EARTHQUAKE RESPONSE OF AVONDALE ROAD BRIDGE DUE TO LIQUEFACTION INDUCED LATERAL SPREADING

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Abstract

This project intends to develop and analyze the computational model of Avondale Road Bridge that spans over the Avon River in Christchurch, New Zealand. The computational model produces the response of the soil structure.

Background/Introduction

On February 2011 at approximately 12:51 P.M. Christchurch, New Zealand experienced a M_W 6.2 earthquake generating peak ground accelerations(PGA) up to 0.80g where the city suffered large amounts of irreparable damages. The large peak ground accelerations exceeded the designs used (NZS 1170.5:2004), causing enormous damage. Many structures were beyond repair and Christchurch was destroyed, Figure 2a, but a huge effort from New Zealand is being put into rebuilding Christchurch for the future. You can see the large amounts of quakes that shook Christchurch in Figure 1 below.

Liquefaction induced lateral spreading was the primary cause of failure to the bridges that crossed the Avon River. The improvement and research of the bridge's foundation is the main purpose of this paper.

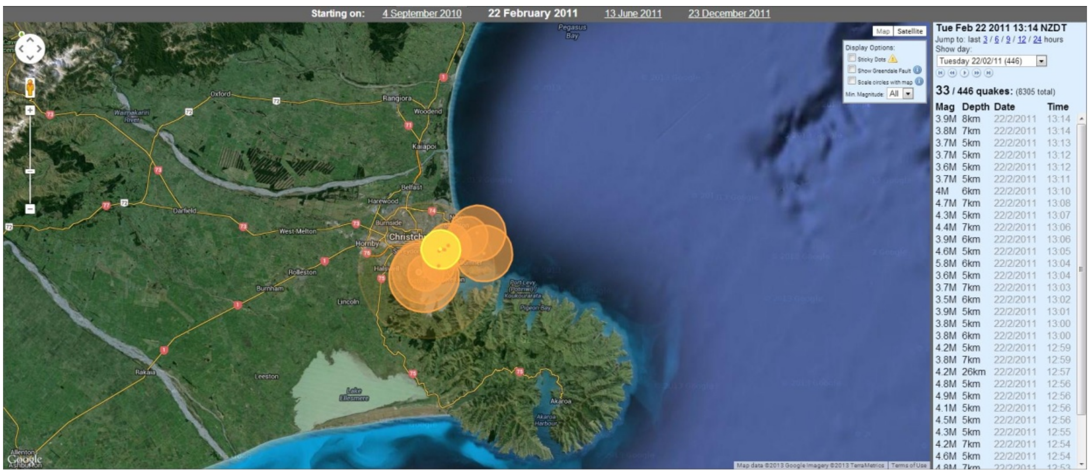


Figure 1: List of quakes in the Christchurch region on February 22, 2011

The Avondale bridge that crosses the Avon River provided on Figure 2b spanned 36 meters and has a width of 12.8 m. The bridge was constructed with 3 spans of precast concrete beams that spanned 12 m, 12.4m and 12m. The complete superstructure is supported by two abutments and two three pile caps . Each abutment is supported by 7 piles, of which 4 are vertical and 3 are at a 14^{o} angle.

Figure 2a: Photo showing the back-tilted southern abutment of the Avondale bridge and the damage to the deck

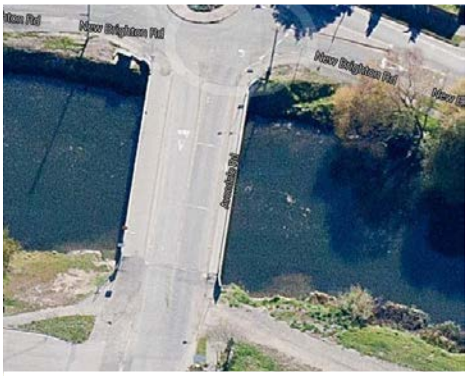


Figure 2b: Overview of the Avondale Bridge

Objective

The objective of this research is to create a computer model of the Avondale Bridge and test its validity by running the analysis of the bridge under the both the Christchurch and Darfield earthquakes with 3 different peak ground accelerations. Upon completion of verifying the initial model, alterations will made on each piles of the bridge to develop improvements of the structure.

Methods

Beam Spring Model

There are various approaches for seismic geotechnical analysis ranging from a simple pseudo-static analysis to sophisticated numerical procedures for dynamic analysis of soil structure systems. In this case, the former was used to analyse the Avondale Road bridge. The model was created in Ruaumoko, a New Zealand based structural analysis program. The pseudo-static analysis method was used on a single-pile model of both the Northern and Southern abutment for the Avondale Road bridge. A single pile was used by dividing the abutment into 4 sections; each section containing a pile, as shown in the Figure 3.

Provided in Figure 4 the pile has a square cross section with a length of 0.4064 meters, Young's Modulus of 29,730,000 kPa, and a moment of inertia of 0.002264 m^4. The moment-curvature characteristics of the pile for a specific case is shown in Figure 6. This moment-curvature behaviour was defined by the hinge properties where the cracking moment occurs at 52.91 kNm and yield moment occurs at 183 kNm and ultimate moment occurs at 205 kNm.

The pile and abutment were modelled as beam elements and the soil was modelled as non-linear spring elements, whose stiffness varied as the soil stiffness varied. The soil-pile model is shown in Figure 5b. The flowchart shown in Figure 7 shows the various soil properties and relationships that are used to determine the soil spring parameters. In order to calculate the parameters, data is gathered through Cone Penetration Testing (CPT) and Standard Penetration Testing (SPT). The calculations were done through Excel and are attached in the Appendix.

In order to test the model, ground displacements were applied to the ends of the soil springs. The displacements follow the shape shown on Figure 6c and vary with the input PGA and M_w. The springs that are not loaded with any ground displacements were made to be fixed because that region is a non-liquefied layer. There are typically 3 layers in a soil profile: crust, liquefied layer, non-liquefied layer. In the case which of the Southern Abutment model, it was loaded with an earthquake of M_W = 6.2 and PGA=0.3, there were 1 crust, 1 liquefied layer, and 1 non-liquefied layer.

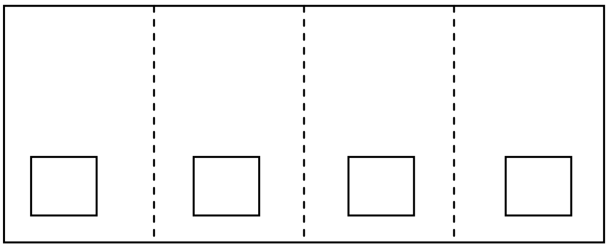


Figure 3: Sections of the Abutment

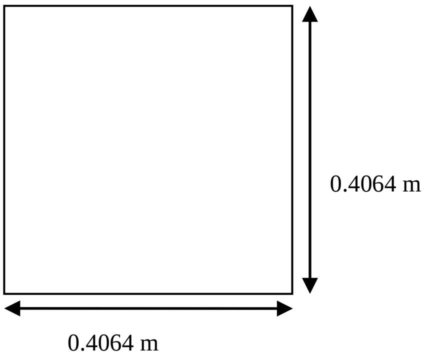


Figure 4: Cross section of the Pile (not to scale)

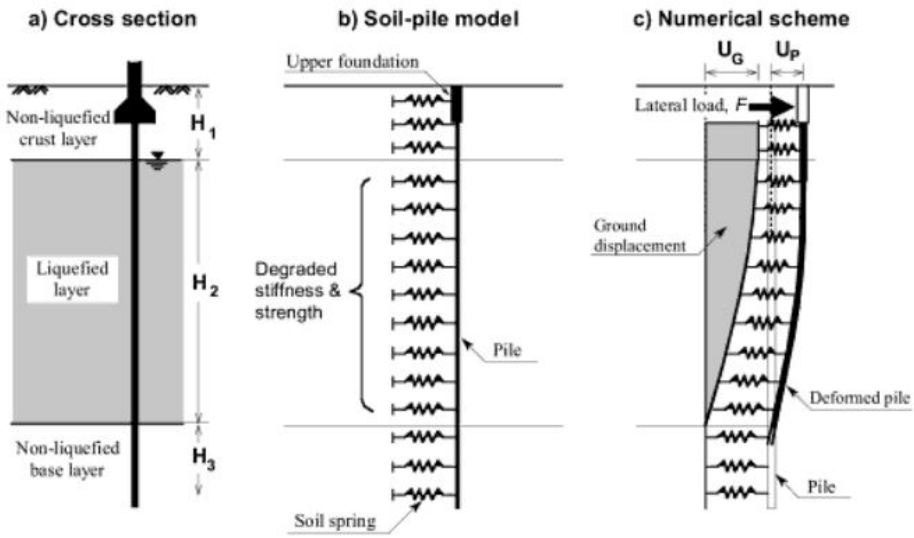


Figure 5: Typical beam-spring models for pseudo-static analysis of piles

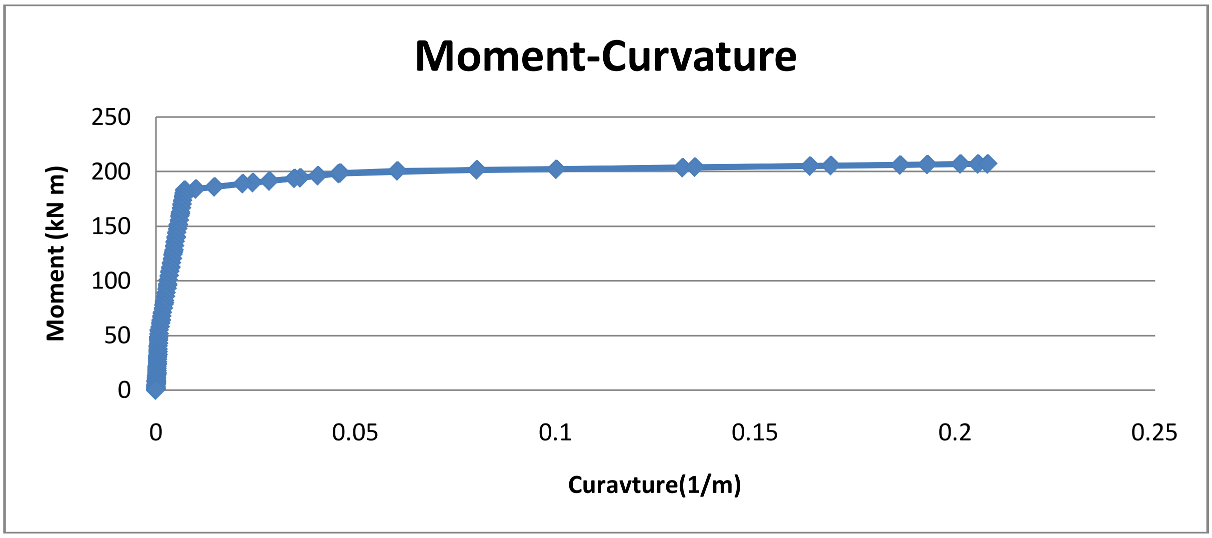


Figure 6: Moment Curvature characteristic of the modelled pile

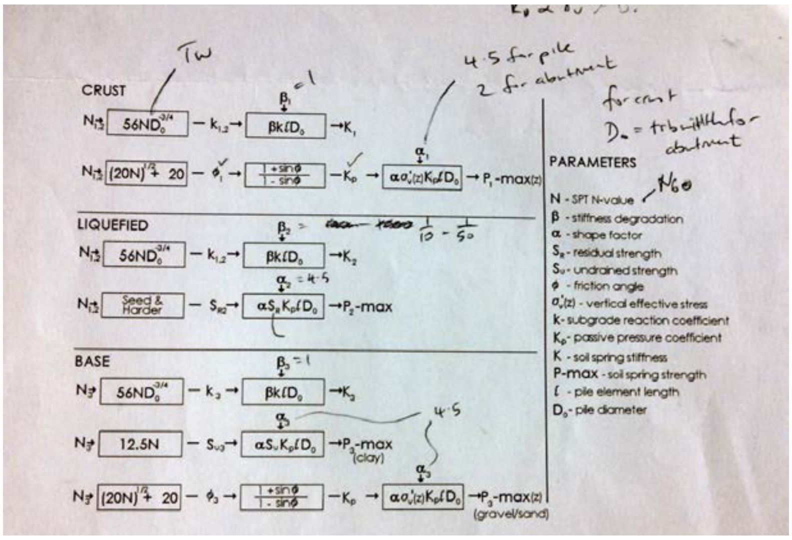


Figure 7: Flowchart showing the various soil properties and relationships that are used to determine the soil spring parameters

Sensitivity Testing

Sensitivity testing was done on the model by applying the upper and lower standard deviation of loads as well as the average. The loads are based on 2 different earthquakes. The Darfield earthquake which had a M_W of 7.1 and an average PGA of 0.18g and the Christchurch earth quake which had a M_W of 6.2 and an average of PGA 0.35g. One standard deviation above and below of the PGA were used for both earthquakes and one standard deviation above and below of the soil profiles for both the Northern and Southern abutment were also used.

Darfield Earthquake M_W 7.1		
Lower PGA 0.13g	Average PGA 0.18g	Upper PGA 0.24g

Christchurch Earthquake M_W 6.2		
Lower PGA 0.25g	Average PGA 0.35	Upper PGA 0.49g

Results

The data outputted from Ruaumoko was processed through Excel and produced the following plots on Figure 8a-e for the Southern Abutment Median Profile. These plots were a product of the model loaded with the Christchurch Earthquake and Average PGA. On Figure 8c you see that where the abutment and pile meet the bending moment far exceeds the cracking moment, where the yellow vertical line is the cracking moment and the red line is the yielding moment. There is also a very large rotation at that same connection.

The results also yielded at which depth the first cracking and first yielding occurred at both the top of the pile and the lower pile. Full results of the moment is shown in the appendix.

