PROPOSED WHARF AND DREDGING PROJECT

RESOURCE CONSENT APPLICATIONS: VOLUME 3

SPECIALIST REPORTS: APPENDIX A - C

PREPARED FOR PORT OF NAPIER LTD

NOVEMBER 2017





Allan Planning & Research Ltd

Stantec

APPENDIX A

NAPIER PORT 6 WHARF PRELIMINARY DESIGN REPORT

NAPIER PORT

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Report

Napier Port 6 Wharf
Preliminary Design Report

Prepared for Napier Port

Prepared by Beca Ltd

14th July 2016

Revision History

Revision Nº	Prepared By	Description	Date
1	Stephen Lee	Draft for review	11/04/2016
2	Stephen Lee	Final	3/06/2016
3	Stephen Lee	Minor amendments	14/07/2016

Document Acceptance

Action	Name	Signed	Date
Prepared by	Stephen Lee		14/07/2016
Reviewed by	Connon Andrews		14/07/2016
Approved by	Connon Andrews		14/07/2016
on behalf of	Beca Ltd		

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1 Introduction

The Napier Port proposes to upgrade the capacity of their container terminal by constructing a new wharf structure. The proposed wharf, to be known as 6 Wharf, is located on the northern side of the current container terminal.

The purpose of this document is to summarise the preliminary design for the wharf and associated revetment slope, including definition of the design philosophy and key design inputs (loads, parameters, materials, etc) adopted for the design.

This document should be read in conjunction with the Preliminary Design Drawings. Details relating to geotechnical aspects of the project may be found in the Geotechnical Interpretive Report. This report highlights the geotechnical conditions and design parameters specific to the site.

2 Description of Preliminary Design Solution

2.1 Wharf Structure

The preliminary design for the proposed wharf structure consists of a cast in situ reinforced concrete deck slab supported on bored reinforced concrete piles, integrated with an armoured revetment slope. The wharf is approximately 350m long x 34m wide, with a deck level of 3.81mCD and berth depth of -14.5mCD.

The deck slab has a typical uniform thickness of 700mm increasing to 1450mm deep beneath the front and rear container crane rails. The wharf is supported on bored reinforced concrete piles, which include permanent steel casings, socketed into the underlying competent (N>50) Mangaheia group sandstone. Piles are typically 900mm in diameter, increasing to 1200mm along the rear edge of the wharf. Piles are arranged on a typical grid spacing of 6.5m longitudinally and 5.95m transversely. The transverse grid spacing was determined to suit the proposed 29.765m gauge between the front and rear container crane rails. The founding sandstone level varies significantly along the length of the wharf, getting progressively deeper towards the western end of the wharf.

Various pile configurations were considered including combinations of 700mm, 900mm and 1200mm diameter piles at various spacing. The controlling aspect in the pile design was accommodating large lateral loads associated with slope/ground displacement under the design seismic event, not purely carrying vertical loads. The selected pile arrangement is based on achieving an efficient and cost effective overall structure, taking into consideration the relative costs of pile and deck construction.

The wharf deck is split into 2 lengths, with a longitudinal expansion/contraction joint provided. A steel cover plate is proposed to allow traffic to run over the gap. A transverse shear key is also provided to retain transverse alignment between the 2 halves of the wharf. The purpose of the expansion/contraction joint is to reduce the demands on the piles generated by longitudinal restraint of thermal movements and concrete shrinkage. Concrete shrinks over time and also expands/contracts due to thermal effects. The wharf shrinks/contracts about its' centre (centre of stiffness) and the ends are pulled in. The longer the wharf is, the greater the amount of shrinkage/thermal movement at each end. The piles try to restrain this movement, generating large loads in the piles; largest at the wharf ends. Reducing the length of the wharf reduces the pile demands and results in a more efficient structure.

The new wharf adjoins the existing 5 wharf. A gap is provided between the structures to limit damage during a seismic event. A steel cover plate could be provided to bridge this gap and enable vehicular access between the wharves, however further investigation is required to assess the existing deck levels and arrangements for this.

A retaining wall is provided at the rear edge of the wharf, where it meets the reclamation (i.e. Northern Container Terminal). The proposed solution adopts a precast concrete wall, connected to underside of the wharf deck. Alternatively, an independent retaining wall solution (i.e. not connected to the wharf), such as an 'L' shaped reinforced concrete wall, could be adopted. In either case, the embedment depth of the retaining wall requires further consideration/refinement to ensure compatibility with the depth/thickness of the selected solution for the revetment armouring.

The deck supports fenders (mounted on precast concrete drop panels) bollards and includes provisions for services. The deck slab is graded to convey surface runoff to a slot drain provided at the rear edge of the wharf. Details on how the slot drain discharges will be resolved by others. However, it is likely that manholes will be provided in the reclamation behind the wharf to provide connection points between the slot drain and the reclamation drainage discharge pipes that run through the rear retaining wall and into the harbour via the existing stormwater outlet locations.

