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GEELONG AND DISTRICT WATER BOARD

JOB NO 26190/01

DATE DUE



TRUCTURAL INTEGRITY ASSESSMENT

OF

NON RIVER OVOID SEWER AQUEDUCT

by

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Taywood Engineering Limited and Maunsell Pty Ltd
In Association

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OVOID SEWER AQUEDUCT
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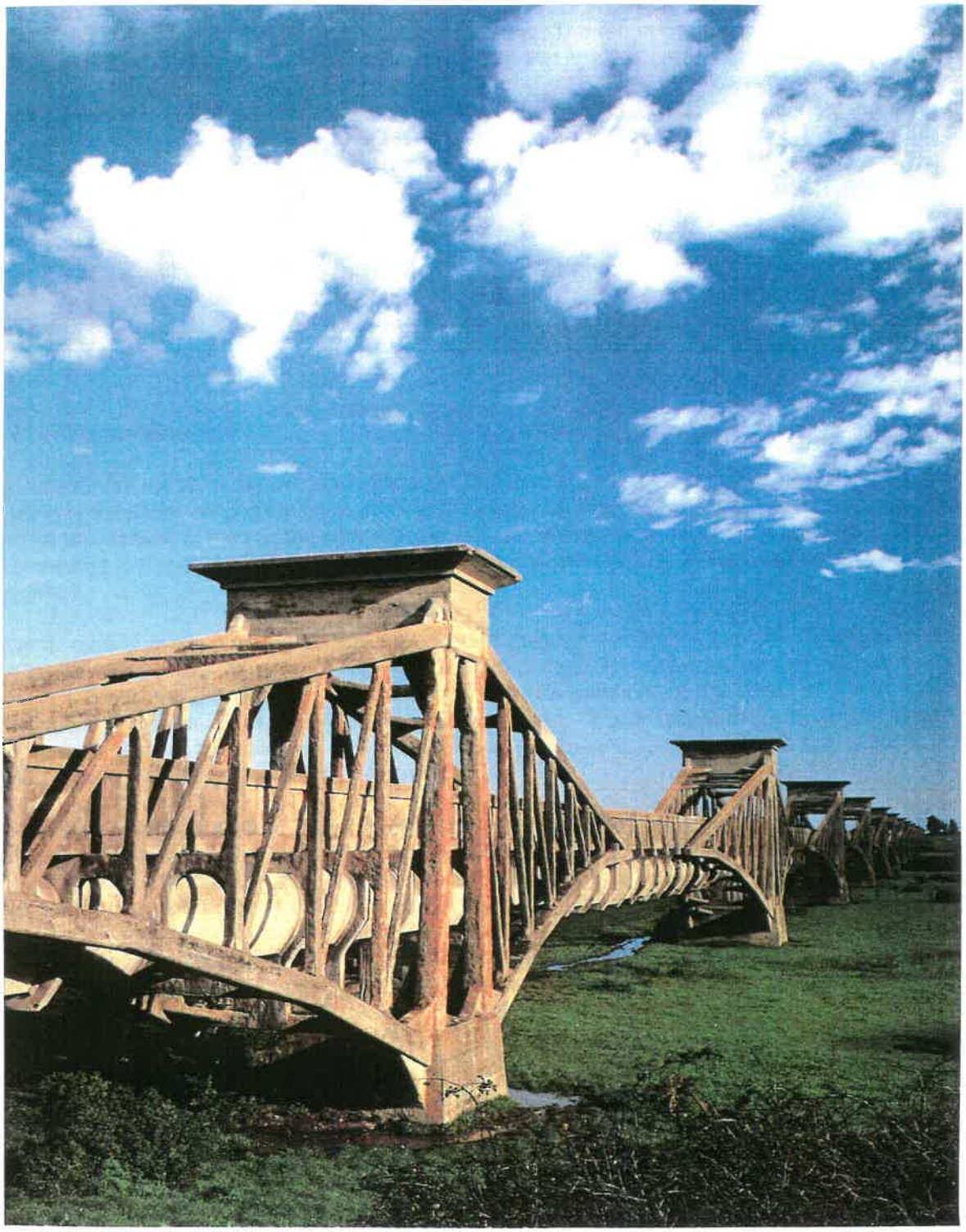
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1. INTRODUCTION

1.1 General

The Geelong and District Water Board commissioned Taywood Maunsell in October 1990 to report on several options for the future management of the Ovoid Sewer Aqueduct in South Geelong. The Board requested that four options be considered; and that the costs of staged repair be considered for each option.

This study has utilised knowledge gained on previous surveys and written records held by the Board to supplement the findings of the most recent investigation. The report produced by Taywood Maunsell in March 1990 is contained in Appendix B, as much of that study is particularly relevant.

The cost estimates are based on the extent of visible damage and a cost rate for performing full patch repairs. The cost rates include allowances for access, materials, labour and falsework but do not include structural propping, site support costs and reinforcement replacement. These costs are included as separate items, as are additional costs such as tender document preparation and contract administration.

The use of realkalinization has been listed as an alternative to a full patch repair approach as its suitability can only be determined by conducting trials on this particular structure. It is probable that this repair method is the only cost effective way that any of the original cover concrete can be retained in the long term.

1.2 Survey Methods

To supplement the work conducted in the initial Taywood-Maunsell investigation, Appendix B Section 1.1, the entire above ground structure was visually assessed. Each element was given a damage rating and an assessment of the surface area affected was made as a percentage of the total area. This data is presented in Appendix A and provides the basis for the preservation options and costs presented in Section 5.

Specific testing was conducted at randomly selected areas to confirm the causes of damage reported on in the March 1990 document, Appendix B Section 1.2. This included measuring the depth of concrete cover and the extent to which the concrete has carbonated, the taking of electropotential readings and the testing of the resistivity of the concrete. These test techniques are described in Appendix B, Section 2.2.2 pages 4 to 9. An additional explanation on the carbonation process is given in the Glossary of Terms contained in Appendix B.

Samples were extracted and tested in the laboratory for chloride and sulphate content, two elements whose presence in concrete above a certain concentration have deleterious effects on embedded steel, in the case of the former, and the concrete itself, if the latter. A brief description of chloride and sulphate attack is contained in Appendix B, Sections 2.3.1 and 2.3.2.

To assess the condition of the piles excavations were made adjacent to Piers 3 and 8 which gave access to two piles below the pier bases. The results of the testing are discussed in section 2.6 of this report.

2. MATERIAL ASSESSMENT

2.1 Cantilever Trusses

The major proportion of previous repair expenditure has been devoted to truss elements; in particular the tension members. The repairs have generally been conducted on an "as needed" basis and the techniques used and quality of these repairs have varied.

The extent of the repair works and the long period over which maintenance has been conducted makes it hard to visually differentiate between original concrete and subsequent repairs in many areas.

To fully categorise the different cementitious materials which have been applied to the aqueduct, their effective performance, the extent of environmental penetration, and their likely future performance would require a very extensive study. Such an investigation would be invaluable in terms of research in this field, and probably unique due to the range and age of materials present on the one structure, but could not be justified on the basis of the benefit to the structure itself.

The site investigation found that wherever the original concrete still provided cover to the reinforcing steel, the depth of carbonation exceeded the depth of steel. The typical depth of carbonation was measured to be 55 mm, which on the compression and tension members would leave only approximately 20% of the concrete unaffected.

More than 50% of the concrete in the main chords would be similarly affected, and it should therefore be assumed that any steel surrounded by original concrete has been depassivated. This was supported by the potential readings taken on visually good members, one member from each pier being tested. The results, particularly those from top and bottom chords, indicated that corrosion activity was likely to be occurring at various locations along the entire length of most visibly sound members.

The repaired tension members showed no sign of activity when potential readings were taken except in the region adjacent to the top and bottom chord joint details. Here the electro potential gradients were pronounced indicating that near these details active corosion may be occurring.

As previous repairs were conducted on the basis of deterioration having progressed to a stage that it was visible, then the majority of reinforcing steel in repaired locations would have been corrosively active until being repassivated by the application of the repair material. However that will only be the case where the repair procedure involved the removal of the carbonated concrete from behind the bars as well as the damaged cover material.

The shotcrete treatment to most elements on Piers 1 and 2 has kept those members in a reasonable visible condition, however testing revealed that though the shotcrete had only carbonated approximately 5 mm, the majority of the steel is lying in carbonated areas of the original concrete. Pier 2 is showing signs that deterioration is starting to accelerate as records indicate that both were in a reasonable condition in the early 1980's. The top chord of the south east cantilever has a small area of drummy material maybe 5% of the surface area, but otherwise no indication of damage. The electropotential assessment would be that in the near future up to half the total surface concrete will be damaged by the underlying reinforcing steel if it is left in its present condition.

The prognosis for all other top and bottom chords is generally similar, or worse. Of the 56 trusses, 49 top chords show some sign of damage and 34 bottom chords are similarly affected. Of the remaining 29 chords, eight were tested. Electropotential readings predicted that all eight had significantly larger areas of corrosion activity than was indicated by surface defects. The rate at which damage will propogate and the time it takes to become visible will vary depending upon the corrosion cells set up within each element (eg anode to cathode ratio) the depth of cover, the resistivity of the concrete, the composition of the concrete and the effects of previous repairs where appropriate. The resistivities were tested and found to be low, generally less than 10,000 Ohm.cms and this would suggest that once a corrosion cell has been initiated the rate of corrosion will be rapid.

The reinforcing steel size and distribution will generally result in the helical wire ligatures corroding prior to the main bars. Except in regions where the concrete cover is low, less than 20 mm, it appears that the development of visible damage is determined by the corrosion of the main bars. This is due to the fact that the density of the concrete and the detailing of the reinforcement allows the helical ligatures to corrode without causing high spalling pressures. The primary concern is that by the time the main bar corrosion causes visible deterioration, the ligatures will have deteriorated to a stage that they require replacing. On some members this will require careful sequencing of repair works to ensure structural adequacy.

2.2 Tower Structure

This incorporates the pier base and the capping arrangement of the 4 main columns, the columns themselves are not included as these were considered to be components of the cantilever trusses.

The pier bases above ground level are in good condition and, with random cover readings indicating an average depth of cover to reinforcement of 60-70 mm, the concrete is providing the reinforcing steel with protection.

Minor damage has occurred and this typically coincides with random areas of low cover.

Below ground level the two pier bases inspected revealed no signs of damage despite the aggressive nature of the groundwater. It is likely that the concrete is saturated to such an extent that there is insufficient oxygen to support the corrosion process.