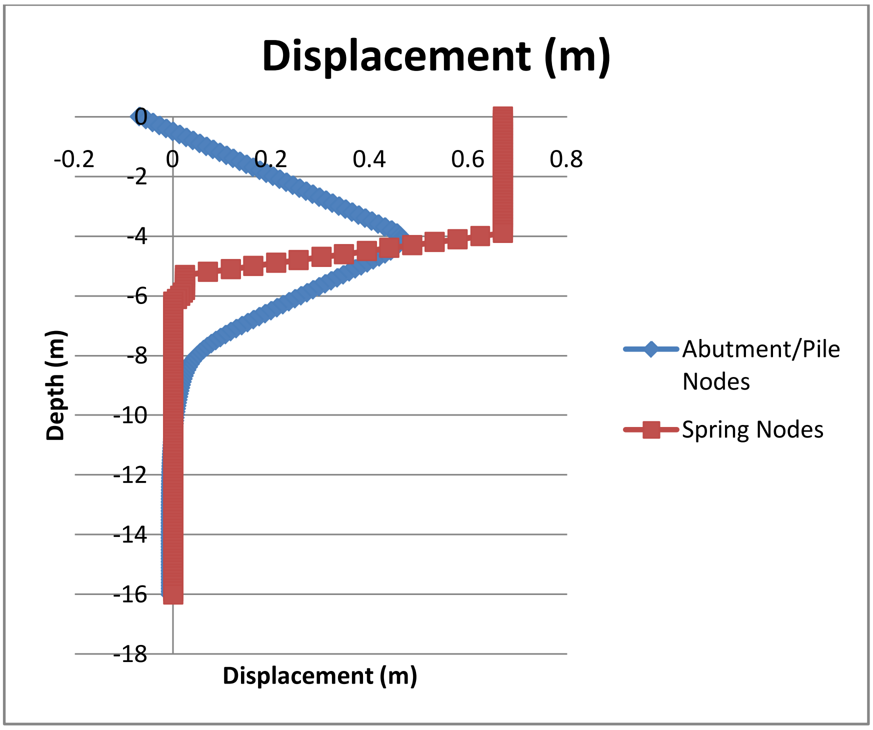


Figure 8a: Soil spring displacements and abutment/pile displacements

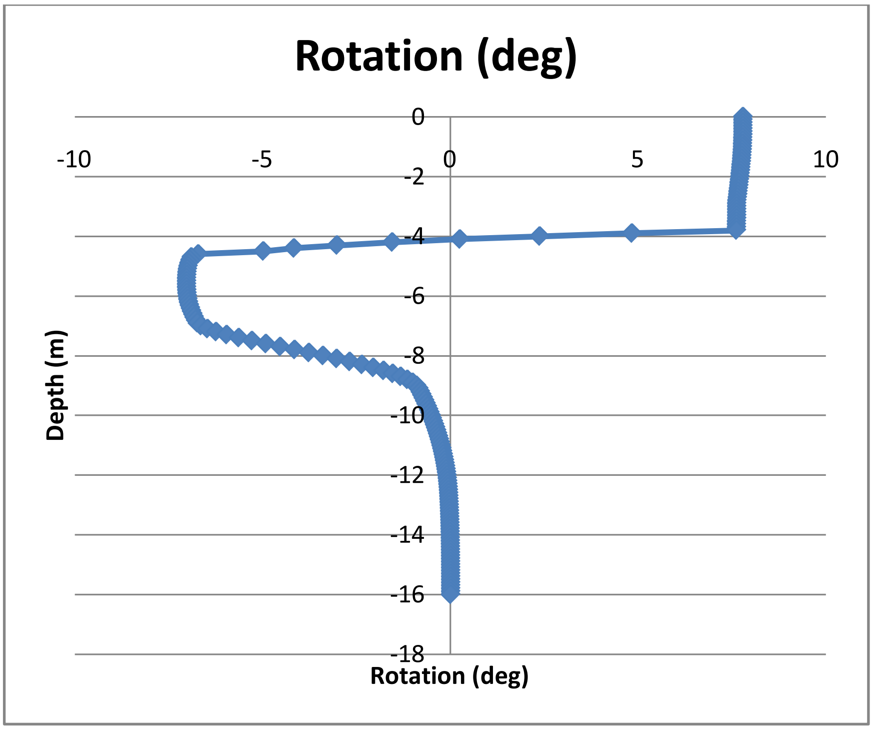


Figure 8b: Abutment/Pile rotation in degrees

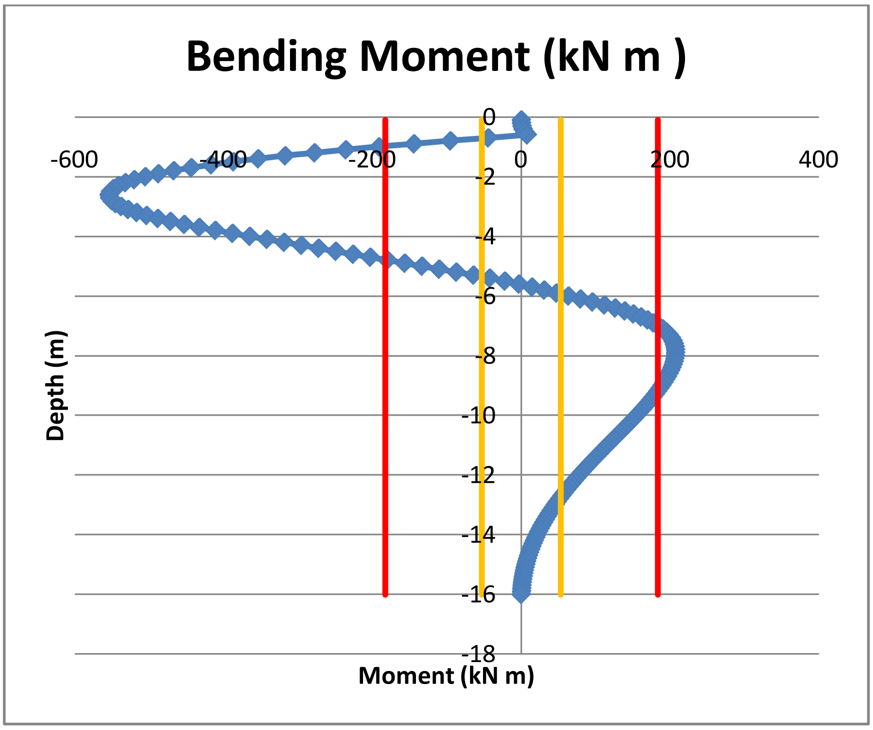


Figure 8c: Pile/Abutment bending moment

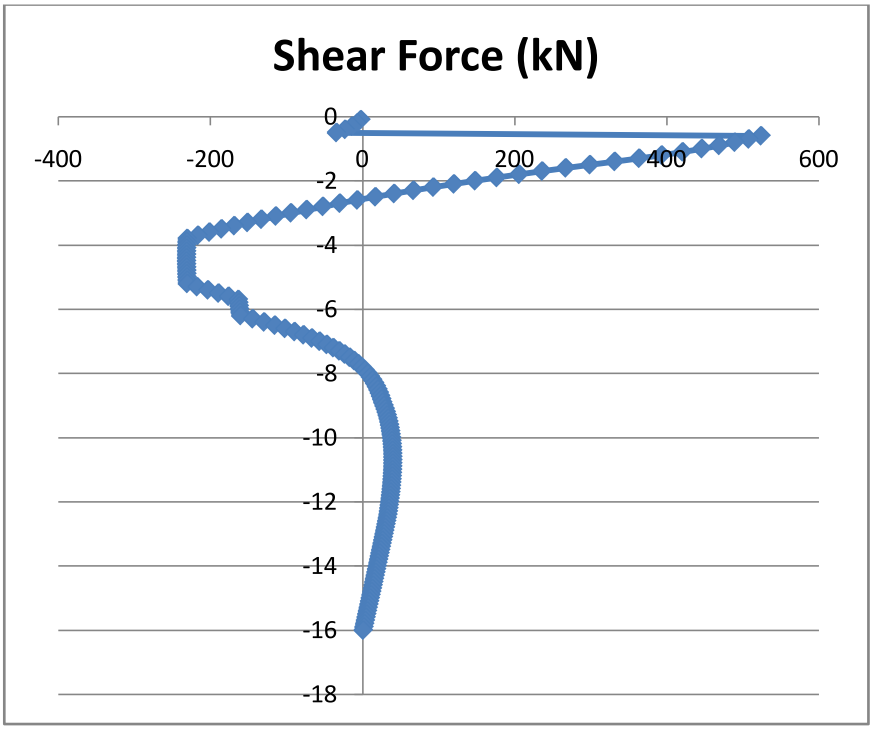


Figure 8d: Pile and Abutment Shear Force

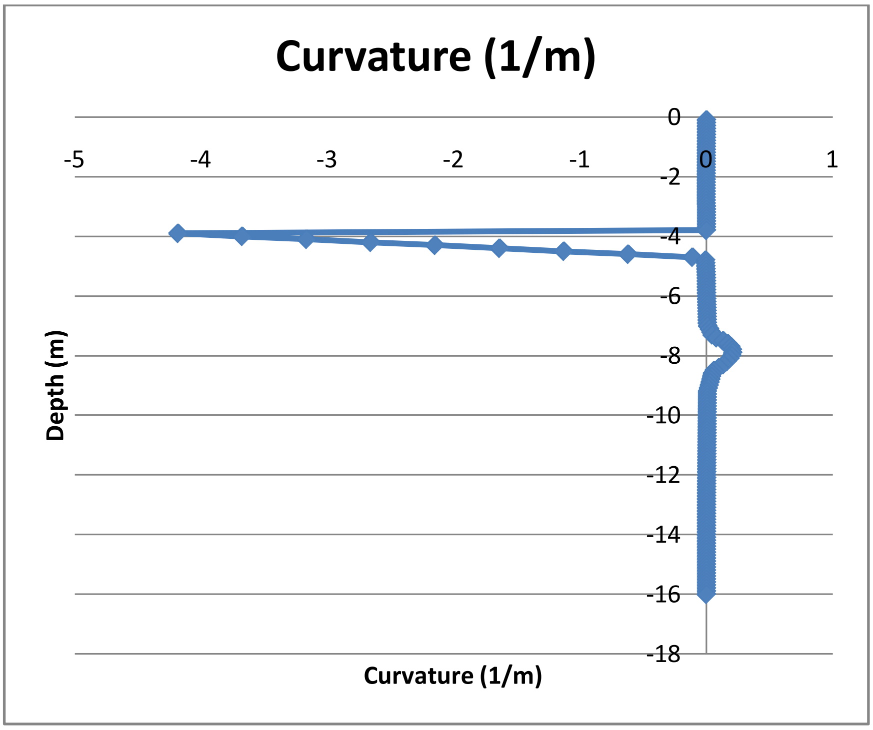


Figure 8e: Pile and Abutment Curvature

Conclusion

The series of earthquakes that shook Christchurch in February 2011 caused significant liquefaction induced lateral spreading. Much of the lateral spreading occurred along the Avon River, affecting the city business district, CBD.

Using the data collected from SPT and CPT testing and ground level displacements of each earthquake and PGA case, the following results were recorded down:

- Bending Moment
- Shear Force
- Horizontal displacement
- Rotation
- Soil Axial Force

Sensitivity analysis was done on the model in order to get a range of results for the model, increasing the sample size.

- For both Southern and Northern Abutments, 3 profiles were created: Lower, Median, Upper bound profiles
- Testing was done based on two different earthquakes: Christchurch M_W 6.1 and Darfield Earthquake M_W 7.1
- For both earthquakes, the Lower, Median, and Upper bounds were used for the peak ground acceleration

By considering the moment equilibrium of the abutment about the abutment-pile connection, major improvements can be made to the stability of the superstructure. As shown in the Bending Moment and rotation graph in the results, there is a large moment and rotation about the connection. Retrofitting has already begun on the Avondale Bridge, improving on the bridge joints. The plan details are shown in the appendix.

References

1. Google Maps
2. http://www.christchurchquakemap.co.nz/february
3. Cubrinovski, M. & Bradley (2008) Assessment of seismic performance of soil-structure systems. Soil-Structure Interaction Vol. 34, Issue 1. August 2013
4. Haskell, Jennifer. "Lateral Spreading-induced abutment rotation in the 2011 Christchurch earthquake: Observations and analysis". August 2013.
5. Idriss, I.M. and Boulanger, R.W. (2007). SPT- and CPT-based relationships for the residual shear strength of liquefied soils. *Lectures*, K. D. Pitilakis, ed., Springer, the Netherlands, 1-22. August 2013.
6. Hayden J. Bowen, Misko Cubrinovski, Pseudo-Static analysis of piles in Liquefiable Soils: Parametric evaluation of liquefied layer properties, No. 4, August 2013

Appendix

Soil Profile Characteristics

WT	3.46	m
Abutment Height	3.8	m
Abutment Top RL	3.71	m
Pile Width	40.6	cm
Spacing	0.1	m

gammadry 17; gamma 19

(Zhang et al. 2004) $D_r = 16\cdot\sqrt{(N_1)_{78}} - 14\cdot\sqrt{(N_1)_{60}} \quad [(N_1)_{60} \leq 42]$