2.2 Revetment Slope

The revetment slope has a 1.84H:1V or 1.5H:1V finished profile, depending on the armouring solution adopted (refer revetment armour section below), and to achieve acceptable factors of safety for static and seismic global slope stability. The increased depth of weaker material at the western half of the wharf revetment is of significant influence. In particular, the short term global stability factor of safety during construction following dredging at the western end is likely to be marginal. During construction, the construction methodology will need suitable controls put in place to remove potential risk of failure.

Additionally, a number of potentially liquefiable layers were identified at the site, in the reclamation fill, low plasticity zones of the recent marine sediments, and upper zones of the quaternary marine sediments. This was identified in the 6 Wharf Development – Geotechnical Factual Report. Liquefaction susceptibility under design seismic loads (SLS1, SLS2 and ULS) was assessed based on SPT and shear wave velocity data. Liquefaction under SLS1 seismic loading is considered unlikely. Layers within the reclamation fill and marine sediments (recent and quaternary) were identified in the analysis as likely to liquefy under SLS2 and ULS seismic loading.

A preliminary assessment of stability required shear contribution from wharf piles to achieve a reasonable static factor of safety, and reduce seismic displacements under liquefied conditions to less than 500mm. Furthermore, additional geotechnical requirements to reduce seismic displacements included:

- Dredging of recent marine sediments at the reclamation toe where present at the eastern end of the wharf;
- Ground improvement at the western end of the wharf. Ground improvement at the western end of the wharf could consist of a lattice of ‘weak’ concrete trenched walls.

At this preliminary design stage, only limited consideration has been given to transitioning of the revetment slope at each end to tie into the existing slopes. At the eastern end, where the new wharf adjoins the 5 wharf, the sandstone level is comparatively shallow. This will assist with transition by allowing steeper slopes or benching to be cut. At the western end, the sandstone level is deeper and overlayed with weaker material. Accordingly, the new (deeper) revetment slope will need to extend for some distance beyond the end of the wharf to allow a more gradual transition back to the existing slope. This is indicated on Napier Port drawing 5341-SHT405-REV F. Some further thought is required on the implications of this on any future extension, as the associated armouring will make the future installation of piles challenging.

2.3 Revetment Armour

Two options were considered for the under wharf revetment armour due to the wave climate and rock supply issues. The first option is conventional rock armour and the second option is single layer concrete armour units.

Hydraulic stability of the under wharf revetment is governed by wave action, not wash produced by the ship's main propeller or bow thrusters. This means that analysis of different ship sizes and berthing directions is not required.

2.3.1 Rock Armour Option

The relatively high wave period means that larger rock is required and this is exacerbated if locally available limestone rock is assumed with an apparent mass density of about 2.2 t/m^3. A primary armour rock mass (M_{50}) of about 14t is required. Allowing for an under-layer and two layer rock thickness means that a total layer thickness of about 5.4m is required. If a denser rock is assumed (such 2.6-2.8 t/m^3), rock size can be significantly reduced.

A revetment slope of 1.84H:1V is assumed and geotextile is allowed for between the bund material and the rock under-layer. A 6m wide toe trench is allowed for to support the slope rock. Where the Mangaheia Rock (N>50) is located above -14.5m CD, the armour can be anchored into the rock without a toe trench.

2.3.2 Concrete Armour Unit Option

An option to reduce the under wharf revetment thickness is to use single layer concrete armour units. Options are Core-Loc, Accropode or Xbloc units, licensed by Concrete Layer Innovations (France) or Delta Marine Consultants (The Netherlands). Subject to detailed design, we assess that about $2m^3$ units which are 1.8m thick would be suitable, overlying a rock under-layer and geotextile. A rock toe support to the first row of units is required. Where the Mangaheia Rock (N>50) is located above -14.5m CD, the concrete units can be anchored into the rock without a toe support.

Typically the revetment slope for concrete armour units is steeper than rock (i.e. 1.33H:1V) which increases unit stability and reduces material quantities. However for global geotechnical stability reasons, we have used 1.5H:1V. Another issue is the number of units required on the slope. A nominal 20-row maximum is recommended subject to detailed design, which is exceeded in this case. Options are to increase the unit size, include a rock berm and/or steepen the slope. A rock berm obviously cannot intrude into the berth pocket area.

2.3.3 Scour Protection in the Berth Pocket

Scour protection in the berth pocket is provided by rock armour based on wash from the ships main propeller. Two layers of rock over geotextile is provided. The typical scour protection width is 30m from the quay face. No protection is required where the Mangaheia Rock (N>50) is located above the berth pocket depth of -14.5m CD. The extent of protection at the eastern end of the wharf is to be assessed based on the actual rock profile in this location.

2.4 Wharf Services and Drainage

Services on the wharf have been provided as per the indicative plans provided by Napier Port, and are as follows:

2.4.1 Electricity and Telecommunications

Electrical supply and telecommunications ducts are suspended beneath the waterside wharf deck edge with a conventional bracket system. Access pits are provided in the waterside deck edge at approximately 50m centres.

