ESPERANCE TANKER JETTY STRUCTURAL ASSESSMENT

Prepared for:
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Structural Analysis Report

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1 EXECUTIVE SUMMARY

BG&E Pty Ltd was commissioned by the Shire of Esperance to undertake a structural assessment of the Esperance Tanker Jetty, which is located within the main beachfront adjacent to the Esperance town site in the Great Southern region of Western Australia. The purpose of the investigation is to establish the overall condition and structural integrity of the jetty. Individual elements comprising the jetty structure have been assessed to determine their condition state as documented within the previous Detailed Inspection Report(1). Structural analysis has been based on the detailed inspection information, and this report establishes the capacity of the existing jetty structure and identifies structurally deficient elements that require remedial works.

Construction of the Tanker Jetty commenced in 1934 and was completed in 1935. The jetty originally comprised 192 piers, but was reduced to 143 piers with an overall length of 656m following a large storm in 1988. A portion of the original jetty head remains, but is isolated from the main structure by a 210m gap. Various repairs to the jetty structure have been undertaken since 1987.

Structural analysis of the existing jetty structure indicates that it is suitable for ongoing pedestrian usage in the short-term, subject to remedial works to specific elements identified as deficient and control of crowding. The jetty capacity is not adequate to achieve compliance with the relevant Australian Standards for publicly accessible facilities which are subject to crowd loadings, and for this reason its suitability for ongoing access can only be considered short-term (up to 3 years) subject to future re-assessment of structural condition and risk of over-loading. The jetty decking does not have adequate capacity for vehicular traffic.

The capacity of the jetty to withstand lateral loads arising from storms (wind and wave action) and earthquake events is very low. Analysis indicates that the jetty is not wholly capable of resisting the design wave for a 1 yr return period storm event with an appropriate safety margin. In addition the jetty is unable to meet the minimum earthquake design requirements in accordance with Australian Standards.

To address the structural limitations of the jetty in the short-term it is recommended that access management strategies are implemented to control crowding during planned events eg. annual jetty birthday celebrations, and to prevent vehicular access onto the deck. It is also recommended that the jetty be closed to public access when storm warnings are present. Three localised areas of jetty structure require priority pile repairs, and regular inspection of these areas for progressive signs of distress is recommended until repairs can be undertaken.

The limited lateral load resistance of the jetty structure is primarily due to inadequacies in the existing pier element sizes and connectivity, rather than the actual condition of the individual elements. The existing structure is not capable of achieving any significant improvement in the lateral load capacity from remedial works to isolated elements.

The works required to upgrade the existing structure to achieve appropriate capacity for pedestrian crowd loads, wave action and earthquake are very extensive, requiring replacement of all halfcaps, piles and ironwork to the retained superstructure (corbels, stringers and decking). Given the large cost of such works and the limited working life of the retained superstructure elements, upgrade or refurbishment works are not considered cost-effective or practical. The most appropriate long-term approach to provision of a recreational jetty facility would be for reconstruction of the jetty in accordance with current Department of Transport and Australian Standard design criteria, in much the same manner as the works recently undertaken for the similarly historic Busselton Jetty.

References:

(1) Esperance Tanker Jetty Structural Assessment – Detailed Inspection Report, 20 October 2010, BG&E

2 INTRODUCTION

The Esperance Tanker Jetty comprises a 656m long timber structure with 4.5m wide deck located within Esperance Bay projecting out from the shoreline in a south easterly direction. The jetty abutment is positioned adjacent The Esplanade and Norseman Rd intersection within the Esperance townsite. The Tanker jetty was originally constructed in 1934/35 by the Public Works Department of Western Australia to serve the shipping needs of the Esperance region. Freight was transported along the jetty using a steam train on narrow gauge rail tracks.

Usage of the Tanker jetty reduced soon after the construction of a new jetty located near the Taylor St port area in 1976. The Tanker jetty has subsequently been used for recreational pursuits by pedestrians, and is no longer used for rail transportation or vessel berthing.

Due to storm damage in 1988 the Tanker jetty was reduced in length from 192 piers to 143 piers. A remnant portion of the jetty head section remains isolated from the main length of jetty.

Ownership of the Jetty was transferred from the Western Australian State Government to the Shire of Esperance in 1990. Since this time the Shire has undertaken significant repair and refurbishment works, including installation of replacement timber piles, concrete encasement and protective wrapping to piles at the tidal zone, installation of steel cross-bracing to piers, reinforced concrete deck overlay and installation of new steel balustrading.

The purpose of this report is to summarise the structural analysis results for the jetty based on its condition state as defined in the Detailed Inspection Report(1). The structural analysis considers the effects of deterioration in the structural members to determine the allowable deck loads, and to determine the maximum storm and earthquake events which can be reliably resisted.

This report also identifies the jetty load requirements based on Australian Standards, and how these requirements compare with the structural analysis results. Commentary is provided regarding the extent to which repairs may be undertaken to improve the loading capacity and design life.

Reference has been made to a wave study undertaken by JFA Consultants Pty Ltd in their report dated January 2011 for assessment of the wave conditions associated with various storm events. The wave forces applied to jetty elements were determined by JFA Consultants for use in the structural analysis.