Kayen et al. (1992) $C_N = 2.2/(1.2 + \sigma'_{vo}/P_a)$

Hatanaka & Uchida (1996) $\phi' = [15.4(N_1)_{60}]^{0.5} + 20^\circ$

RL (m)	Depth (m)	N (blows/300mm)	N60 (blows/300mm)	gamma (kN/m3)	sigv0 (kPa)	u (kPa)	sigdashv0 (kPa)	Fines Content (%)	Plastic (y/n)	Liquefiable (y/n)	Dr (%)	N1 (blows/300mm)	N160 (blows/300mm)	Friction Angle (degrees)	Modulus of Subgrade Reaction (kN/m3)
3.8	0		9	17	0	0	0	30	n	N	56.8683	0	16.5	35.94051442	30705.1
3.7	0.1		9	17	1.7	0	1.7	30	n	N	56.4747	0	16.27243226	35.8302071	30269.63
3.6	0.2		9	17	3.4	0	3.4	30	n	N	56.0893	0	16.05105634	35.72215849	29846.96
3.5	0.3		9	17	5.1	0	5.1	30	n	N	55.7116	0	15.83562293	35.61629255	29436.53
3.4	0.4		9	17	6.8	0	6.8	30	n	N	55.3414	0	15.62589592	35.51253677	29037.81
3.3	0.5		9	17	8.5	0	8.5	30	n	N	54.9786	0	15.42165155	35.41082197	28650.29
3.2	0.6		9	17	10.2	0	10.2	30	n	N	54.6227	0	15.2226776	35.3110821	28273.52
3.1	0.7		9	17	11.9	0	11.9	30	n	N	54.2737	0	15.02877267	35.21325406	27907.03
3	0.8		9	17	13.6	0	13.6	30	n	N	53.9313	0	14.83974549	35.11727755	27550.41
2.9	0.9		9	17	15.3	0	15.3	30	n	N	53.5953	0	14.65541429	35.02309489	27203.27
2.8	1		9	17	17	0	17	30	n	N	53.2656	0	14.47560624	34.93065089	26865.22
2.7	1.1		9	17	18.7	0	18.7	30	n	N	52.9418	0	14.30015685	34.83989271	26535.9
2.6	1.2		9	17	20.4	0	20.4	30	n	N	52.6238	0	14.12890955	34.75076971	26214.97
2.5	1.3		9	17	22.1	0	22.1	30	n	N	52.3115	0	13.96171516	34.66323339	25902.12
2.4	1.4		9	17	23.8	0	23.8	30	n	N	52.0047	0	13.79843148	34.57723721	25597.04
2.3	1.5		9	17	25.5	0	25.5	30	n	N	51.7033	0	13.63892289	34.49273654	25299.43
2.2	1.6		9	17	27.2	0	27.2	30	n	N	51.407	0	13.48305996	34.40968853	25009.03

Figure A1a: Soil Profile Characteristics used for calculations

Liquefaction Triggering Calculations

Mw	6.2	
PGA	0.25	g
MSF	1.627924	(Andrus & Stokoe 2000)

Cyclic Stress Ratio (Youd el at 2001)

$$CSR = \frac{\tau_{av}}{\sigma'_v} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right)r_d$$

(Youd el al 2001)

$\alpha = 0$ for FC ≤ 5%; $\alpha = 5.0$ for FC ≥ 35%; $\beta = 1.0$ for FC ≤ 5%; $\beta = 1.2$ for FC ≥ 35%

Confining Stress Hynes & Olsen (1999)

$f = 0.7-0.8$ for D_r ~40-60%

$f = 0.6-0.7$ for D_r ~60-80%

(Youd et al 2001)

(Youd et al 2001)

CRR/CSR

Cyclic Stress Ratio: Depth (m)	r_d	CSR	$CSR_{7.5}$	Cyclic Resistance Ratio: alpha	beta	N160cs	f	Kσ	$CRR_{7.5}$	$CRR_{7.5}$ Kσ	FOS - Liquefaction	Liquefiec
0.1	1.000671	0.162609	0.099887	4.706238	1.154317	23.48978	0.717626	1	0.173105	3	3	N
0.2	1.000208	0.162534	0.099841	4.706238	1.154317	23.23424	0.719554	1	0.170794	3	3	N
0.3	0.999585	0.162433	0.099779	4.706238	1.154317	22.98556	0.721442	1	0.168563	3	3	N
0.4	0.99889	0.16232	0.09971	4.706238	1.154317	22.74347	0.723293	1	0.166406	3	3	N
0.5	0.998156	0.1622	0.099636	4.706238	1.154317	22.50771	0.725107	1	0.16432	3	3	N
0.6	0.9974	0.162078	0.099561	4.706238	1.154317	22.27803	0.726886	1	0.162302	3	3	N
0.7	0.996631	0.161953	0.099484	4.706238	1.154317	22.0542	0.728631	1	0.160347	3	3	N
0.8	0.995854	0.161826	0.099407	4.706238	1.154317	21.83601	0.730343	1	0.158452	3	3	N
0.9	0.995074	0.161699	0.099329	4.706238	1.154317	21.62323	0.732023	1	0.156615	3	3	N
1	0.994292	0.161573	0.099251	4.706238	1.154317	21.41567	0.733672	1	0.154833	3	3	N
1.1	0.993512	0.161446	0.099173	4.706238	1.154317	21.21315	0.735291	1	0.153104	3	3	N
1.2	0.992734	0.161319	0.099095	4.706238	1.154317	21.01548	0.736881	1	0.151424	3	3	N
1.3	0.991958	0.161193	0.099018	4.706238	1.154317	20.82248	0.738442	1	0.149792	3	3	N
1.4	0.991187	0.161068	0.098941	4.706238	1.154317	20.634	0.739976	1	0.148205	3	3	N
1.5	0.99042	0.160943	0.098864	4.706238	1.154317	20.44988	0.741484	1	0.146662	3	3	N
1.6	0.989657	0.160819	0.098788	4.706238	1.154317	20.26996	0.742965	1	0.145161	3	3	N

Figure A1b: Liquefaction Trigger Calculations

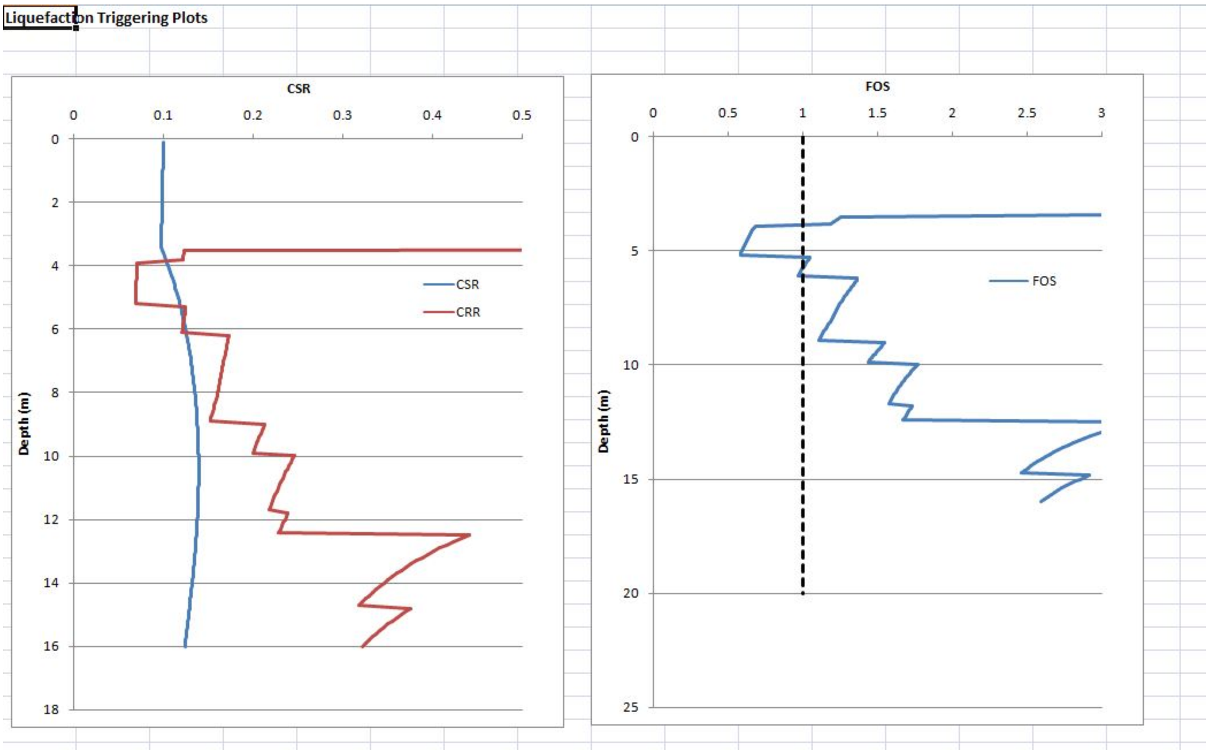
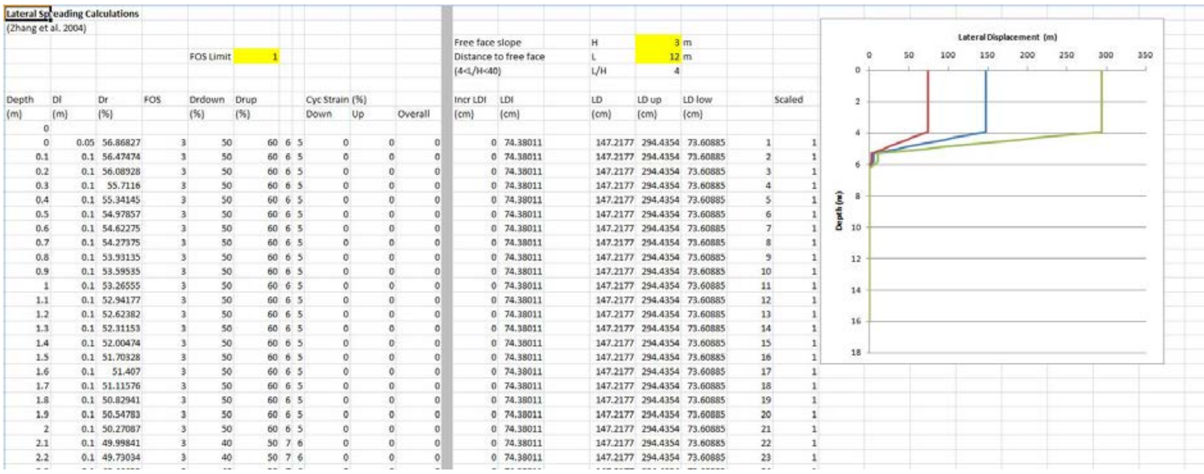


Figure A1c: Liquefaction Triggering Plots



Lateral Spreading Calculations

(Zhang et al. 2004)

FOS Limit 1

Free face slope H 3 m

Distance to free face L 12 m

(4<L/H<40) L/H 4

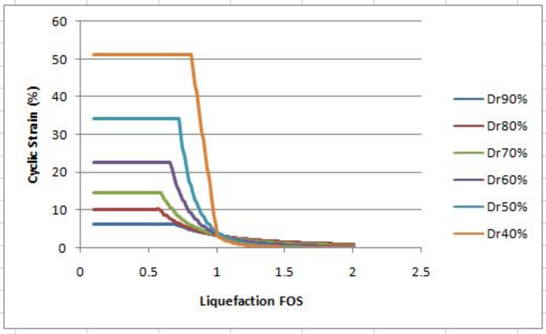
Depth (m)	Dl (m)	Dr (%)	FOS	Drdown (%)	Drup (%)			Cyc Strain (%) Down	Up	Overall	Incr LDI (cm)	LDI (cm)	LD (cm)	LD up (cm)	LD low (cm)		Scaled
0																	
0	0.05	56.86827	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	1	1
0.1	0.1	56.47474	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	2	1
0.2	0.1	56.08928	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	3	1
0.3	0.1	55.7116	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	4	1
0.4	0.1	55.34145	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	5	1
0.5	0.1	54.97857	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	6	1
0.6	0.1	54.62275	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	7	1
0.7	0.1	54.27375	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	8	1
0.8	0.1	53.93135	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	9	1
0.9	0.1	53.59535	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	10	1
1	0.1	53.26555	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	11	1
1.1	0.1	52.94177	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	12	1
1.2	0.1	52.62382	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	13	1
1.3	0.1	52.31153	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	14	1
1.4	0.1	52.00474	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	15	1
1.5	0.1	51.70328	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	16	1
1.6	0.1	51.407	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	17	1
1.7	0.1	51.11576	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	18	1
1.8	0.1	50.82941	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	19	1
1.9	0.1	50.54783	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	20	1
2	0.1	50.27087	3	50	60	6	5	0	0	0	0	74.38011	147.2177	294.4354	73.60885	21	1
2.1	0.1	49.99841	3	40	50	7	6	0	0	0	0	74.38011	147.2177	294.4354	73.60885	22	1
2.2	0.1	49.73034	3	40	50	7	6	0	0	0	0	74.38011	147.2177	294.4354	73.60885	23	1

Figure A1d: Lateral Spreading Calculations

Cyclic Strain Relationships

(Ishihara & Yoshimine 1992, Seed 1979)

	Dr (Relative Density)					
FOS	90	80	70	60	50	40
0.1	6.2	10	14.5	22.7	34.1	51.2
0.11	6.2	10	14.5	22.7	34.1	51.2
0.12	6.2	10	14.5	22.7	34.1	51.2
0.13	6.2	10	14.5	22.7	34.1	51.2
0.14	6.2	10	14.5	22.7	34.1	51.2
0.15	6.2	10	14.5	22.7	34.1	51.2
0.16	6.2	10	14.5	22.7	34.1	51.2
0.17	6.2	10	14.5	22.7	34.1	51.2
0.18	6.2	10	14.5	22.7	34.1	51.2
0.19	6.2	10	14.5	22.7	34.1	51.2
0.2	6.2	10	14.5	22.7	34.1	51.2
0.21	6.2	10	14.5	22.7	34.1	51.2
0.22	6.2	10	14.5	22.7	34.1	51.2
0.23	6.2	10	14.5	22.7	34.1	51.2
0.24	6.2	10	14.5	22.7	34.1	51.2
0.25	6.2	10	14.5	22.7	34.1	51.2
0.26	6.2	10	14.5	22.7	34.1	51.2
0.27	6.2	10	14.5	22.7	34.1	51.2
0.28	6.2	10	14.5	22.7	34.1	51.2
0.29	6.2	10	14.5	22.7	34.1	51.2
0.3	6.2	10	14.5	22.7	34.1	51.2
0.31	6.2	10	14.5	22.7	34.1	51.2
0.32	6.2	10	14.5	22.7	34.1	51.2
0.33	6.2	10	14.5	22.7	34.1	51.2
0.34	6.2	10	14.5	22.7	34.1	51.2
0.35	6.2	10	14.5	22.7	34.1	51.2



$$\text{if } D_r = 90\%, \quad \gamma_{max} = 3.26(\text{FS})^{-1.80} \quad \text{for } 0.7 \leq \text{FS} \leq 2.0 \qquad (8)$$

$$\text{if } D_r = 90\%, \quad \gamma_{max} = 6.2 \quad \text{for } \text{FS} \leq 0.7 \qquad (9)$$

$$\text{if } D_r = 80\%, \quad \gamma_{max} = 3.22 \cdot (\text{FS})^{-2.08} \quad \text{for } 0.56 \leq \text{FS} \leq 2.0 \qquad (10)$$

$$\text{if } D_r = 80\%, \quad \gamma_{max} = 10 \quad \text{for } \text{FS} \leq 0.56 \qquad (11)$$

$$\text{if } D_r = 70\%, \quad \gamma_{max} = 3.20 \cdot (\text{FS})^{-2.89} \quad \text{for } 0.59 \leq \text{FS} \leq 2.0 \qquad (12)$$

$$\text{if } D_r = 70\%, \quad \gamma_{max} = 14.5 \quad \text{for } \text{FS} \leq 0.59 \qquad (13)$$

$$\text{if } D_r = 60\%, \quad \gamma_{max} = 3.58 \cdot (\text{FS})^{-4.42} \quad \text{for } 0.66 \leq \text{FS} \leq 2.0 \qquad (14)$$

$$\text{if } D_r = 60\%, \quad \gamma_{max} = 22.7 \quad \text{for } \text{FS} \leq 0.66 \qquad (15)$$