2.4.2 Water Supply

Water supply main has been suspended beneath the waterside wharf deck edge with a conventional bracket system, shared with electrical/telecommunications ducts where applicable. Access pits for hydrants are provided at approximately 50m centres.

2.4.3 Stormwater Drainage

The deck slab has been graded to convey surface runoff from the front of the wharf to the stormwater collection system at the rear. Crane rail recesses have been detailed with drainage holes to discharge surface runoff captured by the crane rail recesses.

2.5 Future Wharf Extension

A future extension of the wharf at the western end may be considered by Napier Port to achieve an overall deck length of 400m. The future extension is assumed to be an independent structure with a longitudinal movement joint provided between it and the 350m long wharf. The western edge of the deck has also been detailed to accommodate a future transverse shear key.

2.6 Mooring Dolphin

Two mooring dolphins are proposed at the western end of the wharf to facilitate vessel mooring. The structures each consists of a square 18m long x 16m wide cast in situ deck slab and supported by 9 no. 1800mm diameter bored piles with permanent steel casings. The steel casings are typically driven to competent underlying Mangaheia group sandstone then bored to the nominated founding level. Steel casings are then infilled with concrete. Access to the mooring dolphin from the wharf is provided by an access gangway. These are assumed to consist of proprietary aluminium gangways. Three no. 150t bollards been provided on each mooring dolphin, and motorised capstans could be added if required.

The 'inner' dolphin may be designed and detailed such that it can be incorporated into a future extension of the wharf. At this stage, we have indicated these potential connection/incorporation details but note that work is required to develop the design of the wharf extension in order to understand this aspect fully.

Options considered in the preliminary design of the dolphin included:

- A single pile system was considered however the pile diameter exceeded 4m. Adopting a larger pile casing may prove difficult to drive and require a customised piling rig. Furthermore, under mooring loads the dolphin structure was found to be quite flexible and exhibited large deflections and tip rotations.

- A 6 pile system reliant on frame action to resist load was also considered however this also returned large diameter piles and large deflections under mooring loads. Pile socket lengths into rock were also quite long under this arrangement to resist the tension/compression demands generated under frame action.

3 Design Inputs

3.1 General Arrangement

3.1.1 Wharf

The proposed 6 Wharf structure is to be located on the existing northern boundary of the container terminal. The wharf will be 350m long with an allowance for a future expansion of 50m and will cater for vessels up to 350m LOA. The wharf deck will have a uniform fall towards the landside edge where it will tie into the existing ground level of +3.81m. The structure will be approximately 34.0m in width to allow for a container gantry crane gauge of 29.765m.

3.1.2 Dredging

The berth pocket will be required to be dredged to an ultimate design depth of -14.5m. This will require a short term dredge depth to approximately -13.0m to allow for the placement of berth scour protection and slope toe protection.

3.1.3 Fenders

A fender system will be provided to protect the wharf from damage and help absorb kinetic energy of a berthing vessel.

3.1.4 Bollards

Bollards will be provided along the seaward face of the wharf at 12-14m centres.

3.1.5 Ladders

Ladders will be provided along the seaward face of the wharf at 50m centres maximum. The bottom rung will be positioned at minimum 300mm below lowest astronomical tide (LAT).

3.1.6 Kerbs

No kerbs will be provided along the seaward edge of the wharf.

3.1.7 Drainage

The deck slab is graded to convey surface runoff to a slot drain provided at the rear edge of the wharf. Details on how the slot drain discharges will be resolved by others. However, it is likely that manholes will be provided in the reclamation behind the wharf to provide connection points between the slot drain and the reclamation drainage discharge pipes that run through the rear retaining wall and into the harbour via the existing stormwater outlet locations.

3.1.8 Crane Rails

The wharf deck will include recesses to accommodate the future installation of a rail assembly for container gantry cranes. The rail assembly will consist of a steel sole plate, continuous welded rail track, crane rail clips and crane rail pads.

3.1.9 Tidal Levels

The following tidal fluctuations will be considered for the site.

Table 1 – Design Tidal Levels

Parameter	Level (m CD)
Highest Astronomical Tide (HAT)	1.98
Mean High Water Springs (MHWS)	1.90
Mean Sea Level (MSL)	0.95
Mean Low Water Springs (MLWS)	0.02
Lowest Astronomical Tide (LAT)	-0.04

4 Design Philosophy

4.1 Design Standards and References

The design, detailing and construction of the structures will be undertaken in accordance with the following standards:

- NZBC — New Zealand Building Code
- AS 4997-2005 — Guidelines for the design of maritime structures
- BS6349: Parts 1, 2 and 4 — Code of Practice for Maritime Structures
- AS/NZS 1170.0-2002 — Structural design actions
- AS/NZS 1170.1-2002 — Permanent, imposed and other actions.
- AS/NZS 1170.2-2011 — Wind actions
- NZTABM 3rd Edition — New Zealand Transport Agency Bridge Manual 3rd Edition
- NZS 1170.5-2004 — Earthquake action
- NZS 3101-2006 — Concrete Structures Standard
- NZS 3404-1997 — Steel Structures Standard
- NZS 3109-1997 — Specification for Concrete Construction
- AS 1657 — Fixed Platforms, Walkways, Stairways and Ladders
- PIANC – Design of Port Structures for Seismic Actions
- PIANC – Guidelines for the Design of Fender Systems 2002
- CIRIA C683: The Rock Manual – The use of rock in hydraulic engineering (2nd Edition), 2007
- PIANC: Guidelines for protecting berthing structures from scour caused by ships (Report No. 180-2015)
- AS/NZ 4671 — Reinforcing Steel
- NZS 3404.1:2004 — Steel Tube Piles

4.2 Materials

4.2.1 Concrete

a. Exposure Classification

The following exposure classifications have been adopted for the design in accordance with NZS3101-2006 Table 3.1. These are based on achieving a 100 year design life for durability.

Table 2 – Concrete Exposure Classification

Element	Exposure Classification	Concrete Grade	Cover
Piles (concrete infill)	B2	30 MPa	70 mm
Deck Slab	C	50 MPa	60 mm
Abutment	C	50 MPa	60 mm

b. Concrete Mix Design

The concrete mix for the wharf and dolphin structure shall contain as a minimum:

- Not less than 400 kg/m³ cement content

- Not less than 30% Fly Ash or 8% Amorphous Silica
- Water/cementitious products ratio not greater than 0.4

4.2.2 Reinforcing steel

Reinforcement shall have minimum yield strength of 500MPa and be manufactured using the microalloy process. All reinforcing steel shall comply with the requirements for Grade 500E as outlined in AS/NZS 4671.

4.2.3 Steel Tube Piles

All steel tube piles are assumed to be grade 250MPa. Design life will be achieved through allowance for sacrificial corrosion losses. Corrosion rates have been applied to all exposed surfaces of the steel piles in accordance with NZS3404.1:2009 C5.3.2.1. Corrosion rates adopted for design are summarized below:

Table 3 Corrosion Rates

Location	Depth (Chart Datum)	Corrosion Rate (mm/year)
Splash zone	≥ +1.9 m	0.075
Tidal zone	+1.9m to +0.1m	0.035
Low water zone	+0.1m to -0.1m	0.075
Immersion zone	-0.1m to -14.5m	0.035
Embedment zone	≤-14.5m	0.015

4.3 Design Life

The new wharf structure will be designed for a 100 year design life.

4.4 Inspection and Maintenance

The wharf structure requires regular inspection and maintenance due to the aggressive environment. In addition to repairs from accidental damage to the wharf, maintenance of wharf furniture will generally be required every 10-15 years (fenders, bollards, secondary steelwork, navigation aids, kerbs, services and crane rails).

5 Geotechnical Design Parameters

For details refer to the Geotechnical Interpretive Report and Geotechnical Factual Report.

6 Design Loads

Design loads are in accordance with the following:

▪ NZBC	New Zealand Building Code
▪ AS 4997-2005	Guidelines for the design of maritime structures
▪ BS6349: Parts 1,2 and 4	Code of Practice for Maritime Structures
▪ AS/NZS 1170.0-2002	Structural design actions
▪ AS/NZS 1170.1-2002	Permanent, imposed and other actions.
▪ AS/NZS 1170.2-2011	Wind actions
▪ NZTABM 3rd Edition	New Zealand Transport Agency Bridge Manual 3rd Edition
▪ NZS 1170.5-2004	Earthquake action

6.1 Permanent Loads

6.1.1 Dead Loads

Dead loads have been derived from material weights and structural component dimensions used for construction. The following densities have been considered for this design package:

▪ Precast Concrete	26.5 kN/m3
▪ Reinforced concrete	25.0 kN/m3
▪ Steelwork	78.5 kN/m3
▪ Soil	18.0 kN/m3
▪ Wood /Timber	10.0 kN/m3
▪ Rock	20.0 kN/m3

6.1.2 Superimposed Dead Loads

A general allowance of 0.25 kPa has been made for services in accordance with the New Zealand Transport Agency Bridge Manual 3rd Edition Clause 3.4.2. This load has been applied to the complete wharf deck.

6.2 Live Loads

6.2.1 Container Loads

A container crane UDL load of 50 kPa will be adopted based on AS4997-2005 Guidelines for the Design of Maritime Structures. The UDL will be applied over the entire length or a patch loading to produce the most severe effect.

6.2.2 Reach Stacker Loads

Allowance will be made for existing forklifts with up to 120t axle loading. The following axle configuration will be adopted.

