Hamilton City Council

Ruakiwi Reservoir Detailed Seismic Assessment

22 August 2025

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Ruakiwi Reservoir Detailed Seismic Assessment

Hamilton City Council

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REV	DATE	DETAILS
1	2025/08/07	FINAL
2	2025/08/22	MINOR REVISIONS

	NAME	DATE
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ABBREVIATIONS

HCC	Hamilton City Council
SSSHA	Site Specific Seismic Hazard Assessment
DSA	Detailed Seismic Assessment
IL	Importance Level
UB	Universal Beam
RC	Reinforced Concrete
GL	Ground Level
EA	Equal Angle
RHS	Rectangular Hollow Section
SHS	Square Hollow Section

ENGINEERING ASSESSMENT SUMMARY REPORT

The following summary report follows the template proposed when undertaking seismic assessments using "*The Technical Guidelines for Engineering Assessments"* which can be found at www.eq-assess.org.nz

TABLE A-1. BUILDINGS INFORMATION – RUAKIWI RESERVOIR

A. BUILDINGS INFORMATION	
Building name/ Description	Ruakiwi Reservoir
Street Address	Located opposite 3/14 Ruakiwi Road – Hamilton Lake, Hamilton 3204
Territorial Authority	Hamilton City Council

TABLE B. ASSESSMENT INFORMATION

B. ASSESSMENT INFORMATION	
Consulting Practice	WSP New Zealand Limited
CPEng Responsible, including: Name CPEng number A statement of suitable skills and experience in the seismic assessment of existing buildings[1]	Reviewed By: Matthew Crake, CPEng 1022679, Senior Engineer Civil Structures | Team Leader – experienced in seismic assessments and water retaining structures. Approved By: Lewis Thomas, CPEng 1029326, Technical Principal – Civil Structures – experienced in seismic assessments and water retaining structures.
Documentation reviewed, including: date/ version of drawings/ calculations[2] previous seismic assessments	Refer Section 1.1.3. Drawings: — New High-Level Reservoir for Borough of Hamilton dated 20 December 1929. — Main Water Tower Repairs dated 30 September 1948. — Reservoir Roof Replacement dated August 1978. Previous Seismic Assessments: — Aurecon – Water Reservoir Structures Seismic Review dated 20 June 2009. — Stantec – Ruakiwi Reservoir Roof Assessment Seismic Review dated May 2017.
Geotechnical Report(s)	Ruakiwi Reservoir – Future Use Options Assessment / DSA Geotech Input – 3 July 2025.
Date(s) Building Inspected and extent of inspection	Exterior Condition of Structure was inspected – 9 July 2025.
Description of any structural testing undertaken and results summary	N/A
Previous Assessment Reports	Opus International Consultants Ltd – Condition Assessment of Dinsdale, Maeora, Fairfield and Ruakiwi Reservoirs dated September 2002. Altex Coatings Ltd – Hamilton Water tank Ruakiwi Reservoir for HCC Spot Check Report – Maintenance dated 25 September 2019. CMW Geosciences – Site Specific Hazard Assessment dated 28 February 2025. WSP Limited – Site Visit Record for Ruakiwi Reservoir dated 9 July 2025.
Other Relevant Information	-

[1] This should include reference to the engineer's Practice Field being in Structural Engineering, and commentary on experience in seismic assessment and recent relevant training.

[2] Or justification of assumptions if no drawings were able to be obtained.

Table C: Summary of Engineering Assessment Methodology and Key Parameters Used

C. SUMMARY OF ENGINEERING ASSESSMENT METHODOLOGY AND KEY PARAMETERS USED	
Occupancy Type(s) and Importance Level	No stored water and restricted pedestrian access throughout. Hamilton City Council has secured government funding to build infrastructure to enable growth. With this funding a new reservoir will be constructed, and the existing reservoir will be decommissioned and repurposed. Once the structure has been emptied of water, the structure is no longer considered a storage tank (IL3 – IL4 structure). Therefore, the decommissioned reservoir is considered to be an Importance Level 2 structure, as confirmed with HCC.
Site Subsoil Class	D (confirmed by desktop assessment)
<u>For a DSA:</u>	
Summary of how Part C was applied, including: — the analysis methodology(s) used from C2 — other sections of Part C applied	**Step 1:** Assess the structural configuration and load paths to identify key structural elements, potential structural weaknesses (SWs) and severe structural weaknesses (SSWs). **Step 2:** Calculate the relevant probable strength capacities for the critical elements. **Step 3:** Determine seismic demands on all critical elements in accordance with DZ TS 1170.5.
Other Relevant Information	Concrete, f'c: 25 MPa Reinforcement, fy: 200 MPa Steel, fy_s: 250 MPa Material strength values were derived as per previous reporting (refer to Appendix), experience, or the age of the structures according to Section C5 in *"The Seismic Assessment of Existing Buildings"* guidelines. Concrete compressive strengths and steel bar reinforcement yield strengths were informed by the Aurecon Water Reservoir Structures Seismic Review –issued on 20 June 2009, refer to Appendix C.

Table D-1. Assessment Outcomes

D. ASSESSMENT OUTCOMES	
Assessment Status (Draft or Final)	Final
Reservoir	Ruakiwi
Assessed *%NBS* Rating	< 34%NBS – DZ TS 1170.5:2004
Seismic Grade and Relative Risk (from Table A3.1)	Grade D *10-25 times greater risk relative to a new building.*
<u>For a DSA:</u>	
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	N/A
Describe the Governing Critical Structural Weakness (CSW)	The connection between the reservoir walls and the base slab is a critical structural weakness. This detail consists of the wall sitting on a steel expansion guide with no positive connection between concrete. Therefore, the only sliding resistance across this plane is friction between the steel plates of the expansion guide. Failure of this connection due to out-of-plane moment exceeding the friction capacity of the steel plates will result in the lateral movement of the walls relative to the base slab and could lead to structural failure.
If the results of this DSA are being used for earthquake-prone decision purposes, <u>and</u> elements rating <34*%NBS* have been identified (including Parts)[3]:	The reservoir has been identified as an earthquake prone structure as per the NZSEE Guidelines and Building Act. Hamilton is considered to be an area of medium seismic risk as per Clause 133AD of Subpart 6A of the Building Act. Once decommissioned, the Ruakiwi Reservoir is not considered a priority building (IL3 or IL4). Therefore, there is a legislative requirement to strengthen or demolish the structure within the next 25 years as per Clause 133AM of Subpart 6A of the Building Act.

[3] If a building comprises a shared structural form or shares structural elements with other adjacent titles, information about the extent to which the low scoring elements affect, or do not affect the structure.

EXECUTIVE SUMMARY

WSP carried out a Detailed Seismic Assessment (DSA) of Ruakiwi Reservoir for Hamilton City Council (HCC). The structure was assessed following the NZSEE Seismic Assessment Guidelines.

The following table identifies the critical structural elements of the structure and their assessed %NBS ratings for the required load derivation method:

STRUCTURE	ITEM	%NBS IL 2 (DZ TS1170.5)
Ruakiwi	Overall structure	**< 34%**
	NZSEE Seismic Grade	**D**
	Roof-to-wall connection	> 100%
	Reservoir Uplift	> 100%
	Base-to-wall connection	Vulnerable to sliding failure (< 34%)
	Bearing on soil	> 100%

The analysis of the structures was carried out under the following critical assumptions:

- There is no significant deterioration of internal structural elements inside the reservoir.

Ruakiwi Reservoir has been identified as having a Seismic Resilience Class of D, with a New Build Standard ratio less than 34%NBS. The structure is defined as an Earthquake Prone Building as per NZSEE guidelines and the Building Act.

Because this structure is to be decommissioned and not used to store water in the future, the structure is considered to be a monument (that can be entered by a person) under the Building Act 2004 and is included in the Earthquake Prone Building Provisions within the Building Act. This requires the structure to be strengthened to > 34%NBS or demolished within 25 years of HCC receiving this DSA as per Clause 133AM of Subpart 6A of the Building Act.

The NZSEE guidelines recommend minimum strengthening to 67%NBS with consideration of additional works required to reach 100%NBS or above. The additional cost required to strengthen Ruakiwi Reservoir to 100 %NBS relative to >34 %NBS is not considered to be significant.

1 PROJECT BACKGROUND

1.1 BACKGROUND

1.1.1 ENGAGEMENT

WSP New Zealand Ltd (WSP) was commissioned by HCC to carry out a Detailed Seismic Assessment (DSA) for the reinforced concrete reservoir at Ruakiwi Road, in Hamilton. The reservoir is to be decommissioned and emptied of water and will be kept as a heritage monument with potential to be repurposed as a commercial structure in the future.

1.1.2 SITE LOCATION

The Ruakiwi Reservoir is located opposite 3/14 Ruakiwi Road – Hamilton Lake, Hamilton 3204. Refer to Figure 1-1 for the location of the reservoir site.

Figure 1-1. Location of Ruakiwi Reservoir

1.1.3 EXISTING DOCUMENTATION

WSP has been provided with multiple documents that detail information about the reservoir structure and condition. The following documents were used to inform this DSA.

Drawings

- New High-Level Reservoir for Borough of Hamilton dated 20 December 1929.

  The original drawings of Ruakiwi Reservoir. These drawings were used to determine the dimensions of the reservoir and its structural properties (concrete reinforcement, section thicknesses, etc.)

- Main Water Tower Repairs dated 30 September 1948.

  These drawings provided information about upgrades completed to the expansion joints within Ruakiwi Reservoir.

- Reservoir Roof Replacement dated August 1978.

  The structural drawings for the new steel roof for Ruakiwi Reservoir. These drawings provided the structural details of the roof structure, including member layout and section dimensions.

Seismic Assessments

- Aurecon – Water Reservoir Structures Seismic Review dated 20 June 2009.

  A seismic review for multiple reservoirs under the jurisdiction of the HCC. This document provided accurate assumed material properties for Ruakiwi Reservoir.

- Stantec – Ruakiwi Reservoir Roof Assessment Seismic Review dated May 2017

  A memo confirming that the previous Aurecon review mentioned above remains as the latest requirements for reservoir structures in the Hamilton area.

- CMW Geosciences – Site Specific Hazard Assessment dated 28 February 2025.

  A report detailing the geotechnical investigations completed by CMW in 2025. The critical information from this report was summarised by WSP into a geotechnical memo (see Appendix B) and used to inform the seismic demands on Ruakiwi Reservoir.

Condition Assessments

- Opus International Consultants Ltd – Condition Assessment of Dinsdale, Maeora, Fairfield and Ruakiwi Reservoirs dated September 2002.

  An assessment report covering multiple reservoirs under the jurisdiction of HCC. This report was used to inform WSP of structural deficiencies withing Ruakiwi Reservoir.

- Altex Coatings Ltd – Hamilton Water tank Ruakiwi Reservoir for HCC Spot Check Report – Maintenance dated 25 September 2019.

  A report documenting the condition of the internal steel liner within Ruakiwi Reservoir. This report was used to inform WSP of any structural deficiencies in the internal steel liner within the structure.

- WSP Ltd – Site Visit Record for Ruakiwi Reservoir dated 9 July 2025.

  An internal document completed by WSP that details any structural deficiencies on the exterior of Ruakiwi Reservoir. This record was used to inform WSP of any structural deficiencies that have persisted since the 2002 condition assessment.

1.2 SCOPE OF WORKS

The PSP IFS Variation Form issued on 27 June 2025 had the following deliverable to cover the integrity and resilience of the Ruakiwi Reservoir as a standard structure:

— Detailed Seismic Assessment Report for the structure (Geotechnical Interpretive Technical Memo to be included in Appendix).

The following scope of works was agreed with HCC to be included in the DSA:

— %NBS of critical structural elements.

— Commentary on areas of structural deficiency.

— High level description of any strengthening works identified as being required.

— Commentary on potential works required to be carried out if a future change of use was implemented.

The following scope of works was agreed with HCC to be omitted from the DSA:

— Assessment of any appurtenant structures (e.g. lift pump building).

— Analysis of any access structures (e.g. stairs).

— Preliminary or detailed design of strengthening or modification works.

1.3 THE SEISMIC ASSESSMENT OF EXISTING BUILDINGS GUIDELINES

1.3.1 GENERAL

Following the significant earthquakes in New Zealand between 2010 and 2016, the Seismic Assessment Guidelines were developed to provide clarity on earthquake risk and to provide a framework to ensure seismic assessments were applied consistently.

1.3.2 PRIMARY OBJECTIVE – LIFE SAFETY FOCUS

As per the 'The Seismic Assessment of Existing Buildings':

"*These guidelines focus on the assessment of life safety issues as the primary objective. This means that the earthquake scores or ratings are based primarily on life safety considerations rather than damage to the building or its contents unless this might lead to damage to adjacent property. The earthquake rating assigned is, therefore, not reflective of serviceability performance and the reporting should warn of this.*"

This indicates the main outcome of a seismic assessment is to determine the immediate life safety risk to people. A significant life safety hazard is defined as "*an unavoidable danger that a number of people are exposed to*".

1.3.3 IMPORTANCE LEVEL 2 STRUCTURES

An IL2 structure is defined in AS/NZS 1170.0:2002 as "*normal structures and structures not in other importance levels*". HCC has informed WSP that the structure may serve as a potential attraction for commercial use once the reservoir has been decommissioned.

Therefore, the reservoir is not required to remain functional post-disaster and is not required to store water and thus, has been considered as a monument (that is capable of being entered by a person) under the

Building Act 2004. The Ministry of Business, Innovation and Employment states in clause B1.3.1 of the New Zealand Building Regulations that "*Buildings*, *building elements* and *sitework* shall have a low probability of causing loss of *amenity* through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives, or during *construction* or *alteration* when the *building* is in use".

As such, we have assessed the elements of the reservoir that will result in the highest risk to life safety following a ULS seismic event.

1.4 DETAILED SEISMIC ASSESSMENT

1.4.1 GENERAL

WSP performed a DSA of the reservoir by calculating the capacity of the major structural elements and comparing this with the seismic demands expected to occur in accordance with current design standards:

— Ministry of Business, Innovation and Employment – Building Regulations 1992

An assessment was completed based on the seismic loading derivation method as described below:

— DZ TS1170.5:2024 – Public consultation draft standard for seismic actions

The draft edition of the new seismic standard was chosen to ensure the structure would be acceptable for the updated seismic code when it is released.

Where information was not available assumptions were made based on previous experience with similar structures and guidance from Part C5 (Concrete Structures) of *"The Seismic Assessment of Existing Buildings"* guidelines.

The assessment was undertaken in accordance with *"The Seismic Assessment of Existing Buildings"* which is available from the website www.eq-assess.org.nz and discussed in Section 1.3.

1.4.2 STRUCTURAL CAPABILITIES

The structural demands and the capacities of the reinforced concrete elements were determined using the following codes:

— AS/NZS 1170.0:2002 Structural Design Actions Part 0: General Principles
— AS/NZS 1170.1:2002 Structural Design Actions Part 1: Permanent, Imposed and Other Actions
— DZ TS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand
— NZS 3101.1:2006 Concrete Structures Standard
— NZS 3404: Part 1:1997: Steel Structures Standard

1.4.3 STRUCTURAL IMPORTANCE LEVEL

At the request of HCC, the Ruakiwi Reservoir is considered as an IL2 structure despite being considered as an IL4 structure in past seismic assessments. The designation of IL2 is based on the structure not being used as a water source for the Hamilton area, and thus not required to resist a disaster level seismic event. These considerations are in accordance with Tables 3.1 and 3.2 of the AS/NZS 1170.0:2002.

1.4.4 SEISMIC ASSESSMENT PARAMETERS

The seismic design parameters for the Ruakiwi Reservoir site are listed in Table 1-1.

A design working life of 50 years was chosen to calculate the demands. This is not an assessment of the remaining life of the structure or the durability. The 50 years is used to determine the likelihood of a future seismic event and informs the choice of the annual probability of exceedance (AEP) for the seismic demands.

Table 1-1. Seismic Assessment Parameters

STRUCTURE	**RUAKIWI RESERVOIR**
CHARACTERISTIC	**VALUE**
Design Working Life	50 years (assumed)
Importance Level	2
Soil Class	D
Assumed shear wave velocity	266 m/s (based on geotechnical desktop assessment)
Site Class	IV (based on geotechnical desktop assessment)
Sas (ULS)	0.44
PGA (ULS)	0.2
N (T, D)	1.0
Annual Probability of Exceedance, ULS	1 / 500 Year Return Period
Annual Probability of Exceedance, SLS1	1 / 25 Year Return Period

1.5 PERCENTAGE OF NEW BUILD STANDARD (%NBS)

The percentage of the New Build Standard (%NBS) is a quantifiable measure of the structural capacity of a building compared to the codified demand. The %NBS is determined by taking the capacity of each element and dividing it by the demand on that element to express it as a percentage. The overall %NBS of the structure is governed by the lowest %NBS of components that pose a life safety risk should they fail.

It is important to understand that the %NBS of a structure has no bearing on its functionality.

A value of >100%NBS, shows that an element has a capacity greater than the demand as determined by the current Design Standard. A value less than 100%NBS indicates that for a design level demand the capacity of that structural element will be exceeded. This is summarised as per Table A3.1 in *"The Seismic Assessment of Existing Buildings".*

Table A3.1: Assessment outcomes (potential building status)

Percentage of New Building Standard *(%NBS)*	**Alpha rating**	**Approx. risk relative to a new building**	**Life-safety risk description**
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low to Medium risk
34-66	C	5-10 times greater	Medium risk
20 to <34	D	10-25 times greater	High risk
<20	E	25 times greater	Very high risk

Several factors need to be considered and evaluated when making decisions on values less than 100%NBS. Refer to Section 1.6 for the discussion on Earthquake Prone Structures.

1.6 EARTHQUAKE PRONE STRUCTURE

The earthquake prone building definition from MBIE:

"A building, or part of a building, is earthquake-prone if it will have its ultimate capacity exceeded in a moderate earthquake, and if it were to collapse, would do so in a way that is likely to cause injury or death to persons in or near the building or on any other property, or damage to any other property."

A moderate earthquake is an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site.

The earthquake prone guidance specifically relates to buildings and has the terms "*collapse and cause injury or death*". This reservoir has been analysed with the assumption that pedestrian access will be restricted throughout. despite this the reservoir is situated across the road from residential housing, and it is possible that a failure of this reservoir would cause injury or death to the people adjacent to it. Thus, life safety has been considered as the most likely safety issue during a seismic event.

2 RUAKIWI RESERVOIR

2.1 SITE DESCRIPTION

The Ruakiwi Reservoir is located opposite 3/14 Ruakiwi Road – Hamilton Lake, Hamilton 3204. Refer to Figure 2-1 for the locations of the reservoir site.

Figure 2-1. Site Layout of the Ruakiwi Reservoir Site

2.2 ASSUMPTIONS

The following assumptions were made to undertake the structural assessment of the Ruakiwi Reservoir:

- All concrete capacities have been determined assuming all concrete sections are uncracked.
- Assumed conservative values for material yield strengths (refer to Table 2-4).
- Assumed allowable ground bearing pressure of 300 kPa under a ULS seismic event (refer to Appendix C).

- All dimensions based on provided drawings are assumed to be true and accurate.
- Seismic load present from soil will be resisted by the curtain wall and columns and will not affect the demand on the main reservoir wall.
- Seismic load present from soil will not travel upwards due to the small aspect ratio between the soil height (~2.75 m) and the column height (17.5 m).

2.3 DESCRIPTION OF STRUCTURE

The Ruakiwi Reservoir is a 12,000 m^3 circular reservoir, constructed in the early 1930s.

Figure 2-2. General elevation site photograph of the Ruakiwi Reservoir

The reservoir is split into three main sections: the roof, the upper walls, and the lower walls.

- The roof is made up of two steel truss systems and Brownbuilt steel purlins that support the Dimondek 16' roofing material. The truss systems are supported at each end by rectangular hollow sections that run vertically and connect to the concrete structure.
- The upper walls are made up of reinforced concrete sections that were poured in vertical stages and span from a top RC ring beam to a bottom RC ring beam. These walls are supported on the inside circumference by 20 vertical UB members with steel truss supports that span from the base of the upper wall to the roof, as well as rectangular stiffening ribs span that form a walkway from the upper ring beam around the exterior of the upper walls.

— The lower RC walls are also poured in vertical stages and span from the lower ring beam to the base slab. The upper and lower walls are connected through the ring beam, that is supported by 20 tapered hollow RC columns that are around the outside of the lower walls. The entire interior of the reservoir (base slab, upper and lower walls) is lined with steel which extends past the upper ring beam to the roof, forming a top steel tank. The base of the reservoir is embedded ~ 2.5 m deep in soil around its entire circumference but from site investigations it appears to be embedded less in certain areas compared to the drawings.

Refer to Table 2-1 for the reservoir properties, and Figure 2-3 for an elevation view of the reservoir.

Table 2-1. Key dimensions of the Ruakiwi Reservoir

CHARACTERISTIC	VALUE
Volume	12,000 m^3
Roof Height (Above Base Slab)	25.9 m
Upper Wall Height	4.3 m
Upper Internal Diameter	25.6 m
Lower Wall Height	18.6 m
Lower Internal Diameter	23.6 m
Column Outside Diameter	(1,118 – 940 mm)
Column Inside Diameter	685 mm
Column Height	17.5 m
Base Slab Diameter	26.8 m
Base Slab Thickness	762 mm

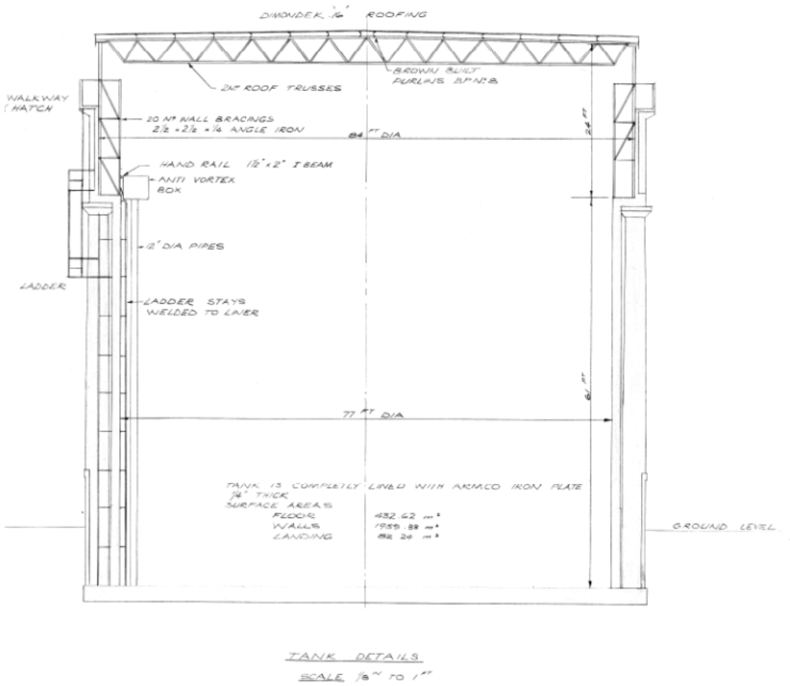


Figure 2-3. Elevation view of Ruakiwi Reservoir

2.3.1 STEEL LINER

The interior of the reservoir is lined with a 6.35 mm steel liner. The liner covers the interior surfaces of the slab, lower walls, upper walls and up to the roof level. The liner coating the base slab is made up of 1.8 m square plates that are welded to horizontal tee sections embedded into the slab and around all edges as shown in Figure 2-4.

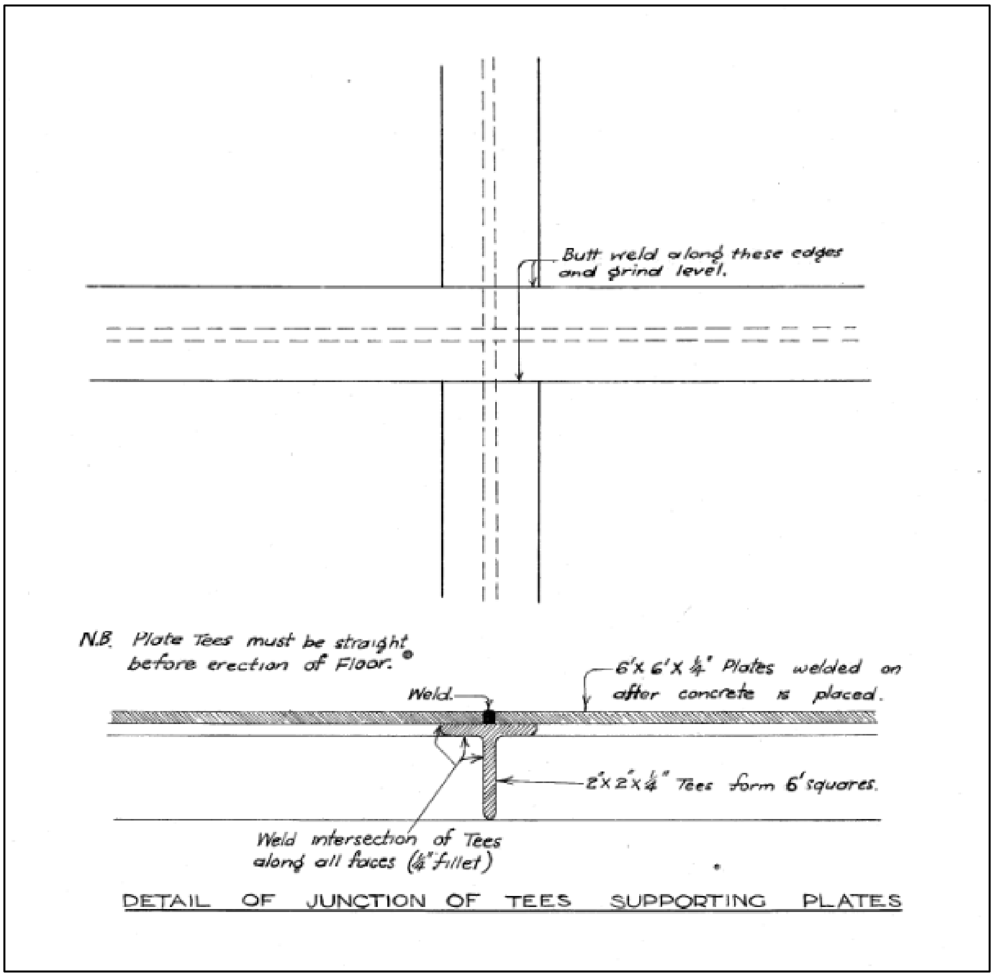


Figure 2-4. Weld detail between horizontal tees in base slab and steel plate

The liner coating the lower walls is made up of steel plates that are welded to the lower concrete wall through 60 vertical 63.5x63.5x10 steel tee members that are embedded into the concrete wall at approximately 1.2 m centres around the circumference of the tank and run the entire height of the lower wall.

The steel liner is attached to the lower ring beam by welding to the 20 vertical UB members around the circumference of the reservoir and a weld to the vertical steel plates connected to the lower concrete wall. The steel liner coating the upper wall runs behind the vertical UB members and is connected to the concrete through welds between the vertical 50x6.35 steel reinforcing plates at approximately 1.3 m centres.

A condition assessment of Ruakiwi Reservoir was completed in 2019 by Altex Coatings Limited. The assessment stated that the steel liner coating the walls has multiple cases of pitting corrosion as deep as 4 mm in some areas. Multiple areas of patch repairs were identified as having poor workmanship and needed repair. The assessment stated that "*of 200 panels, 35 would require full abrasive blast and 150 panels to varying spot repairs estimated at about 25% of the tank wall surface to be blasted and painted*".

The floor lining was in good condition but requires localised patch repairs. Refer to Figure 2-5 for images of the steel liner condition in 2019.

