
SCALES REDUCED ACCORDINGLY
SCALE OF INCHES
0 1 2 3

A
5192-9

1/5463/163/7914/1