The capping details generally show advanced deterioration on the slab soffits and a high percentage have damage occurring on their external walls. It appears unlikely that this deterioration has been accelerated by the effusion of hydrogen sulphide gas from the sewer vents located beneath these elements. The dispersion of the gas would be rapid making the penetration of carbon dioxide the controlling process in this environment.

It is apparent that these elements have been patch repaired at some stage, but the deterioration of the work suggests that improper treatment of the reinforcement and insufficient additional cover was provided.

The detail at the tower top is a parapet wall surrounding the slab and this is likely to result in the slab concrete remaining moist during wet periods. Even if drain holes can be kept clean, minor debris and poor crossfalls will ensure that some moisture will penetrate the concrete, reducing its resistivity, and increasing its rate of corosion.

2.3 Walkway

The walkway component of the structure was assessed in terms of 4 elements. The parapet beams, the balustrade walls, the lower beams and the slab.

(a) Balustrade Walls

The assessment of the balustrade walls has been complicated by the alteration of a large proportion of these elements by the application of a shotcrete material. This has not only filled in the gaps between the balustrades but has added approximately 30 mm of cover. It is likely that this work was conducted in the 1950's when major repair of the tension members was also conducted. The general visual appearance of the continuous walls is good and the overlay provided good protection to the steel reinforcing when these sections are compared to those left in the original configuration.

The underlying portions of the original balustrades were not treated prior to the application of the overlay. The original concrete was fully carbonated prior to this work and the steel is therefore still depassivated. The overlayer has restricted the supply of moisture and oxygen sufficiently to restrict corrosion, but when bars were exposed it could be seen that corrosion was occurring.

The average depth of carbonation measured in the overlay material was approximately 5 mm, and with a coefficient of sorptivity of 0.102 it can be expected that the underlying steel will continue to corrode, but at a slow rate.

The condition of the original balustrade sections is typically poor with a large percentage having lost cover concrete due to reinforcement corrosion, assisted in some cases by vandalism.

There is no evidence visually or from the sulphate analysis that the balustrade elements adjacent to the sewer vents have suffered from sulphate attack. This assessment was achieved by sampling from an area directly above the pipe which was displaying sulphate products and comparing this to the results obtained from core and spall samples taken from other sections of the structure. The sulphate content of the former was an order of magnitude greater than any of the values measured elsewhere. Due to the relative size of the hydrogen sulphide molecule compared to that of carbon dioxide it is unlikely that sulphate attack would be the catalyst for damage in this particular environment.

(b) Parapet Beams

Reinforced with a similar steel arrangement to the truss chord elements, the condition of the parapet beams is generally good, although the concrete is carbonated to a depth below the reinforcement.

Where deterioration does occur it generally takes the form of sheet spalling, due to the density of the helical wire ligatures. Deterioration has not yet progressed to the stage that ligatures have failed at any location.

(c) Lower Beams

The lower beams, at the base of the parapet, are generally in a poorer condition than the parapet beams. This is probably the result of a greater density of steel reducing the effectiveness of concrete placement and these elements taking longer to dry out. The corrosion process will therefore continue for a longer period after rain.

In some instances ligatures have corroded through but as yet there is no indication that the unconstrained main bars have bowed.

(d) Walkway Slab

The top surface of the slab is showing little sign of distress. Most irregularities are a result of intentional works conducted to cover some vent outlets. There are some cracks but none show any sign of recent movement.

The soffit is showing signs of advanced deterioration in some areas. The distribution of damage is random and it commonly takes the form of sheet spalling with exposed reinforcing bars exhibiting signs of relatively advanced corrosion. However there is no visual evidence of major section loss.

2.4 Pipe Bracket Supports

These elements describe the concrete components which provide support to the sewer pipe from the walkway and include the beams and associated propping supports which carry the pipe loadings to the lower chords of the trusses.

The condition of the elements varies considerably from those which have lost most of their cover and exhibit advanced stages of rebar corrosion to others which display no visible sign of deterioration. This variation of conditions is random and is most likely to be a product of low depths of cover rather than the result of aggressive local environments due to pipe leaks.

The depths of carbonation measured 55 mm on average, with a maximum of 108 mm and one reading through a repaired area) of 7 mm. These results indicate that all steel in the suspended collar brackets and the beams and prop sections has been depassivated. The only instance where the cover readings indicated that the steel was at a depth greater than 55 mm was on one 200 mm wide face. All measured covers on both adjacent 560 mm faces were less than 30 mm.

Where cracking of the pipe has allowed moisture to affect brackets the lowering of resistivities will be important in accelerating the rate of deterioration.

2.5 Pipe

It is not possible to properly assess the condition of this element while it is still operating. This is because the major deterioration mechanism likely to be operating is sulphate attack, and the invert of the pipe is where this will occur. The deterioration visible on the outside of the pipe comprises, in the main, horizontal cracks, typically 2 to 3 metres in length, but the precise cause of these cracks cannot be stated with certainty at this stage.

Leaks at some joints indicate that horizontal movement has caused damage to fixings and the expansion details have not continued to operate efficiently.

The micro environment present within the pipe will alter dramatically once it ceases to carry sewage. The threat of carbonation occurring will increase as the concrete dries and this will tend to put all the rebar in the pipe at risk. The pipe will probably go through a stage where visible damage does not propogate as the sulphate attack slows and the carbonation process starts to accelerate. This period will allow the formulation of a suitable restorative aproach and could well be extended by the early application of an anticarbonation coating.

2.6 Piles

The condition of the reinforced concrete piles which distribute loads from the pier footing to the bedrock was assessed to ensure that the long term performance of the material of the piles did not pose a threat to the structure. Piling was not required in the case of 3 piers as bedrock was encountered just below ground level. The testing included the measuring of the cement content and chloride levels, the taking of potential readings and the visual inspection of reinforcement from one pile from Pier 3 and one from Pier 8.

From construction records, the piles at Pier 3 are the longest and being some distance from the river may be susceptible to changes in the water level. With Pier 8 being close to the river, it was expected that the piles would be in an environment that was permanently moist.

The analysis indicated that the cement content is approximately 8%, which translates to there being only 200 kg of cement per cubic metre of concrete. As the mix was rated to achieve a design load of 14 MPa it is unlikely that the cement content was in excess of 250 kg/m^3. The test result does not indicate whether leaching poses a problem, but it is likely that some loss has occurred.

The chloride results reveal that the ground water is saline and that significant levels of salt have built up within the structure of the concrete. At Pier 8 the sample extracted from the pile indicated a constant level of approximately 2.5% by weight of cement to at least a depth of 65 mm. This result when combined with the electro potential readings taken on the pile below pier 3 where constant values of - 590 mV were recorded and those at pier 8 all of which were consistently - 880 mV, would suggest that a major problem existed. However, the visible condition of the bars at each location, the lack of electropotential gradients and the consistency of chloride results indicates that the concrete pores are saturated with moisture to such a degree that there is insufficient oxygen present to support the corrosion process and that there is no cause for concern over the durability of the piles.

3. STRUCTURAL ASSESSMENT

3.1 Superstructure

The strength of the bridge superstructure has deteriorated considerably due to spalling and the corrosion of the reinforcement. To assess the safety of the structure and to determine the reduction of reinforcement area that could be accepted in a rehabilitated structure, assessments of each truss member have been undertaken.

The members have all suffered some corrosion, and the cover to the reinforcement has spalled in many locations. Even in areas which appear to be sound, closer examination shows that the cover concrete has separated from the core of the member. The capacity of the members has therefore been assessed on the basis that the cover concrete is ineffective. Assessments have also been made to determine how much longitudinal reinforcement can be lost due to corrosion before the member capacity becomes inadequate. These assessments assume that the ligatures or helicies are repaired to conform to current design practice.

The cantilever trusses were analysed using the PC-based general frame program SPACEGASS using a three dimensional model of the structure, but ignoring the flexural strength of the pipe and walkway. The structure was analysed for dead loads and a pedestrian live load of 3 kPa, which is based on the loading requirements contained in Australian Standard 1170, part 1, but without any sewage loading in the pipe. Wind loads were not taken into account in the analysis as their effect on the trusses is small.

The calculated ultimate member loads are given in Tables 3.1 and 3.2 for the compression and tension members respectively. They are compared with the calculated ultimate capacities of each member assuming no cover concrete and that a certain proportion of the steel has corroded away. The calculation of member capacities assumed concrete compressive and steel yield strengths of 14.0 MPa and 250 MPa respectively. The buckling and ultimate capacities of the truss members were estimated based on details shown on the original working drawings of the aqueduct, and the lower of the two values was tabulated.

Tables 3.1 and 3.2 show that most members have a significant reserve of strength and can sustain a substantial loss of strength prior to failure. The critical members are the two vertical compression members closest to the tower (i.e. members 1-1' and 2-2' Fig 3.1), which only just satisfy current design standards in their "as-designed" condition.

The tabulated acceptable loss of reinforcement gives an indication of the work that may need to be done on individual members to restore the main reinforcement. For all members, particularly the compression members, the ligatures will need to be reinstated to conform with AS3600 (e.g. 6 mm ligatures at 180 mm spacing in the vertical compression members and at 330 mm spacing in the bottom chord compression members).

The effect of end moments in the members was considered but did not affect the results given above.

In examining the condition of the bridge, the continuing deterioration of the bottom chord of the south eastern truss at pier 14 was noted. The member has lost most of its cover concrete, the ligatures have corroded away and the longitudinal steel can be observed to have buckled away from the core concrete. It is reasonable to assume that none of the longitudinal steel is still effective, and Table 3.1 indicates that it is therefore 20% under strength for zero pipe loading, and worse for the current condition with the pipe carrying sewage. In its current condition the strength of the core concrete may be starting to deteriorate as well. The proposed repairs to this chord are urgent and should not be delayed.

Without the sewage loading the situation is still critical as corrosion damage will propagate. The concern is that due to the nature of the damage affecting the top and bottom chords there is likely to be structural failures within 10 years if no remedial work is undertaken. Pier 14 will still be an immediate concern without sewage loading.