Figure 2-5. Overview of internal steel liner with spot corrosion (Altex Coatings, 2019)

2.3.2 ROOF

The roof is a pitched Dimondek 16 Roofing system that is supported by fourteen lengths of Brownbuilt metal purlins (B.P No. 8) at 1.8 m centres. The purlins are supported by eight lengths of 13 mm diameter bracing at approximately 2.6 m centres running perpendicular to the purlins. Refer to Figure 2-6 below for a plan view of the purlin and bracing layout.

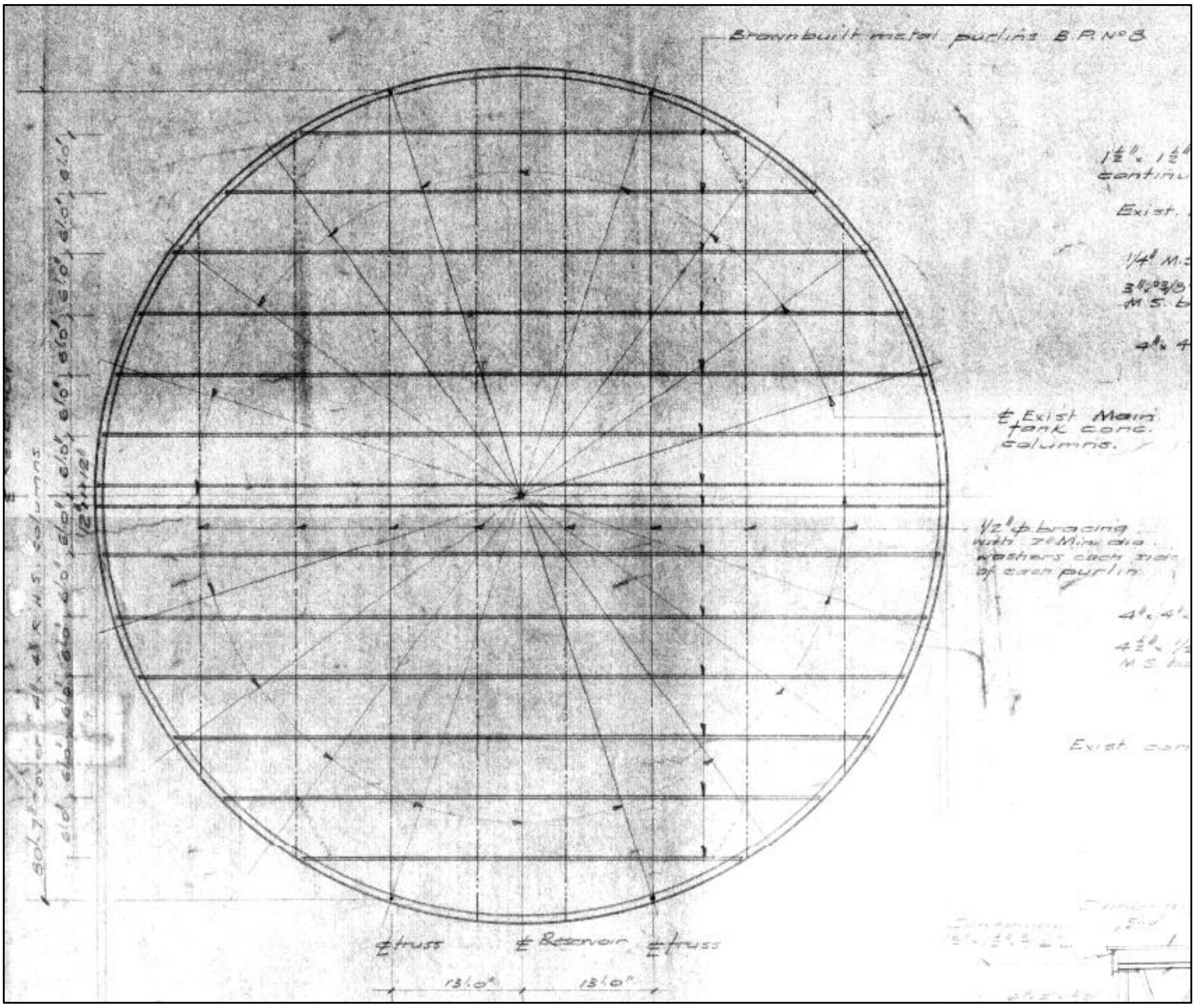


Figure 2-6. Plan view of purlin and bracing layout

There are two pitched steel trusses that run 24.5 m parallel to the 13 mm bracing located 4 m from either side of the centreline of the reservoir. The diagonal bracing members are made up of 2 No. 50x50x5 EAs that are welded to the bottom and top chord members. Every second diagonal brace member is connected at the midspan by a 100x100x6.35 steel plate that is welded to each EA. The chord members of the truss are made up of 2 75x75x6.35 EAs that are welded at the toes. At each end of the truss the chord members are replaced by a 1.5 m long 150x75x10 RHS that is welded to the EAs at one end and a 100x100x6.25 steel bracket at the other end. The steel bracket is welded to a vertical 100x100x6.35 SHS that appears to sit on the concrete structure.

Refer to Figure 2-7 for an elevation view of the truss system and Figure 2-6 for the location of each truss.

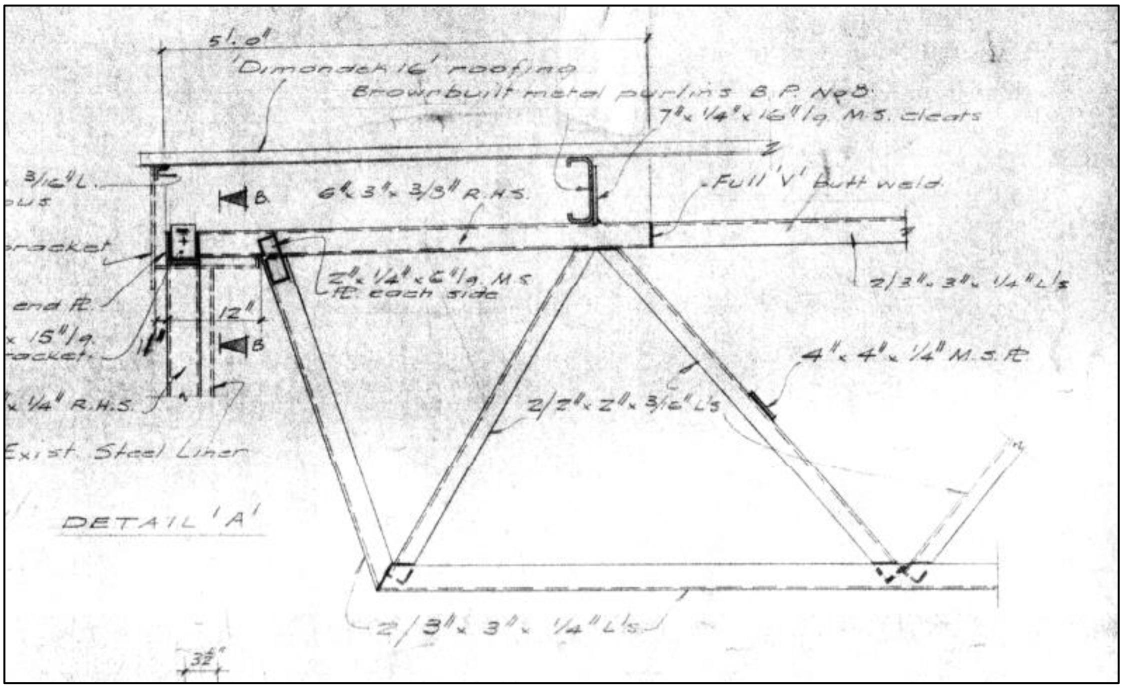


Figure 2-7. Elevation view of the truss system

WSP has not been provided with any information about the condition of the roof structure itself, it has been assumed that there is no significant deterioration within the roof.

The roof system appears to only be able to resist gravity loads as there was no observed means of transferring lateral forces in the provided drawings. Thus, the roof system was not able to transfer any lateral loading to other sections of the structure and was not analysed as a part of the lateral resistance of the structure. There appears to be no positive connection between the roof and reservoir as shown in Figure 2-8, meaning the reservoir roof can displace upwards vertically independent of the reservoir structure.

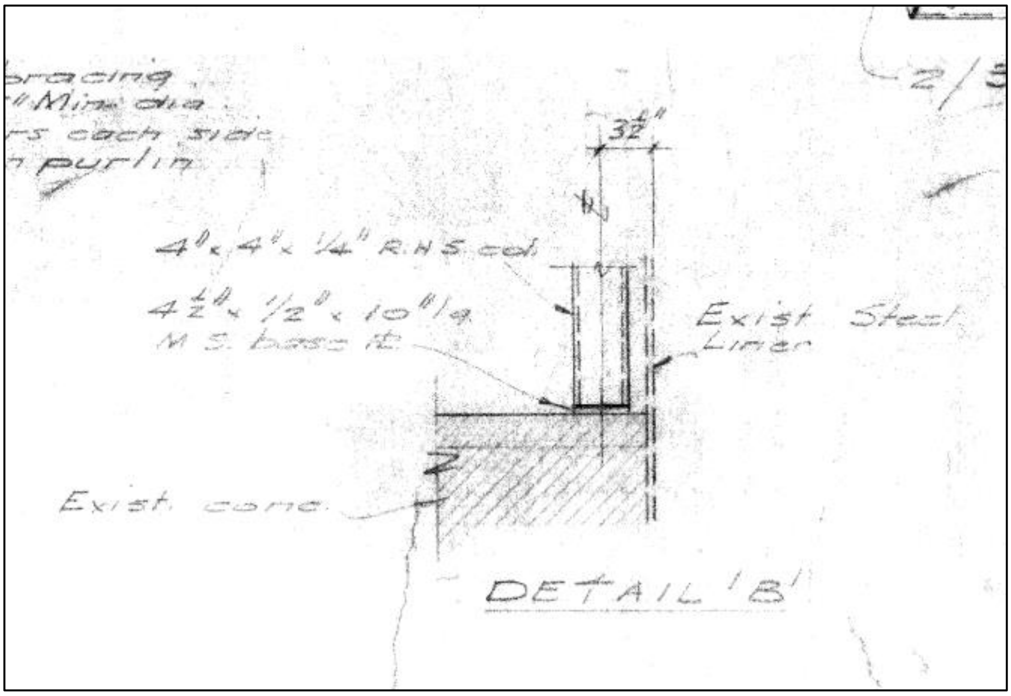


Figure 2-8. Connection between vertical RHS and reservoir

2.3.3 UPPER WALLS

The upper walls are made up of 178 mm thick reinforced concrete that has been poured in vertical stages. The wall is 4.27 m high and is reinforced with “arc-welded reinforcement”. The wall is horizontally reinforced by a layer(s) of 50x6.35 steel plates at the base of the wall that run around the circumference of the reservoir walls and is vertically reinforced by 50x6.35 steel plates at 1.3 m centres that are welded to each horizontal plate by a 6 mm fillet weld. Each vertical reinforcement member is welded to the steel liner.

There are 20 vertical 150x75 UB members 7.3m long at 4 m spacing running from the base of the wall to the roof. The UB members are welded to the steel liner. There are 20 vertical stiffening ribs with overflow pipes that run the entire height of the wall and form a walkway around the exterior of the reservoir. The ribs are vertically reinforced with 2 19 mm square bars and confined by 6.35 diameter rods at 300 mm centres.

A plan and elevation view of the upper section of Ruakiwi Reservoir is shown in Figure 2-9 and Figure 2-10 respectively.

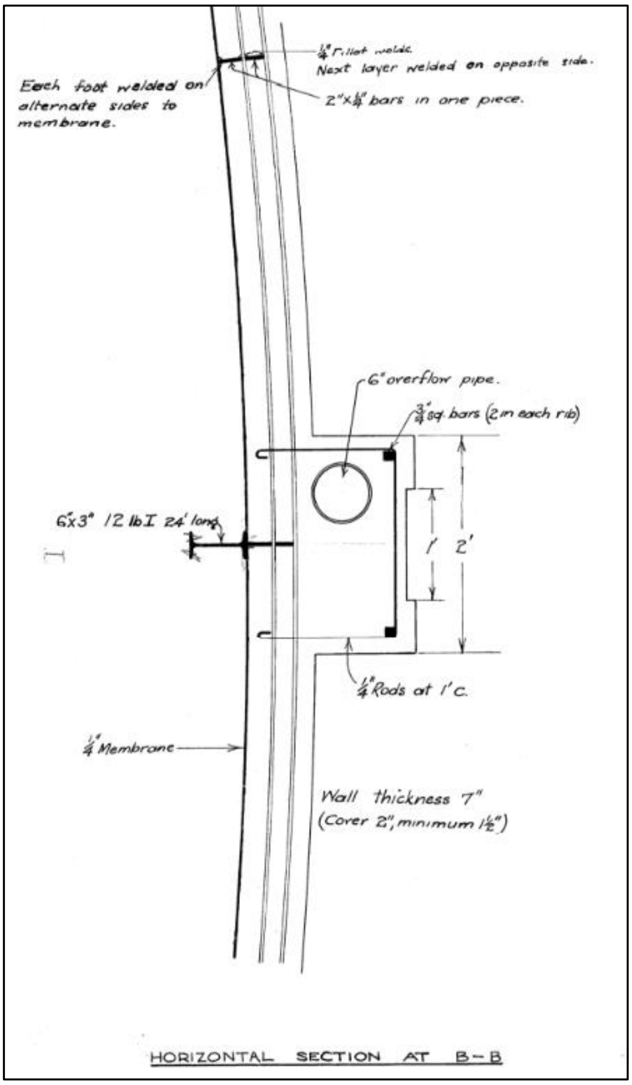


Figure 2-9. Plan view of upper wall section

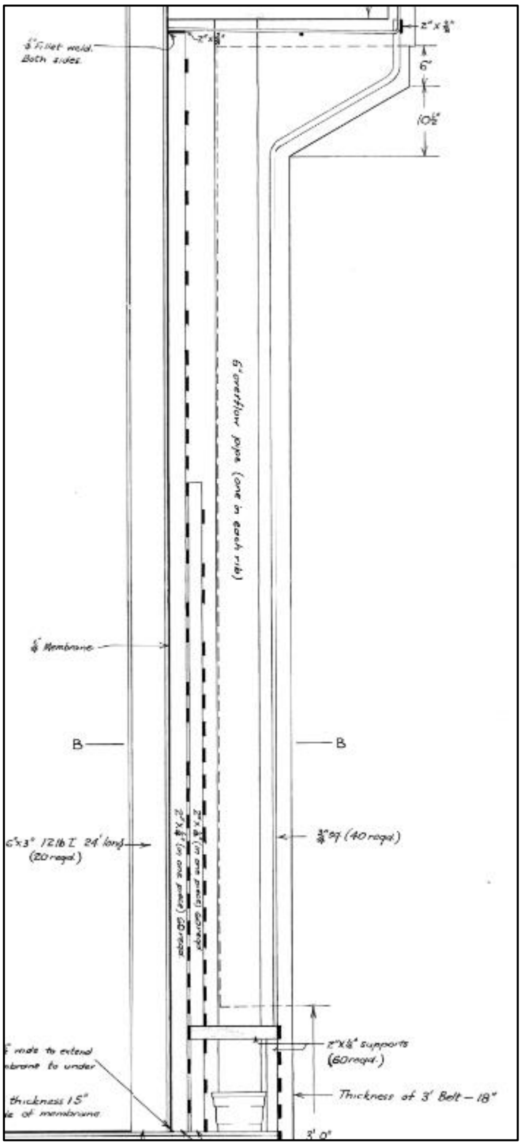


Figure 2-10. Elevation view of upper wall section

As stated in the 2002 Condition Assessment Report completed by Opus, the upper walls have no significant defects that are evident. There was evidence of leakage through construction joints, but they have not caused any significant defects. This was further observed in the Site Visit Record completed by WSP in July 2025.

The 2002 Condition Assessment report also made note of reinforcement corrosion in five of the ten stiffening ribs due to a lack of cover and spalling. This damage results in a minor reduction in structural capacity. Evidence of wall leakage and reinforcement corrosion is shown in Figure 2-11.

Figure 2-11. Evidence of leaking and reinforcement corrosion in the upper wall and rib (2025)

2.3.4 LOWER RING BEAM AND COLUMNS

The lower ring beam that connects the upper and lower walls is comprised of 120 horizontal 300x125 UB members that are 1.5 m long at 600 mm spacing. There are 4 no.19 mm square rods that are welded to the bottom flange of the UBs and run radially around the circumference of the reservoir. The square bars are encased in concrete that connects to the hollow RC columns below through the column reinforcing. An elevation view of the lower ring beam and the column to ring beam connection is shown in Figure 2-12.

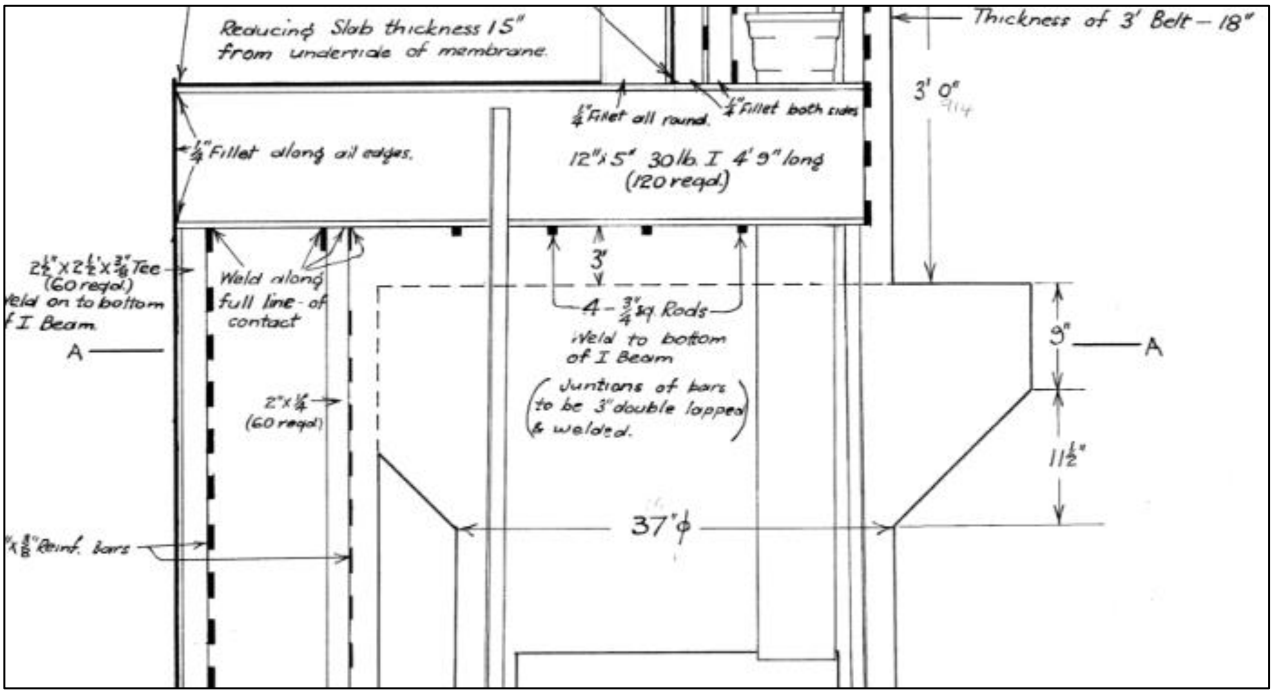


Figure 2-12. Elevation view of ring beam section.

The RC columns are 19 m tall hollow cylinder sections. The top of the column is comprised of a 520 mm thick 1.5 m square RC section that supports the ring beam. The top square section tapers to the diameter of the column over a 300 mm height. The column diameter widens over an 8 m height from 940 mm to 1118 mm, where it spans 8.8 m at a constant diameter. The base 600 mm of the column is a square 1.118 m section that connects to the base slab through the column reinforcement. The hollow diameter of the column is a constant 686 mm that stops at the square sections at the top and bottom of the column.

An elevation view of the top of the column is shown in Figure 2-12 and the lower section of the column into the base slab is shown in Figure 2-13.

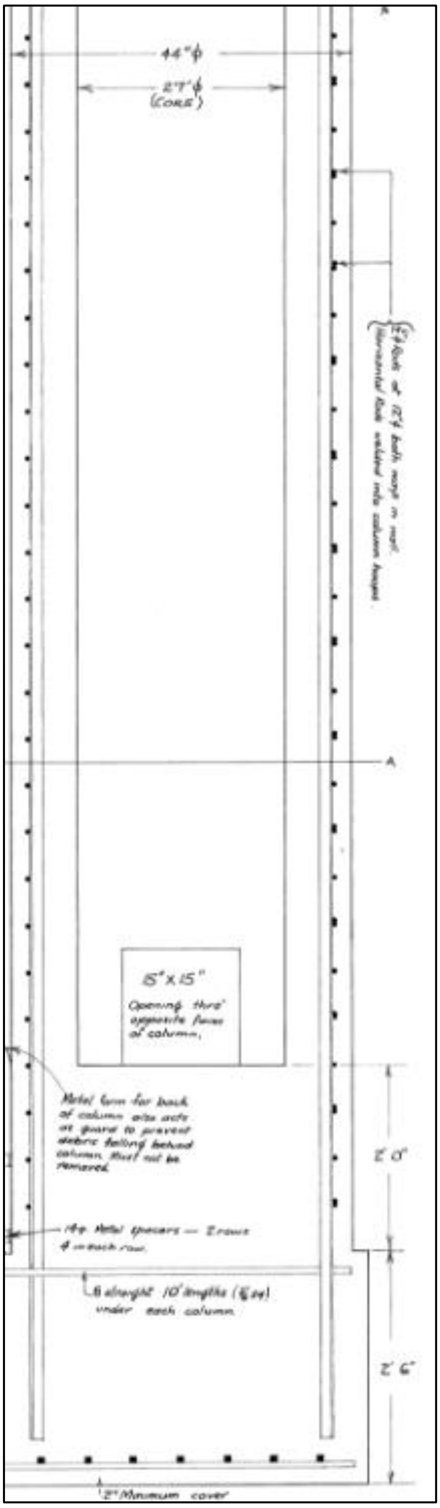


Figure 2-13. Elevation of base of RC column

The column is vertically reinforced by 10 No. 38x38x6.35 undeformed steel tees around its circumference and confined by 12 mm diameter hoops at 150 mm centres as shown in Figure 2-14.

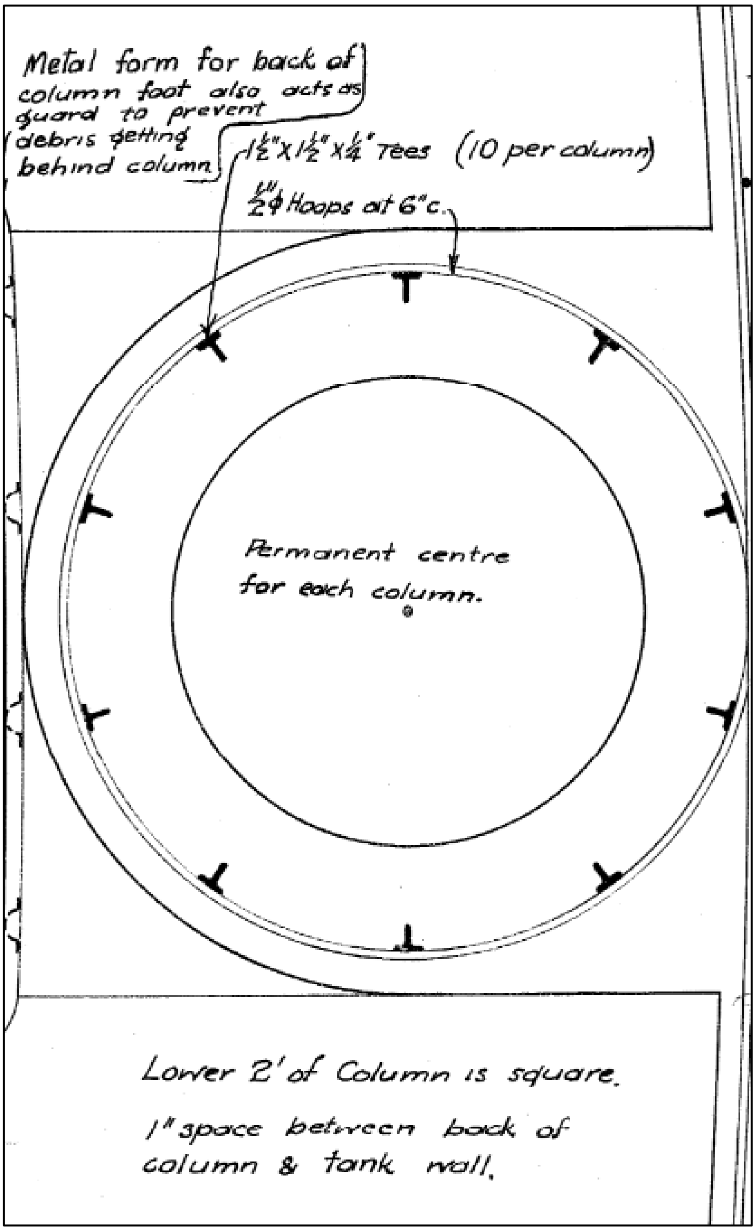


Figure 2-14. Cross section of RC columns

The columns are designed to carry vertical load from the ring beam to the base slab. The connections between the top and bottom of the columns of the reservoir structure appear to only provide resistance for lateral force based on the reinforcement detail.

The connections between the top and bottom of the column and the reservoir structure are comprised of undeformed tees that extend from the column into the reservoir structure. The connection appears to lack any form of defined anchorage depth.

The columns conceal downpipes that drain the walkway from any excess water. As noted in the 2002 Condition Assessment Report completed by Opus the columns display significant water leakage in the mid-height horizontal construction joint and are “*extensively affected by cracking and spalling as a result of reinforcement corrosion*”.

2.3.5 LOWER WALL

The lower wall is made up of two sections that have both been poured in vertical stages. The top lower wall section is 432 mm thick and is 14.8 m tall while the bottom lower wall section is 483 mm thick and is 3.8 m tall. The lower wall has been reinforced using the same method as the upper wall, comprising of seven (bottom) or six (top) layers of horizontal 50x10 mm steel plates at 50 mm centres and 5 (top) or 6 (bottom) layers of vertical 50x6 mm steel plates at 1.2 m centres that are welded to the horizontal reinforcement.

The horizontal plates run radially around the entire circumference of the reservoir and linearly reduce as they travel up the height of the wall until there are 3 layers at the top of the wall. The vertical plates reduce from five layers to one layer at the same segments as the horizontal reinforcement. The vertical reinforcement on the inside face of the wall is attached to the steel liner along its entire length.

An elevation view of the uppermost section of the lower wall is shown in Figure 2-12. An elevation and plan view of the bottom section of the lower wall is shown in Figure 2-15 and Figure 2-16 respectively.