Figure A1e: Cyclic Strain Relationships

Beta	1	Alpha (abut)	2	Tw(m)	1.95
Beta(liq)	0.05	Alpha (pile)	4.5	Pile Width	0.406
				Spacing	0.1

			Crust					
Depth	Liquefied?	Liquefied?	k1,2	K	phi1	phi(rad)	Kp	P1max
0.1	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	2.288388
0.2	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	4.576775
0.3	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	6.865163
0.4	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	9.15355
0.5	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	11.44194
0.6	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	13.73033
0.7	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	16.01871
0.8	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	18.3071
0.9	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	20.59549
1	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	22.88388
1.1	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	25.17226
1.2	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	27.46065
1.3	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	29.74904
1.4	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	32.03743
1.5	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	34.32581
1.6	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	36.6142
1.7	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	38.90259
1.8	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	41.19098
1.9	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	43.47936
2	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	45.76775
2.1	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	48.05614
2.2	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	50.34453
2.3	Abut	N	305.425	59.5578744	33.41640786	0.583226341	3.45156491	52.63291

	Liquefied							
Depth	k1,2	K2	phi	phi(rad)	Kp	Sr2	P2max	k3
0.1	305.425	2.977894	33.41641	0.583226	3.451565	70.07667	212.2446	
0.2	305.425	2.977894	33.41641	0.583226	3.451565	68.34437	206.9979	
0.3	305.425	2.977894	33.41641	0.583226	3.451565	66.67929	201.9548	
0.4	305.425	2.977894	33.41641	0.583226	3.451565	65.07795	197.1047	
0.5	305.425	2.977894	33.41641	0.583226	3.451565	63.53707	192.4378	
0.6	305.425	2.977894	33.41641	0.583226	3.451565	62.05362	187.9448	
0.7	305.425	2.977894	33.41641	0.583226	3.451565	60.62473	183.617	
0.8	305.425	2.977894	33.41641	0.583226	3.451565	59.24772	179.4464	
0.9	305.425	2.977894	33.41641	0.583226	3.451565	57.92006	175.4253	
1	305.425	2.977894	33.41641	0.583226	3.451565	56.6394	171.5465	
1.1	305.425	2.977894	33.41641	0.583226	3.451565	55.4035	167.8033	
1.2	305.425	2.977894	33.41641	0.583226	3.451565	54.21027	164.1893	
1.3	305.425	2.977894	33.41641	0.583226	3.451565	53.05773	160.6985	
1.4	305.425	2.977894	33.41641	0.583226	3.451565	51.94404	157.3254	
1.5	305.425	2.977894	33.41641	0.583226	3.451565	50.86742	154.0646	
1.6	305.425	2.977894	33.41641	0.583226	3.451565	49.82623	150.9111	
1.7	305.425	2.977894	33.41641	0.583226	3.451565	48.8189	147.8602	
1.8	305.425	2.977894	33.41641	0.583226	3.451565	47.84396	144.9073	
1.9	305.425	2.977894	33.41641	0.583226	3.451565	46.90001	142.0483	
2	305.425	2.977894	33.41641	0.583226	3.451565	45.98572	139.2792	
2.1	305.425	2.977894	33.41641	0.583226	3.451565	45.09985	136.5961	
2.2	305.425	2.977894	33.41641	0.583226	3.451565	44.2412	133.9955	
2.3	305.425	2.977894	33.41641	0.583226	3.451565	43.40866	131.4739	

FigureA1f: Soil Stiffness Calculations

Time	100.00
Mcrack	52.90761
Myield	183
Abut Rot	7.5

Abutment Rotation	
Time (s)	53
Rotation (deg)	7.620338675

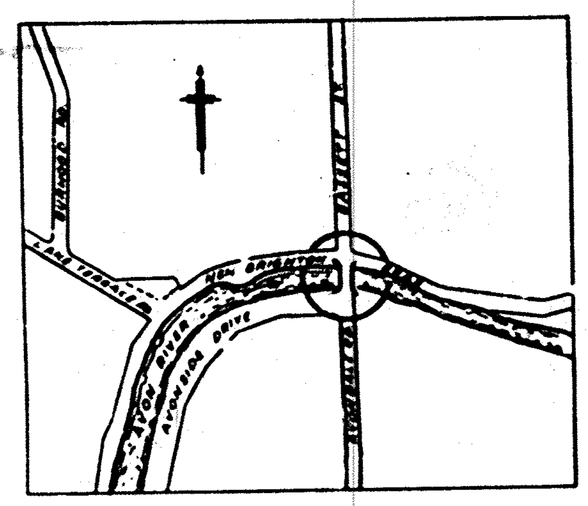
Top of Pile			
	Depth (m)	Time Step (s)	Moment (kN m)
First Cracking	-3.9	4	-56.04
First Yield	-3.9	15	-187.6
Lower Pile			
First Cracking	-10.1	17	54.47
First Yielding	-10	35	183.3

		Abutment/Pile Nodes		Spring Nodes	
Node	Depth (m)	Displacement (m)	Rotation (deg)	Node	Displacement (m)
1	0	-0.1533	17.566886	162	1
2	-0.1	-0.1226	17.566886	163	1
3	-0.2	-0.09198	17.566886	164	1
4	-0.3	-0.06132	17.566886	165	1
5	-0.4	-0.03066	17.566886	166	1
6	-0.5	0	17.566886	167	1
7	-0.6	0.03067	17.566886	168	1
8	-0.7	0.06134	17.566886	169	1
9	-0.8	0.092	17.56115642	170	1
10	-0.9	0.1227	17.56115642	171	1
11	-1	0.1533	17.55542684	172	1
12	-1.1	0.184	17.54969726	173	1
13	-1.2	0.2146	17.54969726	174	1
14	-1.3	0.2452	17.53823811	175	1
15	-1.4	0.2758	17.53250853	176	1
16	-1.5	0.3064	17.52677895	177	1
17	-1.6	0.337	17.52104938	178	1
18	-1.7	0.3676	17.50959022	179	1
19	-1.8	0.3982	17.50386064	180	1
20	-1.9	0.4287	17.49240149	181	1
21	-2	0.4592	17.48094233	182	1
22	-2.1	0.4897	17.47521275	183	1
23	-2.2	0.5202	17.4637536	184	1
24	-2.3	0.5507	17.45229444	185	1
25	-2.4	0.5811	17.44083528	186	1
26	-2.5	0.6116	17.43510571	187	1
27	-2.6	0.642	17.42364655	188	1
28	-2.7	0.6724	17.41218739	189	1
29	-2.8	0.7028	17.40645782	190	1
30	-2.9	0.7331	17.39499866	191	1
31	-3	0.7635	17.39499866	192	1
32	-3.1	0.7939	17.39499866	193	1

ABUTMENT

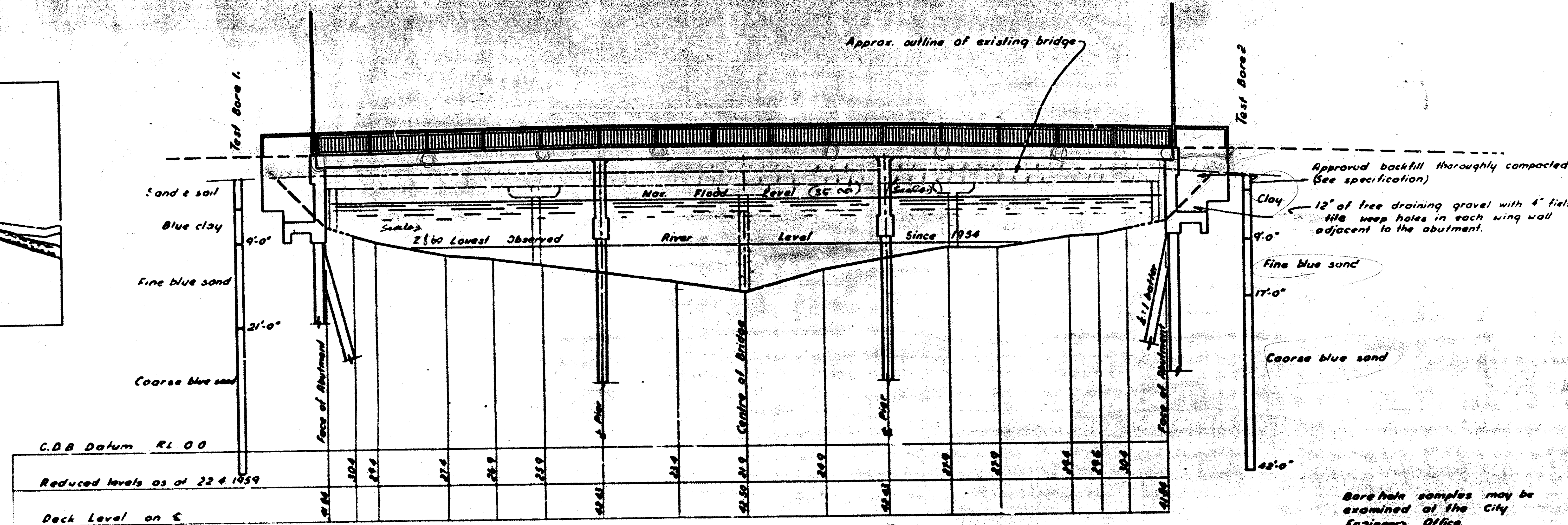
Abutment/Pile Frame Elements					Spring Elements		
Element	Depth (m)	Shear Force (kN)	Bending Moment (kN m)	Curvature (1/m)	Element	Depth (m)	Force (kN)
1	-0.1	-2.295	3.69E-13	1.122E-18	161	0	-2.295
2	-0.2	-6.878	0.2295	0.000000698	162	-0.1	-4.583
3	-0.3	-13.75	0.9173	0.00000279	163	-0.2	-6.871
4	-0.4	-22.91	2.292	0.000006971	164	-0.3	-9.159
5	-0.5	-34.35	4.583	0.00001394	165	-0.4	-11.45
6	-0.6	548.9	8.018	0.00002439	166	-0.5	-13.73
7	-0.7	532.9	-46.87	-0.0001426	167	-0.6	-16.02
8	-0.8	514.6	-100.2	-0.0003046	168	-0.7	-18.31
9	-0.9	494	-151.6	-0.0004611	169	-0.8	-20.6
10	-1	471.1	-201	-0.0006114	170	-0.9	-22.89
11	-1.1	445.9	-248.1	-0.0007546	171	-1	-25.17
12	-1.2	418.5	-292.7	-0.0008903	172	-1.1	-27.46
13	-1.3	388.7	-334.6	-0.001018	173	-1.2	-29.75
14	-1.4	356.7	-373.4	-0.001136	174	-1.3	-32.04
15	-1.5	322.3	-409.1	-0.001244	175	-1.4	-34.33
16	-1.6	285.7	-441.3	-0.001342	176	-1.5	-36.61
17	-1.7	246.8	-469.9	-0.001429	177	-1.6	-38.9
18	-1.8	209.2	-494.6	-0.001504	178	-1.7	-37.66
19	-1.9	173.3	-515.5	-0.001568	179	-1.8	-35.84
20	-2	139.3	-532.8	-0.001621	180	-1.9	-34.03
21	-2.1	107.1	-546.8	-0.001663	181	-2	-32.21
22	-2.2	76.69	-557.5	-0.001695	182	-2.1	-30.39
23	-2.3	48.11	-565.1	-0.001719	183	-2.2	-28.58
24	-2.4	21.35	-570	-0.001733	184	-2.3	-26.76
25	-2.5	-3.592	-572.1	-0.00174	185	-2.4	-24.95
26	-2.6	-26.73	-571.7	-0.001739	186	-2.5	-23.13
27	-2.7	-48.05	-569.1	-0.001731	187	-2.6	-21.32
28	-2.8	-67.56	-564.3	-0.001716	188	-2.7	-19.51
29	-2.9	-85.26	-557.5	-0.001696	189	-2.8	-17.7
30	-3	-101.2	-549	-0.000026	190	-2.9	-15.89
31	-3.1	-115.2	-538.9	-0.00002552	191	-3	-14.09
32	-3.2	-127.5	-527.3	-0.00002498	192	-3.1	-12.28

Figure A2: Data processed for median southern abutment loaded with the Christchurch Earthquake and PGA of 0.35