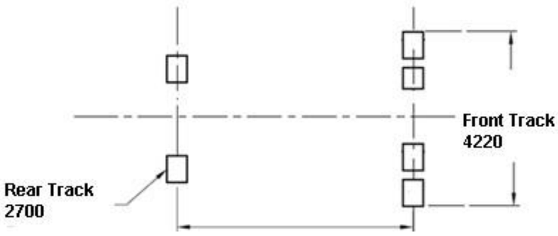

Figure 1 – Fork Lift Axle Configuration

6.2.3 Mobile Crane

The wharf deck will make allowance for the following mobile crane units:

- Existing 6 Series Terex Gottwald Mobile Harbour Crane without restriction
- Potential future 7 series Terex Gottwald Mobile Harbour Crane
- Potential future 8 series Terex Gottwald Mobile Harbour Crane

The mobile crane loadings and specifications are attached in appendix A. It will be assumed that the mobile cranes will use stabiliser pads consistent with the attached specifications.

6.2.4 Container Gantry Crane Loads

To allow for a future container gantry crane without exact specifications the design will adopt the recommendations of AS4997:2005.

A general design load of 750 kN/m will be adopted. This is comparable to 75 tonnes per wheel.

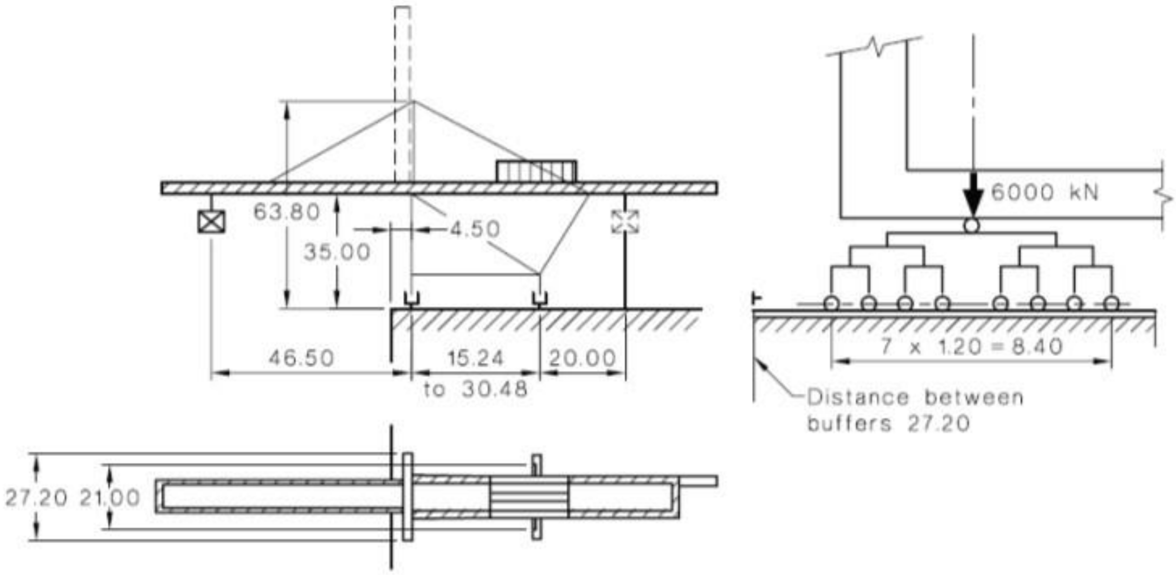

Figure 2 – AS4997 Gantry Crane Configuration

Design actions on the crane stop block will be determined form the kinetic energy relationship using the following equation and parameters:

Force to stop = $\frac{1}{2} v^2 m / s$

Where:

v travel speed, 45m/min was adopted
m total mass, 800tonnes allowed representing 16 No 50 tonnes per wheels (DL only)
s distance to stop, 250mm adopted for the assessment (max. buffer stroke of 500mm)

6.2.5 Vessel Berthing Loads

The approach velocity of a vessel is a major factor in determining the kinetic energy required to be absorbed by the fender system during vessel berthing. The fender system will be designed to accommodate the following vessel berthing configuration and meet the following criteria:

(a) a single large container vessel
(b) two smaller container vessels berthed back-to-back

Table 4 - Design Vessel Criteria

Criteria	Small Vessel	Large Vessel
Vessel type	General Cargo	Container Vessel
Dead weight (DWT)	10,000t	104,000t
Displacement tonnage	16,200t	143,000t
Length overall (L_{OA})	153m	340m
Length between perps (L_{PP})	144m	330m
Maximum beam (B)	23.7m	42.8m
Draft (D)	8.4m	14.5m
Maximum berthing angle	3º	3º
Maximum berthing velocity	250mm/s	100mm/s
Water Cushion Effect C_c	1.0	1.0
Softening Effect C_s	1.0	1.0
Block Coefficient C_b	0.85	0.85

Notes:

1. Vessel berthing velocity has been based on ‘easy berthing’ and ‘exposed’ navigation conditions. It assumes all berthing is tug-assisted. Velocity has been derived in accordance with AS4997:2005 Appendix B Figure B1
2. Vessel berthing is assumed at quarter points of the vessel

6.2.6 Vessel Mooring Loads

The wharf will be used by large container ships and general cargo vessels. A bollard load of 150t (1500kN) will be used for the design. The bollards will be placed along the seaward face of the wharf at 12-14m centres. A maximum of three adjacent bollards are assumed be fully loaded simultaneously for preliminary design purposes.