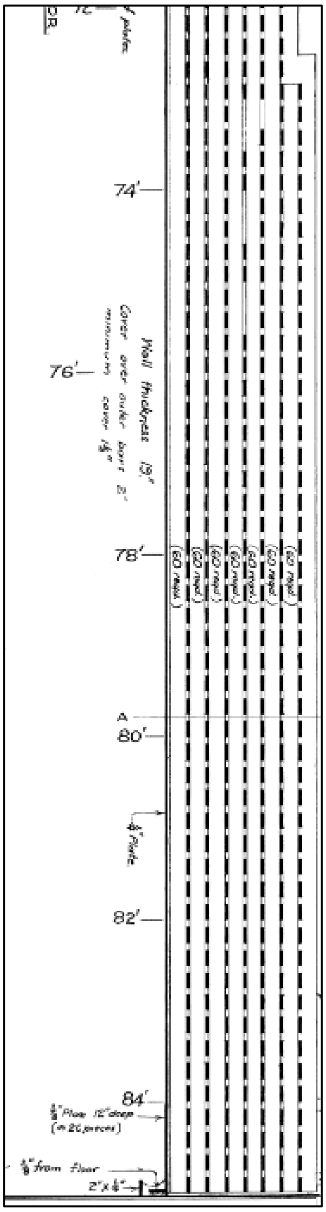


Figure 2-15. Elevation view of base of lower wall section

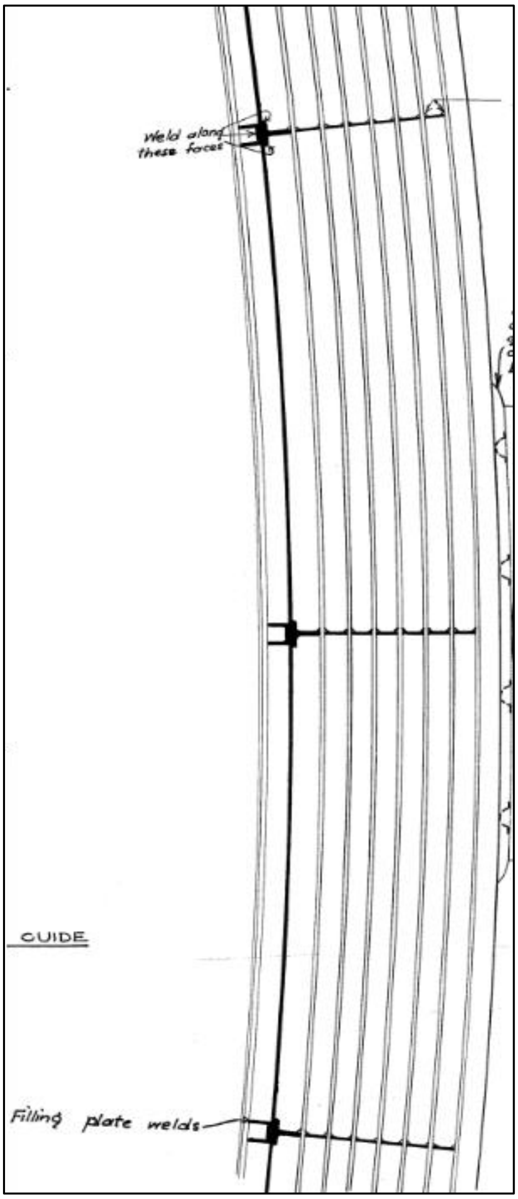


Figure 2-16. Plan view of base of lower wall section

The wall does not appear to have any meaningful connection to the base slab to resist lateral loading. The vertical reinforcement is welded to an expansion guide which allows the reservoir walls to move independently of the base slab. The guide is made up of a 60x10 mm steel plate at the base of the wall and is slotted in between two 50x10 mm steel plates that are coated in stainless steel to allow sliding. The outside plates are then welded to the steel liner on the base slab. There is an expansion joint at each expansion guide location at the base of the wall that has been upgraded in 1948.

A detail of the expansion guide and joint is shown in Figure 2-17 and Figure 2-18 respectively.

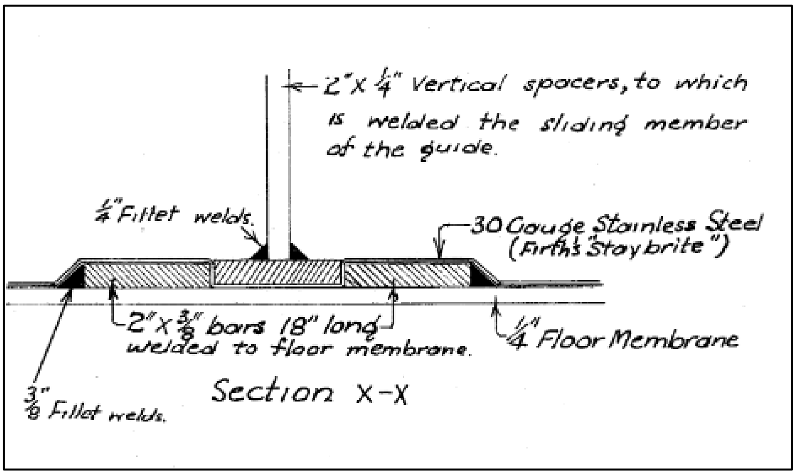


Figure 2-17. Detail of expansion guide

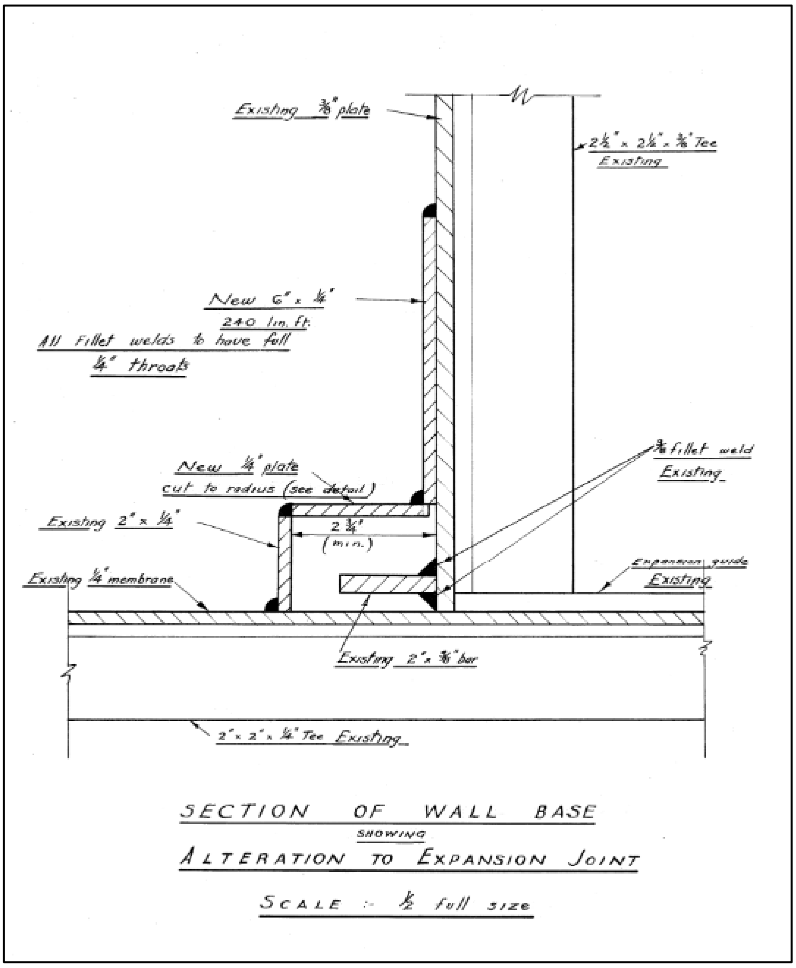


Figure 2-18. Detail of expansion joint upgrade (1948)

The lack of any meaningful connection between the reservoir wall and the base slab to resist lateral loading as indicated by the provided structural drawings means that the connection between the wall and base slab has been assumed to allow full lateral displacement in both directions. The connection will provide a nominal level of resistance, but accurate quantification of this requires on site testing and is outside the scope of this DSA. The nominal resistance has been ignored to provide a conservative estimate of the reservoir's behaviour under seismic loading. The base of the reservoir appears to be embedded in soil around the circumference at approximately 2.5 m depth.

There is evidence of leaking that has occurred in the construction joints of the lower wall as noted in the 2025 WSP Site Visit Record (refer to Appendix A). There was confirmation from HCC that efforts had been made to repair leaks within the walls.

The 2002 Condition Assessment Report completed by Opus also notes that there is carbonation within the concrete of a maximum depth of 62 mm which exceeds the cover of the concrete. The report states that there are only traces of reinforcement corrosion due to the wall sections remaining relatively dry up until that point. There are significant vertical offsets between construction joints at the lower sections of the wall due to poor formwork alignment. The walls were noted to generally be in good condition in both documents (2025, 2002).

2.3.6 BASE SLAB

The foundation of Ruakiwi Reservoir is comprised of a 762 mm thick RC slab with 20 mm square bars at 150 mm centres each way with 50 mm cover from the bottom of the slab. There are 8 No. straight 3 m lengths of 20 mm square bars under each column and 12 No. 3 m lengths of 20 mm square bars in between each column. There are 50x50x6.35 steel tees that act as supporting plates for the steel liner at 1.8 m centres each way that form 1.8 m squares. A view of the slab section under the column and between the column is shown in Figure 2-19 and Figure 2-20 respectively.

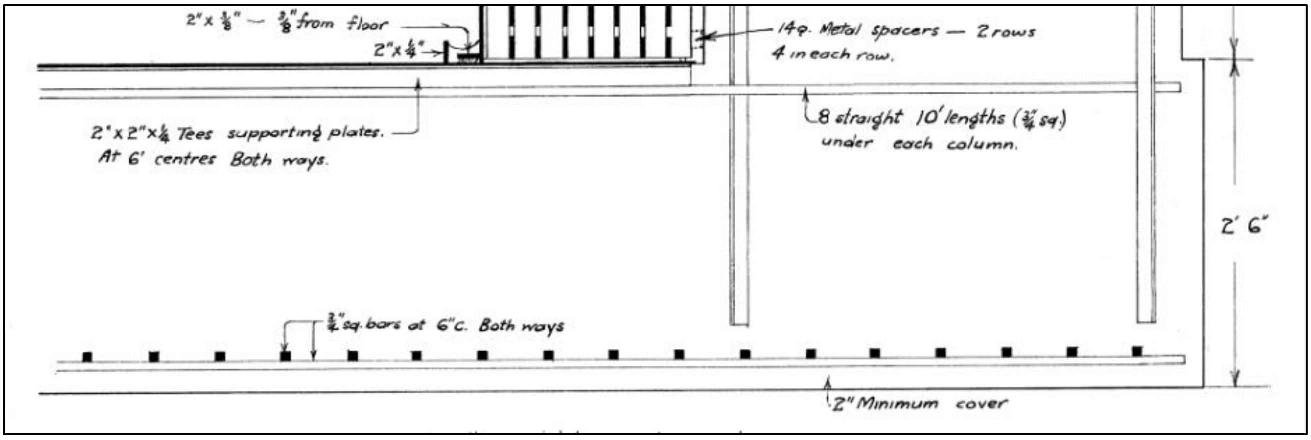


Figure 2-19. Radial section of base slab under wall and column

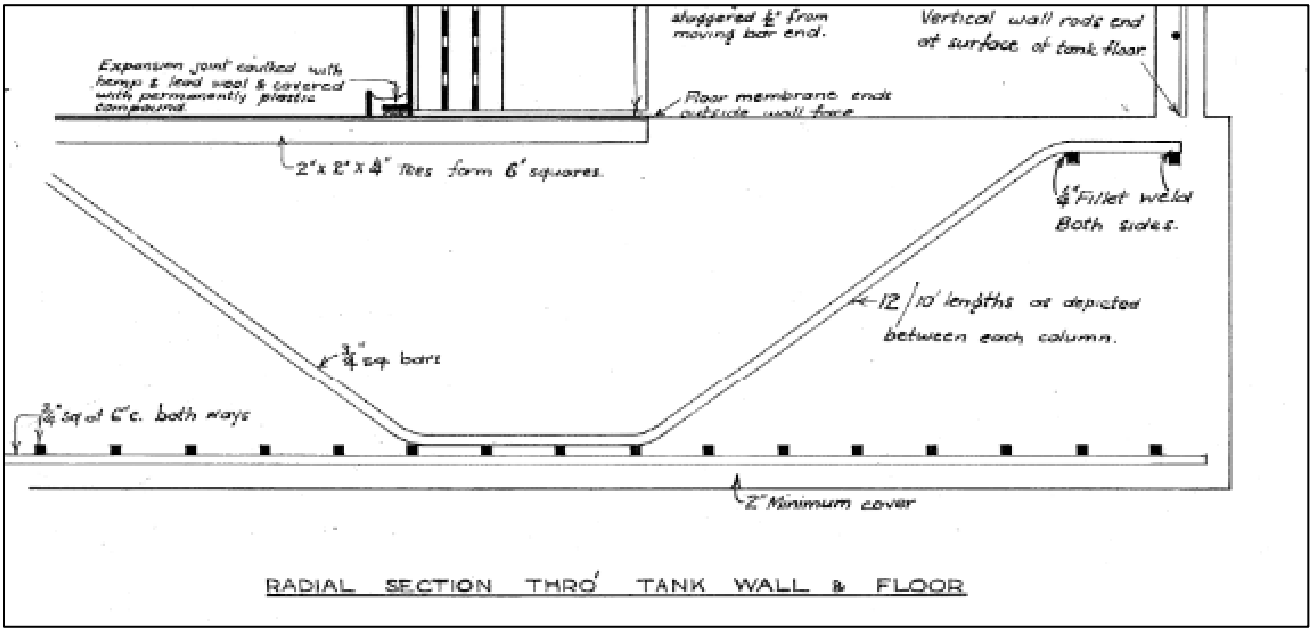


Figure 2-20. radial section of base slab under wall in between columns

There is no access to the base slab for visual inspection, and no condition assessment of the base slab was provided to WSP because the structure is partially embedded in the ground and is internally lined with steel.

2.4 STRUCTURAL ASSESSMENT

The structural assessment for Ruakiwi Reservoir was completed assuming the structure would not be retaining any water or other fluids. Table 2-2 shows the key characteristics of the structure used to determine the expected seismic demand on the reservoir in accordance with DZ TS1170.5 and NZ1170.0.

Table 2-3 shows the seismic load inputs for the structure and Table 2-4 shows the assumed material properties used for the structure.

Table 2-2. Key Specific Structural Characteristics of Ruakiwi Reservoir

CHARACTERISTIC	**VALUE**
Period (T_1)	0.3 seconds
Ductility, ULS (concrete)	1.25
Ductility, ULS (steel)	1.00
Site Class	IV

Table 2-3. Seismic load inputs for Ruakiwi Reservoir (ULS)

CHARCTERISTIC	**DZ TS1170.5 – IL2**
$C(T_1)$ (horizontal spectra)	0.44
$C_v(T_1)$ (vertical spectra)	0.31

Table 2-4. Material Properties for Ruakiwi Reservoir

CHARACTERISTIC	VALUE
Concrete Strength, f'c	25 MPa[1]
Reinforcement Yield, fy	200 MPa[1]
Steel Yield, fy_s	250 MPa[1]
Friction Coefficient (Between Steel Surfaces)	0.4

[1] As the drawings that were available of the structure do not indicate the material strengths, values were derived as per the 2009 Aurecon Water Reservoir Structures Seismic Review, experience or the age of the structures according to Section C5 in "The Seismic Assessment of Existing Buildings" guidelines.

Table 2-5. Summary of Capacities of Critical Structural Items, Haywards A Reservoir

MAIN STRUCTURAL COMPONENT	%NBS1 IL 2 (NZS170.5)
Roof-to-wall connection	>100%
Reservoir uplift	>100%
Wall-to-base connection	Vulnerable to sliding failure (< 34%)
Bearing on soil	>100%

Note: These values are based on assuming the seismic actions have not comprised the structural capacities.

A Finite Element model of Ruakiwi Reservoir was created in SAP2000 based on the information in the provided drawings to determine how the structure would perform under the design loading. The reservoir and roof structure were modelled separately for simplicity. The model for the RC structure is shown below in Figure 2-21 and the model of the roof structure is shown below in Figure 2-22.

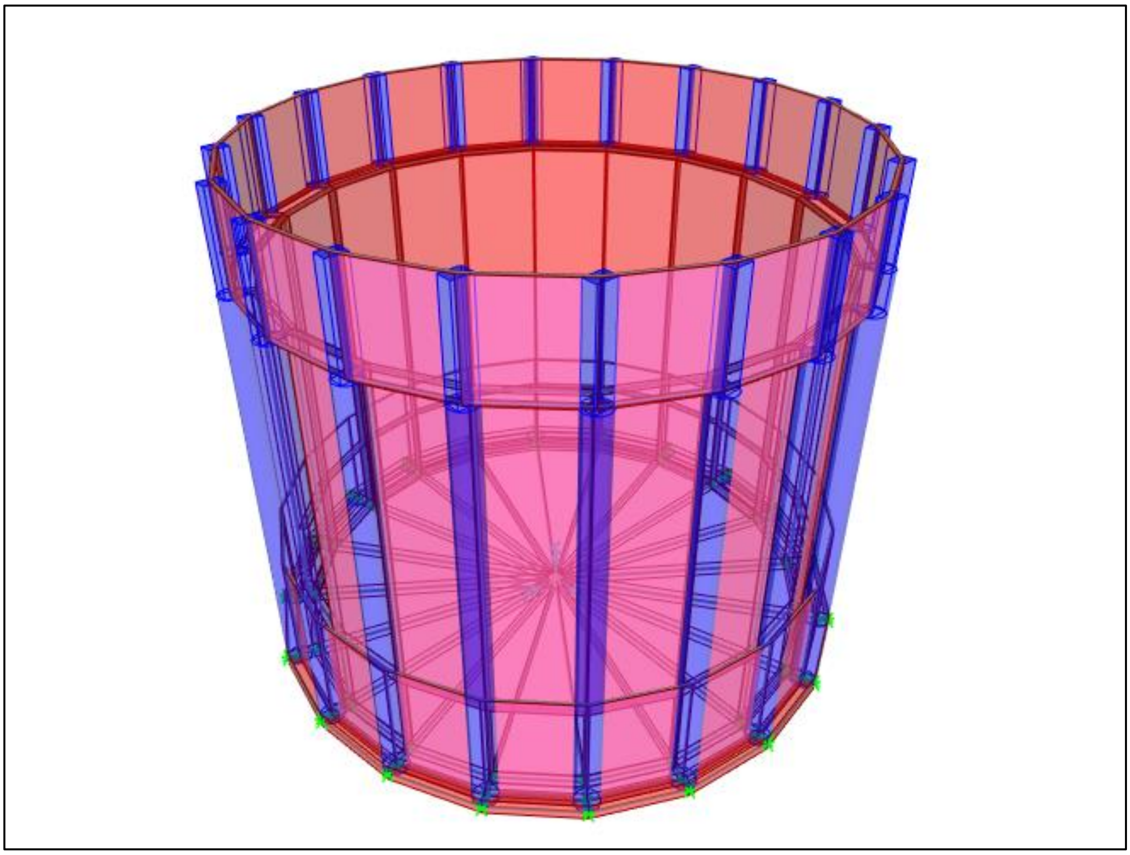


Figure 2-21. Finite Element Model of Ruakiwi Reservoir

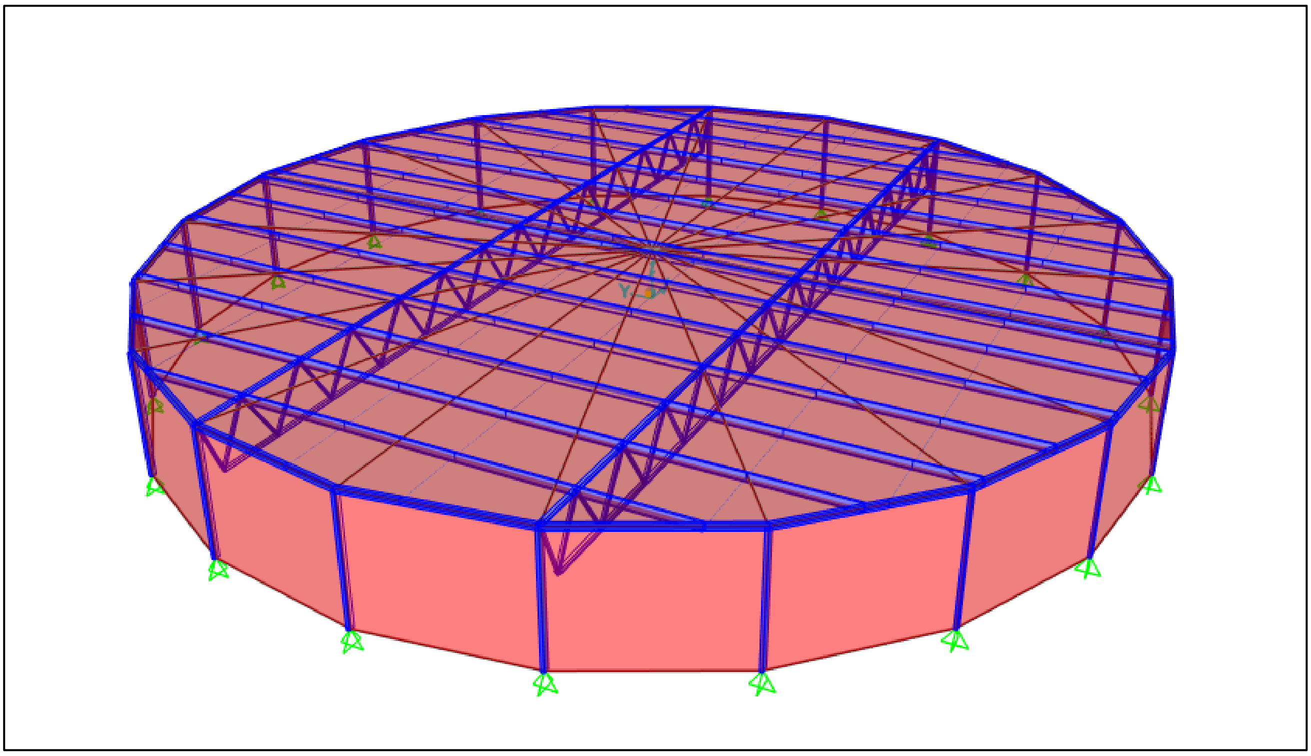


Figure 2-22. Finite Element Model of Ruakiwi Reservoir roof structure

The critical structural weakness (CSW) within Ruakiwi Reservoir is the connection between the reservoir wall and the base slab. There is no meaningful connection between the wall and the base slab, and the shear at the base of the wall is entirely resisted by friction resistance.

The roof structure has been assumed to only be capable of resisting gravitational loads and provides no lateral support against seismic loads from the structure. A %NBS rating for the roof was omitted from this DSA as it does not contribute to the seismic performance of the overall structure.

2.4.1 ROOF-TO-WALL CONNECTION

The roof structure is an entirely steel structure and is light weight and generates low seismic demands on the structure. There is no positive connection between the roof structure and the reservoir, meaning the roof will be able to displace upwards vertically in a seismic event. However, the vertical seismic demands acting on the structure are not significant enough to induce uplift in the roof. The roof is restrained laterally by the 20 vertical I-beams that run around the circumference of the reservoir wall and the upper concrete wall and will not cause significant demands within the reservoir structure.

2.4.2 RESERVOIR UPLIFT FROM BASE SLAB

The weight of the reservoir structure has been confirmed to be sufficient to resist the vertical seismic demands induced by an IL2 event. This means there is no significant risk of uplift from the reservoir structure.

2.4.3 WALL-TO-BASE SLAB CONNECTION

The reservoir wall has no mechanical connection to the base slab. A conservative assumption was made in this DSA that the only resistance for the reservoir structure overturning on the base slab is its own weight and the hollow core columns connected to the base slab. A conservative assumption was also made for the lateral resistance against the reservoir structure sliding off the base slab because of the absence of a mechanical connection between the wall and the base slab, and the lack of quantifiable resistance from the hollow-core columns and the soil profile.

The assumption was made that the resistance against sliding would be entirely provided by friction between the wall and base slab, and the nominal resistance provided by the columns and soil would be ignored.

2.4.3.1 OVERTURNING

During a seismic event the wall and the base slab will move independently of each other, with vertical restraint against uplift being provided by the weight of the structure with nominal support from the hollow core columns. The combined weight of the structure has been determined to be sufficient to resist the seismic overturning demand without the additional support from the column connection and reaches a >100 %NBS rating.

2.4.3.2 SLIDING

Lateral restraint against sliding due to seismic demands will be provided by friction between the steel expansion guide plates underneath the wall and a nominal amount of resistance by the hollow-core RC columns and soil profile. The structural connection between the columns and the reservoir do not provide quantifiable lateral resistance against design loadings.

The lateral resistance provided by the columns was ignored in this DSA.

The friction between the wall and base slab and the passive soil resistance provide the entirety of the lateral restraint against seismic demand. The capacity provided is not sufficient to overcome the seismic demand, and the reservoir structure above the base slab would be expected to significantly displace relative to the base slab in an IL2 ULS seismic event.

2.4.3.3 BEARING ON SOIL

The ground bearing demand is expected to be vertical loading of 100 kPa under a ULS event based on the section sizes provided from the existing structural drawings, and the NZS1170.5:2004 seismic load derivation for an IL2 structure.

No settlement will occur after a design magnitude seismic event based on the assumption that the ground underneath the reservoir is assumed to have a ULS bearing capacity of 300 kPa (based on existing drawings and previous seismic assessments).

This small demand relative to the bearing capacity of the ground is due to the analysis considering the tank as being empty of water, significantly reducing the weight and bearing demand from the structure.

3 STRENGTHENING WORKS

The only element within Ruakiwi Reservoir that requires strengthening is the base to wall connection. The connection could be strengthened to either 34%NBS, 67%NBS or 100%NBS, depending on HCC's preference. The NZSEE guidance recommends strengthening all structures to 67%NBS with consideration of additional works required to reach 100%NBS or above.

The strengthening required would be to implement a reinforced concrete perimeter ring beam that would travel around the inside circumference of the base of the lower reservoir wall. The ring beam would be fixed to both the reservoir wall and base slab through drilled dowels to ensure an adequate structural connection between the two sections.

A typical example of the recommended strengthening is shown below in Figure 3-1.

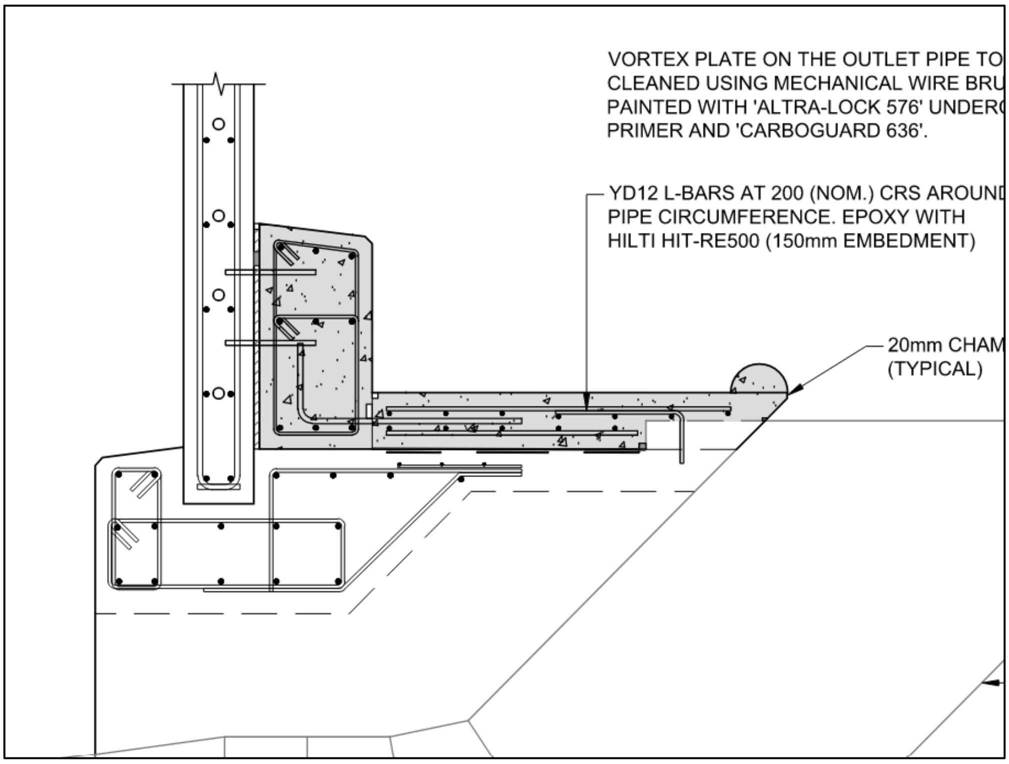


Figure 3-1. Example of typical RC ring beam detail

The cost of this approach would be similar for each %NBS rating, as the only additional expense would be added materials to strengthen the ring beam. A rough estimate of the cost and timeframe of the required strengthening is $100,000 - $150,000 with a time frame of approximately 1 month.