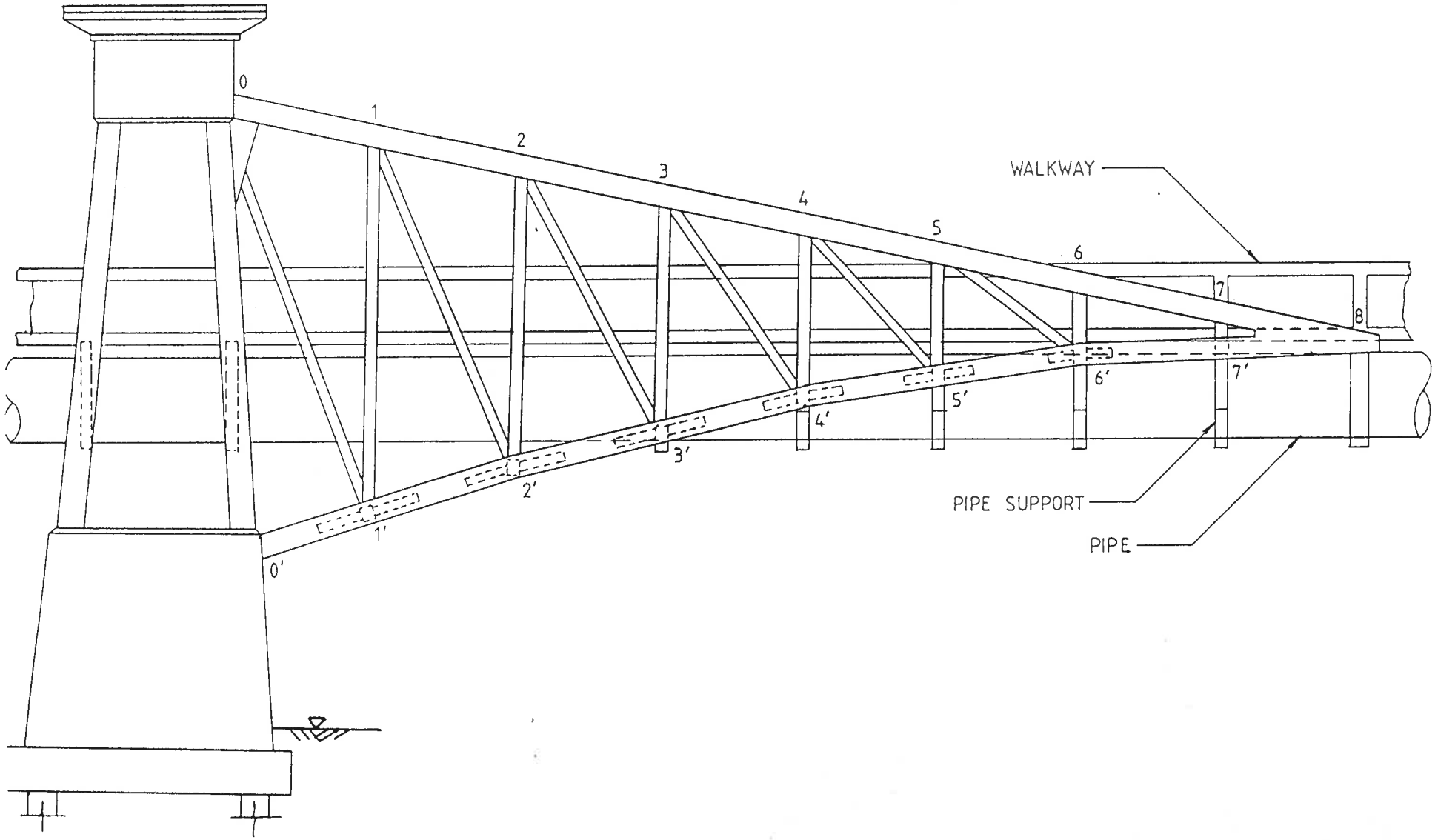


FIGURE 3·1
CANTILEVER TRUSS ARRANGEMENT

TABLE 3.1

OVOID SEWER AQUEDUCT
ULTIMATE COMPRESSION MEMBER LOADS AND CAPACITIES

MARKS	ULTIMATE AXIAL LOAD (kN)	MEMBER CAPACITY (kN) FOR % OF STEEL AREA				
		100%	80%	60%	40%	20%
9' - 1'	852	1031	952	874	796*	718
1' - 2'	809	948	875	804*	732	660
2' - 3'	763	822	759*	697	635	572
3' - 4'	721	782	722	663*	603	544
4' - 5'	682	712	657*	604	550	496
5' - 6'	634	688	635	583*	531	479
6' - 7'	577	462*	426	391	357	322
7' - 8'	457	440*	406	373	340	306
0 - 0'	423	1222	1121	1007	777	635
1 - 1'	84	91	64*	46	31	24
2 - 2'	99	104	73*	53	36	28
3 - 3'	60	124	88	63	42*	33
4 - 4'	71	186	169	151	134	116
5 - 5'	37	186	169	151	134	116
6 - 6'	22	186	169	151	134	116
7 - 7'	25	186	169	151	134	116

* Signifies Critical Bar Loss

TABLE 3.2

OVOID SEWER AQUEDUCT
ULTIMATE TENSION MEMBER LOADS AND CAPACITIES

MARKS	ULTIMATE AXIAL LOAD (kN)	MEMBER CAPACITY (kN) FOR % OF STEEL AREA				
		100%	80%	60%	40%	20%
0 - 1	791	1568	1254	941	627*	314
1 - 2	760	"	"	"	"*	"
2 - 3	722	"	"	"	"*	"
3 - 4	694	"	"	"	"*	"
4 - 5	647	"	"	"	"*	"
5 - 6	615	"	"	"	"	"*
6 - 7	593	"	"	"	"	"*
7 - 8	469	684	547	410*	274	137
0 - 1'	74	228	182	137	91	46*
1 - 2'	64	170	136	102	68	34*
2 - 3'	76	170	136	102	68*	34
3 - 4'	43	68	54	41*	27	14
4 - 5'	63	120	96	72	48*	24
5 - 6*	32	68	54	41	27*	14

* Signifies Critical Bar Loss

3.2 Walkway

The walkway now acts as a 'U' - section girder to span between the ends of the trusses, but between member 6-6' (towards the ends of each truss) the walkway spans between each panel point without using the parapet structurally. In this region the parapet has penetrations and in many areas has deteriorated very badly.

The floor slab of the walkway spans between the sides of the walkway structure in all areas. The soffit has spalled badly in many locations, but is adequate provided that just 21% of the original steel remains effective.

Where the walkway spans longitudinally between the individual panel points of the trusses, the edge beams carry the longitudinal moments but need only 23% of their steel to be effective in bending. The shear capacity of the beam is ample without relying on the ligatures.

Where the walkway spans longitudinally between the ends of the trusses, the whole "U" shape of the walkway contributes to the flexural strength. There is evidence that the sides of the walkway consist of an original parapet with penetrations, and a later cover of concrete to create a solid wall. The wall will nevertheless act as a solid member and has been checked as such. It has conservatively been checked as a simply supported structure between the ends of the trusses.

As a full 'U' section, only 15% of the longitudinal flexural steel is required to remain effective, and the shear capacity is ample. The lateral stiffness of the handrail is also ample for the flexural loadings.

3.3 Pipe Supports

Where the pipe is hung from the walkway the pipe support bracket consists of two hangers from the walkway parapet plus a beam under the walkway soffit.

The proportion of reinforcing steel that is required in the hangers to the walkway parapet is 38%, the proportion of the steel required in the beam under the walkway is 45% and the proportion of steel beside the pipe itself is 18%.

3.4 Walkway Supports

Where the pipe and walkway are supported off the truss the support system vary at each node as a result of the varying geometry and the varying loads.

The supports at the tower are satisfactory by inspection since they use struts directly off the pile cap. The supports at nodes 1 and 2 but particularly at node 1 appear, from the drawings, to be considerably under strength. They will need to be supported during repair and strengthened by the addition of external reinforcement bolted to the side of the top cross members near their bottom.

At nodes 1', 2' and 3'of the trusses the pipe and walkway are supported by crossbeams which run beneath the soffit of the walkway ("Top Beam") and also the soffit of the pipe ("bottom beam"). These members were analysed. The acceptable losses of steel section for each crossbeam are summarised in Table 3.3

TABLE 3.3

OVOID SEWER AQUEDUCT
PERMISSIBLE LOSSES OF STEEL IN WALKWAY AND PIPE SUPPORT MEMBERS

NODE DESIGNATION	PERMISSIBLE LOSSES OF STEEL SECTION TOP BEAM	BOTTOM BEAM
1	0% (under capacity)	0% (under capacity)
2	0% (under capacity)	45% (pipe empty) 20% (pipe full)
3	0% (under capacity)	25% (pipe empty) 0% adequate (pipe full)
4	0% adequate (pipe empty)	N/A
5	10% (pipe empty)	N/A
6	0% (under capacity)	N/A
7	0% (under capacity)	N/A
8	0% (under capacity)	N/A

3.5 Foundation

The piled foundation for the tower structure consists of a 6096 x 4877 x 762 mm pile cap which rests on a group of 8 piles. The piles are precast octogonal and are of dimension 457 mm across the section. All piles were driven to rock, some 1 to 8 metres below the ground level. The piles are working to approximately 22% of their ultimate load capacity. The pile/soil capacities are unknown due to lack of geotechnical information, however some of the trusses have settled as evidenced by rotation of the trusses from each other. The pile settlement to date (approximately 0.6 mm) is currently acceptable and is likely to remain acceptable provided the expansion joints which are in compression are not relieved.

3.6 Articulation and Expansion Joints

The bridge articulation consists of an expansion joint arrangement at the northern end of each truss. The central tower of each truss pair is assumed to be rigid and the thermal movements in each truss are accommodated by the joint.

The joint arrangement is shown on figure 3.3 and consists of three pairs of bearings between the walkway and the truss plus a joint in the pipe.

The system has been affected by the settlement of some of the piers and the rotation of some of the towers and trusses, with the result that some expansion joints are not moving. In these cases the thermal movements must be taken up by a slight rotation of the towers, but there is no sign that this effect is causing any distress and it is not considered appropriate to attempt to alter the structure's current behaviour. To ensure that no local damage occurs in the walkway structure at the expansion joint locations, it is proposed that small elastomeric pads be located at the base of the walkway to carry any longitudinal forces between the trusses. (See Fig. 3.2 below.)