The structural analysis has been undertaken based on the relevant Australian Standards, including:

- AS/NZS 1170.0-2002 'Structural design actions – Part 0: General principles'
- AS/NZS 1170.1-2002 'Structural design actions – Part 1: Permanent, imposed & other actions'
- AS/NZS 1170.2-2002 'Structural design actions – Part 2: Wind actions'
- AS 1170.4-2007 'Structural design actions – Part 4: Earthquake actions in Australia'
- AS 1720.1-1997 'Timber structures – design methods'
- AS 4100-1998 'Steel structures'
- AS 4997-2005 'Guidelines for the design of maritime structures'
- SAA HB108-1998 'Timber design handbook'

It is intended that this report will serve as a reference for the Shire of Esperance in assessment of the risks associated with ongoing jetty usage, and for consideration of future jetty refurbishment or replacement works.

References:

(1) Esperance Tanker Jetty Structural Assessment – Detailed Inspection Report, 20 October 2010, BG&E

3 STRUCTURAL ANALYSIS

3.1 Overview

The traditional approach to structural analysis of an existing structure for assessment of load capacity it to identify the weakest component which in turn is deemed to govern the capacity for the overall structure. This approach is not considered practical for assessment of large-scale structures like the Tanker Jetty which incorporate a certain degree of redundancy, since the limitations associated with only one or a small number of discrete elements may not fairly represent the overall structure.

The Detailed Inspection Report provides a condition rating of 1 to 5 for individual jetty elements as defined in Section 6. Elements with a condition rating of 1 or 2 are considered to have only minor deterioration without any loss of strength. Elements with a rating of 3 are subject to a moderate degree of deterioration with reduced strength. Elements assessed as condition state 4 are subject to advanced deterioration with significant strength loss, and elements with condition state 5 have failed.

For structural analysis purposes elements with condition 1 or 2 can develop full strength capacity. Elements with condition 3 and 4 are subject to reduced strength based upon specific assessment of the deterioration observed. Jetty elements with condition state 5 elements have failed and do not contribute to the structure.

Explanation and illustration of the various jetty structure terms and numbering are provided within Section 6 and Appendix A for reference.

3.2 Gravity Load Assessment

Gravity loads comprise the permanent loads or self-weight of the structure, and the imposed live loads applied to the deck associated with use of the jetty, eg. pedestrians, vehicles. Gravity loads act vertically only, as distinct from lateral loads which act horizontally on the structure and are addressed separately in Section 3.3.

3.2.1 Australian Standard Requirements for Imposed Loads

The design imposed loads required by Australian Standards for jetties open to public pedestrian access are as follows:

- Pedestrian uniformly distributed load (UDL): 5kPa (500kg/m^2)
- Pedestrian concentrated load: 4.5kN (450kg)

For structures accessible to light vehicles (up to 2,500kg gross mass) the Australian Standards prescribe a larger concentrated load to represent forces applied through the wheels:

- Light Vehicle Concentrated load: 13kN (1,300kg)

3.2.2 Jetty Structure Capacity for Imposed Loads

Results of structural analysis for the various jetty elements to determine the imposed live load capacity are presented within the following table;

JETTY ELEMENT	WORST ALLOWABLE CONDITION STATE	IMPOSED UDL CAPACITY	IMPOSED CONC. LOAD CAPACITY
Exposed Timber Deck	3½	5kPa	4.5kN
Deck With Concrete Overlay	2½	5kPa	9kN
Outer Stringers	4	5kPa	13kN
Inner Stringers	3½	5kPa	13kN
Outer Corbels	4	5kPa	13kN
Inner Corbels	3½	5kPa	13kN
Halfcaps	3½	3kPa	13kN
Piles(1)	3½	3kPa	31kN

NOTES:

1. Pile capacity has been determined based on structural capacity of the timber pile element only. The geotechnical capacity of the piles is unknown due to the absence of geotechnical information for the site or appropriate pile test data.

Structural analysis for gravity loads indicates that the UDL capacity is primarily governed by the halfcap bending capacity as well as the partial halfcap seating onto the supporting pile with single bolt connection. The concentrated load capacity is governed by the timber decking as highlighted within the preceding table.

The UDL capacity of 3kPa is not adequate to meet the Australian Standard requirement of 5kPa for pedestrian crowd loads. The concentrated load capacity of the deck meets the requirements for pedestrian usage (4.5kN), but is not adequate for light vehicles (13kN).

Whilst the structure does not comply with the Australian Standard requirements for unrestricted public access, the imposed load capacity of the jetty can be considered adequate in the short-term (up to 3 years) for public access provided the deck is not subject to pedestrian crowding or stacked materials exceeding the UDL capacity of 3kPa. The jetty deck capacity is not sufficient for any type of vehicular traffic.

Adequacy of the structure for pedestrian usage beyond a short-term period of three years should be subject to re-confirmation of adequate structural condition and risk assessment regarding the potential for over-loading.

3.3 Lateral Load Assessment

Lateral loads acting on the structure comprise horizontal forces associated with environmental loads (wind and waves), earthquake and general robustness.

Lateral loads associated with vessel berthing and mooring were not considered since the jetty is not intended for such usage.

3.3.1 Australian Standard Requirements for Lateral Loads

The design lateral loads required by Australian Standards for jetties with a design life of 50 years (which is the typical design working life recommended for jetty structures) are as follows:

- Wind: 500 yr ARI
- Waves: 200 yr ARI
- Earthquake: 500 yr ARI
- Robustness: 2.5% of maximum permanent and imposed loads

Note: ARI is an abbreviation for 'Average Recurrence Interval', which is calculated as the inverse of the probability that an event will be exceeded in any one year.

3.3.2 Jetty Structure Capacity for Lateral Loads

Results of structural analysis at various locations along with length of the jetty to assess the lateral load capacity are presented within the following table.