The cost estimate provided is based on actual construction costs of similar recent works. It includes all design, tendering, construction and MSQA costs associated with the proposed works.

While this is based on hard data from similar works, we note that there are likely to be fluctuations in many contributary variables which would affect the construction cost for the recommended strengthening works.

Such variables include location, material costs, time of year, available labour, contractors forward workload etc.

We therefore recommend using a local Quantity Surveyor (QS) to provide a more accurate overall cost estimate upon completion of preliminary and/or detailed design of the proposed strengthening scheme.

4 CONCLUSION

The structure has been identified as having a Seismic Resilience Class of D, with a New Build Standard ratio as less than 34 %NBS. The overall structural capacity of the reservoir is < 34%NBS when considering the DZ TS1170.5 seismic loading derivation method.

Clause 133AA within Subpart 6A of the Building Amendment Act 2016 includes a definition of the structures covered by the earthquake-prone buildings special provisions. This definition notes that storage tanks are excluded from these provisions. However, this structure is to be decommissioned and will no longer be used to store water. Thus, the structure is considered a monument (that is capable of being entered by a person) under the Building Act 2004 and is included in the Earthquake Prone Building Provisions within the Building Act.

The provisions require the structure to be strengthened to > 34%NBS or demolished within 25 years of this DSA as per Clause 133AM of Subpart 6A of the Building Act. The NZSEE guidelines recommend strengthening to 67 %NBS or beyond. However, it is often most cost-effective to target 100%NBS when strengthening a structure. The reservoir %NBS rating is limited by the connection between the reservoir wall and the base slab.

WSP has identified the lack of meaningful connection between the reservoir wall and base slab as the governing critical structural weakness.

WSP recommends the following:

- Strengthen the connection between the reservoir wall and base slab with a RC perimeter ring beam to provide a quantifiable resistance and load path against out-of-plane loads to achieve a >34 %NBS rating as an IL2 structure with a 50-year design life.
  - This must be completed within 25 years of receiving this DSA as per the Building Act. However, it can be undertaken as part of other works to change the use of the structure.
- Carry out routine maintenance to all concrete and steel elements as required.

5 LIMITATIONS

This report ('Report') has been prepared by WSP New Zealand Limited ('WSP') exclusively for Hamilton City Council ('Client') in relation to the Detailed Seismic Assessment of Ruakiwi Reservoir ('Purpose') and in accordance with the PSP IFS Variation Form agreed to by the client on 27th of June 2025 ('Agreement'). The findings in this Report are based on and are subject to the assumptions specified in the Report, the PSP IFS Variation Form agreed to by the client on the 27th of June 2025 and the previously issued design drawings, seismic assessments and condition reports included in Appendix A, B, C, D, E of this report. WSP accepts no liability whatsoever for any use or reliance on this Report, in whole or in part, for any purpose other than the Purpose or for any use or reliance on this Report by any third party.

In preparing this Report, WSP has relied upon data, surveys, analyses, designs, plans and other information ('Client Data') provided by or on behalf of the Client. Except as otherwise stated in this Report, WSP has not verified the accuracy or completeness of the Client Data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in this Report are based in whole or part on the Client Data, those conclusions are contingent upon the accuracy and completeness of the Client Data. WSP will not be liable for any incorrect conclusions or findings in the Report should any Client Data be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to WSP.

APPENDIX A

WSP (2025) - SITE VISIT RECORD

Site Visit Record

Contract: Ruakiwi Reservoir

Day & Date of Visit: 09 JUL 2025	**Arrival Time:** 11am	**Departure Time:** 11:45am
Weather and Site Conditions: cloudy		

Health & Safety and Environmental Compliance (Notify contractor of any non-compliances)
Limited access on site – avoid working at heights and water. - Use the stairs provided and accommodated by the HCC staff on site.

Site Records/Observations

This site visit was to examine the current conditions of deterioration or damage to the existing structure that may impact the results of Detailed Seismic Assessment.

The existing reservoir is a casted reinforced concrete structure that was built at approximately 1928 according to the dates of the existing drawing.

The structure is a cylindrical structure with concrete walls that were poured in vertical stages. Near the top of the reservoir, there appears to be an outer reinforced concrete ring (gravity weight supported by reinforced concrete columns) that is supporting the walls from out of plane forces due water pressure once the reservoir is full. The columns also appear to be hollow in the middle which act as a downpipe to transfer stormwater from the roof to the drainage system in the ground.

The roof is a truss type system and appears to only support gravity loads.

At the time of the site visit, the reservoir still had some water retained (amount unknown) inside and no access was available to examine the inside of the reservoir.

Upon walking around the reservoir, the external conditions of the reinforced concrete were in acceptable conditions where few locations appeared to have signs a leak from the inside the reservoir shown in photo below.

Upon discussing these signs with the staff from HCC, the staff member verbally confirmed that there were previous works to fix the leaks in the past. The staff also explained that there is an approximately 12mm thick steel plate that act as a seal, where the steel cold joints were welded using fusion weld techniques.

There were also signs of leaks in one of the columns acting as a downpipe as shown below.

Furthermore, some cracking of the external lower wall was found along the circumference of the reservoir, however appeared to impose low risk to the main reservoir structure.

In summary, the external condition appeared to be in acceptable condition. However, further intrusive investigation in the inside of the reservoir where a leak has occurred will be required to fully determine the condition of the existing reinforced concrete.

Signature:

APPENDIX B

WSP (2025) - GEOTECHNICAL INPUT MEMORANDUM

MEMORANDUM

To	Lewis Thomas, Kristina Macnaughtan
Copy	Mladen Sigurnjak
From	Kaylee Wu
Office	Christchurch
Date	3 July 2025
File/Ref	3-39777.00/ task 00DSA
Subject	Ruakiwi Reservoir – Future Use Options Assessment / DSA geotech input

1. Site description

The IAF reservoir site is located near the existing reservoir at Lake Domain along Ruakiwi Road, Hamilton Lake, and the proposed site will be built adjacent to the existing reservoir.

2. Review of existing geotechnical investigations data

Based on the available data provided by CMW, the geological profile at the site is expected to comprise the following.

Table 1: Ground Profile Inferred from 2025 report

Geological Unit	Material	Depth to Top of Layer (mbgl)	Thickness of Layer (m)
Fill	Topsoil, sand, silty clay	0	0.4-2
Hamilton Ash	soft to hard CLAY	0.4-2	0.45-4.75
Walton Subgroup	1b: soft to hard clayey SILT; 1b*: stiff to very stiff sandy SILT	5-6.75	6.4-10.5
Walton Subgroup	1c: medium dense to dense SAND; 1c*: dense to very dense silty SAND	7-16.95	13.05-22.9

Groundwater was measured in the existing investigations based on the Borehole log and Hand Auger results provided in the CMW factual report, ranging from approximately 3m to 23.7m bgl. Additionally, Table 5 from the factual report shows that the reservoir groundwater table ranges from 0.8m to 19.6m bgl.

3.6 Groundwater Data

A summary of the measured water tables between March 2024 and February 2025 is summarised below:

Table 5: Measured Water Tables between March 2024 and February 2025

Location	Piezometer	Elevation (RL m)	Screen Depth (m bgl)	Depth min-max (m blg)	Level max - min (RL m)
Reservoir	BH24-01	65.5	0.5-3.5	3.4 - dry	62.1 - dry
			16.5-29.6	18.7 - 19.6	46.8 - 45.9
	BH24-02	59.9	1.0-3.0	0.8 - dry	59.1 - dry
			4.5-5.5	1.7 - dry	58.2 - dry
	BH24-03	61.7	3.0-6.0	5.9- dry	55.8 - dry
			19.0-28.0	17.2- 18.1	44.5 - 43.6
	BH24-04	58.3	1.0-3.5	0.8- dry	57.5 - dry
			20.0-26.0	15.3 - 16.3	43.0 - 42.0
Pump Station	BH24-07	54.8	6.5 - 12.1	10.5 - 11.0	44.3 - 43.8
			14.0-18.0	10.6 - 11.2	44.2 - 43.6
Note: Elevation levels of Hand auger locations have been inferred from existing contour data from Hamilton City Council.					

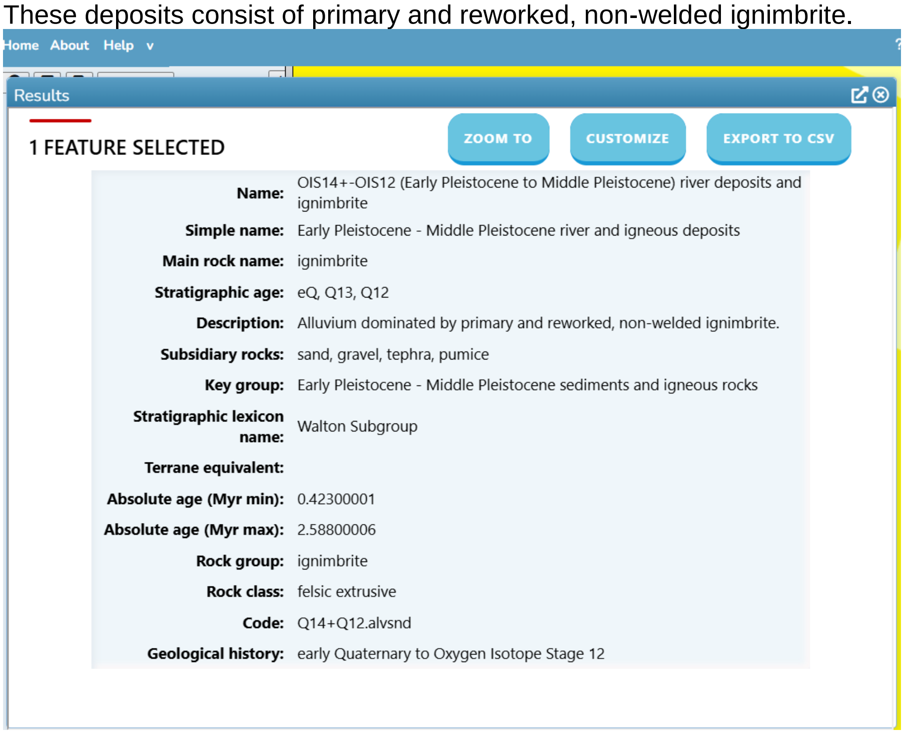
Graphs representing the variation of the water tables over that period are shown in ***Appendix F***.

3. Review of site geology

Based on the CMW site-specific seismic hazard assessment report, the North Island of New Zealand has a complex tectonic setting, with the East Coast bordering the Hikurangi Subduction Zone and the central region home to the Taupo Volcanic Zone. The site is within the Waikato Basin, an alluvial basin with sediments from the Taupo Volcanic Zone, characterised by normal faulting. The closest active fault is the Kututaruhe fault, located 3 km away.

Additionally, according to the factual report, the IAF reservoir site is near Lake Domain along Ruakiwi Road, Hamilton Lake. The proposed reservoir will be built next to the existing one over rolling hills, descending from RL68m to RL56m at a 1:11 gradient, then more steeply to RL38m at the Rotoroa/Hamilton Lake margin.

Moreover, based on the GNS NZ Geology Web Map, the site is likely composed of OIS12 (Middle Pleistocene) and OIS14 (Early Pleistocene) river deposits, classified under the Walton Supergroup. These deposits consist of primary and reworked, non-welded ignimbrite.



Name:	OIS14+-OIS12 (Early Pleistocene to Middle Pleistocene) river deposits and ignimbrite
Simple name:	Early Pleistocene - Middle Pleistocene river and igneous deposits
Main rock name:	ignimbrite
Stratigraphic age:	eQ, Q13, Q12
Description:	Alluvium dominated by primary and reworked, non-welded ignimbrite.
Subsidiary rocks:	sand, gravel, tephra, pumice
Key group:	Early Pleistocene - Middle Pleistocene sediments and igneous rocks
Stratigraphic lexicon name:	Walton Subgroup
Terrane equivalent:	
Absolute age (Myr min):	0.42300001
Absolute age (Myr max):	2.58800006
Rock group:	ignimbrite
Rock class:	felsic extrusive
Code:	Q14+Q12.alvsnd
Geological history:	early Quaternary to Oxygen Isotope Stage 12

4. Site subsoil class, Vs30 estimate

4.1 Soil classification

According to the CMW site specific seismic hazard assessment report, the site subsoil class can be determined as class D - deep or soft soil site in accordance with NZS1170.5 (2004).

4.2 PGA

According to the CMW site specific seismic hazard assessment report, the water reservoir and pump station are Importance Level 4 structures in accordance with NZS1170.0 (2002). The reservoir will have a 100-year design life. The design return periods are as follows:

- SLS1: 1/25 year
- SLS2: 1/1000 year
- ULS: 1/2500 year

Using this information, the ULS and SLS geotechnical peak ground accelerations (PGAs) have been determined in accordance with MBIE Module 1 Guidelines (Ministry of Business, Innovation & Employment, 2021). The geotechnical PGAs are shown in Table 2.

Table 2: ULS and SLS Geotechnical PGAs in accordance with Module 1 Guidelines, Table A1

Limit State	Return Period	Magnitude	Geotechnical PGA
SLS1	1/25	5.9	0.06g
SLS2	1/1000	5.9	0.32g
ULS	1/2500	5.9	0.44g

4.0 DESIGN STANDARDS

The water reservoir and pump station are Importance Level 4 structures in accordance with NZS1170.0 (2002), with post disaster functions. The reservoir will have a 100-year design life and the pump station will have a 80-year design life. The design return periods are as follows:

- SLS1: 1/25 year.
- SLS2: 1/1,000 year.
- ULS: 1/2,500 year.

An updated earthquake design loadings standard, Technical Specification 1170.5 (2024) (Draft)[1] (henceforth DZ TS 1170.5 (2024)) was released as draft for public consultation comment in February 2024. It is anticipated to be released as a final document in 2027, and will be acceptable as an alternative pathway for seismic design until it is legislated into the building code in future. While the draft document specifically states it is not to be used for design, it has since been adopted by some agencies such as NZTA[2] until the final version is released. The new loadings standard is based on the most up to date scientific understanding of the hazard in New Zealand, and is based on the 2022 National Seismic Hazard Model[3].

This PSHA has been completed in accordance with both the existing earthquake design loadings standard[4] (NZS1170.5 (2004)) and DZ TS1170.5 (2024).

Table A1: Peak Ground Acceleration (a_{max}) and Earthquake Magnitude (M) values recommended for Geotechnical Assessment, for Site Classes A, B, C, D and E, for level ground conditions

LOCATION ID NUMBER[a]	TOWN/CITY	PEAK GROUND ACCELERATION (a_{max})[b] AND RECOMMENDED FOR USE FOR ALL SITE CLASSES — RETURN PERIOD 25-YEAR a_{max}(g)	25-YEAR M	50-YEAR a_{max}(g)	50-YEAR M	100-YEAR a_{max}(g)	100-YEAR M	250-YEAR a_{max}(g)	EARTHQUAKE MAGNITUDE (M)[c],[d],[e] VALUES (A, B, C, D AND E) — WITHOUT MODIFICATION[f] 250-YEAR M	500-YEAR a_{max}(g)	500-YEAR M	1000-YEAR a_{max}(g)	1000-YEAR M	2500-YEAR a_{max}(g)	2500-YEAR M	BASIS OF DATA (REFER NOTES BELOW FOR DETAIL)[g]	GROUP ID NUMBER[h]
1	Kaitaia																
2	Paihia/Russell																
3	Kaikohe	0.03	5.8	0.05	5.8	0.07	5.8	0.10	5.8	0.13 (0.19)	5.8 (6.5)	0.17 (0.19)	5.8 (6.5)	0.24 (0.19)	5.8 (6.5)	(1)*	1
4	Whāngarei																
5	Dargaville																
6	Auckland																
7	Warkworth																
8	Manukau City	0.05	5.9	0.06	5.9	0.09	5.9	0.14	5.9	0.15 (0.19)	5.9 (6.5)	0.20 (0.19)	5.9 (6.5)	0.28 (0.19)	5.9 (6.5)	(1)*	2
9	Waiuku																
10	Pukekohe																
11	Huntly	0.06	5.8	0.08	5.8	0.12	5.8	0.18	5.8	0.24	5.8	0.31	5.8	0.42	5.8	(1)	3
12	Ngāruawāhia																
13	Hamilton																
14	Te Awamutu	0.06	5.9	0.09	5.9	0.12	5.9	0.18	5.9	0.25	5.9	0.32	5.9	0.44	5.9	(1)	4
15	Otorohanga																
16	Thames																
17	Morinsville																

4.3 Vs30 (site specific seismic hazard assessment report, page 6)

V_{s30} is the time-average shear-wave velocity over a 30m depth from the ground surface, and is a parameter used as a proxy to capture site response in seismic hazard assessments. It is also the parameter proposed to be used for Seismic Site Classification in the forthcoming TS1170.5 (2024).

Based on the data provided by the CMW site-specific seismic hazard assessment report, five seismic cone penetrometer tests (sCPT) were completed around the reservoir site in 2024. All sCPTs had a target depth of 30m but were refused early on hard strata, where the shear wave velocity measurement did not reach 30m due to equipment refusal.

In accordance with DZ TS1170.5 (2024), uncertainty in Vs30 has been accounted for by applying ±5% for direct measurements (DZ TS1170.5 (2024) Method 1) and ±5-15% for tests less than 25m deep using the Vsz method (DZ TS1170.5 (2024) Method 2). Table 3 shows the summary of reservoir V_{s30} testing.

Table 3: Summary of reservoir V_{s30} testing

Test No.	Depth (mBGL)	V_{s30} Calculation Method	V_{s30}	V_{s30} Lower Bound	V_{s30} Upper Bound
SCPT24-01	21	Vsz	265	252.4	278.3
SCPT24-03	20	Vsz	305	215.2	237.3
SCPT24-04	28	Direct	265	256.2	282.5
SCPT24-05	29	Direct	269	238.7	294.2
SCPT24-06	25	Direct	226	277.3	335.5
Average	-	-	266	248.8	284.5

Table 5.2: Summary of Reservoir V_{s30} Testing

Test No.	Depth (mBGL)	Vs30 Calculation Method[1]	V_{s30}	V_{s30} Lower Bound	V_{s30} Upper Bound
SCPT24-01	21	Vsz	265	252.4	278.3
SCPT24-03	20	Vsz	305	215.2	237.3
SCPT24-04	28	Direct	265	256.2	282.5
SCPT24-05	29	Direct	269	238.7	294.2
SCPT24-06	25	Direct	226	277.3	335.5
Average	-	-	266	248.8	284.5

Notes: [1] V_{s30} from direct measurement has been calculated as per Equation 5.1, V_{s30} for tests <25m deep has been calculated using the Boore (2004)[10] method.

The mean and upper bound V_{s30} fit within DZ TS1170.5 (2024) Site Class IV, while the lower bound result fits within DZ TS 1170.5 (2024) Site Class V. As the mean, upper and lower bound straddle more than one site class, a site class envelope has been adopted across the return periods assessed. The uniform hazard spectra for this envelope, based on the DZ TS1170.5 (2024) spectra for Hamilton for the 1/2,500 year return period, are presented in ***Figure 5.3***. The envelope spectra for the additional return periods assessed are presented in ***Appendix C***. The uniform hazard spectra envelope for all return periods is equal to the Site Class V spectra.

5. Potential geotechnical issues that could affect the reservoirs seismic behaviour.

The mean V_{s30} for the reservoir site is 266 m/s. This corresponds to Site Class IV, indicating a potential for significant ground shaking during seismic events.

Groundwater was encountered at various depths across the site, with some boreholes showing water levels as shallow as 0.8m below ground level. Fluctuating groundwater levels can influence soil strength and stiffness, potentially affecting the seismic response of the reservoir. The reservoir may be subject to buoyancy under seismic conditions. The groundwater level at the reservoir site was identified as potentially varying between 0.8m and 19.6m bgl, Soils could experience an increase in pore water pressure due to liquefaction.

With this said the range of VS30 values can vary significantly, but soils with lower VS30 values (typically less than 200 m/s) are generally more susceptible to liquefaction. Seeing how the mean V_{s30} for the reservoir site is 266 m/s the full liquefaction potential of the site should be explored in more detail.

The site is influenced by nearby crustal faults, including the Kututaruhe Fault, which is 3km away. The seismic hazard is dominated by local crustal faults for short-term hazards and distant subduction ruptures for long-term hazards.

Lateral spreading occurs predominantly in the vicinity of free surfaces, such as slopes or water courses, where soils can laterally displace. The site contains sandy soils and silts with low plasticity indicates a potential for soil liquefaction during seismic events, and lateral spreading might occur.

The site's sloping topography, with gradients up to 1:2.5 (V:H) at the Rotoroa/Hamilton Lake margin, poses a risk of slope instability during seismic shaking. Slope failure could result in significant ground displacement, affecting the site's overall stability.

6. Refinement of the Waikato Regional Hazards Portal risk assessment

According to the Waikato Regional Hazards Portal website, the refinement of the Waikato Regional Hazards Portal risk assessment involves several key aspects:

- Flood Management: Stop banks and floodgates reduce river flooding risk, but there's always a residual risk of overtopping or failure.
- Land Drainage: Schemes drain water from a 10% AEP rainfall event within 3 days to prevent pasture damage.
- Defended Areas: The portal identifies areas defended from flooding by structural defences like stopbanks or floodwalls. These areas still have a residual risk of flooding due to events larger than the design capacity or structural failures.
- Residual Risk Zones: The Waikato Regional Policy Statement includes policies and methods related to residual risk, which are identified in District Plans. The portal currently shows defended areas for Waikato District and Thames Coromandel District, with plans to add more areas as data becomes available.
- Tsunami: Maps show tsunami inundation and safe zones for worst-case scenarios.
- Shoreline Change and Coastal Erosion: The portal includes data on natural processes of shoreline change, including coastal erosion and accretion, which can occur over short or long periods and can be influenced by human development.

Based on the Waikato Regional Hazards Portal, the site has a low potential risk of flood hazard due to its proximity to the Waikato River, which experiences a 1% AEP (Annual Exceedance Probability) rainfall event. Additionally, the site is not within defended areas and is not subject to coastal hazards. Furthermore, the site is classified as an undetermined area regarding liquefaction risk.

7. Liquefaction risk assessment based on the MBIE Module 3, table 5.10 framework

The site contains sandy soils and silts with low plasticity indicates a potential for soil liquefaction during seismic events. Liquefaction can lead to ground settlement, loss of bearing capacity, and lateral spreading, all of which could compromise the reservoir's structural integrity. Also, based on the Borehole and Hand Auger tests from the factual report, the site is considered to be susceptible to liquefaction due to:

- The Walton Subgroup having layers of medium dense to dense sand, dense to very dense silty sand. Sand and silty sand exhibit properties that are typically associated with liquefiable soils.
- The proximity of groundwater to the ground surface near the reservoir site.

Table 5.1: General performance levels for liquefied deposits

	EFFECTS FROM EXCESS PORE WATER PRESSURE AND LIQUEFACTION	CHARACTERISTICS OF LIQUEFACTION AND ITS CONSEQUENCES	CHARACTERISTIC F_L, LPI, LSN
L0	**Insignificant**	No significant excess pore water pressures (no liquefaction).	$F_L > 1.4$ LPI=0 LSN <10
L1	**Mild**	Limited excess pore water pressures; negligible deformation of the ground and small settlements.	$F_L > 1.2$ LPI = 0 LSN = 5 – 15
L2	**Moderate**	Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results in relatively small differential settlements.	$F_L \approx 1.0$ LPI < 5 LSN 10 – 25
L3	**High**	Liquefaction occurs in significant portion of the deposit (say 30 percent to 50 percent) resulting in transient lateral displacements, moderate-to-large differential movements, and settlement of the ground in the order of 100 mm to 200 mm.	$F_L < 1.0$ LPI = 5 – 15 LSN = 15 – 35
L4	**Severe**	Complete liquefaction develops in most of the deposit resulting in large lateral displacements of the ground, excessive differential settlements and total settlement of over 200 mm.	$F_L << 1.0$ LPI > 15 LSN > 30
L5	**Very severe**	Liquefaction resulting in lateral spreading (flow), large permanent lateral ground displacements and/or significant ground distortion (lateral strains/stretch, vertical offsets and angular distortion).	

To complete liquefaction calculations using CLiq software to assess the required Fl, LPI and LSN values, we would need the raw CPT data (in .xls or .xlsx format).

APPENDIX C

AURECON (2009) - WATER RESERVOIR STRUCTURES SEISMIC REVIEW (RELEVANT PAGES)

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Contents

1. Executive Summary

Seismic assessments have been undertaken of the Hamilton City Council water reservoirs located at Dinsdale, Fairfield, Hillcrest, Maeroa, Newcastle, Pukete, and Ruakiwi.

The performance of the existing reservoirs have been assessed against the provisions of the New Zealand Society for Earthquake Engineers (NZSEE) Study Group on Storage Tanks: “Seismic Design of Storage Tanks 2008” (Final Draft) applicable to new storage tank structures (ie as a “percentage of new tank standard” (%NTS)).

Summary Of Reservoir Capacities’ as % NTS

Dinsdale	75% (70%)
Fairfield	75%
Hillcrest	30%
Maeroa	45%
Newcastle	100%
Pukete	30%
Ruakiwi.	20%

Bracketed terms refer to the local impact of access-way penetrations through reservoir walls, installed sometime after the reservoirs’ construction, where these may govern the overall seismic capacity.

A number of recommendations have been provided for improving the performance of the reservoirs under earthquake loading so as to ensure compliance as near as reasonably practicable with performance requirement of the New Zealand Society for Earthquake Engineers (NZSEE) Study Group on Storage Tanks: “Seismic Design of Storage Tanks 2008”. These include:

a. upgrade roof to wall connections to improve shear transfer capacity (Pukete).
b. local strengthening of reservoir walls around the access-way openings using glass or carbon fibre or steel strap reinforcement (Dinsdale, Maeroa, Ruakiwi).
c. strengthening of bending capacity of reservoir walls using glass or carbon fibre or steel strap reinforcement (Maeroa, Ruakiwi).
d. upgrading footing restraint nibs to the internal/external face of reservoir walls to improve the shear capacity at the wall footing interface (Dinsdale, Fairfield, Hillcrest, Maeroa, Ruakiwi).
e. upgrade anchorage of reservoir walls to footing (Ruakiwi).
f. enlargement and strengthening of existing wall ring beam footings (Hillcrest, Ruakiwi).
g. lower the maximum water storage level to preclude damage to the reservoir roof in an earthquake (Fairfield, Hillcrest, Newcastle).
h. removal of earth fill against and over reservoir structure (Pukete).
e. Undertake geotechnical site investigations to confirm available bearing capacities under wall footings (Hillcrest, Ruakiwi).

2. Introduction

2.1 Scope and Limitations

2.1.1 Scope of Report

Hamilton City Council have engaged Aurecon Group (Aurecon)to undertaken seismic assessments of their existing reservoir tank structures locates at Dinsdale, Fairfield, Hillcrest , Maeroa, Newcastle , Pukete, and Ruakiwi. This report presents the findings of the seismic assessments undertaken of the seven reservoir structures.

2.1.2 Limitations

This report has been prepared by Aurecon at the request of Hamilton City Council solely for the use of Hamilton City Council.

Aurecon does not accept any legal liability or responsibility in respect of the use of the report for any purpose other than the purpose specified above.