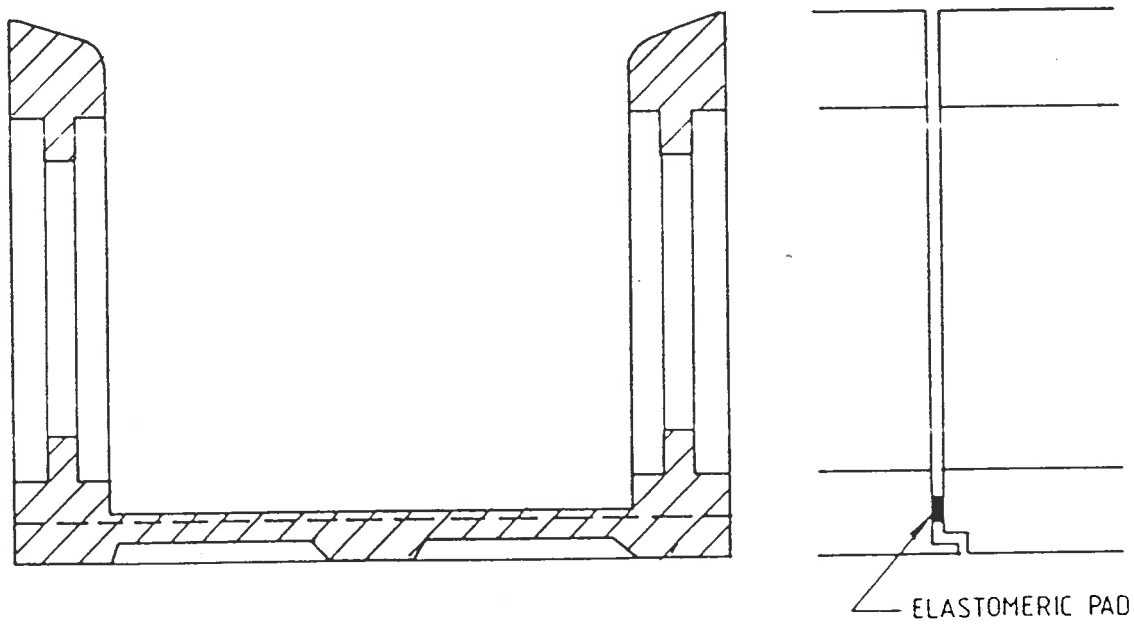


FIG. 3.2 CROSS SECTION OF WALKWAY SIDE ELEVATION

WALKWAY ELASTOMERIC PAD DETAIL

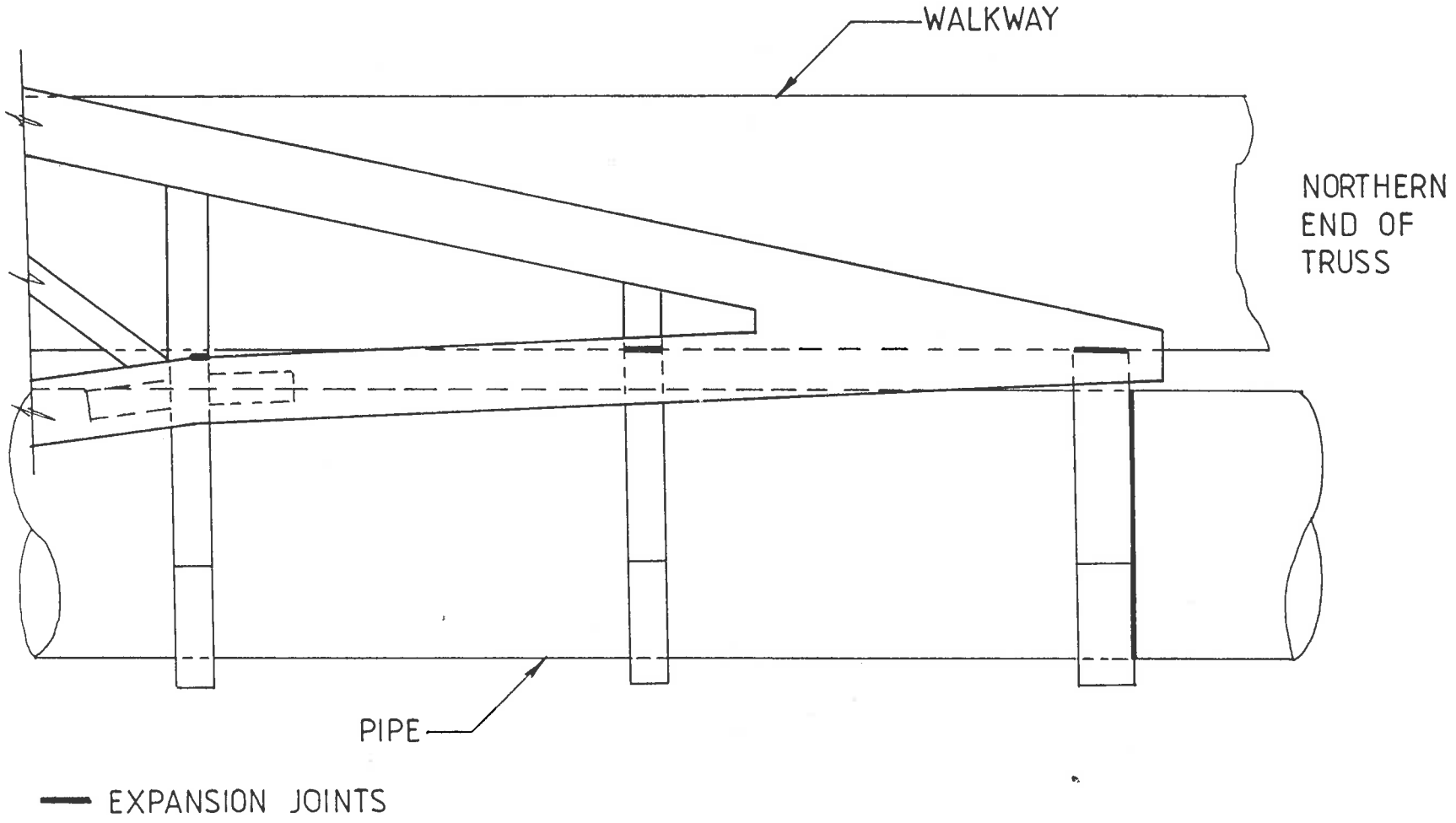


FIGURE 3·3
DIAGRAM OF EXPANSION JOINTS

The expansion joints supporting the walkway consist of pairs of greased steel plates, and these have worked quite well over the years. The steel plates have not corroded to any significant extent, and most of them are adequately greased. To improve their behaviour and to reduce the maintenance requirement, they should be modified by inserting a "sandwich" of stainless steel and teflon surfaced elastomer between the steel planks. These sandwiches would be only about 5 mm in total thickness and would dramatically reduce the friction of the bearings.

Care would be needed in their installation which would involve the following steps:

- Clean the expansion joint in the pipe, providing some clearance
- Jack the walkway off its supports taking care not to damage the pipe
- Clean the steel plates of grease and coat the plates to reduce corrosion
- Insert the proposed stainless steel/teflon/elastomer "sandwiches"
- Fix the sandwiches in position
- Lower the walkway onto the new bearings

4. CURRENT CONDITIONS AND PROGNOSIS

4.1 Cantilever Trusses

Each element of each truss was visually inspected and given a condition rating based on visual defects and drummy soundings. At the same time the extent of such defects as an approximate percentage of an element's total surface area was also noted. The information for each of the 14 piers is presented in Appendix A.

The variation in damage appears to be random and the likely pattern of future damage cannot be predicted because the picture is further distorted by the application of different repairs to different elements at different times.

For example, from a quick visual inspection, the trusses at Piers 1 and 2 appear to be in quite good condition following the application of a shotcrete overlay in the 1950's. However on closer inspection the truss elements at Pier 2, particularly the main chords, are showing signs of damage. This results in a repair cost estimate approximately four times greater for this part of the structure than for Pier 1 where damage is much less. The environment affecting both piers is the same, therefore such a discrepancy must be due to variations in material quality and similarly the tension members, which were the focus of major repair works in the 1950's, generally show up as low cost maintenance items. With such variations it is difficult to formulate a repair works programme except on a member by member basis. What is necessary is to give priority to critical members, which include those with low factors of safety and those which will be extremely expensive to repair if extensive scaffolding is required or structural propping becomes necessary.

Based on the above parameters, for all trusses the key items to repair and maintain in good condition are the top and bottom chords and the compression struts 1 - 1' and 2 - 2'. All other truss members have reasonable capacity and do not require elaborate access. Therefore, a high level of deterioration could be allowed to occur without compromising the structural adequacy of the structure, nor the economies of future repairs.

The site data combined with the structural analysis results allow cost regimes to be developed for the different repair scenarios. These costings are presented in Section 5 based on the conclusions drawn in this section.

The repair method used in the costing is a standard full patch repair which involves breaking out all damaged concrete and exposing corrosion-affected reinforcing steel, cleaning, or replacing that steel, and reinstating the concrete profile with a cementitious material and the application of an approved coating to the entire structure. To effectively clean the steel it may be necessary to remove concrete to at least 25 mm behind the bars. This reduction in the concrete cross section may require temporary propping during breakout or restrict the total area that can be broken out at any particular time. Some allowance for such work has been included in the estimates.

The replacement of main bars can be conducted at the same time as the full patch repair exercise, and by carefully controlling the sequence of removing ligatures and the main bars, their replacement and the reinstatement of concrete cover may not require temporary propping.

Due to the depth to which the concrete has been affected, the extent of the concrete removal required is such that the patch repair method will prove to be expensive. However there are few alternatives which will produce a fully restored structure, so the costings of Section 5 have been based on this approach. It must also be appreciated that although this should fully restore the aqueduct, further damage will occur as areas of the structure which currently show no evidence of damage continue to deteriorate.

A technique that has not yet been used commercially in Australia, but has beeen used in Europe is an electrochemical repair method known as realkalinisation. The basis of the method is that by applying a direct electrical current to the reinforcing steel and a temporary steel surface mesh the alkalinity of the concrete, reduced by the carbonation process, is restored to a level where the steel is repassivated and effectively protected by the concrete. The main physical requirements are that any damaged concrete is repaired and all the steel within any given member is electrically continuous. The type of repairs required are much less rigorous than those proposed for the full break-out approach as the main objective is to eliminate direct current paths from the surface down to the steel reinforcement.

It is estimated that the necessary patch repairs would cost approximately one fifth of the amount calculated for the full break-out method. This reduction is mainly due to three factors:- the total surface area of damaged concrete being treated will be less as it is not necessary to expose steel reinforcement which is corroding beneath undamaged concrete; the depth of breakout will be considerably less as there is no requirement to go beyond the ligatures, and the quality of the materials used to reinstate the concrete cover profile can be lower than those used for an effective patch repair. The realkalinisation is costed at $\$100/m^2$ as a treatment.

Due to the lack of experience in Australia with this technique, it is considered essential that trials be conducted to prove the techniques viability and to establish more accurate repair cost estimates.

Based on the costings shown in Section 5 the realkalinization technique is likely to cost $300,000 - $400,000 less than the full patch repair method. This cost should not be taken as a direct comparison as the realkalinization will prevent damage propagation in areas which the full patch repair will not. The latter will require additional on-going expense over the following ten years which should be included in any cost comparison study. The estimates of the ongoing patch repair costs are included in Section 5.