LOCALITY PLAN

Scale 10 chains to an inch

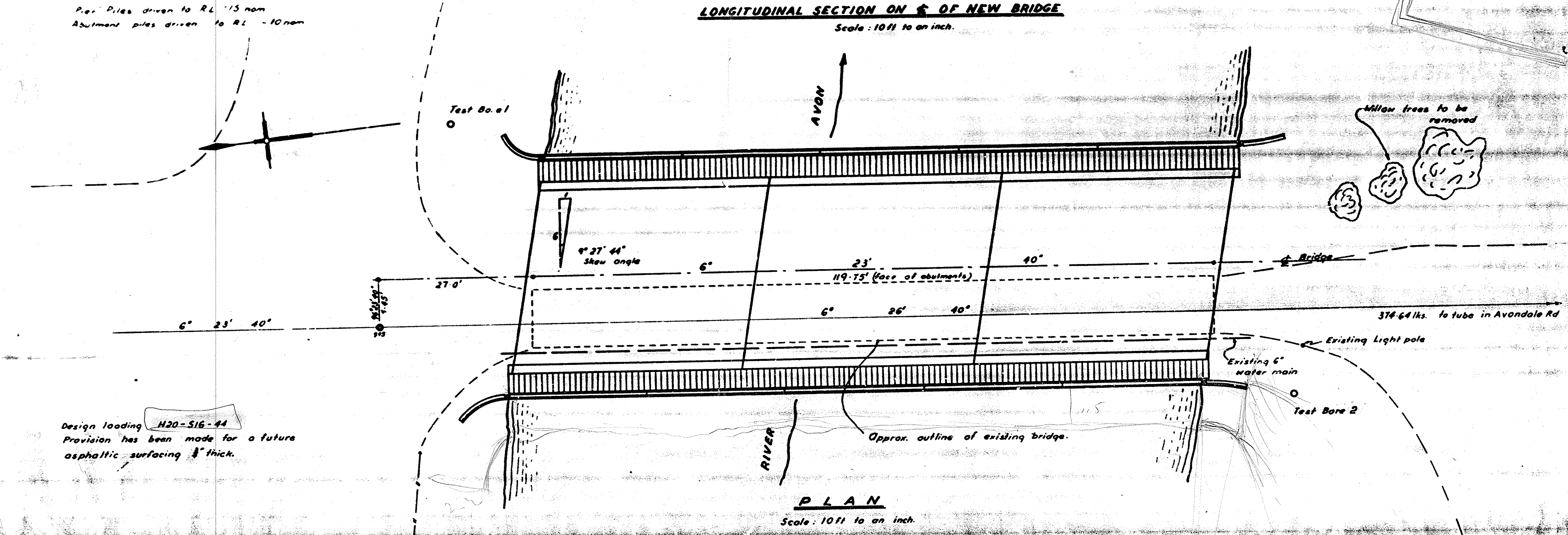


LONGITUDINAL SECTION ON ℄ OF NEW BRIDGE

Scale: 10 ft to an inch.

Bore hole samples may be examined at the City Engineers Office.

Pier Piles driven to RL -15 nom
Abutment piles driven to RL -10 nom



Design loading H20-S16-44
Provision has been made for a future asphaltic surfacing 3/4" thick.

PLAN

Scale: 10 ft to an inch.

Service	Initial	Service	Initial		Amendments	Initial	Date		Book	Page	Initial	Date			Date	Approved
H.P. Water		Land and Survey		A	Lighting standard amended		7.3.61	Surveyed					Designed			
Sewer		Planning						Levelled					Drawn			Design Engineer
S.W. Drainage								B.M. 3845 on iron tube S15.					Traced		2.61	
Gas													Dsg. Chk.			
Cables (M.E.D.)								Compiled from					Dn. Chk.			City Engineer
(P. & T.)																

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

AVONDALE ROAD BRIDGE

SITE PLAN & LONGITUDINAL ELEVATION

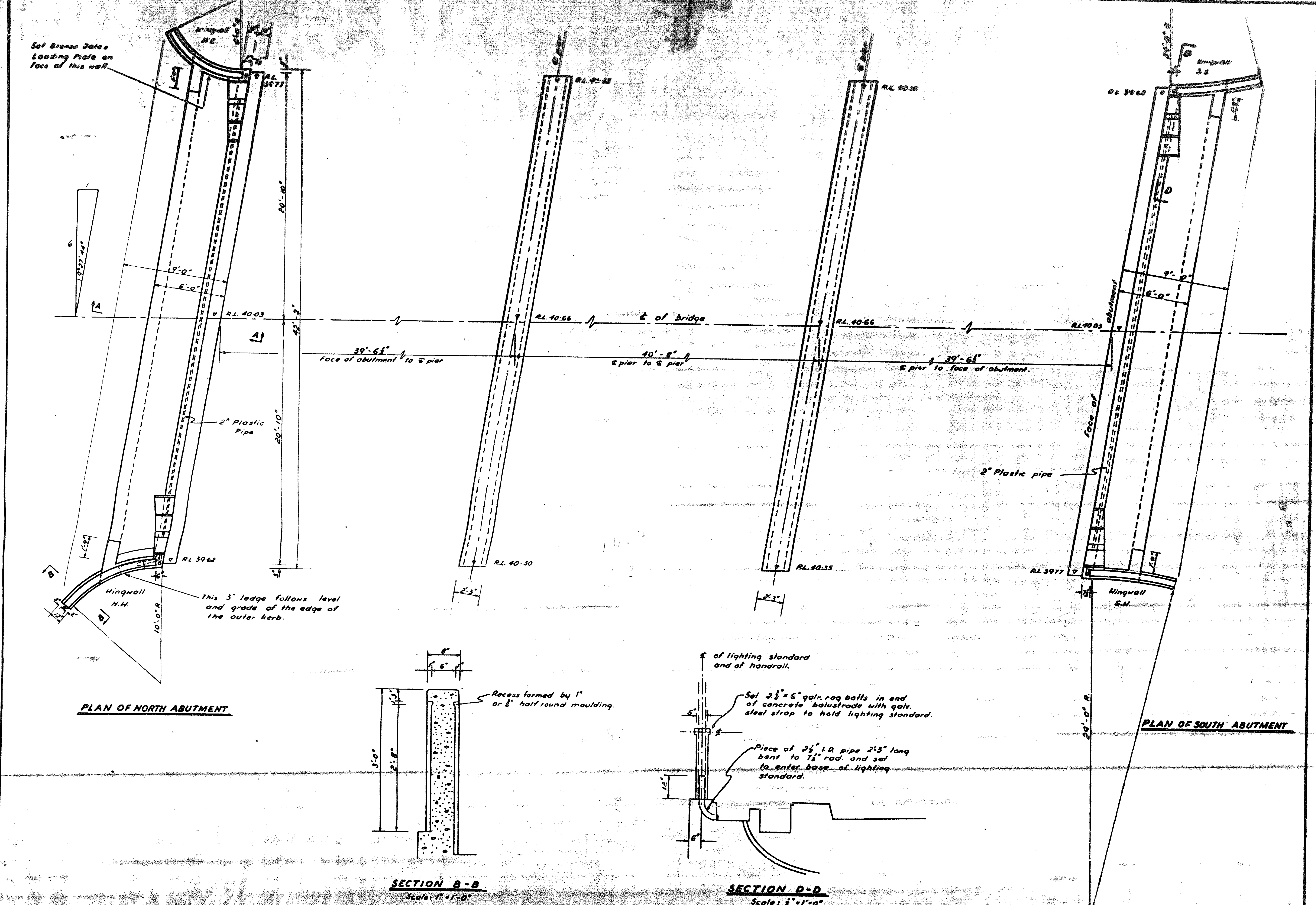
Scale: 10 ft to an inch. 10 chains to an inch.

File No. 0102-3

Issue A

D248

Sheet 1 of 8 sheets



Service	Initial	Service	Initial
H.P. Water		Land and Survey	
Sewer		Planning	
S.W. Drainage			
Gas			
Cables (M.E.D.)			
" (P. & T.)			

	Amendments	Initial	Date
A	Pier levels altered to suit varied bearing pads		17.3.61

	Book	Page	Initial	Date
Surveyed				
Levelled				
B.M.				
Compiled from				

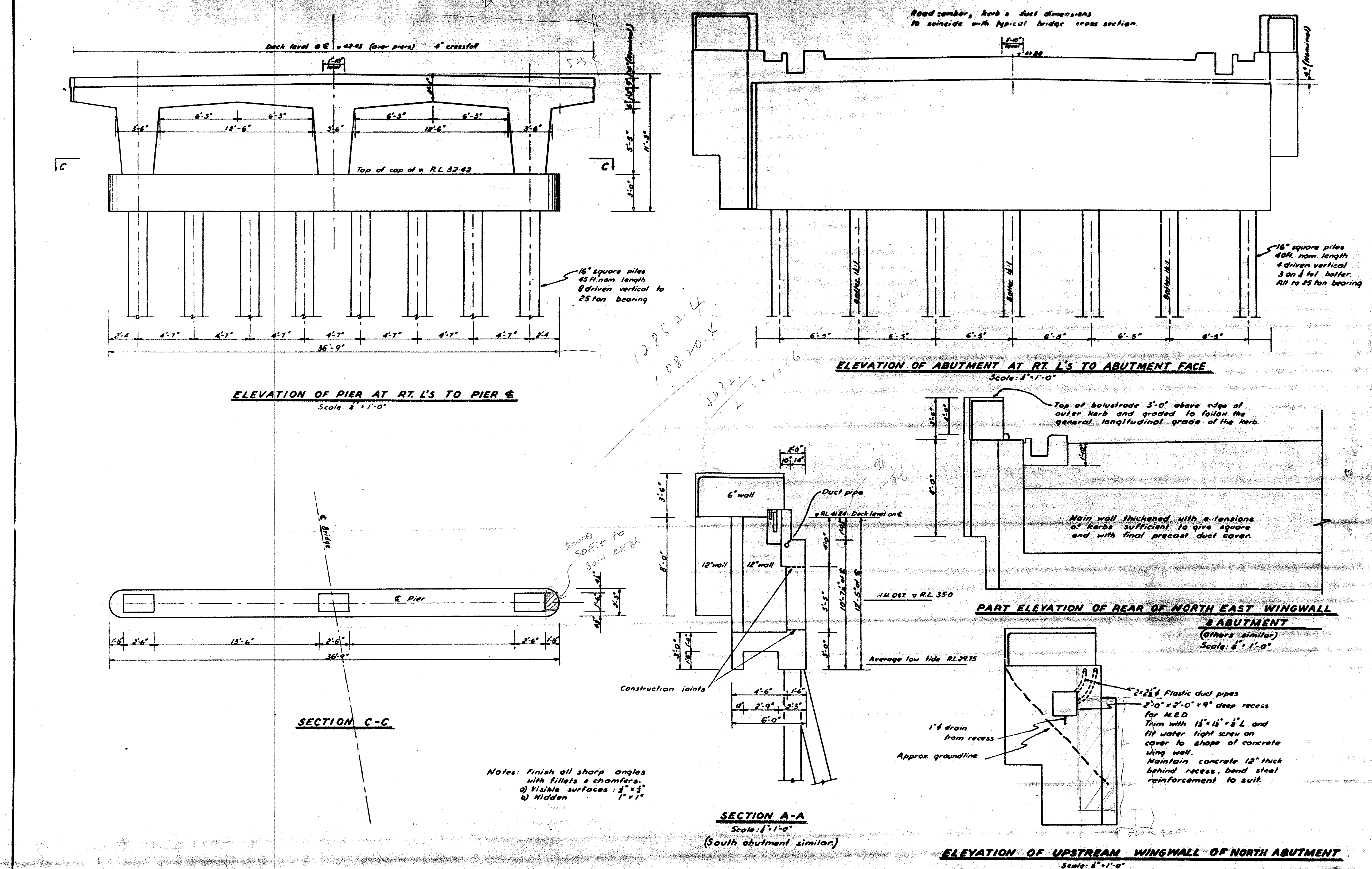
		Date	Approved
Designed	B.A.M.	Aug. '60	Design Engineer 10/3/61
Drawn		Sept. '60	
Traced	R.N.	2.61	
Dwg. Chk.			
Des. Chk.			City Engineer

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

AVONDALE ROAD BRIDGE
LAYOUT OF PIERS & ABUTMENTS

Scale: ¼" = 1'-0", ½" = 1'-0", 1" = 1'-0"

File No. 103-5
Issue A
D248
Sheet 3 of 8 sheets



Service	Initial	Service	Initial		Amendments	Initial	Date
H.P. Water		Land and Survey		A	Details of M.E.D. recess		12.2.61
Sewer		Planning					
S.W. Drainage							
Gas							
Cables (M.E.D.)							
.. (P. & T.)							

	Book	Page	Initial	Date
Surveyed				
Levelled				

B.M. ______

Compiled from ______

		Date
Designed		
Drawn		
Traced		2.61
Dwg. Chk.		
Des. Chk.		