This document has been prepared on behalf of and for the exclusive use of the Client, and is subject to and issued in connection with the provisions of the agreement between Aurecon and the Client. Aurecon accepts no liability or responsibility whatsoever for or in respect of any use of, or reliance upon this report by any third party.

2.2 Methodology

2.2.1 Brief

The scope of the work is as per Aurecon's consultancy proposal dated 4 February 2009. In brief this has comprised the following:

- Review of existing record drawings provided by Hamilton City Council.
- Site walkover inspections to verify the accuracy of the information shown on the record drawings and where possible identify any alterations that may have been undertaken. This survey was limited to visual inspections of the exterior of the reservoir structures where access permitted.
- Structural seismic analyses and assessments of each reservoir structure.
- Identification of key risk elements of each reservoir structure.
- Overall assessment findings and recommendations

The following aspects have not been addressed in the undertaken assessments:

- Geotechnical site stability assessments including global slope stability reviews, liquefaction potential assessments.
- Review of structure for support of gravity load combinations.
- Specific review of structure for latent design defects, unless identified in respect to seismic capacities.
- Site surveys and measure up.
- The impact of piping connections to the reservoirs under earthquake loading.

2.2.2 Geotechnical Information

A nominal level of geotechnical information is available in the provided documentation for some 50% of the sites. This information generally only includes descriptions of the subsoil layers with very little detail of geotechnical parameters.

Based on our own knowledge of subsoil conditions in and around Hamilton City, and the available geotechnical information relating to a number of the reservoir sites, we would expect that ground conditions at all sites to generally compromise layers of silts, sands, sandy clay and clays to some depth. A seismic site hazard spectrum applicable to such deep or soft soils soil conditions (site subsoil Class D of NZS1170:5 –"Structural Design Actions Part 5-Earthquake Actions New Zealand")has accordingly been adopted for this seismic assessment exercise.

2.2.3 Seismic Design Standard

For the purposes of the seismic assessments, we have adopted the recommendations of the New Zealand Society for Earthquake Engineers (NZSEE) Study Group on Storage Tanks: "Seismic Design of Storage Tanks 2008" (Final Draft) (herein known as the NZSEE Recommendations) as the current standard applicable to new reservoir tank structures.

This document is a recent revision of the earlier NZSEE Recommendations for Seismic Design of Storage Tanks (sometimes called the "Tank Red Book"), published in 1986 and widely used in New Zealand and acknowledged internationally. The latest draft incorporates the derivation of seismic loads from the current national seismic loading standard NZS 1170:5 -–"Structural Design Actions Part 5-Earthquake Actions New Zealand" It is viewed that the data presented in these updated recommendations represents the state of the art of seismic design and behaviour of storage tanks.

In tanks under lateral seismic acceleration the fluid in the upper regions tends not to displace laterally with the tank walls. This results in vertical displacement of the liquid adjacent the tank walls, as the result of lateral displacement incompatibility generating convective or anti-symmetric sloshing behaviour. Nearer the tank base the fluid is unable to move out of the way as the tank displaces and acts as an added mass to the inertia mass of the tank, or impulsive behaviour. The guidelines apply a mechanical analogue of response, with the impulsive mass of the fluid rigidly linked to the walls and the convective (sloshing) mass connected to the tank walls by flexible springs.

The performance of the existing reservoirs have been assessed against the provisions of the NZSEE Recommendations applicable to new storage tank structures (i.e. as a "percentage of new tank standard" (%NTS)). %NTS is essentially the assessed structural performance of the reservoir(s) (taking into consideration all reasonably available information) compared with the requirements for a new reservoir structure designed in accordance with the provisions of the NZSEE Recommendations.

2.2.4 Information Gathering and Review

Existing Documentation
Available record drawings of the existing reservoir structures have been provided by Hamilton City Councils for Aurecons use. Our seismic assessments have been on the basis of the information contained in these drawings in respect to the structural arrangements of the reservoir structures, together with any additional information that may have been sourced during our walkover inspections. In most cases the existing drawings were found to be only a partial set of construction documents and some assumptions have needed to be made in respect to structural details. A copy of the available structural drawings utilised for the assessments are annexed in Appendix A.

Walkover Inspections
Aurecon undertook site walkover inspections of each site to verify the accuracy of the information shown on the record drawings and identify, where possible, any alterations that may have been undertaken. This survey was limited to visual inspections of the exterior of the reservoir structures where access permitted. A relevant selection of photographs taken as part of the inspection of the reservoirs is annexed in Appendix C.

3. General Descriptions and Observations

Descriptions of the structural arrangement of each reservoir are given below together with any relevant observations made during the walkover inspections. The descriptions are based substantially on the information in the record drawings made available to us.

Dinsdale Reservoir

Description:
A circular reinforced concrete reservoir constructed in or about 1964, with internal diameter of 30.5m and clear internal height of approximately 6.7m. Wall thickness varies from 457mm over the lower half of the tank height tapering down to 190mm at roof level. The tank walls are located in a preformed slot in the reinforced concrete ring beam footing with no mechanical connection to the footing. The base of the tank comprises a 152mm thick reinforced concrete slab cast on grade.

The roof structure comprises a 102mm flat slab supported on internal reinforced concrete columns on a 4.72 m grid each way and the exterior tank wall. The concrete roof sits directly over the tank wall, with an isolation bearing strip at the wall roof joint with no mechanical connection between the roof and the wall. The roof incorporates an internal down stand nib to the inside wall face to provide restraint under lateral loads. A nominal clearance of approximately 38mm exists between the inside wall face and the down stand nib to the roof which will allow lateral displacement of the roof structure under earthquake loading until the nib is engaged.

Visual Observations:
A reservoir access-way has been installed in the tank lower wall sometime after the reservoir was constructed. An available drawing shows this as a 760mm diameter steel access hatch assembly installed through the reservoir wall. However no calculations or details relating to modifications to the concrete tank wall are available.

A number of small leaks are evident through the base of the wall at a number of locations.

Efflorescence is to be seen at or above a number of the leak locations, indicative of crack self healing.

Two well structures have been added adjacent the tank, one housing a flow meter and the other earthquake valves to the outlet pipes. The excavated well depths are well below the tank footing with no support provided to the exposed soil face. (The need for retention of the exposed cut soil face needs to be investigated.)

Fairfield Reservoir

Description:
A circular reinforced concrete reservoir constructed in or about 1959, with internal diameter of 30.5m and clear internal height of approximately 6.6m. Wall thickness varies from 368mm over the lower half of the tank height tapering down to 178mm at roof level. The tank walls are located in a preformed slot in the reinforced concrete ring beam footing with no mechanical connection to the footing. The base of the tank comprises a 178mm thick reinforced concrete slab cast on grade.

Record drawings shows 2 alternative roof systems, one a flat slab on internal columns and the other a precast slab and beam system. From our visual observations it is believed the first alternative has been adopted, comprising a 114mm thick flat slab supported on internal reinforced concrete columns on a 4.72 m grid each way and on the exterior tank wall. The concrete roof sits directly over the tank wall, with an isolation de-bonding strip at the wall roof joint with no mechanical connection between the roof and the wall. The roof incorporates an internal down-stand nib to the inside wall face to provide restraint under lateral loads. A nominal clearance of approximately 16mm exists between the inside wall face and the down stand nib to the roof which will allow lateral displacement of the roof structure under earthquake loading until the nib is engaged.

Visual Observations:
A number of trees exist immediately adjacent the tank, with heights well in excess of the tank. Whilst there was no evidence of the tree roots affecting the tank foundations it would be prudent to investigate this further.

A small leak was evident at one location at the base of the tank.

Hillcrest Reservoir

Description:
A two level circular reinforced concrete reservoir constructed in or about 1944, each tank with an internal diameter of 10.7m. The clear internal heights to the lower and upper tanks are 7.1 and 5.2m respectively. Wall thicknesses are 216mm minimum with the internal wall face of circular profile and the external face faceted with 18 flat faces. The reservoir walls sit directly on the reinforced concrete ring beam footing, 305mm deep by 990mm wide, and connected to the footing by what is understood to be 18 pairs of 19mm diameter dowels cast into the wall and footing. The dowels incorporate a flexible bitumen sealant surround through the upper depth of the footing. The footing incorporates a modest up-stand nib to the inside wall face, profiled so as to offer very little lateral restraint to the reservoir walls.

The base of the lower tank comprises a 229mm thick reinforced concrete slab cast on grade. The base of the upper tank consists of a 254mm thick reinforced concrete suspended slab spanning onto two internal ring beams supported on reinforced concrete columns and onto a corbel support on the internal face of the tank walls. No mechanical connection exists between the slab and corbel support.

The roof structure comprises a 127mm thick slab similarly supported on internal ring beams over columns and the outer reservoir wall. The slab is cast integrally with the tank walls.

Visual Observations:
Both tanks are understood to currently be unused and empty.

Three cell phone antenna type devices are currently mounted on the upper external face of the reservoir. Cell phone equipment is also present at ground level mounted on pads adjacent the tank.

Maeroa Reservoir

Description:
A circular reinforced concrete reservoir originally constructed in or about 1964, and incorporating a steel trussed roof added in or about 1972. The tank has a minimum internal diameter of 25.1m and clear internal height of approximately 7.2m. Wall thickness varies from 254mm at the base to 203mm to the upper third of the tank. The tank incorporates 20 equally spaced pilaster type columns around the exterior perimeter housing overflow drains from the top of the tank. The tank wall sits directly on the reinforced concrete ring beam footing with very nominal mechanical connection to the footing by what appears to be a copper tubing dowel arrangement. These are expected to offer very little mechanical restraint between wall and footing. The footing incorporates a modest up-stand nib to the inside wall face. The base of the tank comprises a 178mm thick reinforced concrete slab cast on grade.

The roof structure comprises light gauge steel profiled sheet cladding on structural steel radial trusses spanning across the full diameter of the reservoir. The trusses are bolt fixed to the top of the reservoir walls.

Visual Observations:
A reservoir access-way has been installed in the lower tank wall sometime after the reservoir was constructed. An available drawing shows this as a 760mm diameter steel access hatch assembly cast into in-situ concrete surround in 1150x1150mm opening cut through the concrete tank wall. However no calculations relating to the modifications to the concrete tank wall are available.

Evidence of past and present water leakage through construction joints to the tank walls was observed.

A number of trees exist immediately adjacent the tank (typically within 1-2m), with heights well in excess of the tank. Whilst there was no evidence of the tree roots affecting the tank foundations it would be prudent to investigate this further.

Newcastle Reservoir

Description:
A rectangular reinforced concrete reservoir constructed in or about 1991, with internal dimensions of 84.0m x 44.4m and a maximum clear internal height of approximately 6.7m. Wall thickness varies from 300mm to the lower 1.2m to 170 mm minimum above with a vertical ribbed exterior profile. The walls are of precast concrete panel construction with in-situ splices between adjacent panels. The walls have been cast and tied directly into the underlying footings. The base of the tank is recessed into the ground away from the tank wall and comprises a 124mm thick reinforced concrete slab cast on grade.

The roof structure comprises proprietary precast hollow core floor units with in-situ topping spanning on to 3 internal longitudinal beam lines, supported on columns, and the external longitudinal reservoir walls. The roof structure is fully tied to the top of the reservoir walls.

The reservoir provides some retention to adjacent ground to the North and West sides.

Visual Observations:
There is some evidence of water overflowing from the eastern side of the roof down pipe. Water appears to follow the top of the retaining wall towards the main stop valve access way where it is diverted down the bank.

Pukete Reservoir

Description:
A rectangular cast in situ reinforced concrete reservoir constructed in or about 1974, with internal dimensions of 75.6m x 51.2m and an average clear internal height of approximately 6.4m. The tank is totally buried below ground, fill having been place over and heaped around the reservoir after construction. An average depth of fill of 0.65m over the reservoir has been assumed for purposes of the seismic assessment.

Wall thicknesses vary from 762mm at their base to 305 at the wall tops. The walls have been cast integral with the underlying footings. The base of the tank comprises a 229mm thick reinforced concrete slab cast on grade.

The roof structure comprises a 229mm flat slab supported on internal reinforced concrete columns on a 4.88 m grid each way and the exterior tank walls. The concrete roof overhangs the tank walls with the walls set into a rebate or slot in the soffit face of the roof slab. There appears to be no mechanical connection between the roof and the walls with a slip layer of mulseal at the horizontal junction.

Visual Observations:
There is evidence subsidence to the soil banks to the sides of the reservoir. In some locations this subsidence is measurable and up to 100mm. There is also a large amount of ground movement on the stairs adjacent to the pump house.

A police communications mast has been attached to the north facing outside wall of the pump house. This tower is approximately 14m high.

Ruakiwi Reservoir

Description:
A circular reservoir with a concrete outer shell typically reinforced with horizontal steel hoops, originally constructed in or about 1930, with internal diameter of 23.5 to the lower reservoir level and 25.6m to the upper level. The reservoir is fully internally lined with steel plate. External circular hollow core concrete columns to the exterior of the lower reservoir level support the reservoir upper walls and roof. The original reservoir roof was replaced with a metal profile clad steel trussed roof in or about 1972.

The base of the reservoir comprises a 762 mm thick concrete slab reinforced with square bar cast on grade, with a steel liner to the upper surface. The concrete reservoir walls and steel wall liner appear to sit directly on the steel liner to the base of the tank. There appears no mechanical connection between the walls and the reservoir base.

Visual Observations:
An access-way has been cut through the reservoir wall at low level including a major opening through the adjacent external lower skirt wall. Sketch drawings and calculations relating to the access-way through the tank wall have been sourced and reviewed.

The concrete to the outer columns appears bony in places without complete cement paste. There is also evidence of inadequate concrete cover to reinforcing steel. Some patch repairs have been previously undertaken.

4. Modelling Assumptions and Parameters

4.1 Assumed Material Properties

The following material properties have been adopted for assessment purposes, unless alternative properties were note on the provided record drawings:

Soil parameters:

Ultimate Foundation Bearing Capacity	300kPa
Spectral Site Subsoil Classification	Site subsoil Class D of NZS1170:5 (deep or soft soil)
Mass density	1800kg/m3
Poisson's ratio for soil	0.3
Modulus of Elasticity for soil	50MPa

Concrete parameters:

Reinforced Concrete Density	2400kg/m3
Concrete strength -in-situ	25Mpa typically
Concrete strength -precast	35Mpa typically
Friction coefficient between surfaces	0.3

Steel Grades

Reinforcing steel –pre 1970	250Mpa yield strength
Reinforcing steel –1970 -1990	295& 410 Mpa yield strength
Reinforcing steel –1991 -2000	300 &430 Mpa yield strength
Steel bar reinforcement- Ruakiwi	200MPa yield strength

Water

Water Density	1000kg/m3

4.2 Reservoir Water Level

The following water reservoir levels have been adopted for the assessment, these being the maximum measured levels during 2008 and 2009 as advised by HCC, or in the case of the Hillcrest and Newcastle reservoirs, the design water levels noted on the provided record drawings.

Dinsdale	6.17m
Fairfield	6.5m
Hillcrest	5.5m& 4.9m Note 1
Maeroa	6.0m
Newcastle	6.30m Note 2
Pukete	5.3m
Ruakiwi	24.8m

Note 1: Water levels to lower and upper tanks respectively.
Note 2: Maximum depth measured at lowest floor level.

4.3 Loading Parameters

i) The following parameters were adopted from the NZSEE Recommendations:

Tank importance level	IL =4 (Consequence of failure "Serious" as facilities of high community significance with the tanks are to be functional for post disaster water supply)
Return Period Factor	R=1.3 (1/1000 annual probability of exceedance for IL4)
Displacement ductility factor	1.0 for convective and vertical modes of vibration 1.25 for horizontal impulsive modes

ii) The following ultimate limit state load combination has been adopted for assessment purposes:

G + Eu + Flp

Where G = dead load including self weight
Eu= earthquake load (ULS)
Flp= liquid pressure load

Where tanks are partially or fully embedded into the ground, or retain soil, static earth pressures and associated horizontal earthquake pressures have also been considered in the above load combination.

4.4 Material Standards

Assessment of section or member strengths have been for the ultimate limit state loading conditions in general accordance with the relevant principles and procedures for design as set out in the following material standards:
NZS 3101:2005 –Reinforced Concrete Standard
NZS 3404:1999 – Structural Steel Standard

4.5 Assessment Methodology

Adopted design loadings, design actions and general analyses and design principals have been assessed or undertaken in accordance with the provisions of the NZSEE Recommendations.

The following actions have been assessed and existing reservoir capacities reviewed against demands resulting from the NZSEE Recommendations

a. *Roof shear at roof wall connection*. Roof slabs cast integrally at their connection with the reservoir walls generally have sufficient reinforcement to transmit shear. Tank roofs with overhangs and internal and or external down-stand nibs rely on shear transfer over that portion of the circumference where the nib overhangs come in contact with the wall. Some contribution comes from friction resistance across the roof wall interface but this is generally quite small given the tributary area of roof slab loading the walls.

b. *Total shear at base of walls*. Similar to the roof wall connection, walls cast integral with the footing generally have sufficient reinforcement to transmit shear. Where wall panels are located in a slot in the ring footing without any mechanical connection, shear transfer relies on friction at the wall base footing interface and bearing over that portion of the circumference where the wall comes in contact with any up-stand nib. Radial friction between the wall base and the footing is generally non existent where no mechanical restraint is provided.

c. *Total overturning effect.* Where tank walls are cast integrally with the footing, the foundations can contribute to overturning resistance. Where walls are not secured to the foundations, resistance is provided by the mass of the tank walls and local component of roof structure supported by the walls. Uplift or rocking of the walls will occur where global overturning exceeds the available restoring resistance.

d. *Wall vertical bending moments.* Reservoir wall loads are resisted by a combination of vertical bending and hoop forces. Vertical bending moments in rectangular tanks are significant, with walls commonly cantilevering off their footings or horizontally adjacent return walls. Vertical bending is less critical in circular tanks where hoop forces are more dominant.

e. *Wall hoop forces:* as d. above

f. *Convective wave height versus available freeboard:* Wave heights have been calculated to establish effects on the reservoir roofs. If the freeboard is less than the height of the convective (sloshing) waves then hydrostatic pressures will be generated on the roof structure. Resistance from untied roofs is solely by the roofs own self weight. Metal clad roof systems offer little resistance to hydrostatic pressures.

5. Seismic Capacities Summary

The results of the seismic assessment at presented in the tables below for each of the seven reservoirs.

5.1 Dinsdale Reservoir

Component	%NTS
Roof shear at roof wall connection	100%
Total shear at base of walls.	75% Note 1
Total overturning effects	100%
Wall vertical bending moments	100% Note 2
Wall hoop forces	100% Note 2
Convective wave height	575mm
Available freeboard	530mm Note 3

Note 1: Seismic capacity is limited by the shear resistance at the wall footing interface. This relies on frictional resistance on the base of the wall and wall bearing against the raised footing nibs. The shear capacity of the smaller internal nib governs the maximum shear that can be transferred from the wall into the footing.

Note 2: This excludes the impact of the access-way installed sometime after the reservoirs construction. Whilst no details of modifications to the tank wall have been made available, it is likely that the cut penetration through the wall will be similar to those to the Maeroa and Ruakiwi reservoirs where 1.1x 1.1m penetrations or thereabouts were made. The resulting local loss of both vertical and horizontal reinforcement will reduce the wall vertical bending moment and hoop force capacities to the order of 70%NTS at and adjacent the access-way location.

Note 3: Whilst the convective wave will exert some pressure on the roof of the reservoir this will be relatively small and resisted by the weight of the roof.

5.2 Fairfield Reservoir

Component	%NTS
Roof shear at roof wall connection	100%
Total shear at base of walls.	75% Note 1
Total overturning effects	100%
Wall vertical bending moments	100%
Wall hoop forces	100%
Convective wave height	700mm
Available freeboard	190mm Note 2

Note 1: Seismic capacity is limited by the shear resistance at the wall footing interface. This relies on frictional resistance on the base of the wall and wall bearing against the raised external and internal footing nibs. The shear capacity of the smaller internal nib governs the maximum shear that can be transferred from the wall into the footing.

Note 2: Structural damage can be expected to the roof structure as the loading imposed by the convective wave exceeds the roof self weight and the roof slab has insufficient flexural capacity to resist uplift forces through spanning between supports.

5.3 Hillcrest Reservoir

Component		%NTS
Roof shear at roof wall connection		100%
Total shear at base of walls.		30% Note1
Total overturning effects		60% Note 2
Wall vertical bending moments		100%
Wall hoop forces		100%
Convective wave height		820mm
Available freeboard	*Lower tank*	1570mm
	Upper tank	300mm Note 3

Note 1: Base shear capacity is limited by the capacity of the shear dowel connections between the tank walls and ring beam footing. The internal up-stand nib offers little resistance due to its profiled face angled away for the reservoir walls.

Note 2: Overturning capacity is limited by the bearing strength of the ground under the ring beam footing as well as the structural capacity of the ring beam footing in distributing the vertical wall reactions to the underlying ground.

Note 3: Structural damage can be expected to the upper roof structure as the loading imposed by the convective wave exceeds the roof self weight and the roof slab has insufficient flexural capacity to resist uplift forces through spanning between supports.

5.4 Maeroa Reservoir

Component	%NTS
Roof shear at roof wall connection	100%
Total shear at base of walls.	45% Note 1
Total overturning effects	100%
Wall vertical bending moments	95% Note 2,3
Wall hoop forces	100% Note 3
Convective wave height	930mm
Available freeboard	1460mm

Note 1: Wall base shear capacity is limited by the shear resistance at the wall footing interface. This relies on frictional resistance on the base of the wall and wall bearing against the raised internal footing nib. The shear capacity of the nib governs the maximum shear that can be transferred from the wall into the footing.

Note 2: Vertical bending capacity is limited by the available vertical reinforcement in the walls. Some sliding of the base of the wall under hydrostatic loading has been assumed.

Note 3: This excludes the impact of the access-way installed sometime after the reservoirs construction. The resulting local loss of both vertical and horizontal reinforcement will reduce the local wall vertical bending moment and hoop force capacities to the order of 55% and 85%NTS respectively at and adjacent the access-way location.

5.5 Newcastle Reservoir

Component	%NTS
Roof shear at roof wall connection	100%
Total shear at base of walls.	100%
Total overturning effects	100%
Wall vertical bending moments	100%
Wall hoop forces	NA
Convective wave height	1680mm
Available freeboard	300mm Note 1

Note 1: Structural damage can be expected to the roof as the resulting pressure from the convective wave well exceeds the self weight of the hollow core roof structure. The hollow core roof structure will have little flexural capacity to resist uplift forces exceeding self weight by spanning between supports.

5.6 Pukete Reservoir

Component	%NTS
Roof shear at roof wall connection	75% Note 1,2
Total shear at base of walls.	100%
Total overturning effects	100%
Wall vertical bending moments	30% Note 3
Wall hoop forces	NA
Convective wave height	440mm
Available freeboard	1100

Note 1: The soil mass on the roof of the reservoir contributes significantly to the imposed impulsive lateral forces on the reservoir structure. Mobilisation of in-plane walls to resist earthquake loads is limited by the connection between the roof and walls, with no mechanical anchorage present. Consequently significant lateral load is resisted by end walls in cantilever action.

Note 2: The lack of mechanical connections at the roof and wall connections limit the in plane shear that can be transferred from the roof to longitudinal walls whilst the shear capacity of the down-stand roof nibs to the inside and outer faces of the walls limits the shear force that can be transferred into the transverse walls.

Note 3: Vertical bending capacity is limited by both wall footing reinforcement and vertical wall reinforcement. Significant out of plane or cantilever bending result s from lateral forces associated with fill material over and around the reservoir.

5.7 Ruakiwi Reservoir

Component	%NTS
Roof shear at roof wall connection	100%
Total shear at base of walls.	20% Note 1
Total overturning effects	60% Note 2
Wall vertical bending moments	65% Note 3,4
Wall hoop forces	100% Note 4
Convective wave height	900mm
Available freeboard	1100mm

Note 1: Wall base shear capacity is limited by the shear resistance at the wall footing interface. This relies on frictional resistance as the base has no mechanical connections or restraining nibs.

Note 2: Overturning capacity is limited by the self weight of the reservoir structure and lack of meaningful anchorage to the reservoir base as well as the bearing capacity of the underlying ground.

Note 3: The reservoir walls do not appear to incorporate any vertical reinforcement. However some flexural capacity is available utilising the tension capacity of the concrete. Vertical bending moments are not expected to be high as some sliding of the base of the wall under hydrostatic loading is expected. Much of the water pressure on the wall is expected to be resisted by hoop stresses.

Note 4: An 1100mm x1100mm penetration was cut through the reservoir wall at low level during installation of the access-way, cutting through the steel hoops reinforcing the tank wall. However the combined area of available hoop reinforcement in the adjacent wall above and below the opening is sufficient to compensate for the area of hoop reinforcement removed cut during construction of the access-way. Some local vertical bending will be induced either side of the penetration. This has been assessed as exceeding the local flexural capacity of the wall based on the tension capacity of the concrete, reducing the available vertical bending capacity to approximately 35% NTS.

5.8 General Summary

A summary of the overall seismic capacities of the seven tanks based on the results above are given in the following table. These correspond to the lower of the capacities of the items assessed and reported for each of the reservoirs in the tables above. Bracketed terms refer to the local impact of access-way penetrations through reservoir walls, installed sometime after the reservoirs' construction, where these may govern the overall seismic capacity.

The capacities do not consider possible damage to reservoir roof structures where convective wave heights exceed available freeboard. It is assumed that maximum water storage levels will be appropriately reduced to preclude the possibility of such damage under earthquake loading where this currently exists.

Summary Of Reservoir Capacities' as % NTS

Dinsdale	75% (70%)
Fairfield	75%
Hillcrest	30%
Maeroa	45%
Newcastle	100%
Pukete	30%
Ruakiwi.	20%

6. Recommendations

On the basis of the results of the seismic assessments undertaken of the seven reservoirs, the following recommendations are made for improving the performance of the reservoirs under earthquake loading so as to ensure compliance as near as reasonably practicable with performance requirement of the New Zealand Society for Earthquake Engineers (NZSEE) Study Group on Storage Tanks: “Seismic Design of Storage Tanks 2008” (Final Draft) as considered applicable to new reservoir structures.

Dinsdale Reservoir

- Upgrade the footing restraint nib to the internal face of the walls to improve the shear capacity at the wall footing interface
- Source and review the calculations and construction drawings of the access-way penetrations and locally strengthen the walls if required.

Fairfield Reservoir

- Upgrade the footing restraint nib to the internal face of the walls to improve the shear capacity at the wall footing interface
- Lower the maximum water level to preclude damage to the reservoir roof in an earthquake.

Hillcrest Reservoir

- Enlarge and strengthen the existing wall ring beam footing.
- Undertake a geotechnical site investigation to confirm the bearing capacity of the ground.
- Provide an external restraint nib to the wall footing adjacent the outside face of the wall.
- Lower the maximum water level to the upper tank to preclude damage to the reservoir roof in an earthquake.

Maeroa Reservoir

- Upgrade the footing restraint nib to the internal faces of the wall and provide and external restraint nib to the outside face of the wall.
- Install vertical glass or carbon fibre reinforcement strips to the exterior face of the tank wall to improve vertical bending capacity.
- Local strengthen the reservoir wall around the access-way opening using glass or carbon fibre or steel strap reinforcement.

Newcastle Reservoir

- Lower the maximum water level to preclude damage to the reservoir roof in an earthquake.

Pukete Reservoir

- Consider removal of the earth fill on top of the reservoir and reduction in height of earth banks to the sides of the reservoir.

Ruakiwi Reservoir

- Provide restraint nibs to the footing adjacent the external face of the tank wall.
- Provide anchorage detail at junction of wall and footing
- Local strengthen the reservoir wall around the access-way opening using glass or carbon fibre or steel strap reinforcement.
- Install vertical glass or carbon fibre reinforcement strips to the exterior face of the tank wall to improve vertical bending capacity.
- Undertake a geotechnical site investigation to confirm the bearing capacity of the ground.