Key principles adopted for the lateral load analysis are as follows:

- Lateral loads are resisted by cantilever action of the piles projecting from the seabed, and structural analysis is based on a condition state of 2 for piles at seabed level.
- In the absence of any geotechnical data or assessment it has been assumed that piles can develop full fixity (ie. full bending capacity) at a depth of 1.8m below the seabed for a 300mm diameter pile.
- Pier cross-bracing elements are only present on 30% of the piers, and where they do occur they are typically in poor condition and installed without any positive connection to the piles such that they rely on friction for load transfer. The contribution of pier bracing elements has been ignored due to the relatively small number of braced piers and the poor condition and connectivity of the rod braces.
- Imposed loads were not considered to act in conjunction with environmental loads (wind and wave) based on the low likelihood of pedestrians being present on the jetty deck during such significant storm events.
- An imposed deck load of 3kPa UDL was adopted for assessment of earthquake and robustness loads based on results of the gravity load capacity assessment.

JETTY REF.[1]	PIER No.	WIND CAPACITY (ARI)	WAVE CAPACITY(2) (ARI)	EARTHQUAKE CAPACITY (ARI)	ROBUSTNESS CAPACITY
Point 1	49	500 yr	1 yr	Nil	OK
Point 2	75	500 yr	1 yr	Nil	OK
Point 3	106	500 yr	1 yr	Nil	OK
Point 4	124	500 yr	1 yr	Nil	OK
Point 5	143	500 yr	<1 yr	Nil	OK

NOTES:

1. Jetty reference points coincide with specific positions along the jetty with varying seabed levels adopted for assessment of design wave loads within the Wave Study by JFA Consultants. Refer to Section 6.3 'Jetty Plan View' for reference point locations.
2. Wave loads were assessed in combination with 70% of the design wind loads in accordance with AS 4997. The ARI for wind was adopted as 5 yrs for combination with wave forces, which represents the minimum ultimate wind load load prescribed within AS 1170.1, and provides better compatibility with the low wave ARI.

(a) Wind Load Capacity

The existing structure is capable of resisting wind loads to the level required by Australian Standards. It should be noted however that the magnitude of lateral loads is relatively low due to the small area of structure exposed to wind, and it is unlikely that such wind forces would occur without coincident wave loads.

(b) Wave Load Capacity

Analysis of the jetty capacity to resist wave forces has been undertaken using 3-D structural modelling software (Microstran) to provide the most realistic assessment of the jetty structure response to the peak design wave. In particular the 3-D model allows accurate assessment of load-sharing between piers, whereby the most heavily loaded pier at the wave crest can share loads with more lightly loaded piers within the wave trough to optimise the overall structure capacity.

The analysis indicates that an individual pier within the wave crest does not have adequate capacity to resist the applied forces in isolation, and it relies upon distributing longitudinal and transverse loads to adjacent piers. Longitudinal loads act parallel to the length of the jetty and are distributed to successive piers through the stringers. Transverse loads act perpendicular to the jetty and are distributed to adjacent piers via diaphragm action in the decking. Distribution of loads in this manner relies upon the shear capacity of the ironwork, ie. bolts connecting pier and superstructure elements, and decking spikes.

The jetty ironwork is generally heavily corroded with condition state 4½ which limits the extent to which wave loads can be distributed between piers. Particular importance is attached to the single bolt connection between the pile and halfcaps. This bolt is relatively new and in condition state 2, although it is working close to its full capacity to resist wave loads.

Results from the 3-D analysis indicate the jetty can resist the peak design wave occurring as part of a 1yr ARI storm event as far as Pier 124. Beyond Pier 124 the depth to seabed becomes too large such that the pile capacity is not adequate to achieve the level of safety required by Australian Standards.

Remnant pile and bracing elements have been retained at 32 piers in an effort to provide bracing to the replacement piles. The remnant elements have not been effectively connected to the replacement piles and as a result they do not offer any bracing action, ie. the remnant elements are redundant to the structure. Whilst the jetty can resist 1yr ARI wave loads up to Pier 124, the analysis indicates there is no reserve capacity to resist additional drag forces acting on the remnant pier elements. Given these elements are redundant to the structure they should be removed, otherwise there is a significant risk they will overstress the piles when subject to wave action.

The jetty capacity for a 1 yr ARI wave event or less represents a large shortfall when considered against the 200 yr ARI wave event recommended in AS 4997 'Guidelines for the design of maritime structures'. It should be noted however that the recommended design wave event ARI is dependent on the design working life of the jetty. For a design working life of 5 years (the minimum considered by the code), the appropriate design wave ARI reduces to 20 yrs. The wave study indicates that a 20 yr design wave would overtop the jetty deck for the majority of its length by up to 1m in some parts, thereby exerting far greater forces that the existing jetty can resist. On this basis the jetty capacity to resist wave events falls well short of compliance with Australian Standard guidelines.

(c) Earthquake Load Capacity

The lateral loads applied to the jetty deck as a result of the 500 yr ARI earthquake event are represented by a horizontal force of 12% of the gravity loads. The minimum earthquake load required by the code is only slightly lower than this value at 10% of the gravity loads.

Structural analysis indicates that the piles do not have adequate capacity to resist the forces associated with the minimum earthquake loads, and therefore the existing jetty does not achieve compliance with Australian Standards for earthquake design.

(d) Robustness

The robustness requirements within the Australian Standards are intended to ensure structures are provided with a minimum level of horizontal load resistance so that unexpected events (ie. vessel impact) do not result in damage which may be disproportionate to the original cause.