Rather than restoring all elements in an initial, large project another approach would be to make the aqueduct structurally safe and free from the likelihood of spalling concrete. Repair works could then be planned to be completed over a 10 year period. The work could be staged with members to be repaired being selected on their structural capacity, rated condition and extent of damage.

This approach would require the implementation of a maintenance management system which would require regular surveys to be conducted, to ensure that the information it contained was up-to-date and relevant. This would require additional expenditure but would allow accurate forecasts of future budgetary requirements to be made. It would also act as a repository for information on the aqueduct including such information as what type of repairs were conducted, by whom, when, and what materials were used.

The staging of the works would result in higher unit rate costs for the repair work since mobilisation and site establishment costs would be incurred repetitively and by defining limited areas of work the contractor would be able to organise a works programme to take advantage of the economics of scale. Contract administration and site supervision will cumulatively cost more if parts of the works are deferred than if the repairs were conducted within the one contract.

4.2 Tower Structure

The capping details on most piers have suffered significant deterioration, particularly the soffit areas. Although not of structural concern these are of high priority in terms of public safety.

The repair of these areas poses few technical problems as access can be constructed up from the walkway structure. To make these elements safe, by removing any loose concrete, will require high pressure water jetting or use of light mechanical breakers. As deterioration is quite advanced it is recommended that these be repaired within the next 7 years. The area behind the parapet should be kept free of debris and plants and regularly checked to ensure that drainage is adequate. Where drainage pipes have deteriorated, these should be replaced to make sure that the water is discharged effectively to prevent water from collecting inside the towers and causing corrosion to accelerate.

The pier bases are generally in good condition and do not require extensive work to fully restore them. If works are deferred they should be inspected every three years, however it is expected that repairs need not be instigated within the next six years but should be considered thereafter.

4.3 Walkway

The walkway was assessed as four separate components during the visual survey and in the costing exercise to simplify the estimation process.

The structural capacity of the walkway is such that there are no areas at present that require immediate attention. However, the soffit of the walkway slab and the adjacent beams have deteriorated badly over significant areas. The parapet beams and balustrade walls also display areas of advanced deterioration but these are not as extensive and are easier to access than the other two elements.

It is recommended that if repairs are to be staged then the walkway should not be included in the first package of work. It is a high cost item and once the important structural elements have been addressed then the entire walkway should be refurbished. This scenario would be dependent on whether public use of the bridge was considered, and if aesthetic requirements had to be taken in to account. If the work was scheduled purely on durability requirements then the repairs should be conducted after approximately 5 years. An inspection should be carried out after 3 years to review this prediction which may result in a further delay of the works or a change of scope. The latter could be caused by acceleration in the rate of deterioration of those areas currently rated as being in a poor condition and a lack of any further progress in the deterioration of other areas currently rated as good.

However, it would be preferable if possible to restore the entire walkway structure as a single project as cost savings in the access component would be substantial.

4.4 Pipe Support

The support of the pipe where it is hung from the walkway can be addressed at any stage since temporary steel brackets can be introduced to provide support to the pipe.

However, if public access beneath the pipe is expected then the possibility of concrete spalling should be eliminated. As with the tower capping details it would be prudent to require the removal of all loose concrete by water jetting, or mechanical means, during the first phase of repairs. This will allow a better assessment of the actual condition of each bracket to be made and highlight where steel brackets should be installed.

Without the load of the sewage the support requirements for the pipe itself will only result in the installation of steel brackets where substantial loss of steel reinforcement is identified. In the case of the walkway and pipe supports the requirement for the structural strengthening proposed in section 3.3 will necessitate the installation of temporary reinforcing, or propping, during repairs with the additional, permanent reinforcement being installed as part of the remedial process.

4.5 Pipe

The repair of the pipe will be of low priority once it is no longer carrying sewage and the threat of spillage no longer applies.

It is therefore considered that maximum cost benefit will be gained from coating the pipe internally and externally, as early as possible in the restoration process. This will necessitate the cleaning of all surfaces, but need not involve the repairing of cracks.

Without the sewage the deterioration that has resulted in the formation of cracks is expected to slow markedly. This and the application of the coating should ensure that the condition of the pipe will be stabilised allowing repairs to be programmed prior to the first recoat which would occur after approximately 10 years.

4.6 Piles

No requirement for repairs are envisaged on these elements. It is recommended that a visual inspection be conducted on two of the piles of any three piers every 8 years to ensure their continued satisfactory performance.

6. SUMMARY

The aqueduct is a unique 75 year old concrete structure. It has managed to perform a task for the community well in excess of that for which it was designed. However historical records show that it has been susceptible to carbonation attack and has required regular maintenance throughout its life.

The main initial flaw as far as durability is concerned is that the structural efficiency of the design required a concrete with a compressive strength of only 14 MPa. As a guide, a 20 MPa concrete will generally have poor durability characteristics, 32 MPa will have acceptable characteristics and 45 MPa will perform well in most environments. The first indications of problems occurred in the 1920's when quite extensive remedial work was conducted. The next phase of major repairs were performed in the early 1950's and this has been followed by regular low scale repairs to the present day.

The test techniques used in this study have only been developed and widely used in the investigation of concrete deterioration in the last 10 to 15 years. Whilst the nature of carbonation attack has been known for some time the understanding of the corrosion of reinforcing steel embedded in concrete has progressed significantly in the last 20 years. This has resulted in a corresponding improvement in repair materials, remedial techniques and protective coatings during that time. It is important to note that despite advances in knowledge similar deterioration is not uncommon on structures only 15 to 20 years old.

Due to the greater awareness of the high future costs associated with corrosion induced damage greater resources are being committed to maintenance by private and public bodies. However there is a stage where the economic predictions indicate that partial, or full replacement of an asset is more cost effective than escalating maintenance budgets. The aqueduct has reached that stage and due to the inability of the pipe to handle future flow rate requirements a replacement solution was necessary.

The current state of the structure even without the load of the sewage requires some immediate repairs to be conducted on members that are considered to be critical to ensure that structural failures will not occur. The current condition of the bottom chords for Pier 14 and the similarity of the damage appearing on other trusses makes it likely that a structural failure will occur within 10 years. An additional problem which must also be addressed by the owner of the structure is the issue of public safety, not just from a structural point of view but also with regard to possibility of a concrete fragment falling from the structure and causing injury.

It was based on these parameters that the repair options in section 5 were formulated, and the resulting estimated costs are presented in Table 5.1 and summarised below.

The costing options indicate that to fully restore the entire structure would require approximately $5.8 million to be spent on an initial contract and a further $2.2 million on additional maintenance over a 30 year period. Total demolition of structure is estimated to cost $540,000.

If it is decided that only part of the structure will be retained then the corresponding estimated costs are:

	Initial Repair Contract	Demolition	Maintenance
For Piers 7-14 -	~$4 million	~$250,000	~$1.6 million
For Piers 9-12 -	~$2.6 million	~$340,000	~$1.0 million
For Piers 1 & 2 -	~$1.3 million	~$470,000	~$600,000

The deferring of the work does not cause a dramatic escalation in the cost of the work, typically ~ $1,000,000 for the entire structure down to approximately $400,000 for Piers 1 and 2 over the 30 year period.

The benefits cannot be fully ascertained until a method of funding is established and the cost effectiveness of investing against expenditure can be estimated. However the flexibility for scheduling future work will be enhanced by the staged approach.

All costings were based on the patch repair method as it is considered the only viable approach which has been used on a commercial scale in this country. Variations within that approach are numerous, e.g. different cementitious products, coating types, application methods, etc. and will produce less expensive, and more expensive, budgets than those presented here. However it is unlikely that such differences would be significant within the terms of an estimating exercise, as was shown with the indicative costing of the realkalinisation repair where the overall saving is likely to be less than 10% of the predicted patch repair cost. Such a figure is not considered significant enough to warrant the continuation of the estimation exercise for all the other restoration options.

APPENDIX B

MARCH 1990 TAYWOOD-MAUNSELL REPORT

GEELONG AND DISTRICT WATER BOARD

JOB NO 26190

CONDITION ASSESSMENT
of
BARWON RIVER OVOID SEWER AQUEDUCT

by

TAYWOOD-MAUNSELL
Remedial Engineers and
Construction Materials Technologists

Taywood Engineering Limited and Maunsell & Partners Pty Ltd
In Association

MARCH 1990

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FIGURES 2.1 - 2.9 AND 3.1

GLOSSARY OF TERMS

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1.0 INTRODUCTION

1.1 Objective

This report provides an assessment of the current condition of the ovoid sewer aqueduct across the Barwon River in Geelong, and details various repair options to keep the aqueduct in service. The foundations were not investigated and the recommendations of this study are dependent on the substructure being sound.

Representative spans of the superstructure were examined to assess the degree of corrosion in a detailed investigation which included visual inspection, sampling and testing. Structural analyses were carried out to determine the structural adequacy of the main structural members of the trusses. This report also makes recommendations in regard to future maintenance options.

1.2 Summary

The major findings of this report are that the concrete in the structure has been carbonated and is no longer protecting the reinforcing steel. It must be assumed that all the reinforcement is depassivated (and is potentially corrosively active). Even most of the repaired sections of the trusses are continuing to corrode, albeit slowly, because the carbonated concrete has not been removed.

Areas which show little or no sign of damage at present will deteriorate and require remedial work at some stage. It is likely that every existing original member will need to be repaired within 25 years. Of that number approximately 25% will have to be treated in the next 5 years. Some members have deteriorated to a stage where there is already structural concern and those must be given a priority, over and above any general maintenance and repair programme.

A major concern is the damage that has recently occurred to the lower chord of the truss near Tower 14. At this location the structure currently has very little reserve of strength against failure. It is anticipated that serious damage will occur at other locations in the future, and the possibility of collapse cannot be entirely ruled out. It would be advisable to conduct detailed surveys of every top and bottom chord beams to quantify the extent of 'hidden' deterioration. This was not possible within the time frame of this study. Regular inspection and repairs are essential.

The foundations of the structure have also moved, and although there is no need for immediate repair work, major future works may become necessary.

To maintain the superstructure of the aqueduct, the following budget cost estimates have been prepared, based on the length of time for which the Board wishes to use the structure. These costs are:

3 year life:			$500,000
3-10 year life:	Option 1 - Yearly Maintenance		$2,500,000
	Option 2 - Realkalinization		$2,200,000
	Option 3 - Major Patch Repair		$2,600,000
Long term repair:	Structure	$4,000,000	
(Minimal further	Pipe	$800,000	
repairs for 15-20 years)	TOTAL		$4,800,000

The above costs are indicative, and as with any repair work, the final costs could vary considerably. Any costs for repairs to the foundation need to be added to the above.