APPENDIX D

ALTEX COATINGS (2019) – RUAKIWI RESERVOIR CONDITION ASSESSMENT

Project Details

Project:	Hamilton Water Tank Ruakiwi Reservoir for HCC
Client:	Avalon Industrial Services Ltd
Attention:	Doug Sutherland
Consultant/Specifier:	Altex Coatings Ltd
Fabricator:	Existing
Painting Contractor:	Avalon Industrial Services Ltd
Project Location:	Ruakiwi Rd, Hamilton
Size:	
Test Method:	Elcometer 456 gauge
Environment:	Immersion in potable water
Reference Documents *(incl spec #)*	Condition assessment and spot repair report.

Item/s	Coating Specified	DFT (µm)
Tank walls condition assessment	Full coat system – average readings	
	Total Specified DFT	**532.9 µm**
Tank floor condition assessment	Full coat system – average readings	
	Total Specified DFT	**554.6 µm**
Tank wall spot repairs to welded steel patches	Full coat system - Altra~Shield 2000	400 µm
	Total Specified DFT	**400 µm**

Disclaimer

Condition Assessment

Assessed by: Elliot Gaensicke

Element	Condition	Rust Grade (ASTM)	Average DFT
Interior tank lining	Average to poor	2 - Greater than 16 % to 33 %	543.75 µm

Spot Check Results

Inspected by:	Elliot Gaensicke. NACE Coating Inspector Level 1.

Element	Date	Coating	# of readings	Readings		Average
Tank walls Full coat system	25/09/2019	Existing epoxy coating	80	Min	232 µm	532.9 µm
				Max	849 µm	
				Std Dev	138.6 µm	
Tank floor Full coat system	25/09/2019	Existing epoxy coating	54	Min	334 µm	554.6 µm
				Max	1076 µm	
				Std Dev	133.1 µm	
Wall spot repairs Full coat system	25/09/2019	Altra~Shield 2000	22	Min	158 µm	406.9 µm
				Max	786 µm	
				Std Dev	170 µm	

Date	Spot Check Notes/ Comments
25/09/2019	**Wall welded steel patch repairs - spot repairs - Altra~Shield 2000** Surface preparation to steel patches was achieved by mechanical surface preparation. Good brush and roller application with positive cure. Suggested another heavy roller coat over all steel repairs of Altra~Shield 2000 to target 400µm Dry film thickness.

Date	Condition Assessment Notes/ Comments
25/09/2019	**Tank walls - Existing epoxy coating** The bottom strakes are heavily pitted from corrosion as deep as 4mm in places, multiple patch repairs that have poor ugly welds and splatter with sharp edges of welding from weld repairs. Weld joins/ repairs require some re-engineering to smooth off. There is a roller coat at the 6-9 o'clock position about 2m high appears to be an attempt to add extra dry film build to the existing coating. Approximately 200 panels about 35 would require full abrasive blast and about 150 panels to varying spot repairs estimated at about 25% of the tank wall surface to be blasted and painted.
25/09/2019	**Tank floor - Existing epoxy coating** Floor lining is in reasonably good order with good dry film builds the intact coatings are still sound with good adhesion. Spot repairs could be considered. The dry film builds taken from the pipes are good however the condition is poor and require a full abrasive blast and paint.
25/09/2019	**Interior tank lining - 2 - Greater than 16 % to 33 %** Condition of the coating system is near the end of its surface life. With an estimated repair surface removal area of about 25%. At 30% of surface repairs it is more economical to fully abrasive blast and repaint.

Photos

Figure 1 - 6-9 0'clock on wall to 2m roller coat applied coating DFT slightly less than existing

Figure 2 - Pitting corrosion at 6 o'clock closeup

Figure 3 - Patch repair and ugly weld filling, rough edges heavy pitting along bottom strake

Figure 4 - Spot repairs on corrosion pitting about 4mm deep

Figure 5 - Weld join example with spot corrosion returned to power red oxide

Figure 6 - Overview of patch repairs and corrosion cells looking at 3-6 o'clock worst area 2-7 o'clock corrosion pitting

Figure 7 - Example of spot corrosion spaced at 300-400mm apart

Figure 8 - 6 o'clock overview of spot corrosion on liner

Figure 9 - 8 o'clock overview of spot corrosion on liner

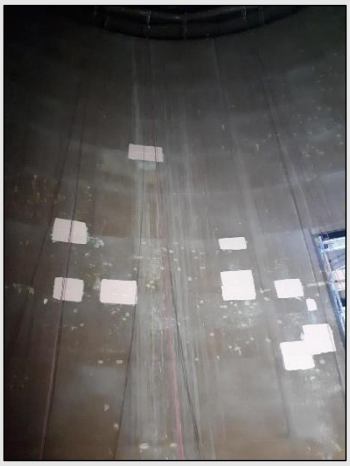


Figure 10 - 4 o'clock estimated surface area for repair at 25%

Figure 11 – Previous spot repairs which have failed

Figure 12 - New steel patch repairs welded and coated in Carboguard 504 and Altra~Shield 2000

Figure 13 – Close up repair at 5 o'clock 3rd strake up full coating system

Figure 14 - Floor Spot corrosion at 4th panel in at 3 o'clock

Figure 15 - Floor overview of 4th panels in at 9 o'clock

Figure 16 - Floor weld at panel 3 in from 6 o'clock

Figure 17 - Floor overview looking at 12.00 o'clock

Figure 18 - Pipes

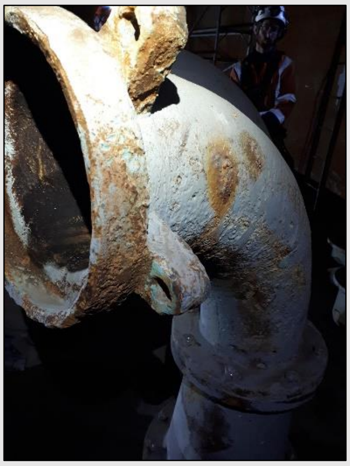


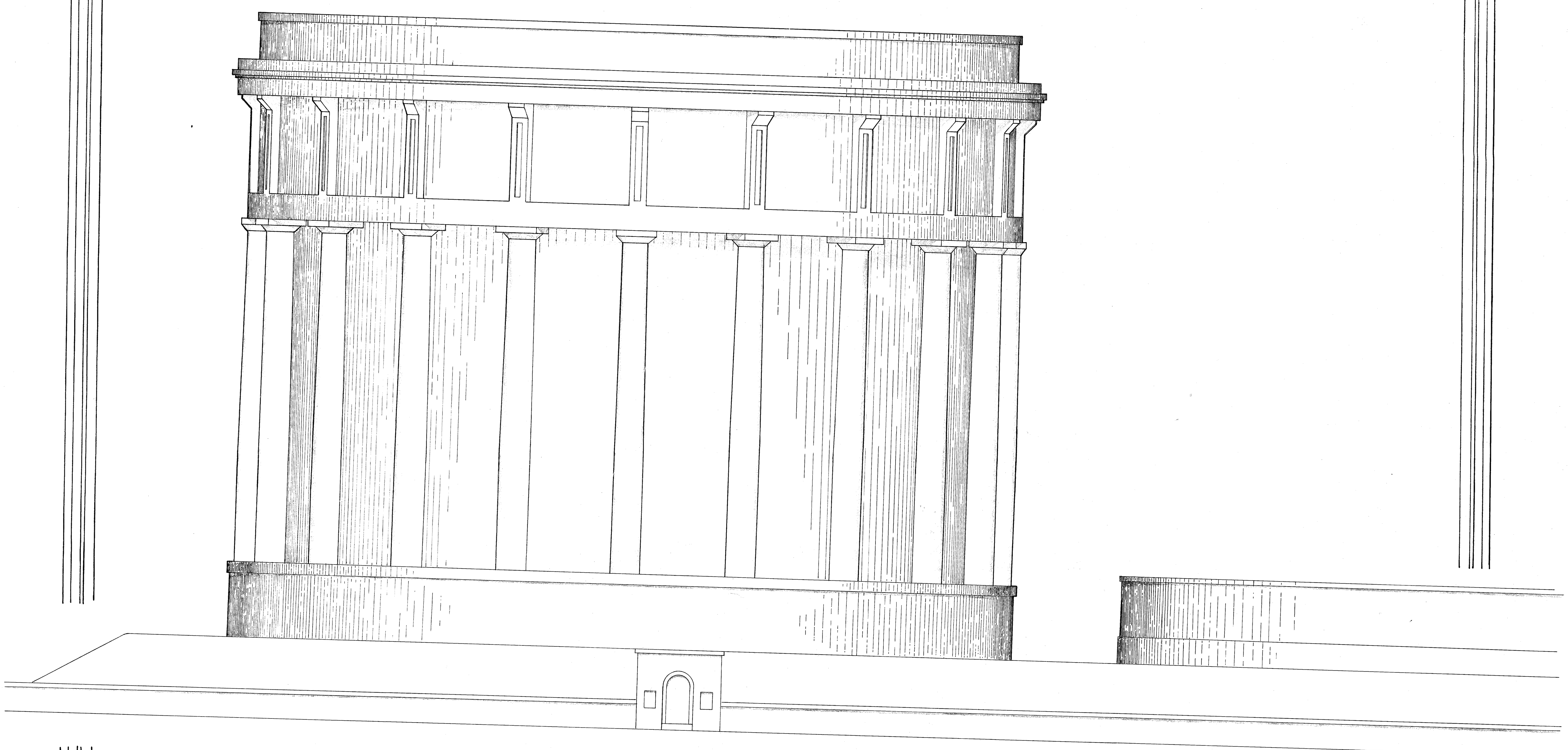
Figure 19 - Close up of pipe

Figure 20 - Pipes stubs

APPENDIX E

RUAKIWI RESERVOIR STRUCTURAL DRAWINGS AND UPGRADES (1932, 1948, 1972) (RELEVANT PAGES)

Re-Order Filemaster "S"
SELF-ADHESIVE PLAN HANGER
Patent Number 137841



NEW HIGH LEVEL RESERVOIR
FOR
BOROUGH OF HAMILTON

TOTAL CAPACITY 2,606,600 GALLONS.
TOP WATER LEVEL 300 FT. ABOVE DATUM.
INSIDE DIAMETER. – UPPER PORTION 84 FT.
" " – LOWER " 77 FT.
TOTAL DEPTH OF WATER 85 FT.
Nº OF COLUMNS 20
DIAMETER OF COLUMNS 44" TAPERING TO 37" DIAM.
FOUNDATION SLAB 88' FT DIAM.

CONSTRUCTION COMMENCED
" COMPLETED 17TH. FEB. 1932.

UNDER THE DIRECTION OF R. WORLEY A.M.I.C.E. BOROUGH ENGINEER.
RESIDENT ENGINEER, DESIGN & CONSTRUCTION, J.R. BAIRD A.M.N.Z.SOC. C.E.
DRAWN BY J.R. BAIRD & R. SMITH.

These are the plans referred to in ~~my~~ our contract dated this 20th day of [illegible] 1929
Signed [illegible]
Witness R. Worley

SHEET Nº 1

BOROUGH OF HAMILTON

DETAILS OF LOWER PORTION OF LOWER TANK WALL & COLUMN.

Scales {Tank Sections 1in. / Enlarged Details 4in.} to 1ft.

SHEET No 2

WW-T3-002

UNDER THE DIRECTION OF R. WORLEY A.M.I.C.E. BOROUGH ENGINEER.
RESIDENT ENGINEER, DESIGN & CONSTRUCTION, J.R. BAIRD A.M.N.Z. Soc. C.E.
DRAWN BY J.R. BAIRD & R. SMITH.

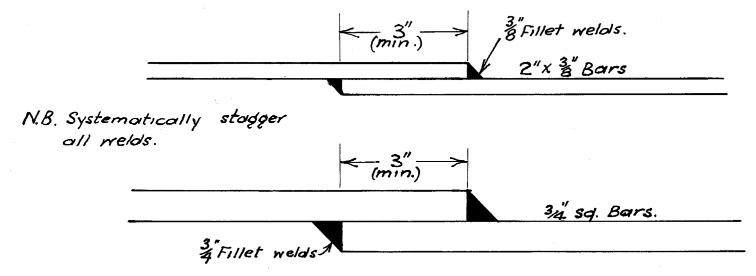
These are the plans referred to in our contract dated this 20th day of [illegible] 192[illegible]
Signed [illegible]
Witness [illegible]

BOROUGH OF HAMILTON

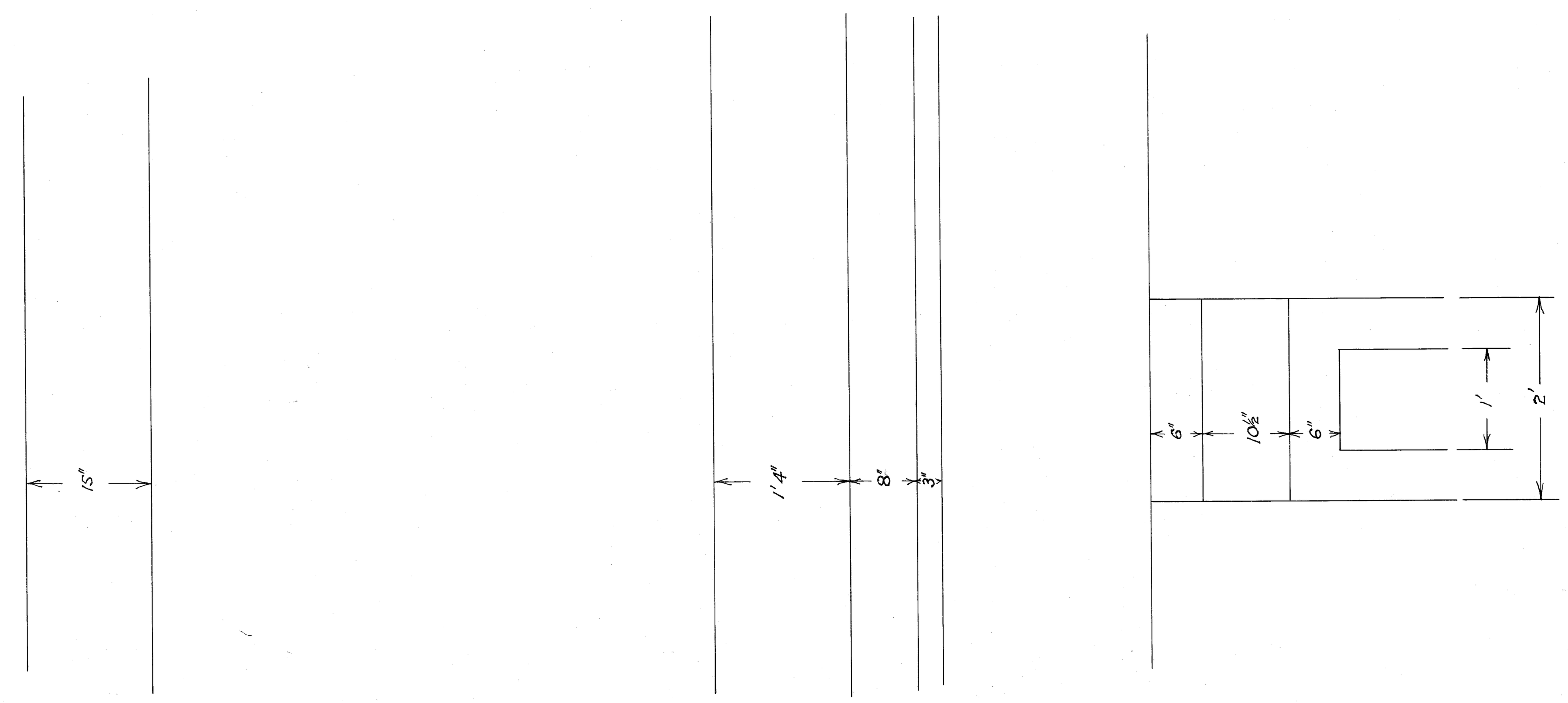
DETAILS OF UPPER TANK WALL FOOTWAY & COLUMN CAP

Scale: 1 in. to 1 ft.

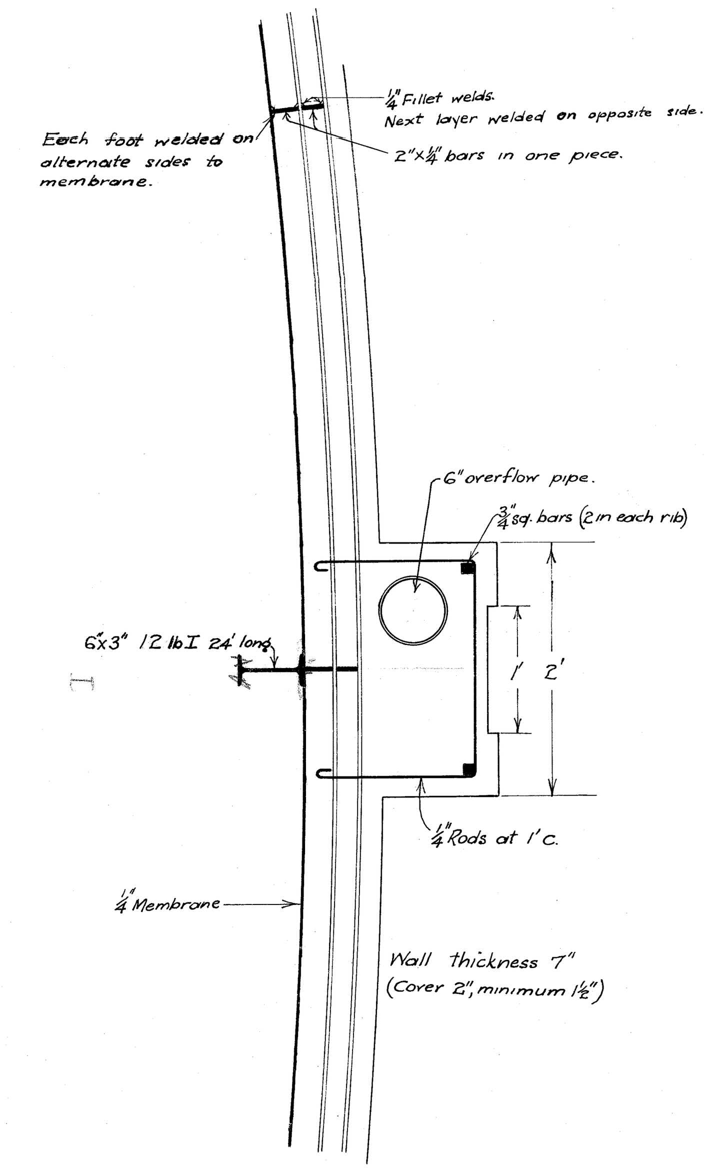
SHEET No 3



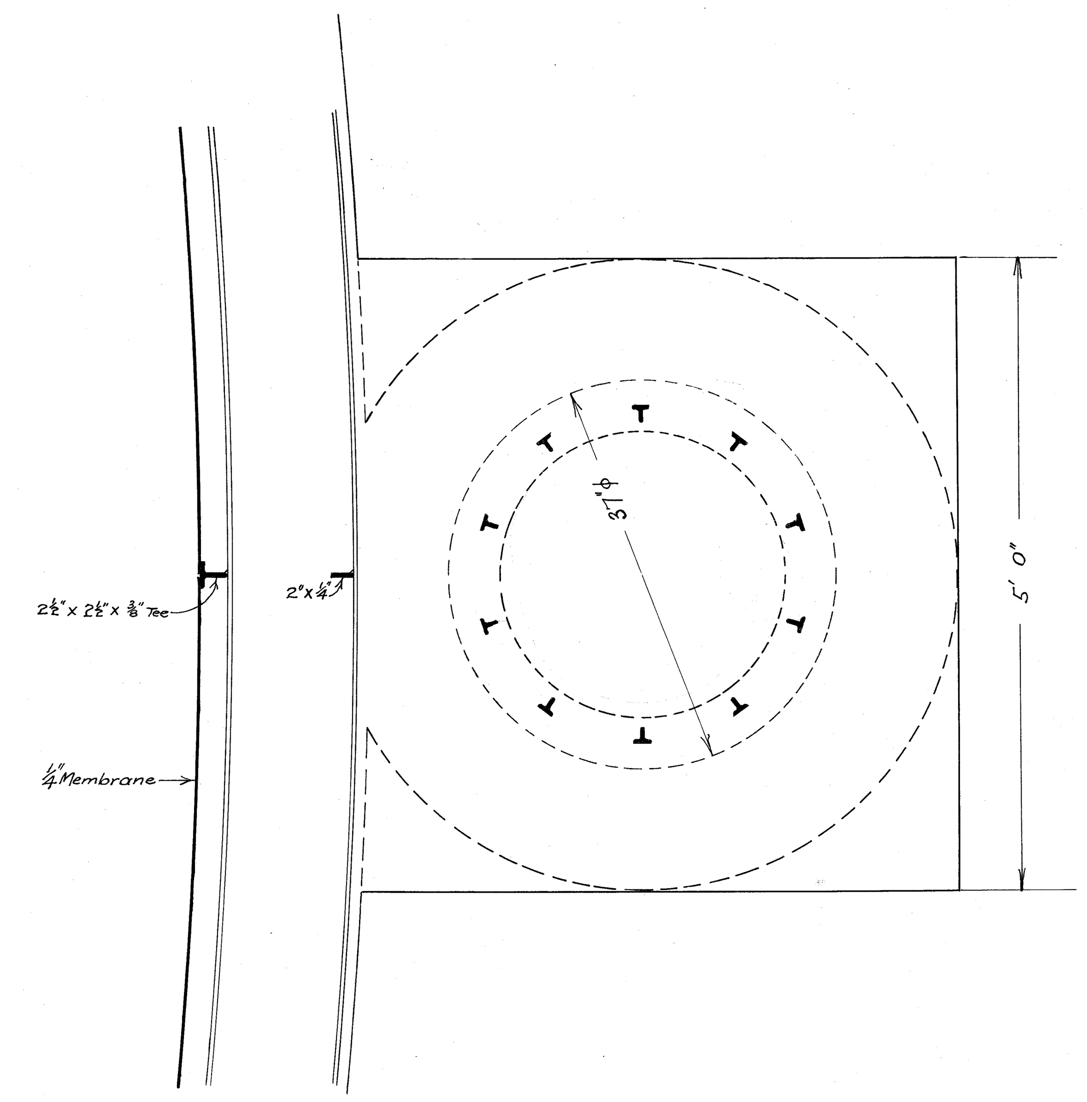
LAPPED & WELDED JUNCTIONS OF 3/4" SQ & 2" x 3/8" BARS.



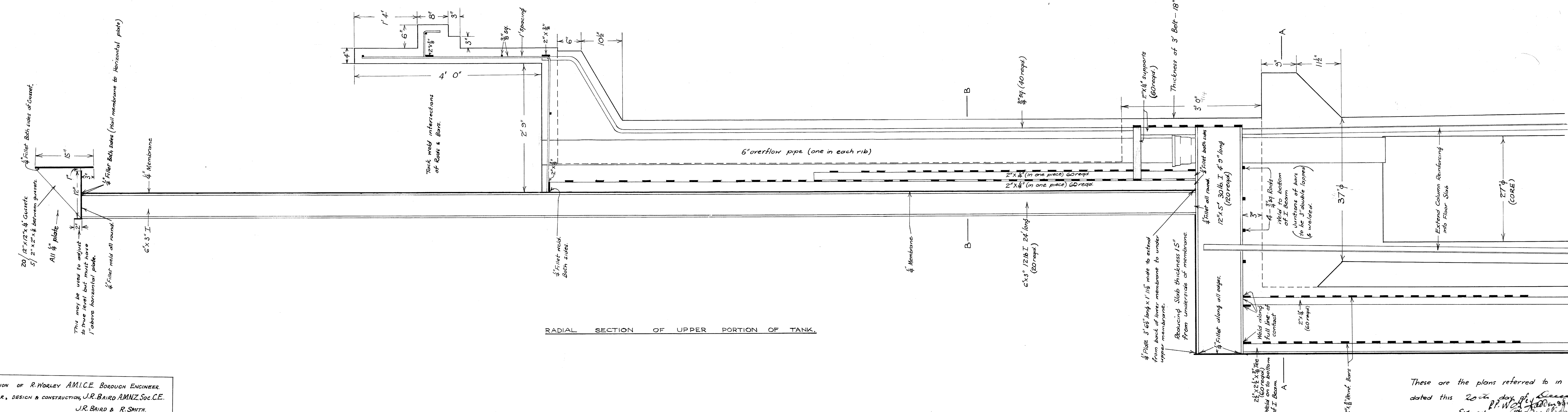
VIEW SHOWING OUTER FACE OF FOOTWAY ETC.



HORIZONTAL SECTION AT B–B



HORIZONTAL SECTION AT A–A



RADIAL SECTION OF UPPER PORTION OF TANK.

UNDER THE DIRECTION OF R. WORLEY A.M.I.C.E. BOROUGH ENGINEER.
RESIDENT ENGINEER, DESIGN & CONSTRUCTION, J.R. BAIRD A.M.N.Z. Soc. C.E.
DRAWN BY J.R. BAIRD & R. SMITH.