The jetty structure has adequate capacity to meet the general robustness requirements of the Australian Standards.

4 REMEDIAL WORKS

4.1 Deficient Jetty Elements

Existing jetty elements which have deteriorated to an extent worse than what is considered representative of the general element condition for the overall structure, or have failed, are identified within the Appendix B - 'Schedule of Deficient Elements'.

Remedial works to reinstate these members to a serviceable condition will ensure the jetty can accommodate imposed pedestrian loads of 3kPa UDL and 4.5kN concentrated load, which is consistent with the general structural capacity of the jetty. The remedial works may take the form of repairs, strengthening or replacement subject to the specific nature and extent of deterioration at a particular element.

The limited lateral load resistance of the jetty structure is primarily due to inadequacies in the existing member sizes and arrangement for the overall jetty, rather than the actual condition of the members. The existing structure is not capable of achieving any significant improvement in the lateral load capacity from remedial works to isolated elements.

The structural analysis has identified that remnant elements retained for pier bracing purposes are effectively redundant, and in fact only serve to attract additional wave drag loads to the structure. Removal of the remnant elements will reduce wave drag on the structure, and therefore the remnant elements have been identified within the schedule of deficient elements.

4.2 Priority Remedial & Access Management Works

4.2.1 Piles

Whilst the piles are the most important elements within the jetty structure, the deterioration of an isolated pile can be accommodated to a significant extent in the short-term due to continuity in the stringers. The stringers are typically continuous across two spans which provides some capacity for deck gravity loads to be transferred to adjacent piles.

In instances where two or more adjacent piles are subject to significant deterioration, then the ability of the structure to redistribute loads is diminished or non-existent. There are three areas where adjacent piles are deficient which should be given priority for remedial works:

(a) Pier 22 & 23: Pile number P2 at each pier has significant section loss at the seabed.
(b) Pier 29 & 30: Steel replacement pile number P1 at each pier has significant section loss at water level.
(c) Pier 39: Pile number P1 has failed completely, and pile P2 has significant section loss at seabed.

Until remedial works can be completed at these pier locations the structure should be monitored on a regular basis to identify any signs of distress, eg. pile movement, deck sag. Pedestrian access to the deck at these areas should be restricted during any public events to prevent overcrowding, eg. annual jetty birthday celebration.

4.2.2 Bollards

The jetty deck is not capable of supporting any vehicular traffic with an appropriate level of safety and bollards should be installed in advance of the abutment to prevent any vehicular access onto the deck.

4.2.3 Closure During Storm Events

The jetty has low resistance to wave action during storms, and it is recommended that the jetty be closed to public access when storm warnings are issued by the Bureau of Meteorology.

5 JETTY ASSET MANAGEMENT CONSIDERATIONS

5.1 Compliance with Australian Design Standards

Compliance of the existing jetty structure capacity with the Australian Standards design criteria is summarised within the following table;

ACTION	DESIGN CRITERIA	JETTY CAPACITY	A.S. COMPLIANCE
Imposed Load	5kPa	3kPa	No
Wind	500 yr ARI	500 yr ARI	Yes
Wave	200 yr ARI	< 1 yr ARI	No
Earthquake	500 yr ARI	< min ARI	No
Robustness	2.5% Gravity Load	2.5% Gravity Load	Yes

5.2 Implications of Non-Compliance

5.2.1 Imposed Load Capacity

The jetty is open to the public with unrestricted pedestrian access and therefore must be capable of accommodating crowd loads. The jetty capacity of 3kPa is significantly less than the 5kPa crowd loading criteria.

The current capacity of 3kPa can be considered adequate for incidental pedestrian access for the short-term, provided appropriate management strategies are employed to control crowding for planned public events or gatherings.

In order to achieve the 5kPa design criteria the following upgrades to the jetty would be required (in addition to the remedial works previously identified for deficient elements):

- Replacement of all halfcap elements with new members fully seated onto piles each end with a two bolt connection and new bolted steel bracket connections to the existing corbels.
- Stringer strengthening or replacement for all stringers with condition rating 4 or worse.
- Pile repair or replacement for all piles with condition rating 3 or worse.

The need to replace all the halfcap elements represents a large scale operation with a significant degree of complexity, since the existing superstructure would need to be temporarily propped at each pier location while the existing halfcaps are removed and new members installed.

Typically halfcap replacement operations for bridgeworks works are only undertaken at isolated locations within the overall structure, since the costs associated with propping and working over water are generally prohibitive when considered on a large-scale or widespread basis.

Whilst the jetty superstructure elements (decking, stringers, corbels) are generally in serviceable condition and capable of accommodating 5kPa crowd loads, the working life of these elements would only be in the order of 5-10 years maximum. Similarly the existing replacement piles are currently in reasonable condition but can only be expected to provide for an ongoing working life of 10-15 years maximum.

Given the limited remaining life of the existing superstructure and pile elements, wholesale replacement of the halfcaps is not considered an appropriate long-term solution, and as a short-term solution it would be cost-prohibitive. This leaves full reconstruction of the jetty as the most appropriate approach to achieving compliance with Australian Standards for a public jetty facility.

5.2.2 Wave Load Capacity

The Australian Standard for design of maritime structures for wave actions serves as a guideline only, and is not mandatory. Responsibility for determining the design wave criteria is vested with the asset owner/operator, which is generally based on consideration of the Australian Standard guidelines and risk assessment.

Structural analysis indicates that existing jetty cannot wholly withstand the design wave for 1 yr ARI storm event with an appropriate factor of safety. The design wave has been determined from the average of the highest 1% of all waves in the storm, coinciding with the peak steady water level to account for tide and storm surge.