2.0 CONDITION ASSESSMENT

2.1 General

This condition assessment quantifies the state of the structure, primarily by investigating the remaining ability of the concrete to prevent corrosion of the reinforcement. It then estimates the extent of repair work that will be necessary in the long term.

The structure has deteriorated to a significant extent and there are large areas of cracked and spalled concrete. This is due to corrosion of the steel reinforcement, caused in turn by the fact that the concrete is no longer protecting and passivating the steel. The Board has been repairing the worst areas of cracking and spalling for many years, but the extent of the problem has been steadily growing.

The affected areas include the truss members, the tower tops, the pipe supports, the walkway and the pipe itself, but to date the Board's repairs have centred on the truss members, with only minor work on the other areas. The extent of repairs will therefore to increase as the other areas deteriorate and demand increased attention.

The effect that the observed deterioration may be having on the adequacy of the structure is discussed in Section 3, which also discusses the visual evidence of foundation movements.

2.2 Corrosion Assessment

The aqueduct consists of 14 towers, each with a pair of trusses, numbered from north to south, with the walkway acting as a structural beam between the trusses. The sewer pipe is supported under the walkway, either from the walkway itself or directly from the trusses. The trusses show varying degrees of corrosion and have been repaired to differing extents and by a variety of methods. To give an overall view of the condition of the aqueduct, the trusses at towers 2, 3, and 14 were chosen for detailed inspection and testing after a general visual inspection of the whole aqueduct had been conducted.

The trusses at towers 2 and 14 were selected because at tower 2 the trusses are in a reasonable condition, while at tower 14 they are in a particularly poor state of repair. The trusses at tower 3 are in reasonable condition, and contain areas of both early shotcrete repair and examples of more recent repair types. The particular test areas were chosen becuase they had little, or no, visible damage, so that the results assist in defining the extent of 'hidden' deterioration.

2.2.1 Visual Assessment of Selected Towers

The condition of each member of the trusses at towers 2, 3 and 14 was visually assessed and is given in tabular form in Figures 2.1 to 2.3. The assessments quantify the extent of the visible damage which can be seen to be considerable, both the members which have been repaired, and in those which have not yet been repaired. The similarity in the general level of damage found on all trusses indicates that the major deterioration on one of the bottom chords of the trusses at tower 14 may well affect similar members on other towers in the near future.

2.2.2 In Situ Testing of Selected Members

The assessment of the cause of damage, the probable extent of non-visible damage and the effectiveness of the different repairs was made by testing selected areas of members in situ and extracting the necessary samples for laboratory analysis.

The site tests performed were:-

(i)	Cover	Measurements to check the depth of concrete covering the steel bars.
(ii)	Depth of Carbonation	Measurement of the extent of diffusion of CO_2 into the porewater. The CO_2 reduces the alkalinity of the concrete and depassivates the steel.
(iii)	Potential Mappings	Use of electrochemical potential surface readings to indicate the possible areas of corroding steel reinforcement without having to break out concrete.

(iv) Resistivity Measurements — Measurements which indicate the degree to which the properties of the concrete will suppress the rate of corrosion.

Nine elements were tested, four from the trusses at tower 2, four from the trusses at tower 14 and one member from tower 3. The latter had recent repairs carried out to its upper half but not the lower. Both halves appeared to be in good condition.

(i) Cover Measurements

The values presented in Table 2.1 show that the concrete cover to the steel reinforcement was uniformly good and that despite the relative slenderness of the members, the cover that was achieved in the original construction would have been acceptable practice until the introduction of more recent codes. Some areas of low cover were noted during the general visual examination, but these did not occur on members chosen for testing.

Table 2.1
Reinforcement Cover Readings

	By Covermeter				By Tape Measure			
	No of Read	Min (mm)	Max (mm)	Av (mm)	No of Read	Min (mm)	Max (mm)	Av (mm)
Large Members								
Main Bars	16	20	89*	48	9	30	55	46
Ligatures	16	15	56*	42	5	20	45	32
Slender Members								
Main Bars	10	22	45	33	4	18	44	28
Ligatures	20	12	44	27	4	11	39	23

* Value includes ~ 25 mm of shotcrete.

The additional cover given to repair areas has been achieved at the expense of the aesthetic appearance of the structure. Many of the areas which have been repaired are likely to be those where the original concrete cover was low.

The importance of cover can be gauged from the effect that a nominal increase has on the rate of penetration of the carbonation front.

(ii) Depth of Carbonation**

Since the early 1980s it has been recognized that carbonation is the prime mechanism initiating corrosion on this structure. Based on the findings of this survey, the current typical depth of carbonation for the original concrete is approximately 55 mm, although the measured depths varied from 45 mm to 65 mm (See Table 2.2).

An accepted expression for determining the depth of carbonation is Klopper's equation where the depth of carbonation $x = \sqrt{2Dt}$.

In this equation t is the time of exposure in years, and D is the Carbonation coefficient (in mm^2/year). D is considered to be a constant for any given concrete mix. Using a 55 mm depth for carbonation, D for the original concrete is 20.2.

Given that the typical cover achieved was approximately 25 mm on slender members and 35 mm on others, then the time to activation would be 15 years and 30 years respectively. The quality of the concrete would indicate that widespread damage was likely to become visible approximately 5-10 years after activation.

Based on these times damage would have become apparent in the early 1940s (after 25 years)and have been widespread by the late 1940s (after 30 years). Damage in some of the main members might not have been visible until the late 1950s or later (after 40 years). These dates agree well with the observations which record visible damage from the mid 1940s, and extensive damage since the early 1950s.

** Refer Glossary for definition

The measured depths of carbonation combined with the findings of the cover survey mean that when predicting the extent of repairs it must be assumed that all the original reinforcing steel is lying in a zone of carbonated concrete, has been depassivated, and is potentially corrosively active. The only exception is where subsequent repairs have involved breaking out of the concrete to behind the steel, effective cleaning of the steel and, reinstating the concrete cover. Partial breakouts and render overlays will not have resulted in repassivation.

Table 2.2
Depth of Carbonation Readings

	No of Readings	Minimum (mm)	Maximum (mm)	Average (mm)
Original Concrete	7	45	65	55
Repair Material	6	3	12	6

Carbonation measurements on the shotcrete repairs conducted on the trusses at tower 2 revealed that in the area tested, the material had been applied to a depth of 25 mm (1") over the original member. Although the shotcrete had only carbonated to a depth of approximately 3 mm in 35 years, the reinforcing steel was lying in the original, still carbonated, concrete. The repairs have relied on the ability of the shotcrete material to prevent the ingress of moisture and stifle corrosion. The shotcrete has provided protection in the manner of a coating rather than of a patch repair. Where the protection has broken down rapid deterioration is occuring since the cycle of damage does not involve the depassivation stage.

More recent repairs (thought to be from the mid 1970s) have tended to carbonate at a higher rate than the shotcrete, typically 10 mm in 15 years. This gives a coefficient of carbonation, D = 3.33, an order of magnitude better than the original material.

The most recent repairs have not been exposed to the environment for long enough to allow a coefficient of carbonation to be determined with any degree of certainty. A value of around 3 could be expected from such material.

(iii) Potential Mapping

This is a technique where corrosion activity can be detected at the surface through the change of the potential reading when measured against a stable electrochemical half cell. For this survey a silver/silver chloride reference electrode was used.

For corrosion which is associated with carbonation of the concrete (as in this case), the steel is activated when the potential (as measured above) is between -290 mV and -570 mV. If there are chloride ions in the concrete, corrison may start at less negative potentials.

Readings taken over a particular surface area are not sufficient on their own to determine with any confidence the state of the underlying reinforcement. However by conducting selective breakouts and inspecting sections of bars, criteria can be developed for the particular structure that allow the prediction of corrosion areas.

The potential results shown in Figures 2.4 to 2.8 indicate that for each particular member, except where a repaired area was also mapped (Figure 2.8), the readings tended not to vary significantly. As each breakout indicated some corrosion activity then it is probable that all the steel in each region tested has been depassivated and is corroding.

The top half of compression member 3-3′ on the north-western truss at tower 3, has been repaired recently. The area chosen was not showing visible signs of distress, however potential readings on the original surface indicate that corrosion is affecting the steel in that region, and visual inspection confirmed that corrosion was reasonably advanced and liable to cause damage in the near future. This would not only affect the old concrete but the repair is also likely to fail, since the interface of the old and new concrete will provide a localised condition under which the corrosion process will accelerate.

(iv) Resistivity

When corrosion has been initiated, the rate at which it proceeds will largely depend on the quality of the concrete. Not only will a more impermeable concrete increase the time to activation, but it will restrict the corrosion rate and in some cases, even though a bar has become depassivated a particularly dense concrete can suppress the corrosion process completely.

Concrete with a cylinder strength of 14 MPa, as used in the construction of the aqueduct, has been shown in laboratory tests and on surveys of other structures, to be unable to impede corrosion, particularly as it is suspected that salt affected aggregates may have been used in the original concrete mix. Results achieved gave typical values of between 6000 and 7000 Ω .cm when dry. This is in a range which will support a high rate of corrosion (see Table 2.3 below) and is likely to shift into the very high range for a period of time following rainfall, particularly if (as is suspected) salt affected aggregates were used in the original concrete mix.

Table 2.3
Resistivity Criteria

Resistivity Ω . cm	Corrosion Rate
< 5,000	Very High
5,000 - 10,000	High
10,000 - 20,000	Low
<20,000	Negligible

2.3 Laboratory Analyses

2.3.1 Chloride Analysis

Steel embedded in concrete is normally prevented from corroding by virtue of the highly alkaline nature of the concrete. Concrete normally has a pH of between 12.5 and 13.5 which leads to the formation of a passive film of oxides on the steel surface, which prevents further corrosion occurring. This protection can be broken down by the penetration of the passive layer by chloride ions. The penetration of chloride ions can therefore lead to corrosion even in conditions of high pH.

Chloride analysis was conducted on samples to investigate whether chloride ions were present in sufficient quantities to affect the deterioration process.

The results given in Figure 2.9 show that all but one sample contained quantities of chloride below the accepted threshold levels required to depassivate the reinforcing bars. The one high result from member O-O′ on the north-eastern truss at tower 2 was a sample of shotcrete again material and the figure is consistent with calcium chloride having been added to the mix to promote setting times. The chlorides in this instance have not diffused into the original concrete, confirming that the shotcrete used was a good quality material.