These are the plans referred to in my/our contract dated this 20th day of December 1929

Signed [signature]

Witness [signature]

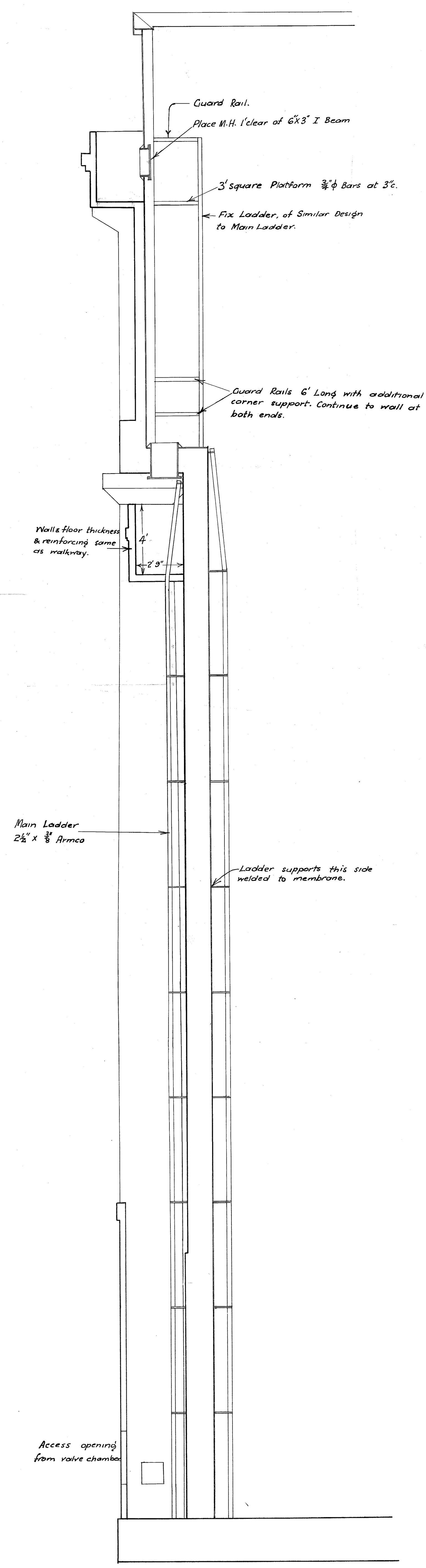
WW-T3-003

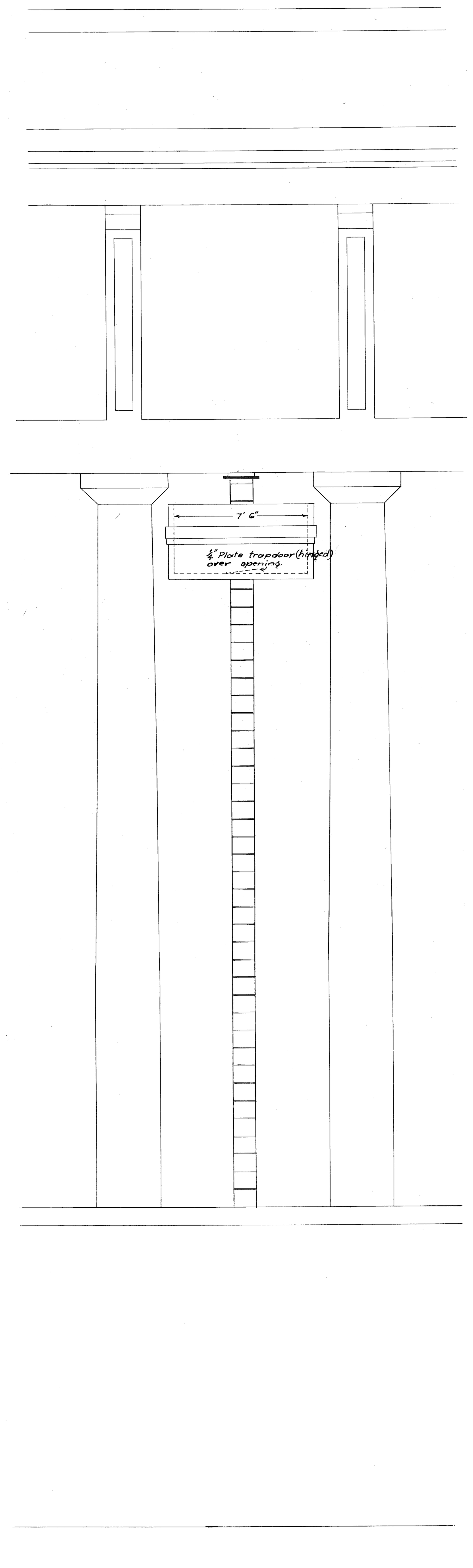
BOROUGH OF HAMILTON

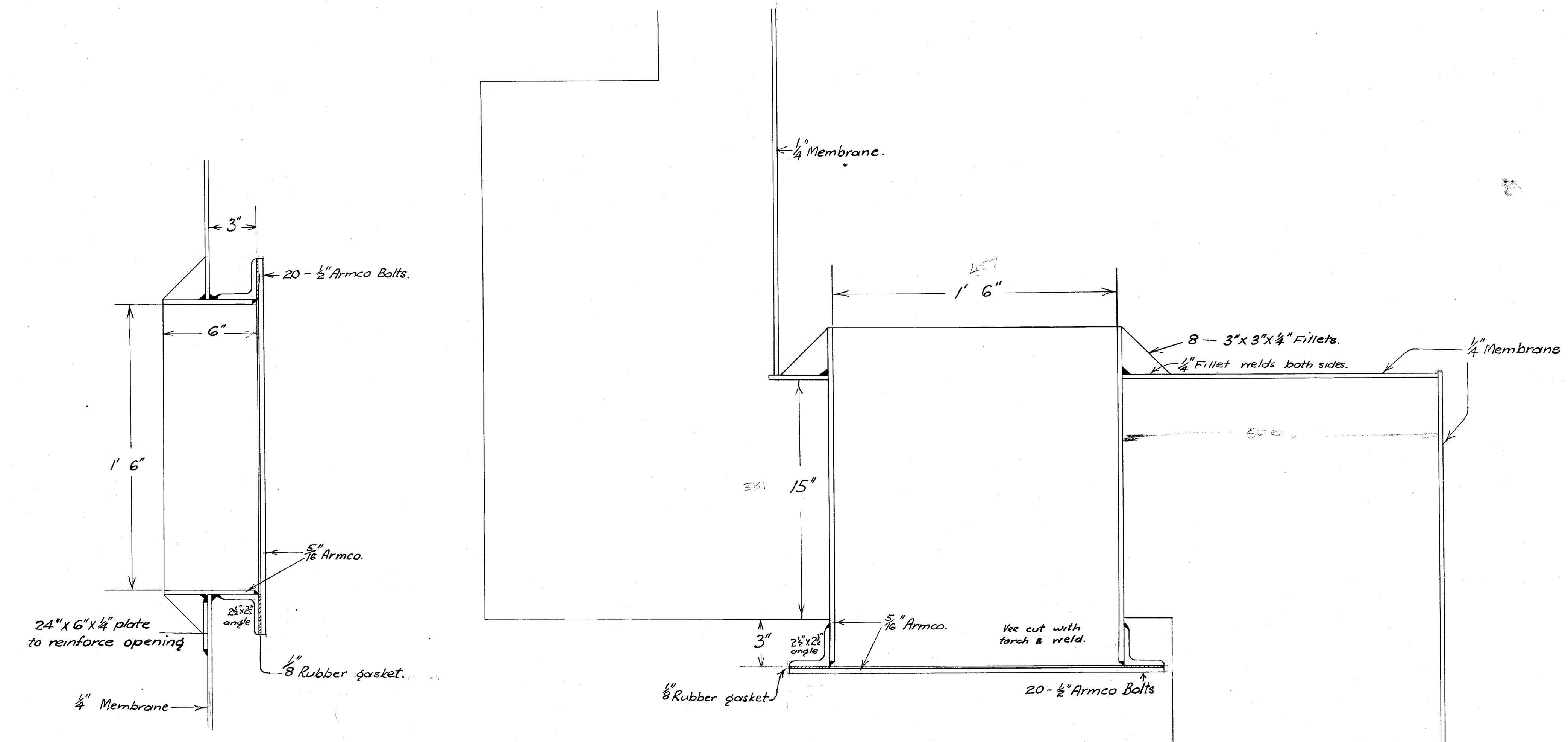
SHEET No 4

DETAILS OF TANK & FOOTWAY ACCESS LADDERS & MANHOLES.

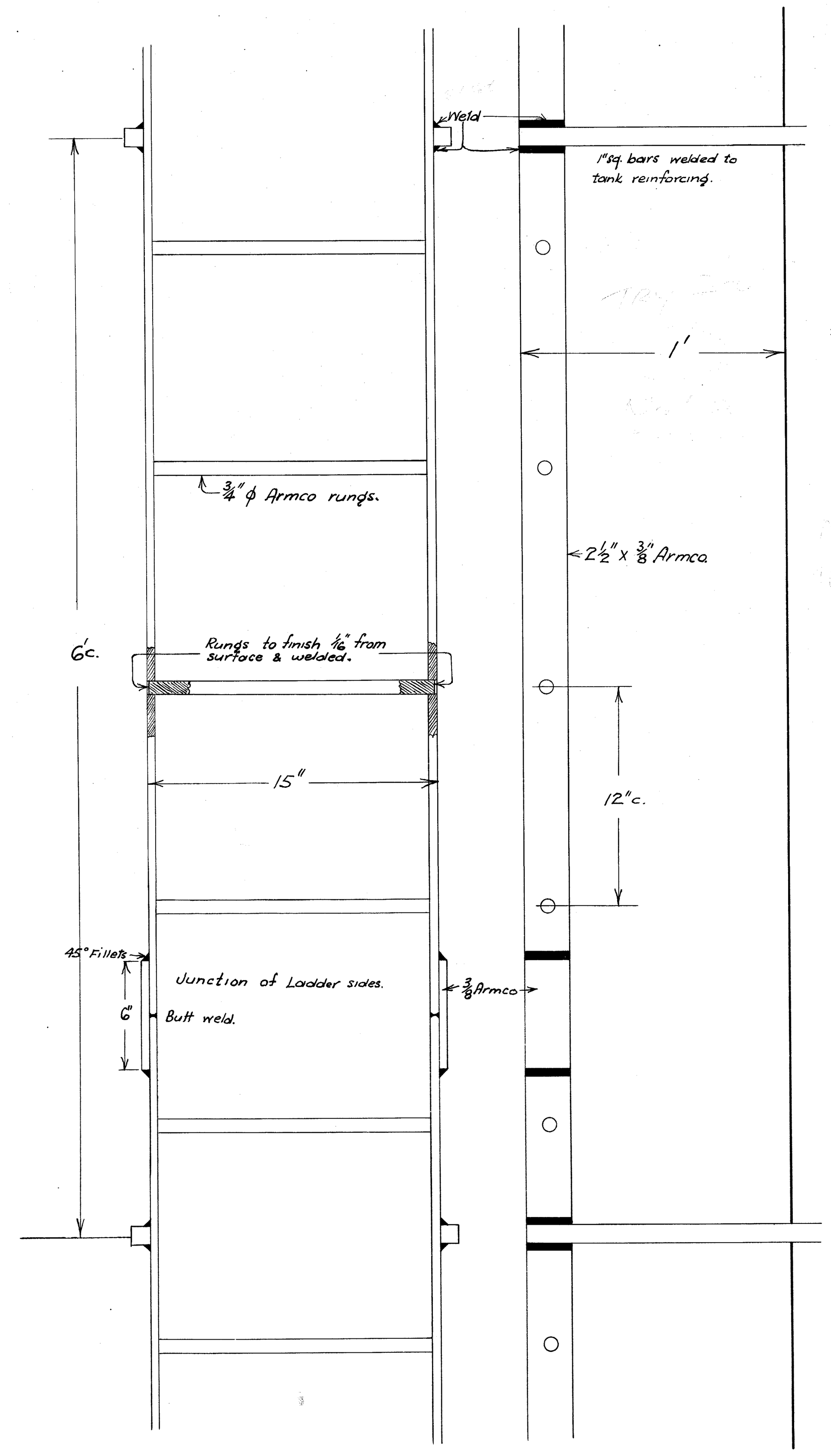
Scales {Tank Section & Elevation 1/4 in. / Enlarged Details 2 in.} to 1 foot.



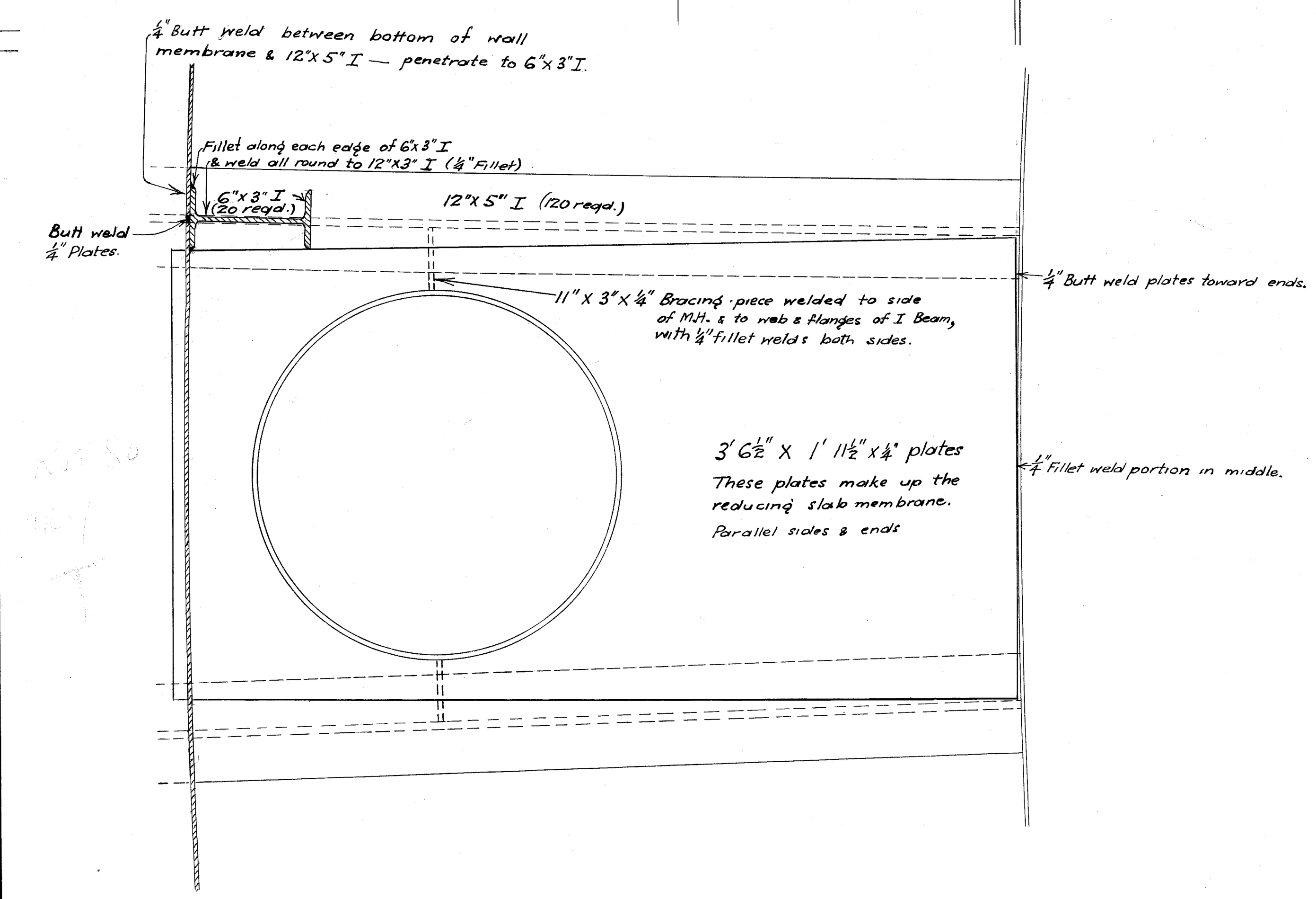




DETAILS OF UPPER MANHOLE.



DETAILS OF ACCESS LADDER.



DETAILS OF LOWER MANHOLE

UNDER THE DIRECTION OF R. WORLEY A.M.I.C.E. BOROUGH ENGINEER.
RESIDENT ENGINEER, DESIGN & CONSTRUCTION, J.R. BAIRD A.M.N.Z. Soc. C.E.
DRAWN BY J.R. BAIRD & R. SMITH.

These are the plans referred to in my/our contract
dated this 20th day of [illegible] 1929
pp W. McFaddin & Son
Signed [illegible]
Witness [illegible]

WW-T3-004

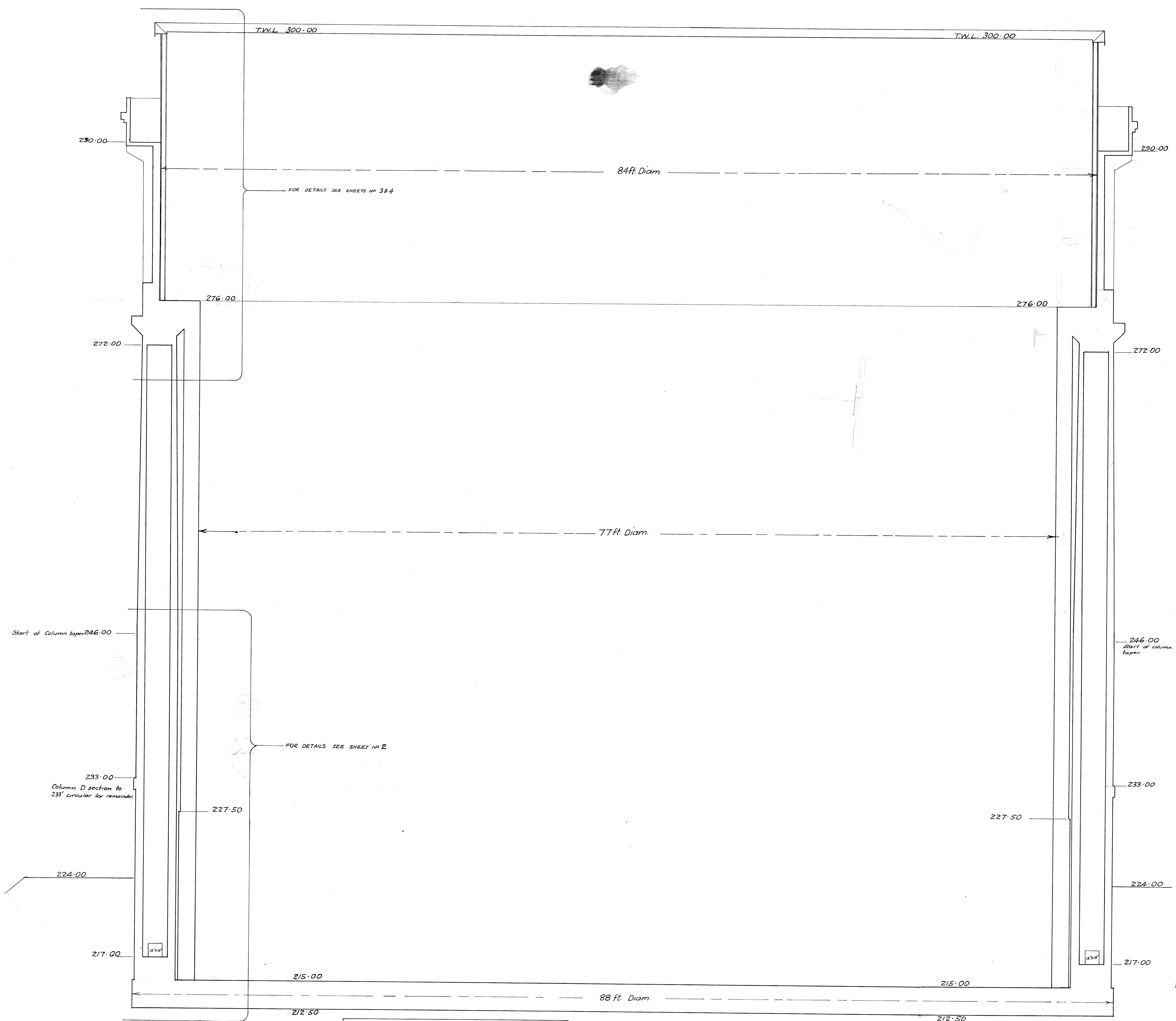
N.W. T/3.004

BOROUGH OF HAMILTON

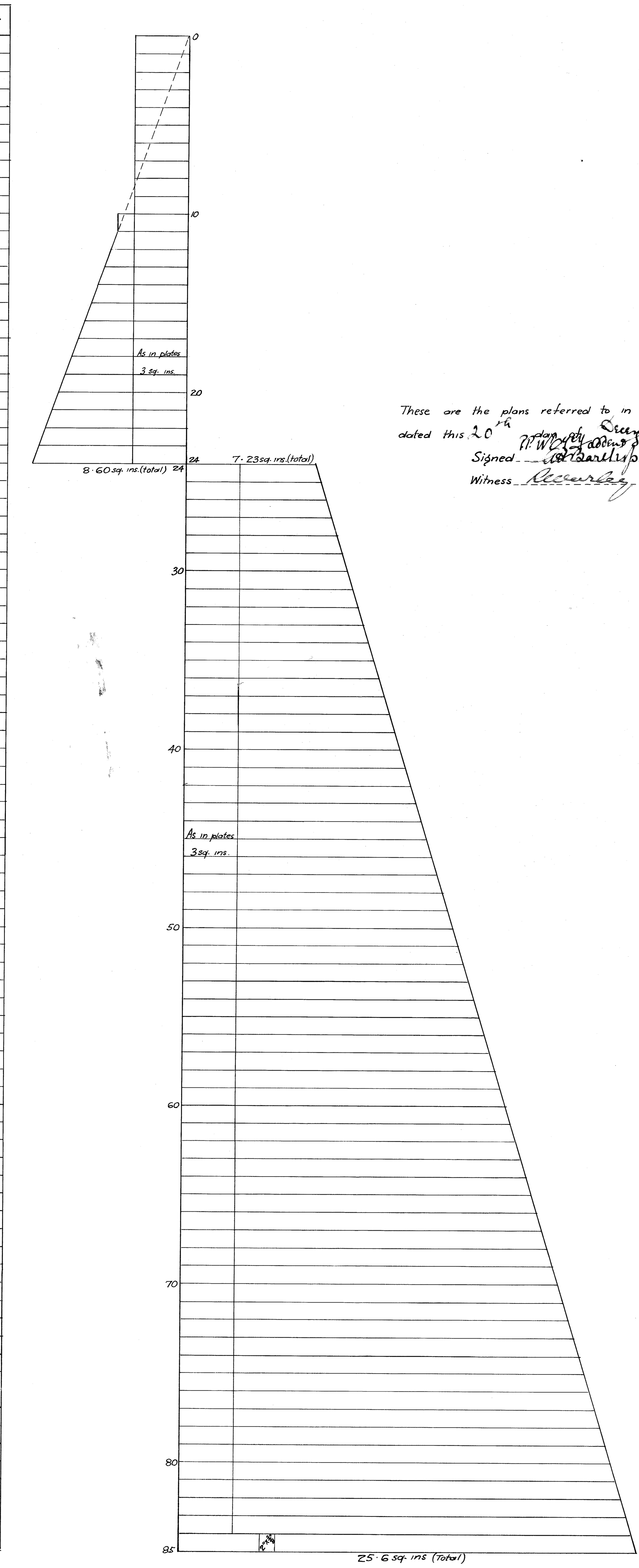
CROSS SECTIONAL ELEVATION OF TANK — HIGH LEVEL RESERVOIR.

Scale: ¼in. to 1ft.

SHEET No 6



Depth	No of Bars	No of Layers	No of Bars per Layer	Error
0				
10	1·00			0
	1·50			+·50
	2·00			0
	2·54			+·46
	3·00			0
	3·40			−·4
	3·88			+·12
	4·40			−·4
	4·85			+·15
20	5·33			−·33
	5·75			+·25
	6·25			−·25
	6·80			+·2
	7·20			−·2
24	5·80	3 1	1 4(¾"□)	+·2
	6·20	2	3	−·2
	6·60	2 1	2 3	+·4
	7·00	2 1	2 3	0
	7·40	2 1	2 3	−·4
	7·80	1 2	2 3	+·2
30	8·20	1 2	2 3	−·2
	8·60	3	3	+·4
	9·00	3	3	0
	9·40	3	3	−·4
	9·80	2 1	3 4	+·2
	10·20	2 1	3 4	−·2
	10·60	1 2	3 4	+·4
	11·00	1 2	3 4	0
	11·40	1 2	3 4	−·4
40	11·80	3	4	+·2
	12·20	3	4	−·2
	12·60	2 1	4 5	+·4
	13·00	2 1	4 5	0
	13·40	2 1	4 5	−·4
	13·90	1 2	4 5	+·1
	14·30	1 2	4 5	−·3
	14·70	3	5	+·3
	15·10	3	5	−·1
	15·50	3	5	−·5
50	15·90	2 1	5 6	+·1
	16·30	2 1	5 6	−·3
	16·70	1 2	5 6	+·3
	17·10	1 2	5 6	−·1
	17·50	3	6	+·5
	17·90	3	6	+·1
	18·30	3	6	−·3
	18·70	2 1 1	4 5 6	+·3
	19·10	2 1 1	4 5 6	−·1
	19·50	2 1 1	4 5 6	−·5
60	19·90	1 2 1	4 5 6	+·1
	20·30	1 2 1	4 5 6	−·3
	20·70	3 1	5 6	+·3
	21·10	3 1	5 6	−·1
	21·50	2 2	5 6	+·5
	21·90	2 2	5 6	+·1
	22·30	2 2	5 6	−·3
	22·70	1 3	5 6	+·3
	23·10	1 3	5 6	−·1
	23·50	4	6	+·5
70	23·90	4	6	+·1
	24·30	4	6	−·3
	24·70	4	6	−·7
	25·10	3 1	6 7	−·1
	25·50	2 2	6 7	+·5
	25·90	2 2	6 7	+·1
	26·30	2 2	6 7	−·3
	26·70	4	7	+1·3
	27·10	4	7	+·9
	27·50	4	7	+·5
80	27·80	4	7	+·2
	28·20	4	7	−·2
	28·70	4	7	−·7
	29·10	4	7	−1·1
	29·50	4	7	−1·5
85	27·00	4	7	+1·0



REINFORCING CHART & GRAPH FOR TANK WALL

These are the plans referred to in our contract dated this 20th day of December 1929.
Signed ....
Witness ....

UNDER THE DIRECTION OF R. WORLEY A.M.I.C.E. BOROUGH ENGINEER.
RESIDENT ENGINEER, DESIGN & CONSTRUCTION, J.R. BAIRD A.M.N.Z. Soc. C.E.
DRAWN BY J.R. BAIRD & R. SMITH.

WW-T3-006

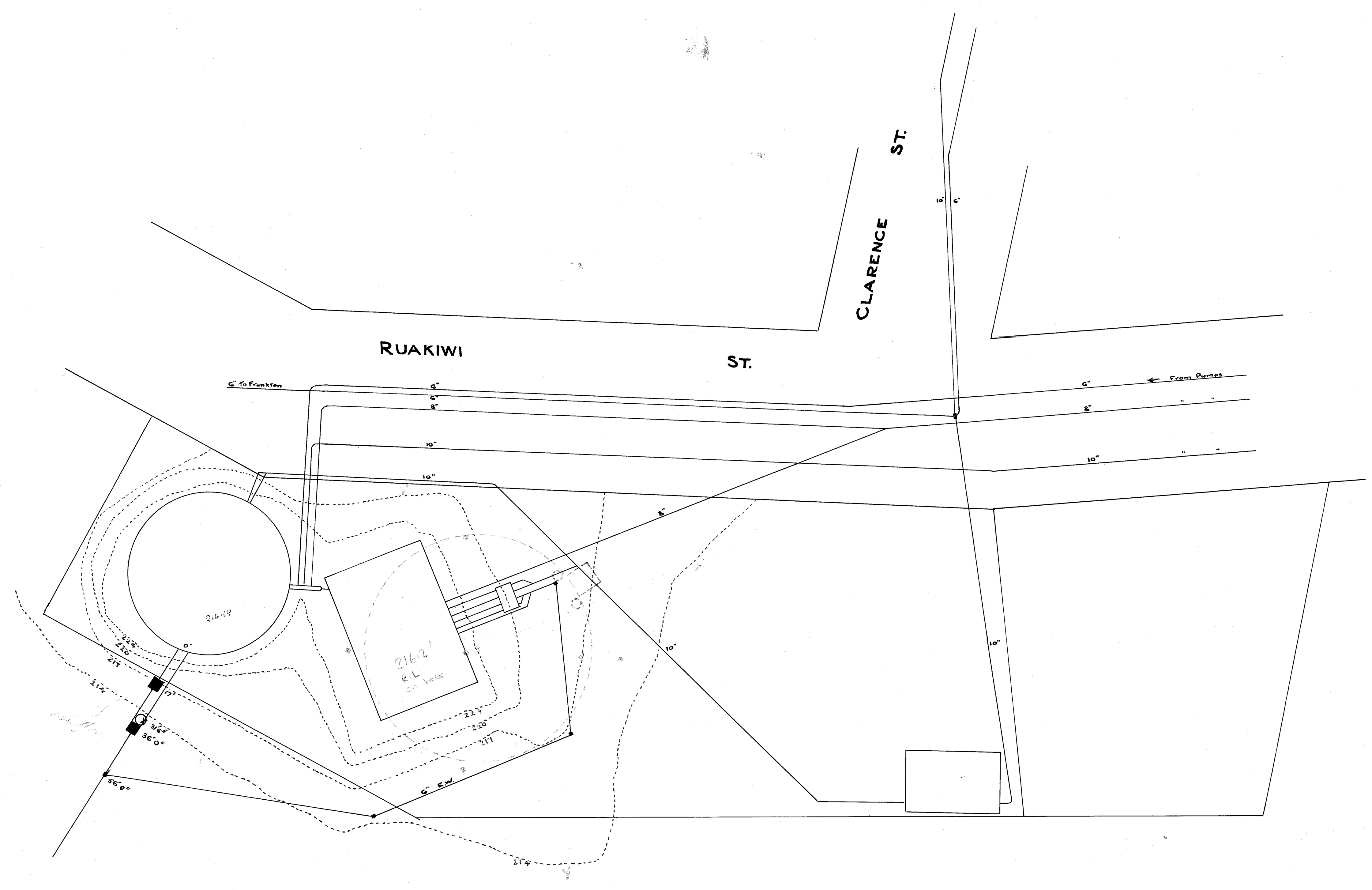
Under the direction of R. Worley A.M.I.C.E Borough Engineer.
Resident Engineer, design & construction J R Baird A.M.N.Z. Soc. C.E.
Drawn by S.E. West.