(a) Wave Study Calibration

JFA Consultants reviewed tide and wave data from a significant storm event on 5 January 2007, where anecdotal observations were that waves approached but did not overtop the jetty deck. JFA advised that this storm represented a 2-4 yr ARI event for wave height, however the storm occurred in a relatively low water level part of tidal cycle, therefore the overall crest height would be less than the design crest height for such an event. A storm with similar intensity can occur in high water level which would result in design crest height. The numerical findings were consistent with anecdotal observations to provide a degree of calibration for the wave study.

(b) Historical Jetty Performance During Wave Action

A key issue to reconcile with the results of the structural analysis for wave load capacity is the observation that the existing jetty has survived relatively intact for over 75 years. The following issues are relevant for consideration of this apparent anomaly:

- The final 4 piers comprising the original jetty head (pier no. 189 to 192) remain intact and isolated from the main jetty length. The original jetty head deck was designed for lateral vessel berthing and mooring loads and incorporates additional raking 'stay' piles, providing the head section with greater lateral capacity than the main jetty.
- Wave crest heights at the remnant jetty head are typically lower than those at the main jetty due to the effect of shoaling where wave height increases when travelling from deep to shallow water.
- The 'neck' portion of jetty, comprising the section of main jetty connecting to the widened head did succumb to wave action during the 1980's which precipitated removal of 210m of jetty for safety reasons.
- The original jetty piers comprised three piles in each pier with cross-bracing and walers. The jetty in its current form however since the 1990's comprises only two piles within each pier typically without any effective bracing, which has lower resistance to wave action than what the original jetty structure could achieve.
- The behaviour of waves and wave-structure interaction in shallow water is complex. The Wave Study by JFA Consultants represents a best practice assessment based on current understanding and established guidelines. It does however provide necessarily conservative results in terms of wave crest height and forces applied to the structure to account for recognised uncertainties associated with the behaviour of waves and their interaction with structures.
- The design wave criteria used as part of the wave study represents an extrapolation of existing data which may introduce an additional degree of conservatism to the assessment.
- The design wave for a specific event is defined by Australian Standards as the highest 1% of waves which occur during the storm. A casual observer of waves during a storm may not bear witness to the peak wave crests to appreciate the most damaging extent of the event.
- The strength properties of timber can be variable and the Australian Standards requires that the strength values adopted for design purposes represent the lowest fifth percentile, ie. the strength value that is obtained by 95% of the material, to ensure an appropriate level of reliability. The actual strength and performance of the timber elements therefore can be expected to be significantly greater than what is determined by structural analysis.
- The capacity of structural elements is reduced by a 'capacity factor' in accordance with Australian Standards to account for material variability, manufacturing control processes, construction tolerances etc, to ensure that design principles are consistently achieved over time. The capacity factor contributes to the overall safety margin against failure. It is likely that the existing structure is stressed beyond the limits allowed by the capacity factor during storm events.
- When piles are subject to horizontal wave loads they are most highly stressed in flexure (or bending) below seabed level, where the horizontal wave loads are transferred from the pile into the soil. If piles are overstressed from wave action then no visible signs of distress are apparent until failure occurs.

In summary, the low wave ARI rating for the existing structure can be reconciled against historical performance as the result of applying an accepted margin of safety in accordance with relevant Australian Standards, through increased wave forces and reduced structural capacity. Based on previous failure of the jetty neck section and recent failure at seabed of a pile at Pier 39, there is evidence to suggest that wave actions applied to the jetty are exceeding acceptable pile stress levels.

(c) Improving Resistance to Wave Forces

The capacity of the jetty to resist wave action relies primarily on the bending capacity of the piles, and to a lesser extent the ironwork to distribute loads to adjacent piers and pier cross-bracing. Installation of new steel rod cross-bracing to the piers would provide a measure of increased lateral load resistance, however the effectiveness of new bracing is limited by the poor connectivity between existing piles and halfcaps. The piles comprising the existing jetty are effectively too small in diameter and too little in number to provide appropriate resistance to wave action.

Significant improvement to the wave resistance of the structure could only be achieved by installing new piles with larger diameter and higher strength, as well as replacement of all the superstructure ironwork. Such works represent a major undertaking, and when considered in conjunction with the halfcap replacement the upgrade works effectively become full reconstruction.

A major benefit of jetty reconstruction is the flexibility to raise the jetty deck level to provide an ‘air-gap’ between the design wave crest and the jetty superstructure. The existing jetty deck was constructed relatively close to the waterline to better serve freight loading and unloading activities with moored vessels. If the jetty is only intended for recreational usage, then the deck level can be raised to avoid wave drag forces on the superstructure which in turn reduces the forces transferred to the piles. The result would be a far more efficient and cost-effective structure.

5.2.3 Earthquake

Accurate assessment of the forces generated in the jetty due to earthquake relies upon geotechnical assessment of the site to classify the seabed soil conditions. For the purpose of this report assumptions have been made regarding the soil conditions to provide indicative earthquake forces.

Structural analysis indicates that the jetty cannot achieve compliance with the minimum requirements of the earthquake standard. The relatively low likelihood of significant seismic events in the Esperance region means that jetty resistance to wave action is primary short-term consideration for the structure. Nonetheless the low lateral load capacity of the overall jetty structure means that it is susceptible to collapse in an earthquake event. Such events often occur with little or no warning, in which case the safety of those on the jetty would be jeopordised.