6.3 Environmental Loads

6.3.1 Seismic Loads

The seismic design will be accordance with NZ1170.5:2004 and NZTMB 3rd Edition. An equivalent static force-based design approach shall be adopted based on the following criteria:

- Design Life: 100 years

- Classification: IL 4
- Limit State Factor: Ultimate Limit State (ULS) R_u = 1.8 for 1/2500 APE
- Site sub soil classification: C (Shallow soil sites)
- Zone Factor: Z = 0.38 (Napier)
- Near-fault Factor: N(T,D) = 1.0
- Structural Performance Factor: S_p = 0.80 except when considering the lateral stability of the whole structure against sliding or toppling S_p = 1.00.

The revetment slope and land reclamation behaviour in seismic events is discussed in the geotechnical interpretive report and accounted for in the structural analysis as appropriate.

The following combinations of orthogonal seismic loads are to be considered for design of the wharf in accordance with the New Zealand Transport Agency Bridge Manual 3rd Edition:

- 100% Longitudinal + 30% Transverse
- 30% Longitudinal + 100% Transverse

6.3.2 Wave Loads

The wharf and revetment preliminary design assumes wave loading in accordance with Advisian memorandum titled "Preliminary estimate of extreme waves for wharf", dated 21 April 2016. The following parameters will be adopted for wave loads:

Table 5 – Design Wave Parameters

Limit State	Significant Wave Height (m)	Mean Wave Period (s)
Ultimate Limit State (1/500 APE)	2.6	9.5
Revetment Armour (1/50 APE)	2.3	9.5
Serviceability Limit State (1/25 APE)	2.1	9.5

Due to lack of guidance in AS4997:2005, wave forces assumed in the design are to be based on BS6349 Part 1 and the publication "Wave-in-deck loads on exposed jetties" (Cuomo et al. 2007).

6.3.3 Tsunami

Tsunami loads have not been considered at this stage and a detailed study would be required to establish an appropriate design basis and loads. However, it is noted that the wharf structure inherently has significant capacity to withstand the uplift pressures applied under the wharf deck due to a Tsunami.

6.3.4 Debris Loads

The wharf will be designed for debris loading as a mat could form against the structure. Debris loads are in accordance with AS4997:2005 Clause 5.6.

6.3.5 Earth Loads

The following earth pressure loads will be considered for the design of the new wharf:

a. Static Condition

At-rest lateral earth pressure acts on the back of the rear retaining wall and piles. Additional lateral loading due to traffic surcharge also act on the retaining walls and piles. A 50kPa vertical surcharge is to be assumed to be applied immediately behind the wharf.

b. Seismic Condition

Under seismic accelerations, reclamation soil restrained by the rear retaining walls and piles will be accelerated into the wall. A 'stiff' condition will be adopted for the wall due to the degree of propping restraint provided by the wharf deck limiting wall deflections. These soil pressures will be applied in accordance with the method recommended by Wood & Elms (1990) for stiff walls as shown below.

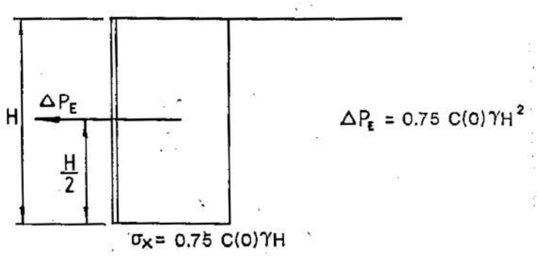

Figure 3 - Earthquake Pressure Increment on a Stiff Wall

Where: H = height of soil above base of foundation
C(0) = peak ground acceleration coefficient

ΔP_E = increment or decrement in active earth pressure force due to earthquake
γ = unit weight of soil

The lateral earth pressures due to static earth pressure to be included in the seismic load case are at rest earth pressures, P_O.

6.3.6 Shrinkage and Creep

Assessment of shrinkage and creep effects has been considered in accordance with AS3600-2009 Concrete Structures with recommended adjustment suggested in New Zealand Transport Agency Bridge Manual 3rd Edition for creep and shrinkage coefficients.

- Relative Humidity: 82%
- Relative Humidity Factor: $k_4 = 0.38$
- Average Final Drying Basic Shrinkage Strain: $\varepsilon^*_{csd,b} = 990\mu$

The wharf is a shrinkage sensitive structure. Consideration will be given to the fact that shrinkage has a range of ±30% from design values.

6.3.7 Temperature Effects

Overall and differential temperature effects will be considered in accordance with the New Zealand Transport Agency Bridge Manual 3rd Edition. Allowance is to be made for forces and movements resulting from an overall temperature change of ±20ºC. Temperature gradient through the depth of the structure is to be taken from the New Zealand Transport Agency Bridge Manual 3rd Edition.