Borough of Hamilton

Sheet N° 7

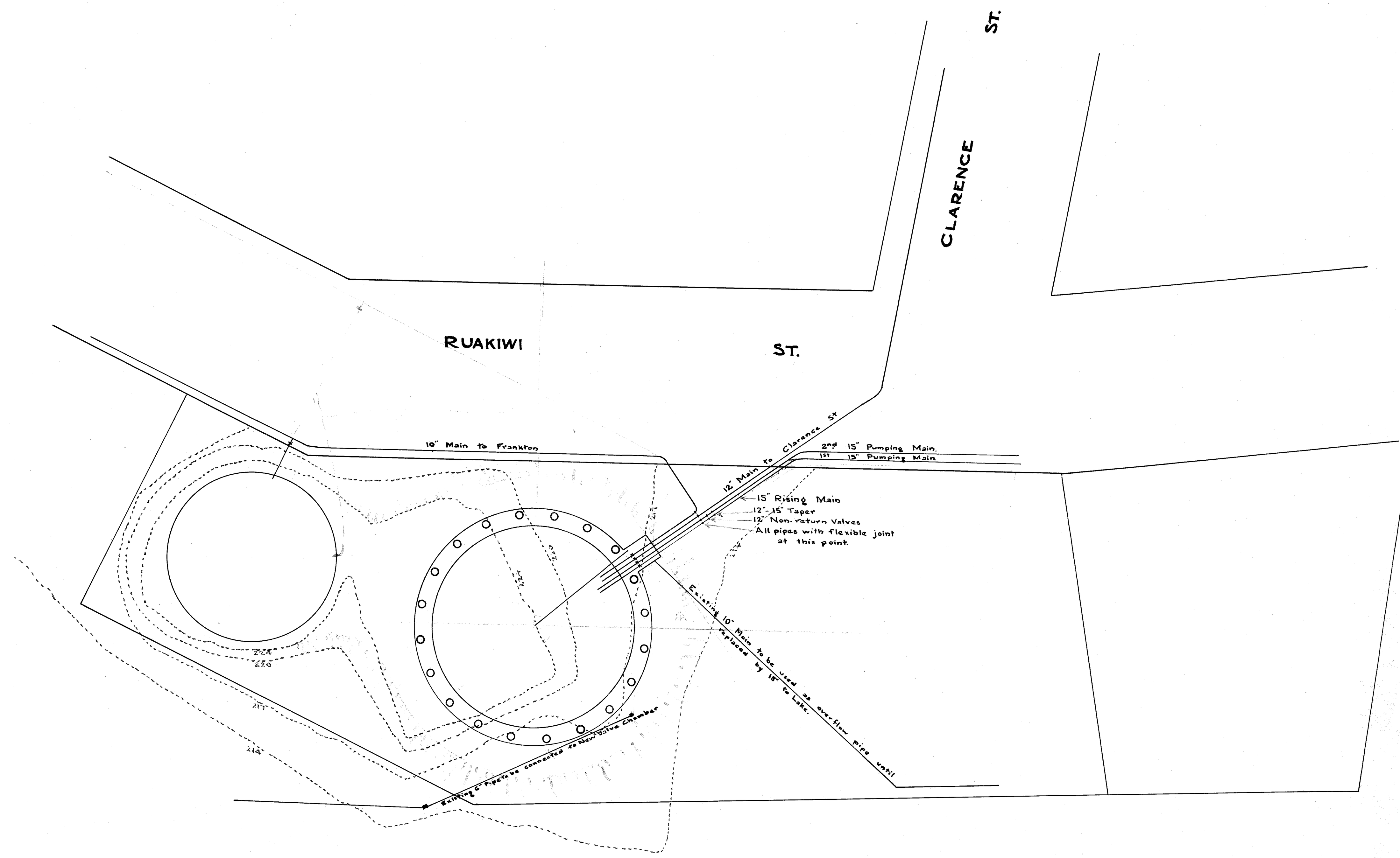
Plans & Cross Sectional Elevations of Site

Scale: 30 feet to 1 inch

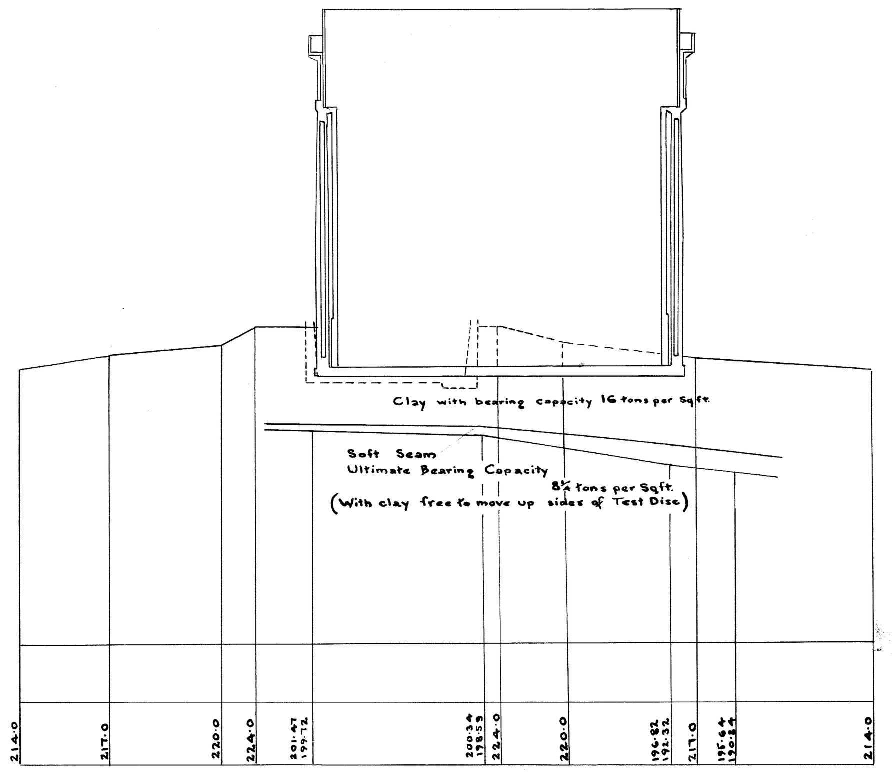


Contour Plan shewing Present Arrangement of Reservoirs, Pumping & Supply Mains, & Filters.

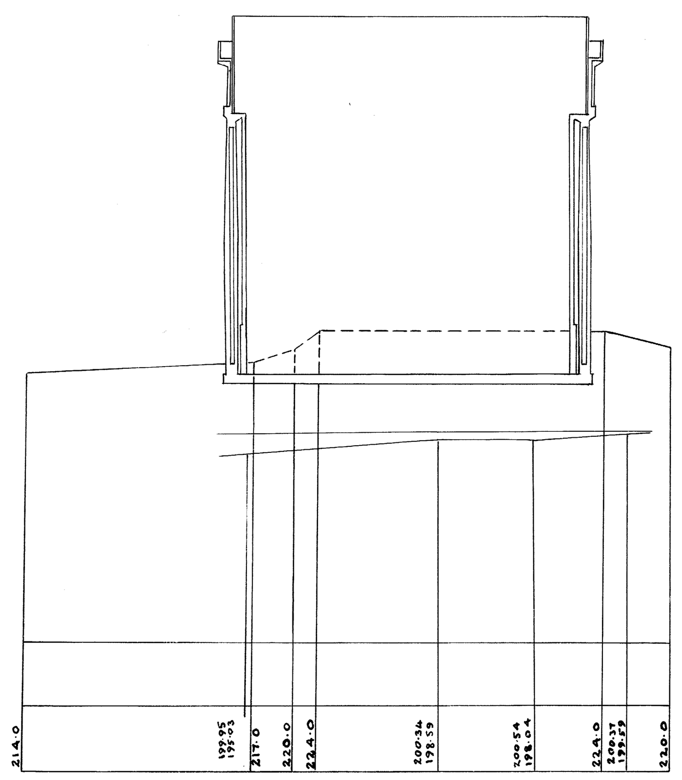
← Denotes bores to determine approx. position of soft seam



Plan shewing New Arrangement

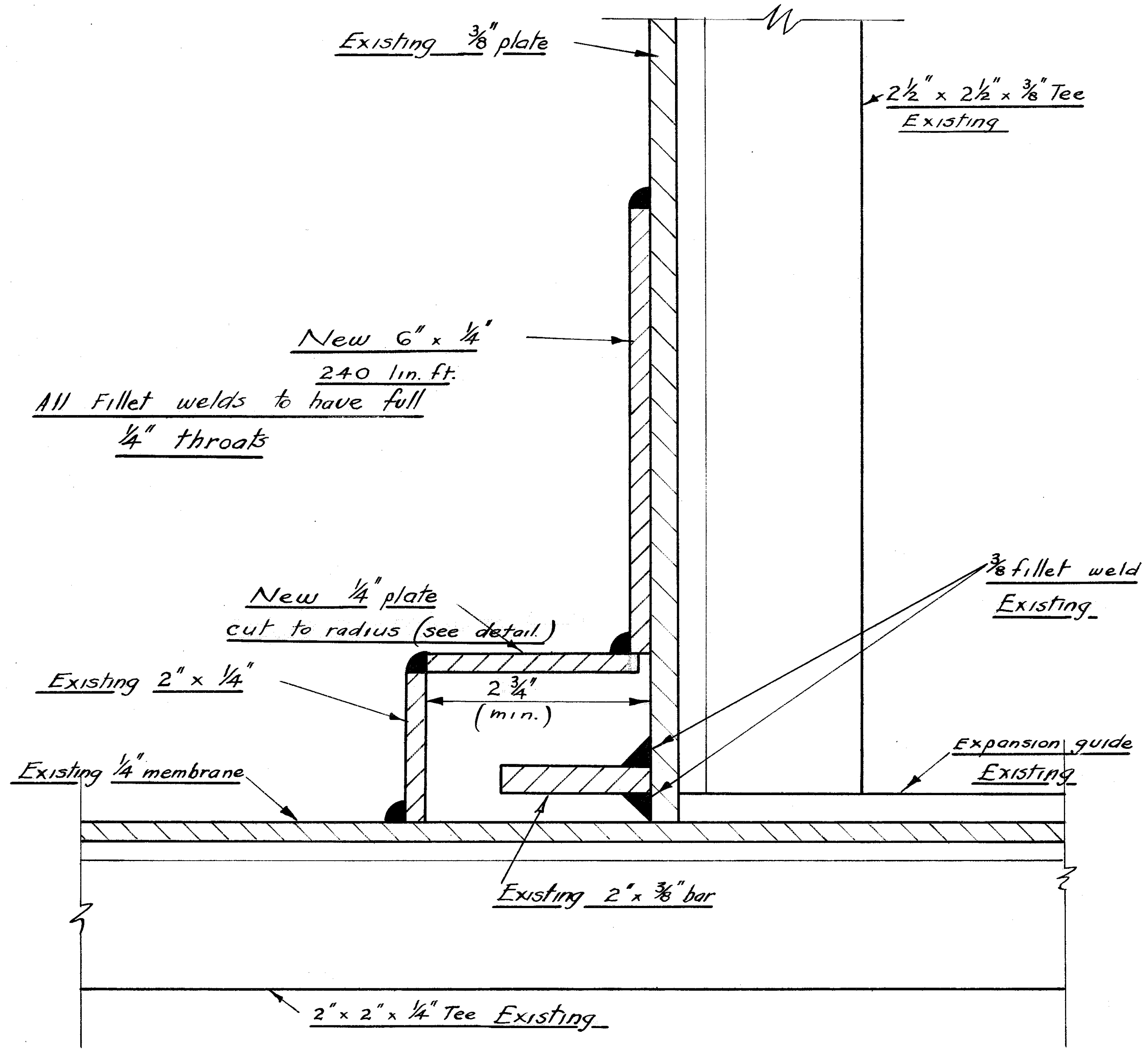


Section along XX'



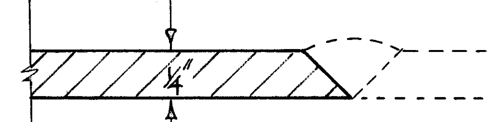
Section along YY'

WW-T3-007

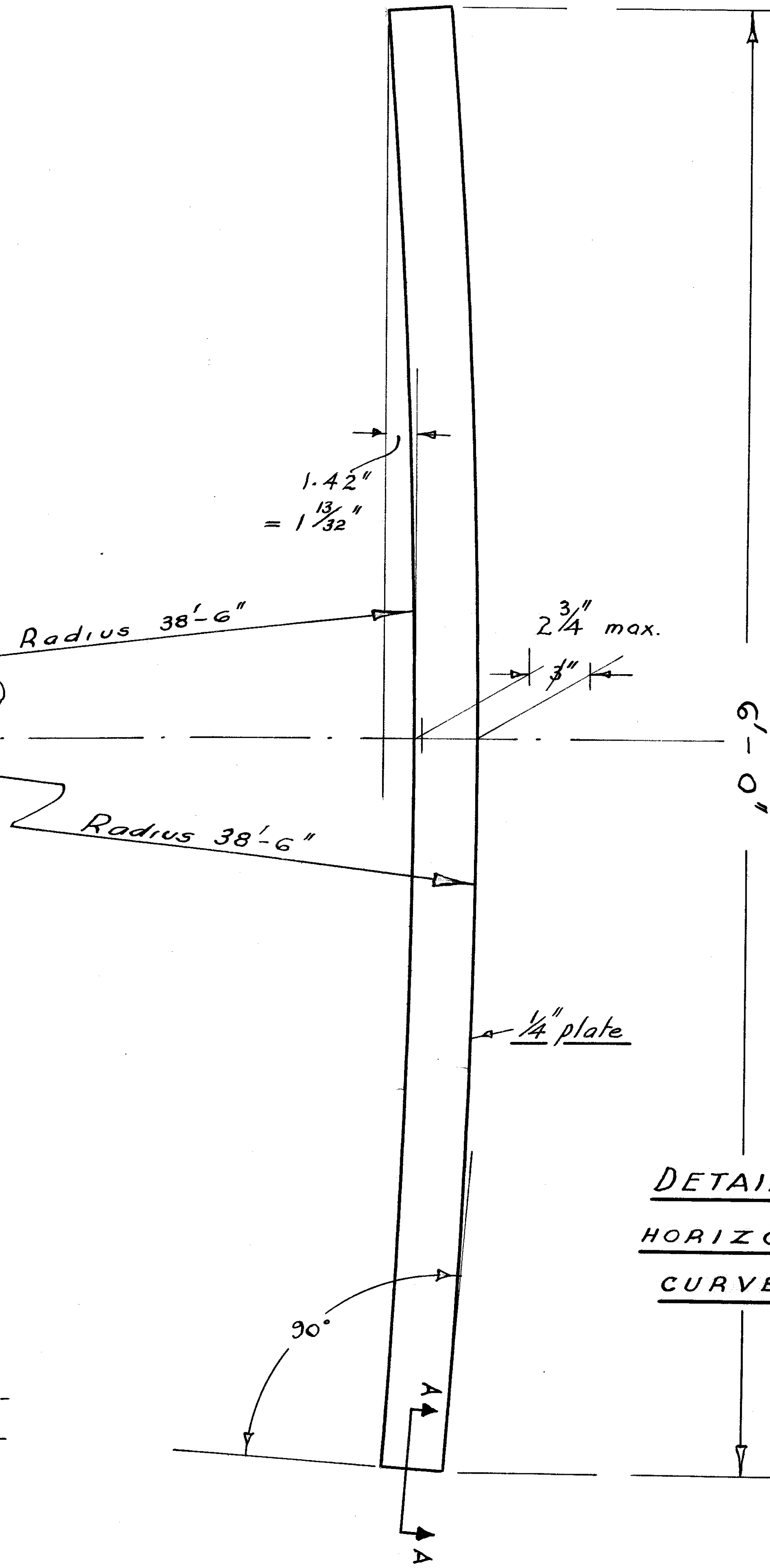


SECTION OF WALL BASE
SHOWING
ALTERATION TO EXPANSION JOINT

SCALE :- ½ full size



Section A-A
Full Size



DETAIL OF HORIZONTAL CURVED PLATE

SCALE : 1½" to 1 ft.

40 Reqd.

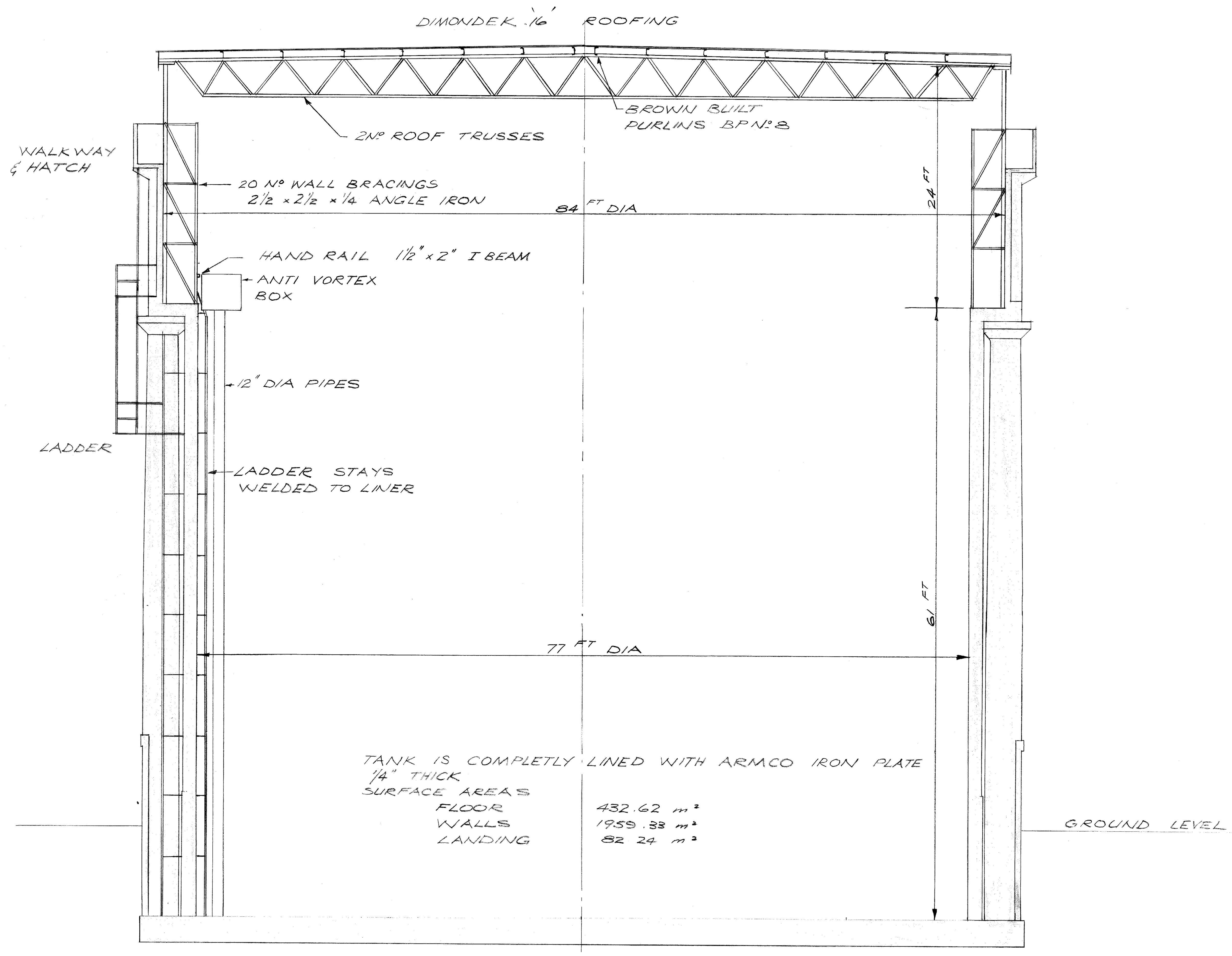
Revised. Width of curved plate 21.9.48 - F.S.

HAMILTON CITY ENGRS DEPT.	SCALES AS SHOWN			D	FILE REF. NO.
MAIN WATER TOWER	APPROVED	DRAWN	F.W.H.S.		WW/T/3.015
REPAIRS TO EXPANSION JOINT		TRACED	F.W.H.S.		
	DATE 16/9/48	CHECKED			

Brownbuilt metal purlins B.P. Nº 8
PLAN OF STEELWORK
93' 9" Internal dia. of existing steel Liner
84' 10" Internal dia. existing steel bracket
℄ truss
℄ Reservoir
13'.0"
'Dimondek 16' roofing
Brownbuilt metal purlins B.P. Nº 8
7" x 1/4" x 16" lg. M.S. cleats
Full 'V' butt weld
Exist. bracket
1/4" M.S. end Pl.
4" x 4" x 1/4" R.H.S.
Exist. Steel Liner
DETAIL 'A'
2/3" x 3" x 1/4" L's
4" x 4" x 1/4" M.S. Pl.
2/2" x 2" x 3/16" L's
℄ Exist. Main tank conc. columns.
1/2" ϕ bracing with 2" Min. dia. washers each side of each purlin.
4" x 4" x 1/4" R.H.S. col.
Exist. Steel Liner
Exist. conc.
DETAIL 'B'
SECT 'B-B'
Weld bracket to col. & following erection weld truss to bracket
4" x 4" x 1/4" R.H.S. col.
TYPICAL SECT. THRU. TRUSS
BASE Pl. FOR 4" x 4" R.H.S. COL.
Terrier fixings in exist. conc.
ELEVATION TRUSS
80' 7" over 4" x 4" cols.
4" x 4" x 1/4" M.S. Pl. welded across face of latticed members where shown.
All latticing 2/2" x 2" x 3/16" L's.
Exist. Steel Liner
Exist. conc.
DETAIL 'C'
DETAIL 'D'
DETAIL 'D' AT RIDGE WHERE BROWNBUILT PURLINS ARE SET DOWN 1"
Dimondek 16' roofing
Brownbuilt metal purlins B.P. Nº 8
Exist. Steel Liner
℄ Reservoir
℄ truss
SECT 'A-A'
13'.0"
ALL WELDS TO BE 1/4" FILLET WELDS, UNLESS NOTED OTHERWISE.
DIAGRAM OF TRUSS
WW/T3/032

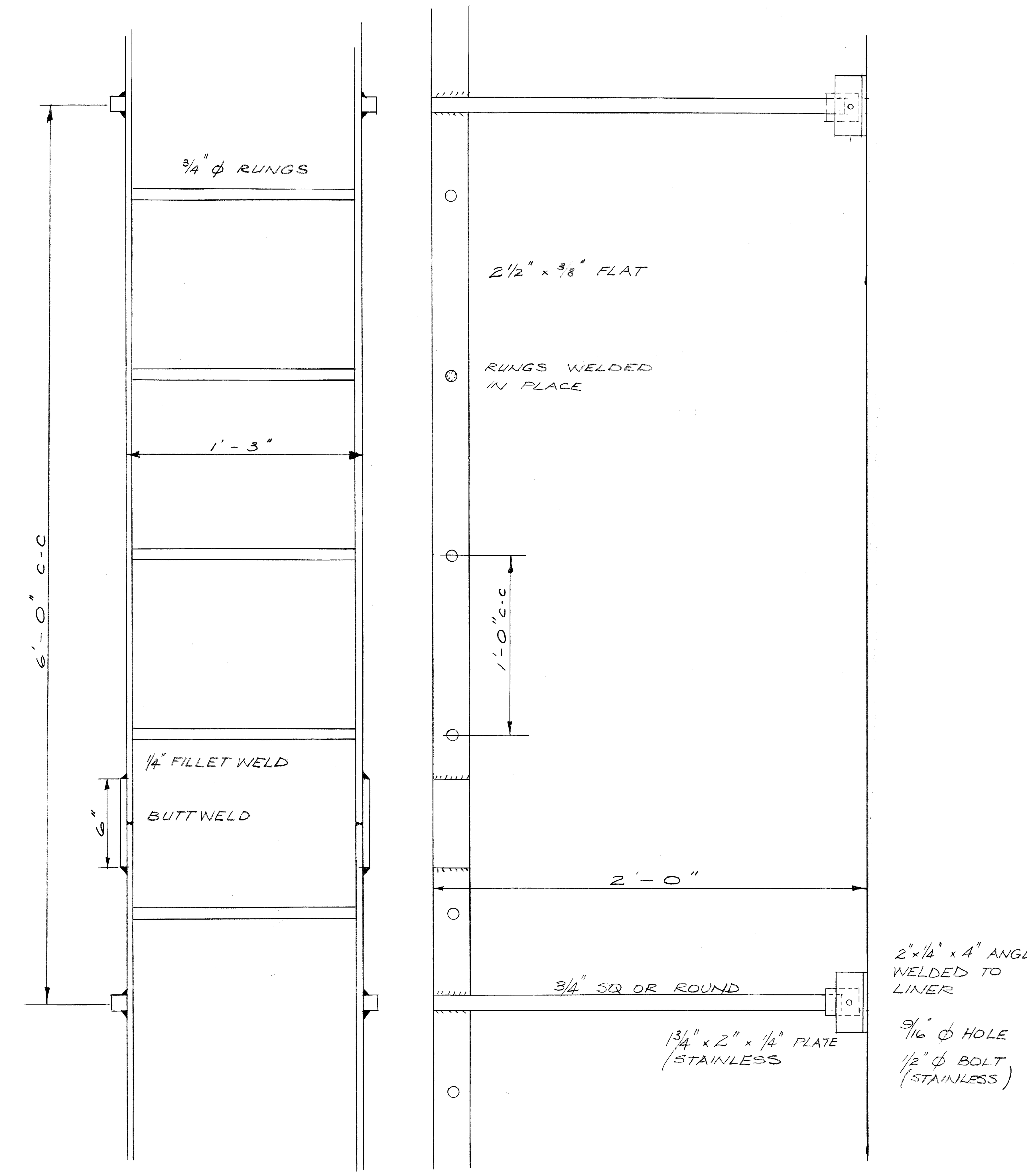
WW/T3/04

CENTRAL RESERVOIR
GEN DETAILS OF TANK



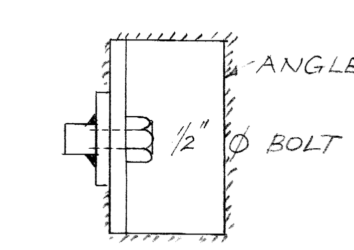
TANK DETAILS

SCALE ⅛ IN TO 1 FT



LADDER DETAILS
TOTAL LENGTH 61'-0"

SCALE 2 IN TO 1 FT



DIA NOT TO SCALE

Graphic Scales :								
	AMENDMENTS	NAME	DATE		NAME	DATE	PROJECT	
A				Survey			APPROVED	
B				Drawn	R UTTING	8/78	J Baird 78	
C				Design	J BAIRD	1929	APPROVED FOR ISSUE	
D				Traced	R UTTING	8/78		
E				Checked			CITY ENGINEER	

HAMILTON CITY COUNCIL : ENGINEER'S OFFICE

CENTRAL RESERVOIR

GENERAL DETAILS OF TANK

SCALES : AS SHOWN

Origin of Levels :

Field Book :

Level Book :

Corr. File :

Consec. No. : 79/

PLAN No. WW/T3/040

Sheet of Sheets

A1

Re-Order Filemaster "S" SELF-ADHESIVE PLAN HANGER
Patent Number 137841