There is no evidence from these results that chlorides were incorporated in the original mix either through the water used, salt affected aggregates or an additive.

Chloride is therefore not a problem in the concrete of the aqueduct.

2.3.2 Sulphate Analysis

The probability that microbiological activity within the sewage pipe is producing H_2S gas raises the possibility that localised acid attack may be occuring. H_2S itself is not harmful, however it forms sulphuric acid when dissolved in water. Acid attack is most likely to occur on the inside of a sewer, above the waterline of the flowing sewage, but it was considered prudent in this instance to check whether truss members had been affected.

Limited analysis was conducted to assess whether harmful gas generated by the serer was affecting the concrete members of the trusses.

The sulphate quantities in the samples were consistent with typical levels measured in unaffected concrete. It appears that only those members which are adjacent to the pipe, or are close to the sewer vents could be affected by the gas, but the visual evidence suggests that even here the governing mechanism of deterioration is carbonation. The pipe support elements are also affected by the occasional leaks which tend to increase the corrosion rate by reducing the resistivity of the concrete, but they do not show visual evidence of acid attack.

2.4 Discussion

2.4.1 General

Site analysis and chemical testing confirm that carbonation is the major deterioration mechanism and at present the typical depth of carbonation for the structure is 55 mm.

It must be assumed that all the reinforcing steel in the original structure has been depassivated and this dictates the extent of the repair works necessary to maintain the structure in a serviceable condition.

Areas which show little or no sign of damage at present will deteriorate and require remedial work at some stage. It is likely that every existing original member will need to be repaired within 25 years. Of that number approximately 25% will have to be treated in the next 5 years. Some members have deteriorated to a stage where there is already structural concern and those must be given a priority, over and above any general maintenance and repair programme.

To date the use of visual assessments to determine repair priorities has meant that the more slender members, compression and tension, are more likely to be the target for repairs as visual damage has tended to appear more rapidly. However our investigations show that the larger main members can suffer major reinforcement corrosion before substantial damage is apparent on the surface.

Visual inspection of the trusses at tower 14 indicates that the closely spaced, spiral wire has produced internal cracking along the plane of the secondary reinforcement. The relative narrow gauge of the wire and the ductility of the concrete apparently produce a situation where the corrosion product from the spiral wires does not produce cracks extending to the surface. It is not until the main bars begin to corrode that damage becomes visible. At this stage the thin ligatures have had their cross section reduced to such an extent that the expansion of the main bars causes them to fracture, and results in sheet spalling of the concrete cover and sections of ligatures. This leaves lengths of the main bars without confining reinforcement. In the case of tower 14, the first significant visual evidence of deterioration became apparent only after internal damage had proceeded to an extent which had immediate and serious structural implications.

The repairs conducted to date have achieved varying degrees of success but based on the documented costs, the Geelong and District Water Board has achieved good value for money in maintaining the structure for the past 40 years. However it has been based on visible damage control rather than progressive restorative repairs, and such an approach is likely to result in a stage being reached where structural integrity can only be ensured by substantially increasing the scope of the repair works.

The damage to the lower chord of the south-eastern truss at tower 14 is indicative of the type of damage which can be expected to become more prevalent in the next 5-10 years. The deterioration in that instance has proceeded to a point where propping is required to prevent the possible failure of vital members. Having to resort to such techniques will produce substantial increases in the cost of maintaining the structure.

The walkway is an integral part of the structure and deterioration to its soffit and beams is widespread. It provides the support for the pipe between the truss cantilevers and assists in distributing the pipe loads to the truss members within the truss cantilevers. Any requirement to keep the pipe in operation for a period of more than 5 years will necessitate extensive refurbishment of the walkway components.

Similar problems exist with the pipe support details. Approximately 40% of these are showing signs of distress due to corrosion. Steel bands, attached to the top horizontal beams of the walkway, have been used in some instances where severe deterioration is visible. Similar fixing systems will have to eventually replace all the concrete supports as they can not be relied upon in the long term, unless effective repairs are conducted.

The pipe itself can be kept in service as long as it is adequately supported, but the level of that service will have to be restricted as the external plugging of cracks and leaks is an unsatisfactory repair practice. Despite improvements in repair materials there is little possibility of finding a system which will provide anything more than a short term solution. Although plugging of leaks and cracks appears economically attractive in the short term, in the long term this approach will prove to be very costly.

If the pipe is to be retained in use for an extended period of time of over 5 years, relining may well be an economic solution.

Possible options are to insert a plastic sleeve or to apply a suitable coating to the internal walls of the existing pipe. The hydraulic requirements of the Board would be the main criteria in assessing possible systems.

2.4.2 Propping Requirements

The condition of the south-eastern bottom chord of tower 14 is critical. Propping should be conducted as soon as possible to remove any possibility of failure (see Section 3.1 below). As discussed previously, the mode of corrosion damage within this member makes any assessment of time to failure very difficult. Further deterioration could develop at any time.

In the following sections on cost estimates, it is envisaged that no other bottom or top chord members will be allowed to deteriorate to the stage that it requires propping. If the more detailed survey recommended earlier in this discussion reveals that several towers have members in the same condition as tower 14, then the possibility of successfully refurbishing the structure economically will be markedly reduced.

3.0 STRUCTURAL ASSESSMENT

The aqueduct structure is suffering from extensive deterioration to various components, and its structural capacity is greatly affected.

The most important components of the aqueduct are the cantilever trusses themselves, since any failure of their individual members would cause a sudden collapse of a significant length of the sewer. Foundation failure is likely to be more gradual, so that remedial action can be undertaken. Local pipe support failures would affect only a limited length of the sewer.

Structural assessment has therefore concentrated on the trusses and only brief consideration has been given to other structural components.

3.1 The Cantilever Trusses

The cantilever trusses were analysed by the computer program 'Space-gass' for a fully-laden pipe. Wind loads were not taken into account in the analyses as their effect on the trusses is small. The resulting loads in the truss members are summarised in Table 3.1 below.

The ultimate capacities of the truss members were estimated based on details shown in the original working drawings of the aqueduct. Due to the lack of information about the concrete used, a concrete characteristic strength of 14 MPa was assumed. This strength is compatible with the typical 1:2:4 concrete used generally in the period when the aqueduct was constructed. These ultimate capacities are those of the members should corrosion not have taken place, and are listed in Table 3.1 for comparison with the member loads.

As a result of reinforcement corrosion, the truss members have all lost their structural strengths to some degree. In the most severely corroded cases, the concrete external to the reinforcement cage has mostly spalled away or is damaged to the extent that it has little or no structural strength. Apart from the bottom chord to the south-eastern truss at tower 14, the main reinforcement bars have not yet lost a significant proportion of their area, and they remain effective. Concrete within the reinforcement cage has also

retained its strength. Given these assumptions, reduced capacities for the members were estimated and they are listed in the same table alongside the member loads (Table 3.1).

It can be seen from Table 3.1 that had corrosion not taken place, there would be more than sufficient reserves in the truss members to carry the maximum loadings. However, the reserves are substantially reduced by the corrosion, with factors of safety for the members approaching unity. In particular, the 2 vertical struts adjacent to the towers would become most critically stressed. Close monitoring of the members with low factors of safety must be made, although currently no visual structural distress in these members has been noted.

There are several members in the aqueduct suffering from much more severe deterioration than the conditions assumed in the above estimation of reduced capacities, the obvious one being the bottom chord of the south-eastern truss at tower 14. The deterioration of this chord member has been discussed in Section 2, and the structural capacity of the chord is approaching its design load. Immediate repair to the chord must be carried out to prevent further deterioration of that section of the bridge and to avoid sudden collapse.

3.2 The Foundations

It was observed that significant movements of the foundations have occurred at a number of piers, causing rotations of the towers and the trusses. Whilst it is not known when these movements took place, the causes of the movement have to be investigated. Further rotation of the piers may cause unsymmetrical loadings and overstress the trusses.

Should it prove necessary to repair the foundation of any pier, a new foundation would need to be built around and beneath that pier. The cost of this might be in the range of $50,000 to $100,000 per pier.

3.3 Others

Other structural components critical to the superstructure are the walkway girder spanning between the ends of the cantilever trusses. Whilst it also suffers from significant deterioration, its present reduced structural

strength remains adequate. Similarly, the pipe support brackets are generally adequate except for those with substantial concrete loss. The current practice of replacing the brackets where necessary with steel brackets should continue.

3.4 Conclusion

Provided that the aqueduct is continuously and adequately maintained, and that adequate repairs are undertaken as soon as major structural damage becomes visible, there is no reason to assume that the structure will lose its capacity to carry the sewer pipe. Repairs to severely corroded members of the trusses, such as those near tower 14, must be carried out immediately to avoid sudden collapse of the aqueduct.

TABLE 3.1
OVOID SEWER AQUEDUCT
MEMBER CAPACITIES AND FACTORS OF SAFETY

Marks	Actual Axial Load (kN)	Full Ultimate Capacity (kN)	Factor of Safety	Reduced Ultimate Capacity (kN)	Reduced Factor of Safety
0′-1′	-948	-1,858	1.96	-1,228	1.3
1′-2′	-903	-1,714	1.9	-1,129	1.25
2′-3′	-853	-1,534	1.8	-979	1.15
3′-4′	-808	-1,460	1.8	-931	1.15
4′-5′	-766	-1,327	1.7	-848	1.1
5′-6′	-714	-1,278	1.8	-819	1.14
6′-7′	-525	-956	1.5	-550	1.05
7′-8′	-360	-912	1.8	-524	1.46
0-0′	-452	-708	1.57	-484	1.1
1-1′	-63	-114	1.8	-57	0.9
2-2′	-81	-149	1.8	-74	0.9
3-3′	-39	-185	4.7	-78	2.0
4-4′	-53	-290	5.5	-123	2.3
5-5′	-34	-355	10.4	-178	5.2
6-6′	-24	-355	15	-178	7.4
7-7′	-28	-355	12	-178	6.4
0-1	+883	+1,568	1.78	+1,568	1.78
1-2	+851	+1,568	1.84	+1,568	1.84
2-3	+810	+1,568	1.9	+1.568	1.9
3-4	+779	+1,568	2.0	+1,568	2.0
4-5	+729	+1,568	2.15	+1,568	2.15
5-6	+694	+1,568	2.25	+1,568	2.25
6-7	+670	+1,568	2.3	+1,568	2.3
7-8	+530	+684	1.3	+684	1.3
0-1′	+78	+228	2.9	+228	2.9
1-2′	+67	+170	2.5	+170	2.5
2-3′	+82	+170	2.1	+170	2.1
3-4′	+46	+68	1.5	+68	1.5
4-5′	+69	+120	1.7	+120	1.7
5-6′	+35	+68	1.9	+68	1.9

Note: The reduced capacity is for a member with all unconfined concrete spalled away.
- Compression
+ Tension
Member marks as shown on Figure 3.1.
Effect of wind on the trusses has not been included.