Approved: Design Engineer 10/2/61 Date; City Engineer

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

AVONDALE ROAD BRIDGE

ELEVATIONS & CROSS SECTIONS OF PIERS & ABUTMENTS

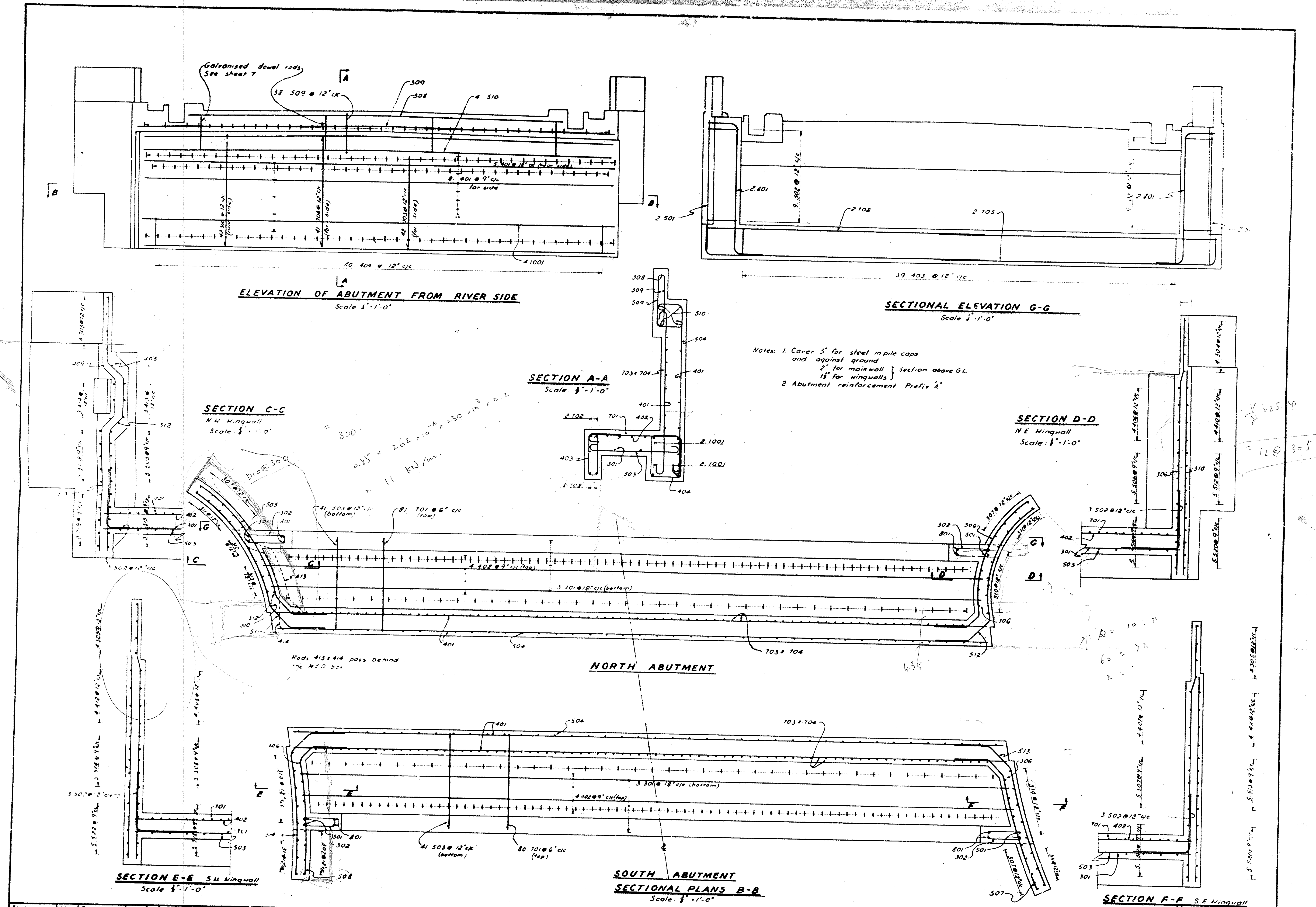
Scales: ¼" = 1'-0"

File No. 103-3

Issue (A)

D248

Sheet 3 of 8 sheets



Service	Initial	Service	Initial	Amendments	Initial	Date
H.P. Water		Land and Survey				
Sewer		Planning				
S.W. Drainage						
Gas						
Cables (M.E.D.)						
.. (P. & T.)						

	Book	Page	Initial	Date
Surveyed				
Levelled				
B.M.				
Compiled from				

		Date
Designed	A.C.F. & B.A.M.	Nov. 60
Drawn		Dec. 60
Traced	R.H.	1/61
Dwg. Chk.		
Des. Chk.		Dec '60

Approved

Design Engineer — Date

City Engineer

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

AVONDALE ROAD BRIDGE
ABUTMENT REINFORCEMENT

Scales: ¾"-1'-0", ¾"-1'-0"

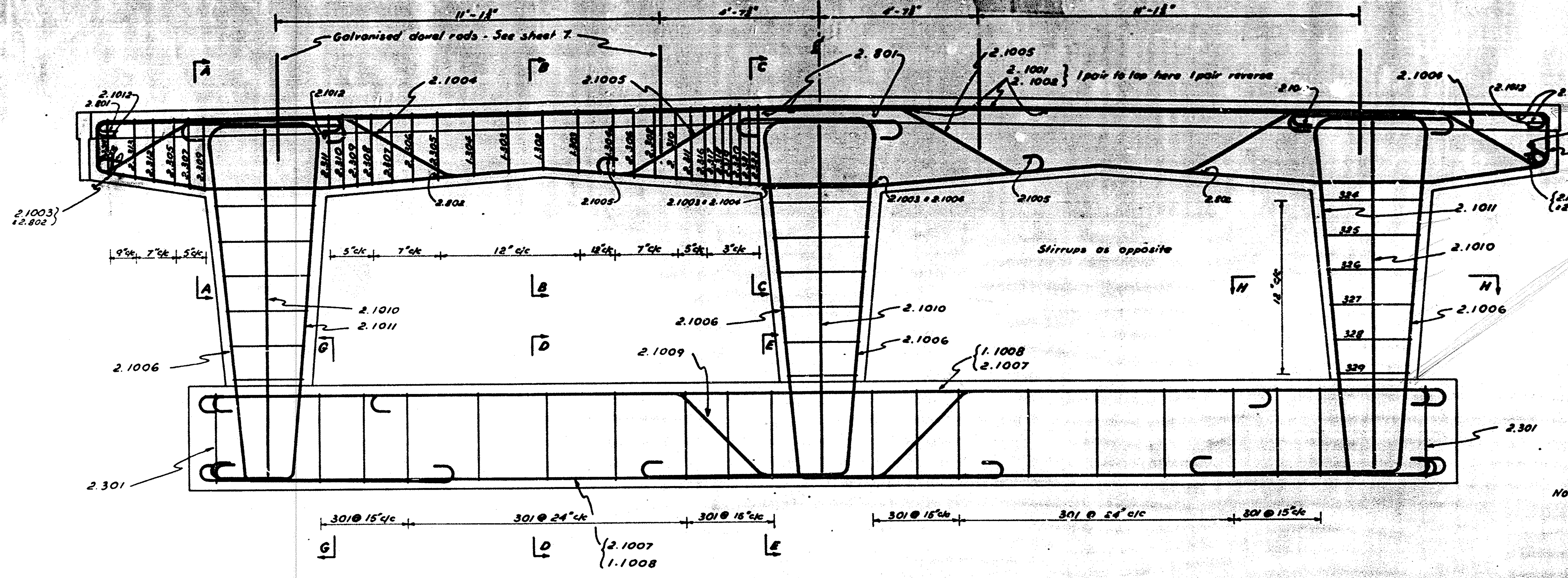
File No. D102-3

Issue

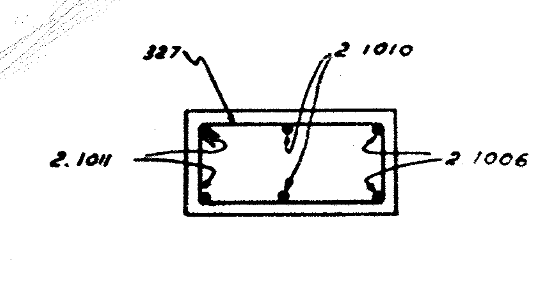
D248

Sheet 4 of 8 sheets

4

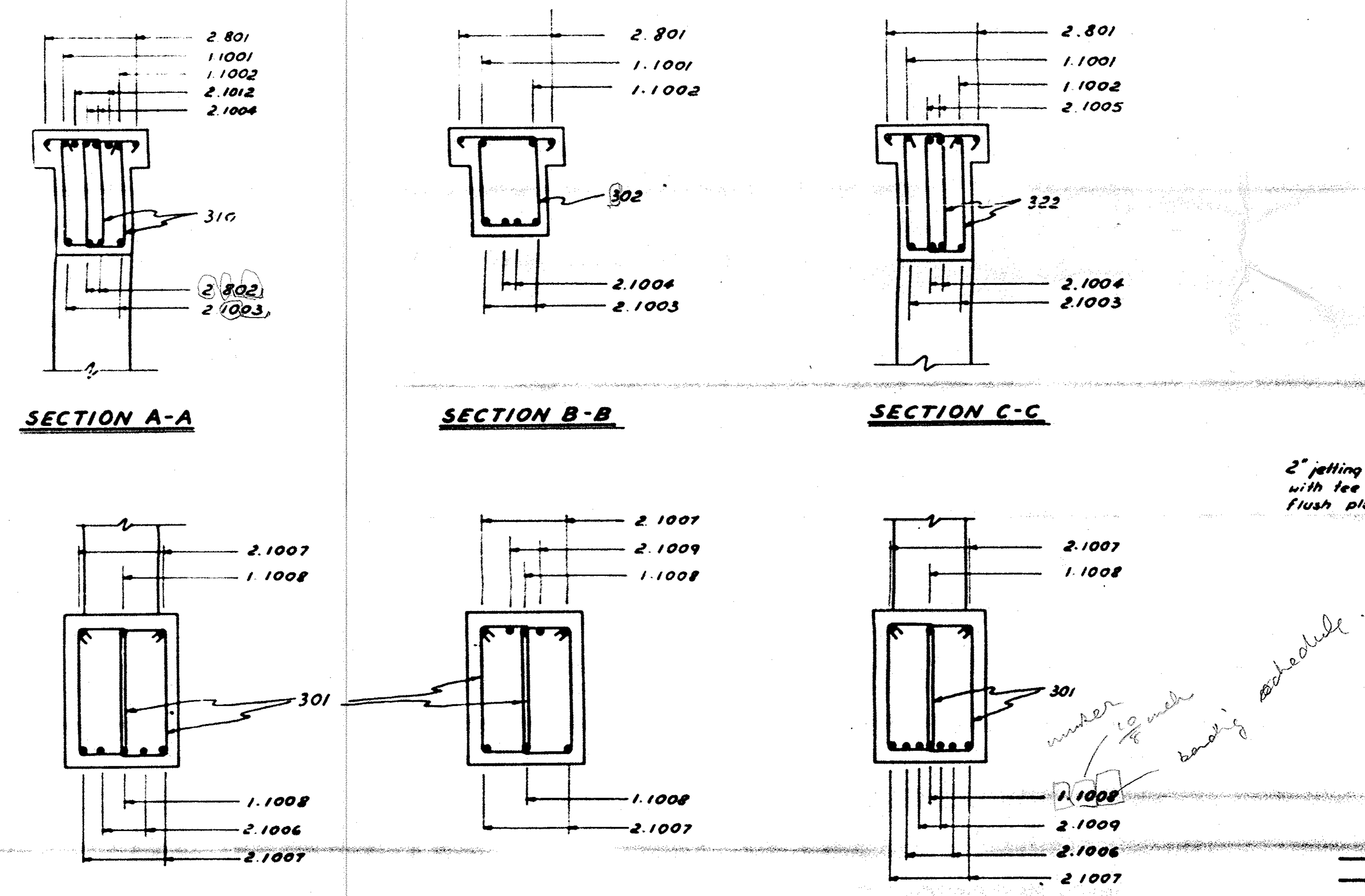


PIER REINFORCEMENT
Scale: 3/8" = 1'-0"



SECTION H-H
(centre column similar)

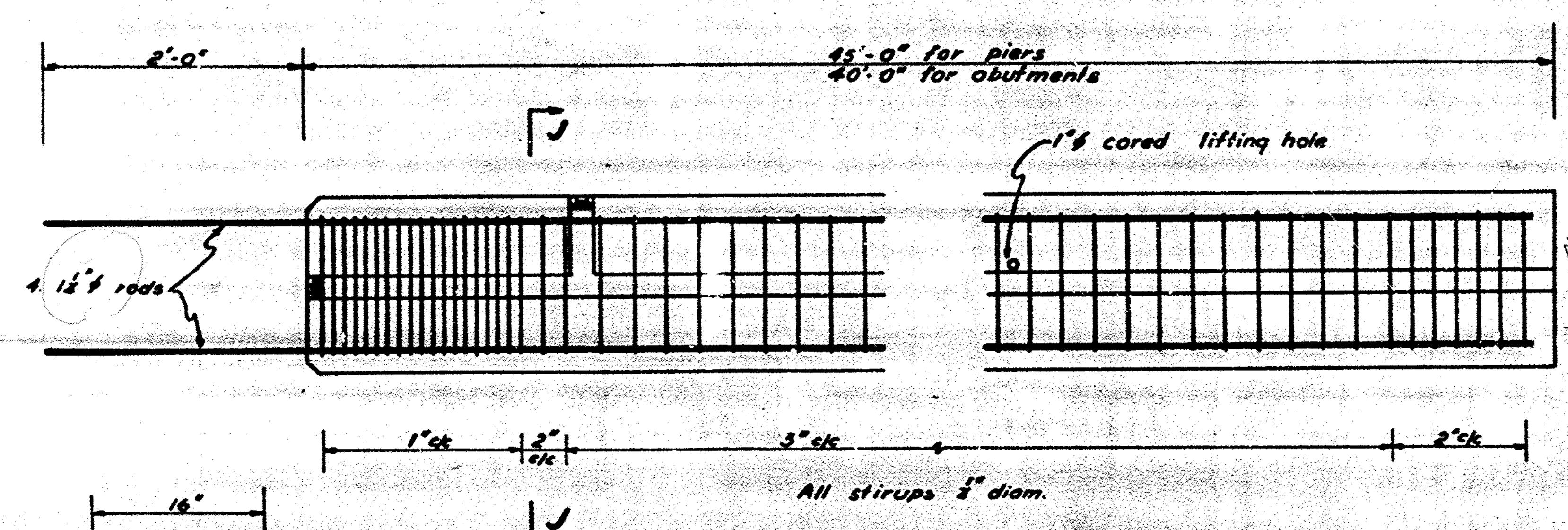
Notes: 1. Cover - 2" for main steel in beams and columns. 3" for main steel in pile cap.
2. Pier reinforcement Prefix "P"



SECTION A-A | SECTION B-B | SECTION C-C

SECTION G-G | SECTION D-D | SECTION E-E

Scale: 3/8" = 1'-0"

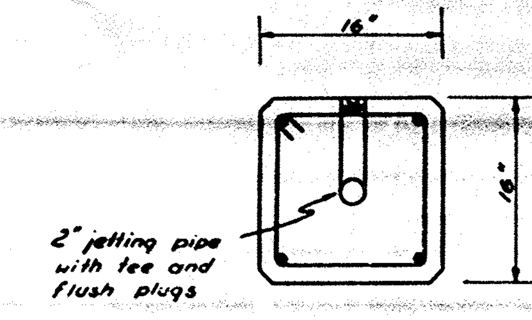


Pile adopted from P.W.D. 143803

SQUARE FLAT ENDED PILE
Scale: 1" = 1'-0"

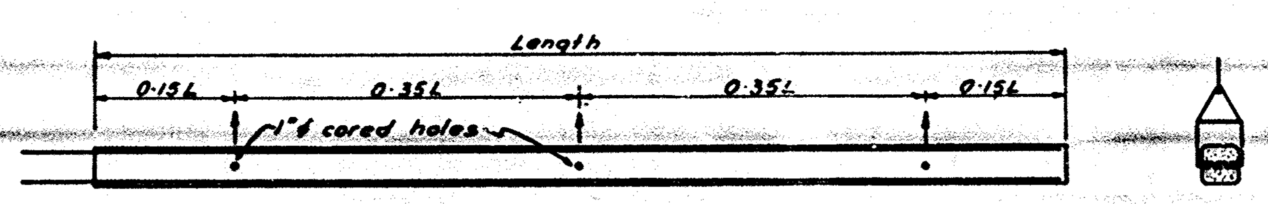
3/8 = 6mm

10/8 x 25.4 =



SECTION J-J

Notes: 1. Cover 1½" for precast piles.
2. 1⅜" ø main steel for piles shall be joined by flash butt welding. The welds shall be staggered in the pile.
3. A suitable helmet shall be provided to permit the pile to be driven without damage to the concrete or to the projecting reinforcing rods.
4. During all lifting & handling piles shall be slung from points shown & the sides shall be kept vertical.
5. Provide spreaders every 5 ft. to hold reinforcement accurately in place.