Upgrading the jetty capacity for earthquake comprises the same approach outlined for improved resistance for wave action in the preceding section. The large scale and cost of such upgrade works is prohibitive. Should jetty reconstructive works be undertaken then a geotechnical assessment of the site would be necessary for earthquake design of the new structure.

6 JETTY ELEMENT NOMENCLATURE & NUMBERING

6.1 Jetty Element Nomenclature

Refer to Appendix A – 'Jetty Nomenclature' for illustrations identifying the various jetty elements and terms.

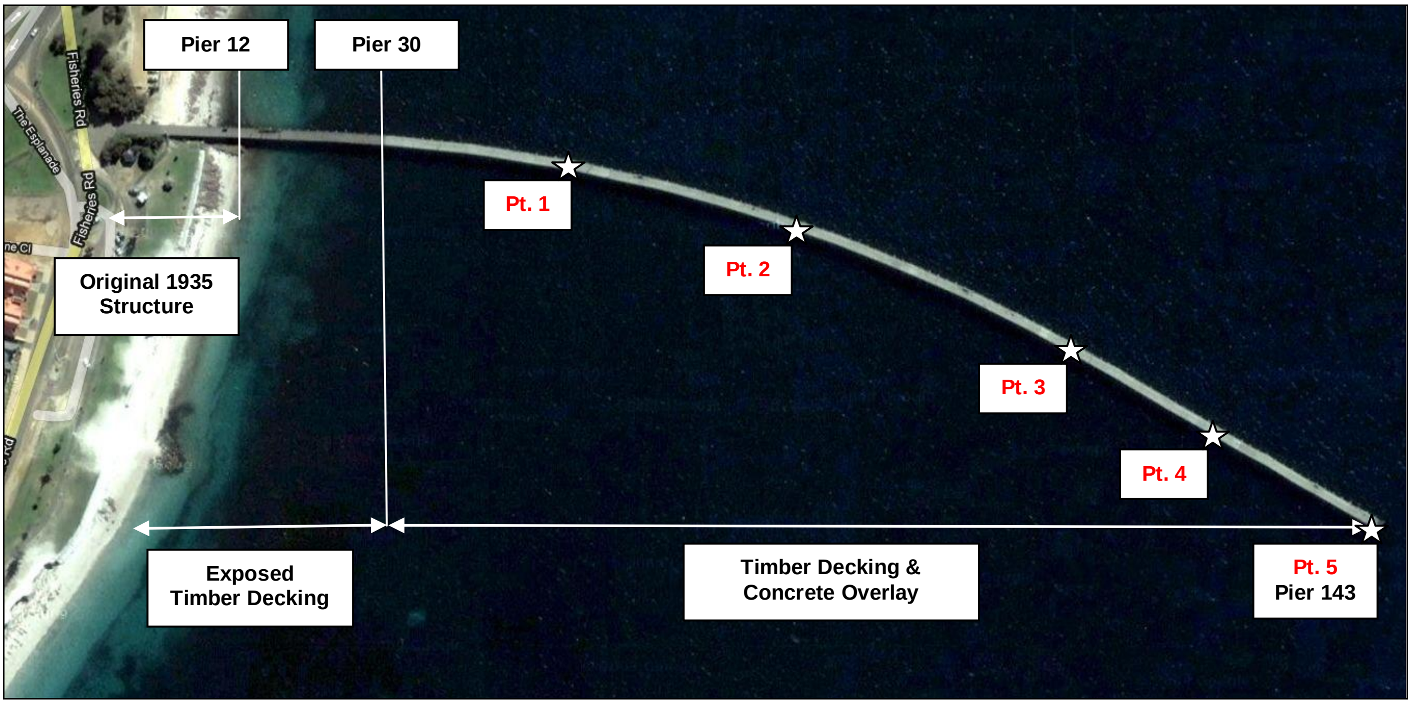
6.2 Element Numbering Conventions

The piers are numbered in sequential order commencing from the abutment at the beach to the jetty head (pier 143).

The piles are numbered from left to right (North to South) within each pier when viewed in the direction of increasing chainage, i.e. looking eastwards towards the head of the jetty.

The span number corresponds to the pier number at the eastern span end. The stringers and corbels are identified numerically from left to right when viewed in the direction of increasing chainage.

6.3 Jetty Plan View



NOTE:

1. Jetty reference points 'Pt. 1 to Pt. 5' represent specific locations along the jetty where wave conditions and forces were determined by JFA Consultants as part of the Wave Study.

6.4 Description of Condition States

The condition state classification numbered 1-5 is based on the ARRB Local Roads Bridge Management Manual, which is provided below for reference.

6.4.1 Timber Elements

Condition State 1 - Timber has little or no rot or decay. There may be minor cracks, splits or checks. Connecting components i.e. Bolts and rivets, are tight with no corrosion.

Condition State 2 - Timber has minor decay and other defects but they do not affect the strength of the member. Connecting components have no cracking, corrosion or loose connections.

Condition State 3 - Timber has moderate decay and large splits or checks, e.g. stringers may have pipe rot up to 50% of the diameter. Connections become loose due to timber decay, and/or steel component corrosion. Manual and/or core testing may be required to verify this level of deterioration.

Condition State 4 - Timber has advanced deterioration, which leads to significant strength reduction. E.g. corbels have more than 50% pipe rot of the diameter, timber stringers have pipe rot up to 70% of the diameter, accompanied by severe splitting, bracing has decay and splitting of ineffective due to deformation, loose joints or other poor connections, and severe damage due to vehicle impact. Manual and/or core testing will be required to verify visual assessment of the deterioration.