6.3.8 Wind Loads

Wind loads will be considered in accordance with AS/NZS1170.2. Wind loads acting on the wharf structure will be determined from the following parameters:

- Terrain Category: 2
- Region A7 (Non-cyclonic)
- Wind Speed Ultimate Limit State (ULS) V_U = 45 m/s
  Serviceability Limit State (SLS) V_{SLS} = 37 m/s
  Normal Operating Conditions Wind Speed V_{OP} = 20 m/s Wind

effects on moored vessels are to be considered as part of mooring loads.

6.3.9 Fatigue Loads

a. Wind and Wave Induced Fatigue

Assessment of wind induced fatigue may be carried out in accordance with AS/NZS1170.2-2011 Clause 2.5.5. Assessment of wave induced fatigue may be carried out by adopting the wave characteristics for the normal operating condition wave based on 10^6 cycles per annum as per AS4997-2005 Clause 5.12.7 in the absence of site specific information. Wind and wave fatigue shall be considered independently. The stress range developed within any fatigue sensitive details is anticipated to be minor and not govern the design.

b. Container Gantry Crane Induced Fatigue

The wharf deck will be assessed for fatigue under the operation of the container gantry crane. Fatigue stress range will be limited to the values outlined in NZS3103-2006 Clause 2.5.2.

6.4 Construction Loads

6.4.1 Construction Live Load

A construction live load of 1.5kPa will be allowed for in design.

6.4.2 Construction Sequence

The design is based on the structure being constructed in accordance with the following general sequence:

a. Carry out ground improvements
b. Perform piling works
c. Construct revetment slope and place rock amour
d. Construct wharf deck
e. Install wharf furniture and services.

For further detail refer to the construction methodology report.

6.5 Deflections

6.5.1 Deflection Limits

The wharf does not have any specific operational requirements nor house any deflection sensitive services or equipment. A horizontal deflection limit of L/150 has been adopted in accordance with

AS4997-2005. Services are to be detailed with flexible joints to accommodate anticipated movements.

Deflections due to slope displacements under the design seismic event will exceed this.

6.6 Load Factors and Combinations

Load combinations are in accordance with AS/NZS1170.0:2002, AS4997:2005 and the New Zealand Transit Authority Bridge Manual 3rd Edition. The following key combinations will be adopted for the design:

Table 6 - Load Combinations

Load Case	Load Combinations											
	1 AS/NZS1170.0		2 AS 4997		3 AS 4997		4 TNZBM 1B		5 TNZBM 2A		6 TNZMB 3A	
	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS
Dead Load	1.00	1.20	1.00	1.20	1.00	1.20	1.00	1.35	1.00	1.20	1.00	1.00
Shrinkage	1.00	1.20	1.00	1.20	1.00	1.20	1.00	1.35	1.00	1.20	1.00	1.00
Creep	1.00	1.20	1.00	1.20	1.00	1.20	1.00	1.35	1.00	1.20	1.00	1.00
Earth Pressure	1.00	1.50	1.00	1.50	1.00	1.50	1.00	1.35	1.00	1.20	1.00	1.35
Live Load												
- Container UDL	0.70*	1.50	0.60	0.60	0.60	0.60			0.40*	0.40*		
- Gantry Crane (DL)	1.00	1.20	1.00	1.20	1.00	1.20	1.00	1.35	1.00	1.20	1.00	1.00
- Gantry Crane (LL)	1.00	1.50	0.60	0.60	0.60	0.60			1.00	1.20		
- Reach Stacker	1.00	1.50	0.60	0.60	0.60	0.60			1.00	1.20		
- Mobile Crane	1.00	1.50										
Temperature							1.00	1.69	1.00	1.20	0.33	0.33
Seismic											1.00	1.00
Mooring			1.00	1.50								
Berthing					1.00	1.50						
Environmental			A	B	A	B						
Crack Width Category	I		II		II		II		II		II	

Notes:

1. The container uniformly distributed load, gantry crane live load, reach stacker and mobile crane are considered as live loads. These have not been included in the earthquake combination as the wharf will not be used for container storage
2. Mobile crane outrigger loads are assumed to act concurrently with container gantry crane loads with a minimum offset between individual loads equivalent to the spacing between piles.
3. Above live loads do not include dynamic effects. A dynamic load factor of α = 1.3 is to be applied as per the NZTBM
4. Symbol '*' denotes where a combination factor has been used for container uniformly distributed live load. This load was conservatively treated as a short-term load with a ψs = 0.7 for service Combination 1. For both service and ultimate Combination 5 a combination factor of ψs = 0.4 was adopted. These factors recognise the unlikely probability of full container live load concurrent with other transient effects
5. Berthing and moorings loads are considered as non-concurrent actions
6. Vessel berthing has been assumed to occur under normal operating conditions. Berthing velocities used for the fender design are assumed to account for wind and wave environmental loads acting directly on berthing vessels. Wind and wave loading acting on the wharf during vessel berthing under normal operating conditions will be significantly lower. For this case a normal operating wind speed and design wave have been considered