LIFTING DETAILS
Not to scale

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

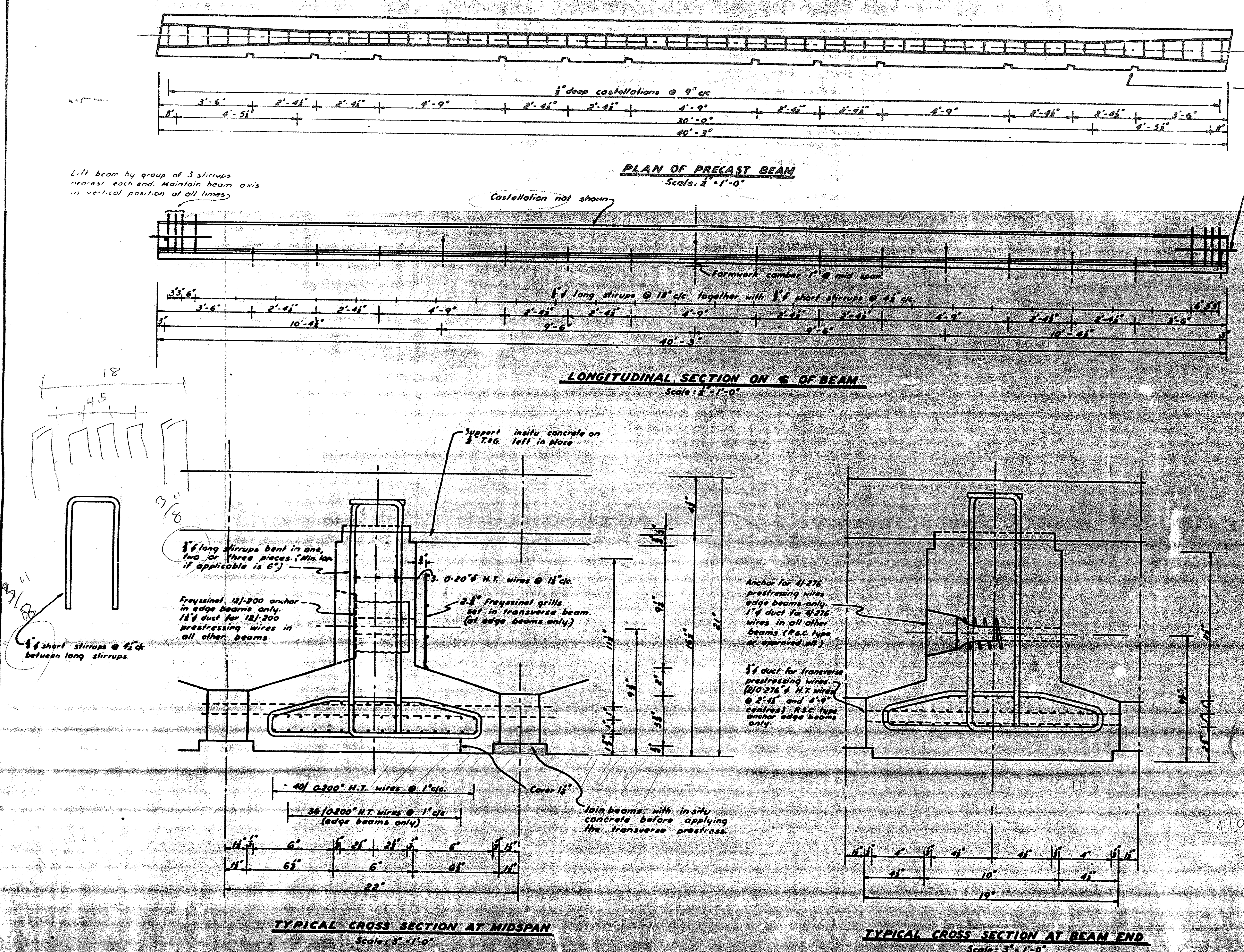
AVONDALE ROAD BRIDGE
PIER REINFORCEMENT
PILE DETAILS & REINFORCEMENT

Scales: 3/8" = 1'-0", 1" = 1'-0"

File No. D102-3

D248

Sheet 5 of 8 sheets



Notes: 1. Beams are to be cast on a cambered bed with midspan 1" higher than the ends. The intermediate points should follow a parabolic curve. Pretensioned wires shall be straight.

2. Edge beams have anchors cast in one side for transverse prestressing wires and modified main wire arrangement. (39 wires)

3. Main beam pretensioning
a) 43/.200 type
Wire stress at transfer = 215000 lb.
b) 39/.200 type
Wire stress at transfer = 193000 lb.

4. After transfer beams shall be supported or lifted at points not more than 9" from the end.
Maintain beams with axis vertical at all times.

5. The beams shall be transversely prestressed before concreting the in situ deck slab.

Wire stress at transfer.

a) 12/.200 wires = 60000 lb.
b) 4/.276 " = 36000 lb.
c) 2/.276 " = 18000 lb.

1 lb = 0.4536 kg

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

AVONDALE ROAD BRIDGE
PRESTRESSED BEAMS DETAILS

Scales: ½" = 1'-0", 3" = 1'-0"

File No. D102-3

D248

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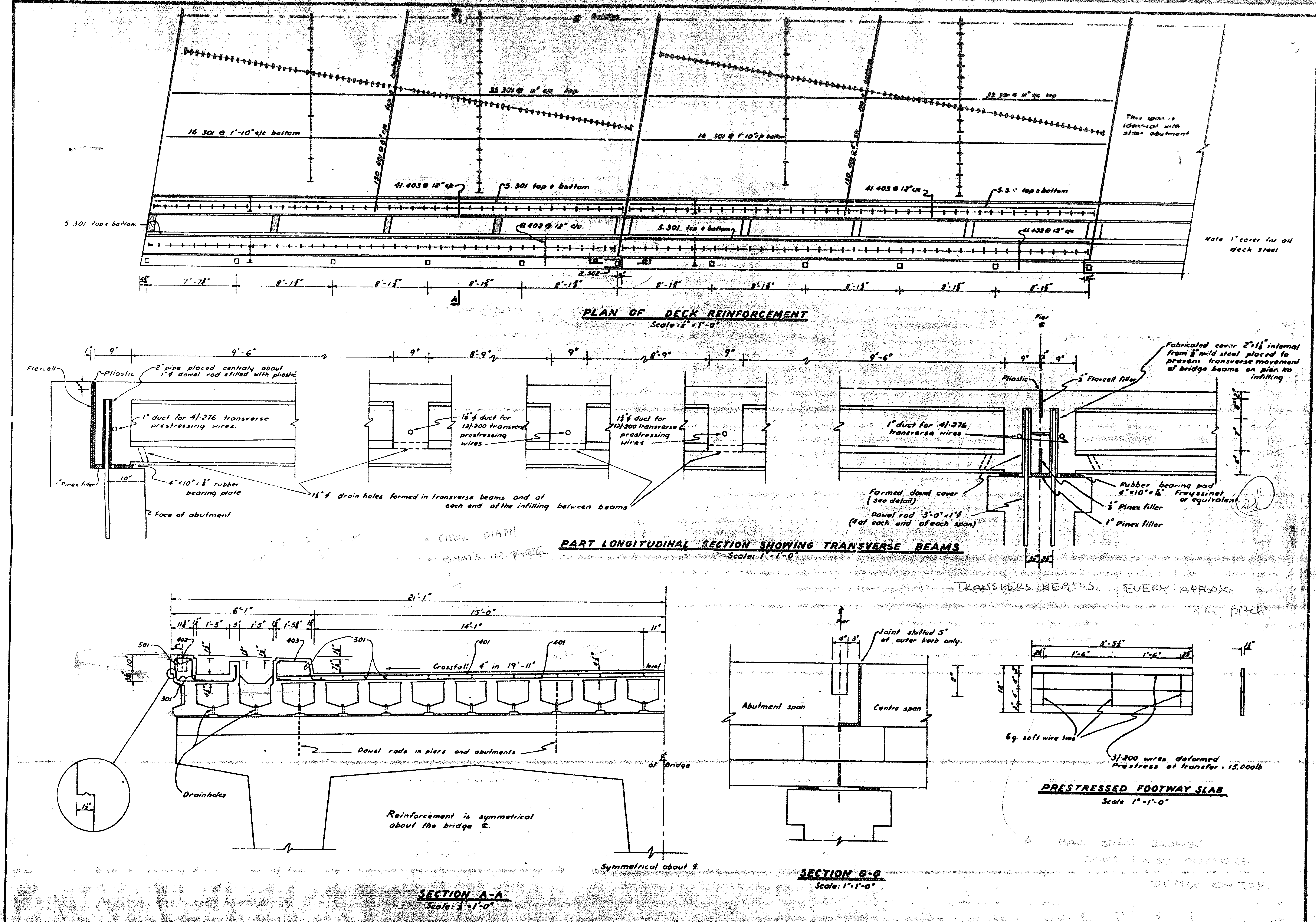


Service	Initial	Service	Initial		Amendments	Initial	Date
H.P. Water		Land and Survey		A	Bearing pads altered		
Sewer		Planning					
S.W. Drainage							
Gas							
Cables (M.E.D.)							
" (P. & T.)							

Surveyed / Levelled / B.M. / Compiled from

		Date
Designed	A.G.C. SEAR	Nov. '60
Drawn		Dec. '60
Traced		2.61
Dsg. Chk.		
Drg. Chk.		Dec. '60