Condition State 5 - Timber has 100% deterioration or has ceased to function due to its condition or loss.

6.4.2 Concrete Elements

Condition State 1 - Concrete is in good condition. It may have some fine cracking due to shrinkage.

Condition State 2 - Fine cracking due to flexural/shear stress and reinforcement corrosion, and there may be a few minor spalls but no rust staining in the cracks.

Condition State 3 - Medium cracking (including flexural cracking) or isolated spalling due to reinforcement corrosion, alkali-aggregate reaction, sulphate attack, movement and strain etc.

Condition State 4 - Heavy cracking and spalling due to heavy corrosion of reinforcement, and advanced alkali-aggregate reaction, salt-attack and sulphate attack, etc.

Condition State 5 - Concrete has no remaining function.

6.4.3 Steel Elements

Condition State 1 - Steelwork has no rust staining on the paint, though there may be minor chalking, peeling or curling. Connecting components, ie bolts and rivets, are tight with no corrosion of coatings, galvanising, not stain or wear signs.

Condition State 2 - Steelwork has spot rusting and the paint is no longer effective, but there is no corrosion of the section. Connecting components have no cracking, corrosion or loose connections. Welds have no corrosion.

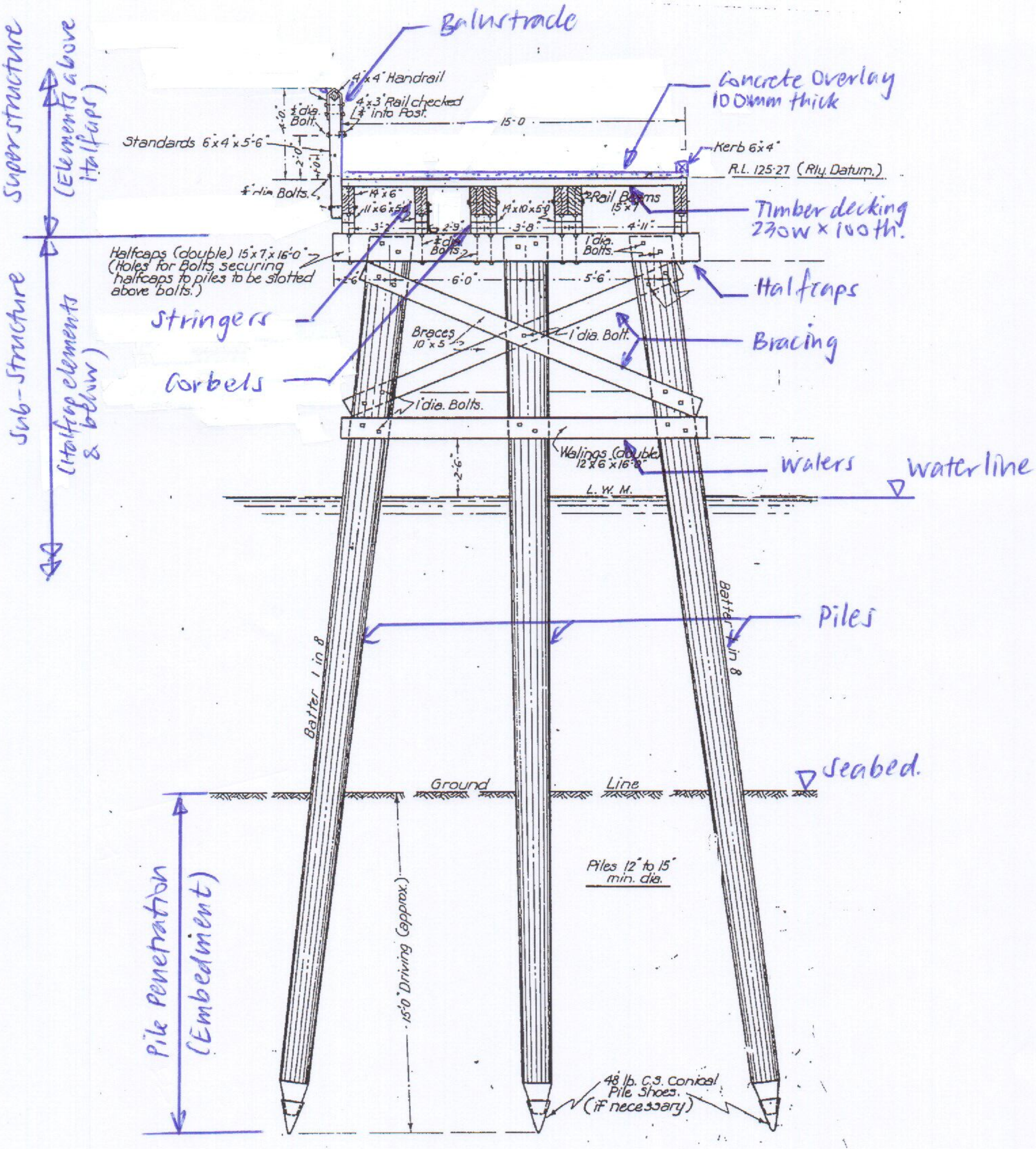
Condition State 3 - Steelwork has medium corrosion with slight loss of section, and paint has completely failed. Smaller components such as bracing may not be effective and connections may be heavily corroded or loose.

Condition State 4 - Steelwork is heavily corroded with obvious loss of section. Connections may be very loose, bracing may be totally ineffective and coatings have failed.

Condition State 5 - Steelwork has failed in section part or whole. Connections are completely loose or corroded or otherwise, causing failure or imminent loss of the element's function.