7. Abnormal berthing shall also be considered in Combination 3 by reducing the load factor to 1.00 and replacing normal berthing with abnormal berthing loads (1.5 and 2.0 times the calculated normal berthing energy for large and small container vessels respectively in accordance with PIANC guidelines). Abnormal berthing condition shall only be considered for ultimate limit state

8. Mooring loads are based on bollard capacity and mooring analysis. These are assumed to account for wind and wave environmental loads acting on moored vessels. Wind and wave loading acting directly on the wharf structure with a moored vessel have been considered

9. 'A' and 'B' denote the following environmental load combinations:

A. Serviceability limit state:
- $W_S + 0.7F_{WAVE,SLS}$
- $0.7W_S + F_{WAVE,SLS}$

B. Ultimate limit state:
- W_U
- $F_{WAVE,ULS}$
- $W_u + 0.7F_{WAVE,ULS} + 1.5F_{DEBRIS}$
- $0.7W_U + F_{WAVE,ULS}$

Appendix A

Mobile Crane Specifications

TEREX Port Solutions

Quay Loadings

G HMK 7408 Mobile Harbour Crane

Main Crane Data:

Total crane weight:	480,0 t
Maximum load:	100,0 t
Maximum load on operation:	580,0 t
Number of axles:	8
Propping base:	15,0 m x 13,0 m
Stabilizer pad size:**	2,0 m x 4,5 m
Stabilizer pads per corner	1

**other sizes on request

Crane in Travelling Mode:

Uniformly distributed load during travelling:

Area covered (16,2 m x 11,1 m)	179,82 m²
Uniformly distributed load (480,0 t / 179,8 m²)	2,67 t/m²

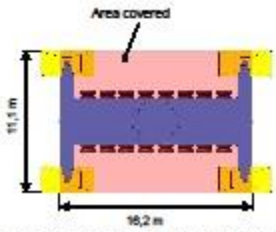

Figure 1 : Area covered by the crane in travelling mode *

Pressure under wheels:

Axle Load:	60 t
Wheels / Axle:	4
Load / Wheel:	15,00 t
Supporting Area / Wheel:	1690 cm²
Pressure under Wheel:	8,88 kg/cm²

Crane in Operation:

Maximum propping forces [Heavy load - 75%]

Boom Position	I	II	III
Load:	98,4 t	98,4 t	98,4 t
Radius:	28 m	28 m	28 m
Stabilizer pad loading:	228,0 t	273,0 t	241,1 t
Pad(s) on which load is exerted:	A, D	A	A, B
Stabilizer Pad Area:	9,00 m²	9,00 m²	9,00 m²
Ground Pressure :	2,53 kg/cm²	3,03 kg/cm²	2,68 kg/cm²

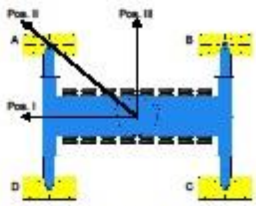

Figure 2: Determination of boom and pad position *

* Images are exemplary and may vary from configured crane

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GOTTWALD Mobile Harbour Crane
G HMK 6408

Quay Loading Data

Main Crane Data:

Total crane weight:	445,0 t
Maximum load:	100,0 t
Maximum load on operation:	545,0 t
Number of axles:	8
Propping base:	14,0 m x 12,5 m
Stabilizer pad size:**	2,0 m x 4,5 m
Stabilizer pads per corner	2

**other sizes on request

Crane in Travelling Mode:

Uniformly distributed load during travelling:

Area covered (22,0 m x 10,5 m)	231,66 m²
Uniformly distributed load (445,0 t / 231,7 m²)	1,92 t/m²

Pressure under wheels:

Axle Load:	56 t
Wheels / Axle:	4
Load / Wheel:	13,91 t
Supporting Area / Wheel:	1690 cm²
Pressure under Wheel:	8,23 kg/cm²

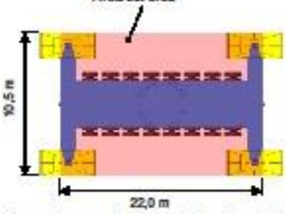

Figure 1: Area covered by the crane in travelling mode *

Crane in Operation:

Maximum propping forces [Heavy load - 75%]

Boom Position	I	II	III
Load:	100,0 t	96,0 t	100,0 t
Radius:	24 m	25 m	24 m
Stabilizer pad loading:	210,8 t	248,2 t	219,7 t
Pad(s) on which load is exerted:	A, D	A	A, B
Stabilizer Pad Area:	18,00 m²	18,00 m²	18,00 m²
Ground Pressure :	1,17 kg/cm²	1,38 kg/cm²	1,22 kg/cm²

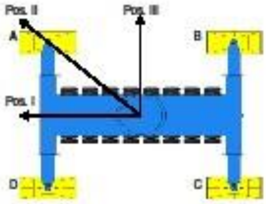

Figure 2: Determination of boom and pad position *

* Images are exemplary and may vary from configured crane

G HMK 6408 - Quay loading data - Heavy load - 75%
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