Approved: Design Engineer, City Engineer

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

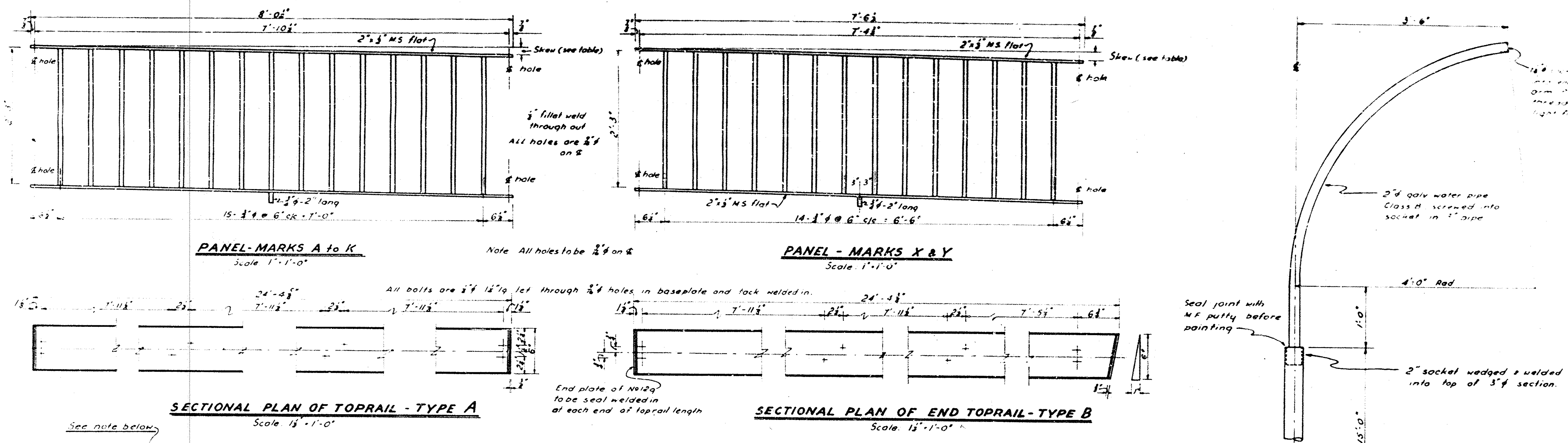
AVONDALE ROAD BRIDGE

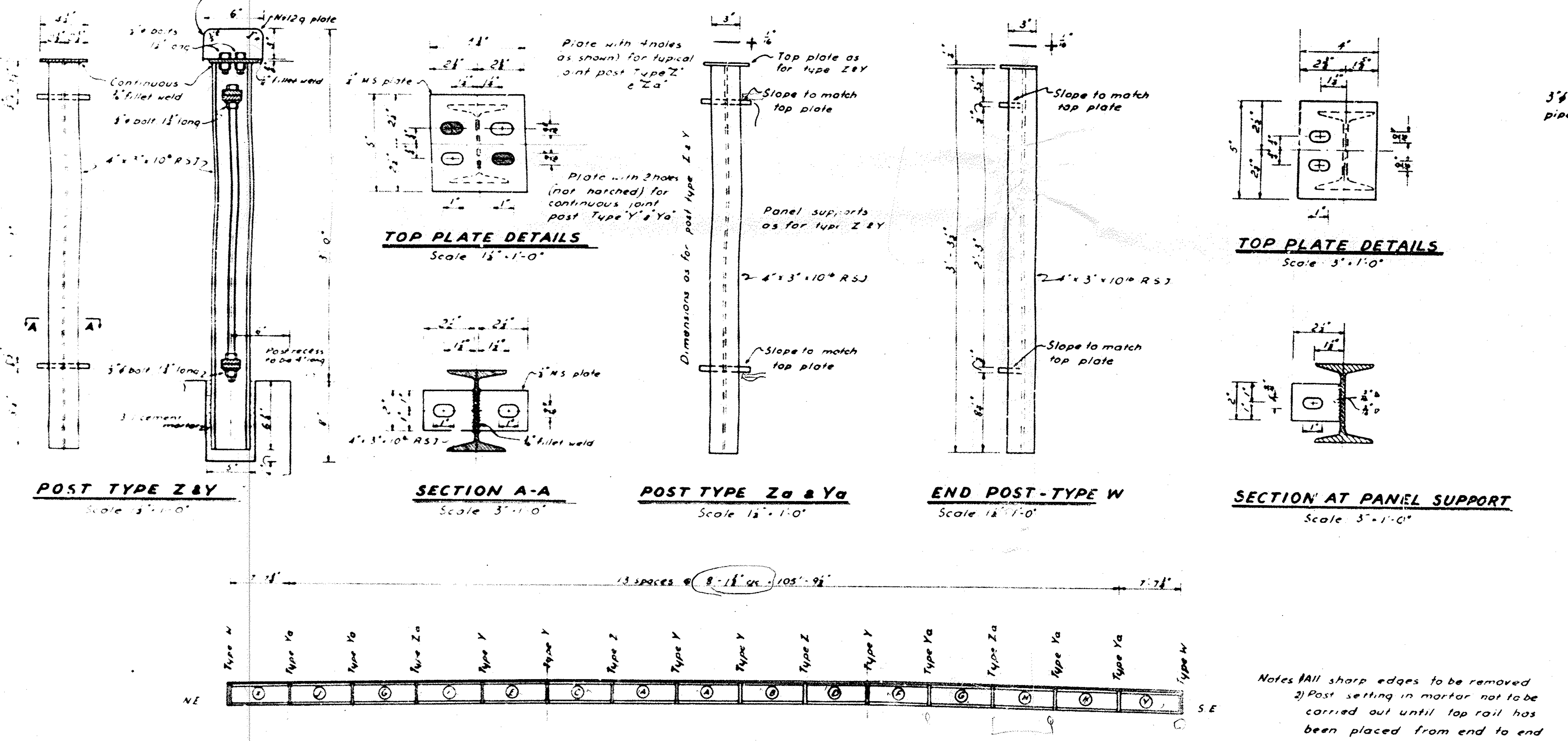
BRIDGE DECK & TRANSVERSE BEAMS: DETAILS & REINFORCEMENT

Scales: 1/4" = 1'-0", 1/2" = 1'-0", 1" = 1'-0"

File No. D103-3

D248





KEY ELEVATION OF HANDRAIL

Scale 10ft to an inch

(A) Note The 6"x3" top rail may be supplied in 6"x3"x¼" Rectangular Hollow Section. Seal ends as above and weld on 5/8" studs by suitable approved method.

Notes 1) All sharp edges to be removed

2) Post setting in mortar not to be carried out until top rail has been placed from end to end of bridge

3) Bolts, nuts & washers shall be hot dip galvanized

SCHEDULE FOR 2 HANDRAILS

Item	Quantity		
Post Type Z	4		
" " Za	4		
" " Y	10		
" " Ya	10		
End post Type W	4		
Toprail Type A	6		
Toprail Type B	4		
		Length	Skew
Panel Type A	4	8'-0½"	1/8"
" " B	2	"	1/4"
" " C	2	"	3/8"
" " D	2	"	5/8"
" " E	2	"	3/4"
" " F	4	"	1"
" " G	4	"	1 1/8"
" " H	2	"	1 3/8"
" " J	2	"	1 5/8"
" " K	2	"	1 1/4"
" " X	2	7'-6½"	1 3/4"
" " Y	2	"	2"

Note. Handrail adapted from P.W.D. 143969

Service	Initial	Service	Initial		Amendments	Initial	Date
H.P. Water		Land and Survey		A	Alternative top rail	RW	30.8.61
Sewer		Planning					
S.W. Drainage							
Gas							
Cables (M.E.D.)							
" (P. & T.)							

Surveyed · Levelled · B.M. · Compiled from

		Date
Designed	B.A.M.	Aug '61
Drawn	B.A.M.	"
Traced	R.N.	3.61
Drg. Chk.		Jan '61
Des. Chk.		

Approved: Design Engineer, Date 10/2/61; City Engineer

CHRISTCHURCH CITY COUNCIL — CITY ENGINEER'S DEPARTMENT

AVONDALE ROAD BRIDGE
DETAILS OF HANDRAIL & LIGHTING STANDARDS

Scales: 1" = 1'-0"; 1½" = 1'-0"; 3" = 1'-0"; 10ft to an inch

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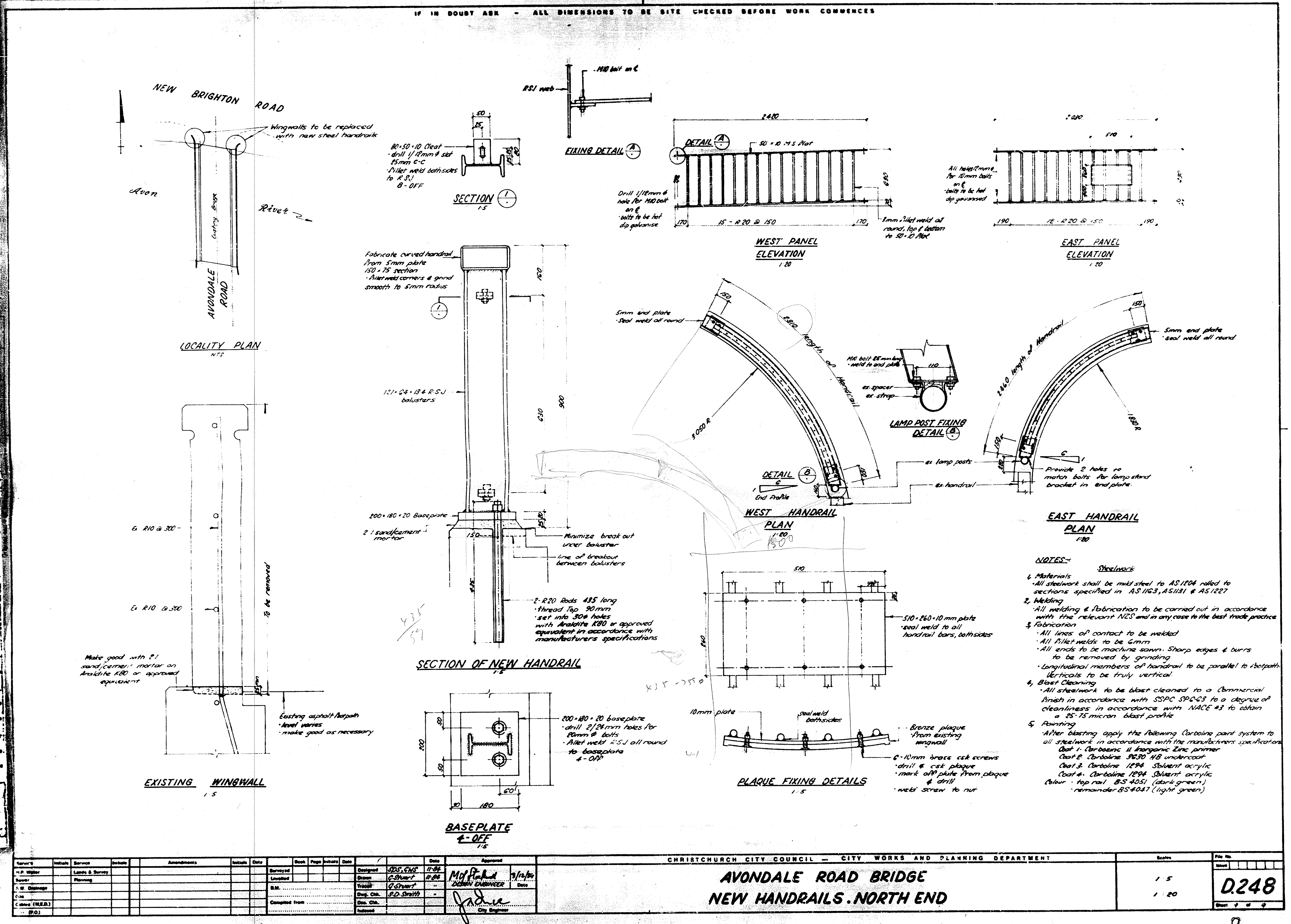

IF IN DOUBT ASK - ALL DIMENSIONS TO BE SITE CHECKED BEFORE WORK COMMENCES
NEW BRIGHTON ROAD
Wingwalls to be replaced with new steel handrails
Avon
River
Existing Bridge
AVONDALE ROAD
LOCALITY PLAN
NTS
80×50×10 Cleat
drill 1/12mm ϕ slot 25mm c-c
Fillet weld bothsides to R.S.J.
8-OFF
SECTION 1
1:5
RSJ web
M10 bolt on ℄
FIXING DETAIL A
DETAIL A
50×10 M.S Flat
Drill 1/12mm ϕ hole for M10 bolt on ℄
bolts to be hot dip galvanise
15 - R20 @ 150
3mm fillet weld all round, top & bottom to 50×10 flat
WEST PANEL ELEVATION
1:20
All holes 12mm ϕ for 10mm bolts on ℄
bolts to be hot dip galvanised
EAST PANEL ELEVATION
1:20
Fabricate curved handrail from 5mm plate 150×75 section
Fillet weld corners & grind smooth to 5mm radius
121×64×13# R.S.J balusters
200×180×20 Baseplate
2:1 sand/cement mortar
Minimize break out under baluster
line of breakout between balusters
2-R20 Rods 435 long
thread Top 90mm
set into 30ϕ holes with Araldite K80 or approved equivalent in accordance with manufacturers specifications
SECTION OF NEW HANDRAIL
1:5
5mm end plate
Seal weld all round
length of Handrail
3050 R
DETAIL B
End Profile
WEST HANDRAIL PLAN
1:20
M10 bolt 25mm long weld to end plate
ex. spacer
ex. strap
LAMP POST FIXING DETAIL B
ex. lamp posts
ex. handrail
5mm end plate
seal weld all round
2460 length of Handrail
Provide 2 holes to match bolts for lamp stand bracket in end plate.
EAST HANDRAIL PLAN
1:20
Ex. R10 @ 300
To be removed
Make good with 2:1 sand/cement mortar or Araldite K80 or approved equivalent
Existing asphalt footpath
level varies
make good as necessary
EXISTING WINGWALL
1:5
200×180×20 baseplate
drill 2/24mm holes for 20mm ϕ bolts
fillet weld RSJ all round to baseplate
4-OFF
BASEPLATE 4-OFF
1:5
510×260×10 mm plate
seal weld to all handrail bars, bothsides
10mm plate
Seal weld bothsides
Bronze plaque from existing wingwall
6-10mm brass csk screws
drill & csk plaque
mark off plate from plaque & drill
weld screw to nut
PLAQUE FIXING DETAILS
1:5
NOTES:-
Steelwork
1. Materials
All steelwork shall be mild steel to AS1204 rolled to sections specified in AS1163, AS1131 & AS1227
2. Welding
All welding & fabrication to be carried out in accordance with the relevant NZS and in any case to the best trade practice
3. Fabrication
All lines of contact to be welded
All fillet welds to be 6mm
All ends to be machine sawn. Sharp edges & burrs to be removed by grinding
Longitudinal members of handrail to be parallel to footpath. Verticals to be truly vertical.
4. Blast Cleaning
All steelwork to be blast cleaned to a Commercial finish in accordance with SSPC SPC-G3 to a degree of cleanliness in accordance with NACE #3 to obtain a 25-75 micron blast profile
5. Painting
After blasting apply the following Carboline paint system to all steelwork in accordance with the manufacturers specifications
Coat 1. Carbozinc 11 Inorganic Zinc primer
Coat 2. Carboline 3630 HB undercoat
Coat 3. Carboline 1294 Solvent acrylic
Coat 4. Carboline 1294 Solvent acrylic
Colour - top rail BS 4051 (dark green)
- remainder BS 4057 (light green)
CHRISTCHURCH CITY COUNCIL - CITY WORKS AND PLANNING DEPARTMENT
Designed SDS, CHS 11-84
Drawn C.Stuart 11-84
Traced G.Stuart
Dsg. Chk. S.D. Smith
DESIGN ENGINEER
City Engineer
3/12/84
AVONDALE ROAD BRIDGE
NEW HANDRAILS. NORTH END
Scales 1:5 1:20
D.248

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DESIGN SERVICES UNIT		INITIALS	DATE		APPROVED	JOB TITLE	DRAWING TITLE	SCALES	C.N.
CHRISTCHURCH THE GARDEN CITY	DESIGNED		4.92	DATUM	20/5/92	AVONDALE ROAD BRIDGE	NEW BRIDGE JOINT DETAILS	1:100	D.4199
The city that shines	DRAWN	PM	4.92	BENCH MK	DATE			1:10	
	TRACED	PM	4.92	SURVEY FB				1:2	
	DRW. CHK.			SURVEY LB					
	DES. CHK		5/92	CONSTN. FB					SHEET 1 OF 1
	INDEXED			CONSTN. LB	DESIGN SERVICES MANAGER				