4.0 RECOMMENDATIONS

4.1 Repairs to the Superstructure

The level at which repairs are conducted will be determined by the future requirements of the Board.

Due to the state of the structure at present the time periods at which the cost levels for repairs will change are; up to 3 years, 3 to 10 years and "long-term".

The costings given are engineering estimates using commercial rates where possible, they include engineering, supervision and other associated costs, as well as a 15% contingency. Additional costs such as a sewage bypass system, which might be necessary during repairs, have not been included. The "long-term" repair option is for the immediate refurbishment of the structure, after which the length of time that the aqueduct can be kept in service is dependent on the maintenance programme that is implemented. The cost of maintenance would be low for approximately 15 to 20 years, with inspections and a recoating being the major items.

4.1.1 Short Term Solution (up to three (3) year life)

If only a 3 year life is required then the Board can expect to manage the structure with minimal expenditure. Based on the findings of the structural analysis those members which have been highlighted as critical should be regularly inspected during that period to detect damage before they become structurally unsound. When repairs are required, externally bonded steel reinforcement would be the cost effective approach although it would affect the aesthetic appearance of the structure. It is perceived to be necessary to provide support to the pipe at each location where the concrete support detail is showing visible signs of deterioration.

The estimated cost for the 3 year period is set out in Table 4.1 below:

Table 4.1
Predicted 3 Year Costs

		Year 1	Year 2	Year 3	Total
1	Design of Repair Bracing	-	$ 10,000	$ 5,000	$ 15,000
2	Installation of Pipe Support Brackets ($450 ea)	$ 67,500	$ 22,500	-	$ 90,000
3	Pipe Repair (on-going patching)	$ 40,000	$ 30,000	$ 20,000	$ 90,000
4	Truss Repairs	$120,000	$ 60,000	$ 30,000	$210,000
5	Detailed Survey of Top & Bottom Chords	$ 20,000	-	-	$ 20,000
6	Monthly Inspections	$ 3,000	$ 4,000	$ 4,000	$ 11,000
7	Contingency - 15%	$ 37,575	$ 18,975	$ 8,850	$ 65,400
	TOTAL	$288,075	$145,475	$ 67,850	$501,400

A total cost of around $12,000 per tower per year.

* These costs are not expected to include any further propping of the kind to be conducted at tower 14 in the first year.

4.1.2 Medium Term Solution (3 year to 10 year life)

After considering possible repair procedures, the 3 following options were favoured. They do not define the repair techniques which would be used in any detail but outline the approaches sufficiently to allow costs to be estimated.

Option 1

If a life of 3 to 10 years is required, one option would be to continue the practice set out in Section 4.1.1. Instead of the trend for costs to drop off after two years, it is expected that they would rise for 7 years and then decrease as the closure of the aqueduct approached.

Table 4.2 (see overleaf) contains the expected costs associated with this time period.

Option 2

Another option for providing a 3 to 10 year life, would be to conduct concrete repairs over the next two to three years which will allow the structure to see out its remaining life.

Such repairs could either consist of standard patch repairs, or a combination of lower quality patches and realkalinization.

The realkalinization technique is an electro-osmosis process which is designed to realkalinise carbonated concrete on a permanent basis. The steel is thereby returned to the passive state it attains in uncontaminated concrete.

The system operates in a similar manner to cathodic protection (CP) but has several advantages as outlined below. It has been researched by various investigators since the mid 1970s but only since the mid 1980s has a system been developed which makes its application commercially viable.

A variety of proven commercial CP systems for wharf repairs are available and they could be applied to this structure. These systems require that:

Table 4.2
Costing for Option 1 - Maintenance Required for a 10 Year Life
(x $1,000)

	Years										
	1	2	3	4	5	6	7	8	9	10	Total
1. Repair Specification & Supervision	5	10	10	14	18	20	20	15	10	5	127
2. Installation of Pipe Support Brackets ($450 ea)	68	35	25	25	-	-	-	-	-	-	153
3. Pipe Repairs	40	48	59	70	70	70	70	60	50	40	577
4. Envisaged Truss Repairs	-	60	60	90	120	140	140	120	90	30	850
5. Walkway Repairs	-	30	30	50	50	50	50	50	30	-	340
6. 4 Monthly Inspections	11	9	9	9	11	9	9	9	9	6	91
7. Contingency - 15%	19	29	29	39	40	43	43	38	28	12	320
	143	221	222	297	309	332	332	292	217	93	2458

- Total expenditure being approximately $17,500 per tower per year.
- All costs as at March 1990.

(a) Delaminations are broken out and repaired.

(b) A permanent groundbed is installed, and

(c) The system is maintained on a permanent basis.

The advantages of electro-osmosis are that:

(a) Only a temporary (3-5 days) groundbed is required. This reduces the long term loading and eliminates problems of level changes.

(b) System maintenance is not required. On completion of repassivation, the system is switched off and the temporary groundbed removed. A seal coat can then be applied to prevent further carbonation.

(c) The cost is considerably less than for CP, and

(d) Real evidence of protection is demonstrable.

Due to the variability of concrete quality and environment, it would be essential to conduct trials prior to installing a full scale system.

The benefits of using the realkalinization technique are that very little undamaged concrete need be removed. The use of cutting equipment would be kept to a minimum and this high cost component of repairs significantly reduced, as would be the potential to induce damage in adjacent members.

The predicted costs of this treatment is:-

(i)	Basic patch repair	-	$12,000 per tower
(ii)	Realkalinization applied to 40% of truss surface areas	-	$24,000 per tower
(iii)	Additional areas of concrete repair (flanges etc)	-	$ 3,200 per tower
	Total cost per tower		$39,200

(a)	Total cost of truss repairs (as above)	$ 548,800
(b)	Design, trial, supervision etc	$ 175,000
(c)	Application of a coating (10 year life)	$ 62,000
(d)	Scaffolding and propping costs*	$ 420,000
(e)	Cost of repairing walkway	$ 337,500
(f)	Pipe Repairs	$ 385,000
(g)	Contingency - 15%	$ 290,000
	Total Cost	$2,218,300

* No allowance for propping the river spans.

This estimate is equivalent to an average cost of approximately $15,800 per tower per year.

Option 3

This is similar to Option 2 but utilizes higher quality patch repairs instead of the realkalinization technique.

Estimated costs are based on 10% of the surface area of the trusses at each tower being treated to provide the required 10 years of service life.

(a)	Cost of patch repairs	$ 945,000
(b)	Specification, supervision etc	$ 125,000
(c)	Walkway repairs	$ 337,500
(d)	Pipe repairs	$ 385,000

(e)	Cost of scaffolding and propping *	$ 560,000
(f)	Continency - 15%	$ 334,000
	Total Cost	$2,561.500

* No allowance for propping the river spans.

This is equivalent to a cost of approximately $18,300 per tower per year.

4.1.3 Long Term Solutions

The prime objective of these repairs would be to restore the structure to a level of service that minimises the possibility of pipe failure and ongoing maintenance costs.

This would require the replacement of the pipe, or the insertion of an impermeable lining, and a new pipe support system.

The repairs to the trusses and walkway structure would have to solve the problem of general steel depassivation. To effectively do this by cutting back to expose all depassivated steel, would necessitate temporary propping systems which would make this work extremely costly, especially for the river spans.

It is recommended that if total refurbishment is required then the approach should be to fully patch repair areas of advanced damage and to reinstate cover where spalling has occurred. Following this, realkalinization should be conducted on every truss member and the walkway structure.

The final process would be to coat the structure with an anti carbonation coating. Such a coating would have a service life of approximately 10-12 years, and the structure would then require recoating.

(a)	The cost of restoring the trusses is estimated to be:		
	(i) Access - special scaffolding requirements per tower	$ 30,000	
	(ii) Patch repairs per tower	$ 67,500	
	(iii) Realkalinization per tower	$ 50,000	
	(iv) Coating per tower	$ 7,000	
	Cost per tower	$ 154,500	
	Total Cost		$2,163,000
(b)	Cost of restoring the walkway:		
	(i) Access - special scaffolding requirements	$ 250,000	
	(ii) Patch repairs	$ 410,000	
	(iii) Realkalinization	$ 225,000	
	(iv) Coating of Walkway	$ 90,000	
	Total Cost		$ 975,000
	Total cost of structural refurbishment		$3,138,000
(c)	Design, specification, trials, supervision etc		$ 250,000
(d)	Cost of renewing the pipe and bracket support system (estimated)		$750,000
(e)	Contingency - 15%		$ 621,000
The total cost of refurbishing the superstructure (estimate)			$4,761,000

GLOSSARY OF TERMS

GLOSSARY OF TERMS

1. Carbonation of Concrete - Concrete which is exposed to the atmosphere will undergo attack by carbon dioxide, which passes into solution in the liquid in the pores of the hardened concrete and subsequently reacts with the calcium hydroxide which is present.

   In solution the reaction between calcium hydroxide and carbon dioxide is rapid, and thus the carbon dioxide in solution is unlikely to penetrate through pores in the concrete which contain calcium hydroxide. This means that the carbonation "front" forms a distinct boundary: on one side there is reduced pH concrete (pH 8.3) with most of the calcium hydroxide neutralised by reaction with the carbon dioxide, while on the other side of the boundary the calcium hydroxide is essentially unreacted (pH <12). The existence of this distinct boundary enables very simple buffered pH indicator solutions to be used to accurately give the depth of carbonation on a freshly broken concrete surface. When the carbonation "front" comes into contact with reinforcing steel, the lowering of the pH results in the breaking down of the passive oxide layer on the surface of the steel, and the onset of corrosion.

2. Passivity of Steel - Concrete, when cast, provides an environment around the steel with a pH of about 12. The alkalinity of concrete arises from the presence of hydroxides produced during the hydration. This high alkalinity causes the formation of an oxide layer on the surface of the steel, protecting it from corrosion.

3. Chloride and Sulphate Analysis - This is performed on pulerverised concrete samples and measures the weight of chloride ions and sulphate present for a given weight of sample. The results are quoted as a percentage of sample weight. The results are compared against known threshold values at which damage is likely to occur.

4. Tower - This bridge consists of fourteen separate towers, each with a pair of trusses cantilevering from it, which in turn support the sewer. Each tower has its own foundation. In this report the term "tower" sometimes refers to the whole structure supported on that footing (ie, tower, two trusses, pipe supports and walkway).

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