Appendix A

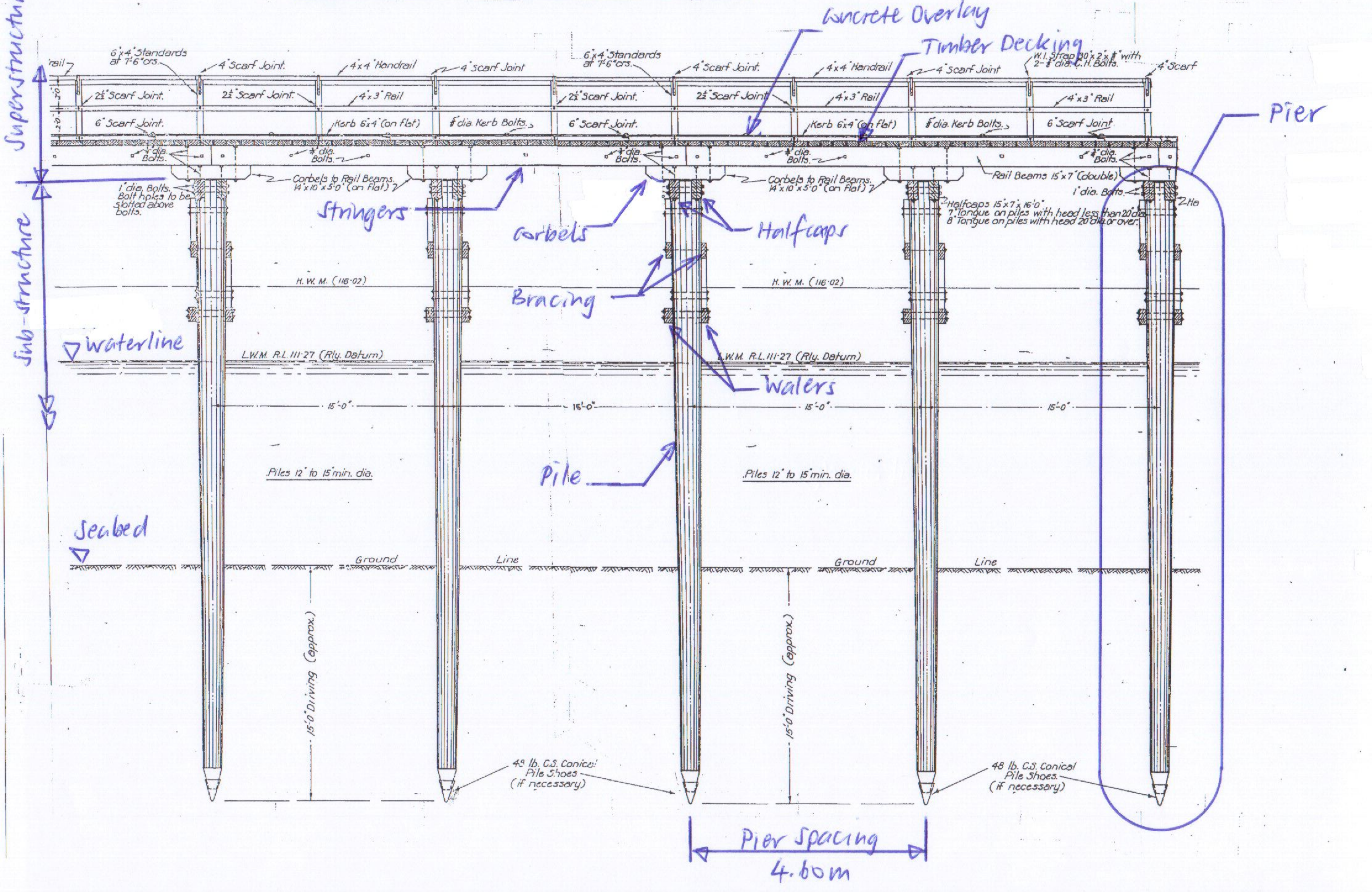
Jetty Nomenclature



Type Cross Section. (Looking Seawards)

TYPICAL PIER ELEVATION

JETTY SIDE ELEVATION



Appendix B

Schedule of Deficient Jetty Elements

SCHEDULE OF DEFICIENT JETTY ELEMENTS

PIER No.	PILE No.	HALFCAPS		CORBELS	STRINGERS	REMNANT PIERS
		West	East			
3		1	1			
4		1	1			
5		1				
6		1	1			
7		1	1	C1		
8				C1		
9				C1		
10				C1		
11				C1		
13				C2		
17	P2					
18		1	1			
20	P1					
22	P2(1)					
23	P2(1)					
24	P1					
29	P1(1)					
30	P1(1)					
37						√
39	P1 & P2(1)					
43						√
45						√
46						√
47						√
48						√
51				C1		
52				C1		
58						√
61	P2					√
62				C2		√
63						√
64						√
66						√
67						√
68						√
69						√

PIER No.	PILE No.	HALFCAPS		CORBELS	STRINGERS	REMNANT PIERS
		West	East			
70						√
71					S2	
72						√
80						√
81						√
82						√
85						√
87						√
92						√
94				C1		
95			1			
96						√
104				C1		√
106	P2					
107						√
113						√
115	P2					
119						√
126				C1		√
128						√
130				C2		
132				C5		√
133				C5		
134				C5		√
TOTAL	**12**	**12**		**16**	**1**	**32**

NOTE:

1. Denotes priority remedial works required as outlined in Section 4.2.