Northport Berth 3 design and construction monitoring

Lucy Coe, Nicola Ridgley, Do Van Toan
Beca Infrastructure Limited, Auckland, NZ

Keywords: retaining wall, deflections, port, sheet piles

ABSTRACT

Berth 3, an extension to Northport Ltd's Marsden Point wharf facility, comprises a twin sheet pile wall to provide support to the new wharf deck and retain the adjacent reclamation. The wall type was selected to enable construction to proceed by walking a crane over the constructed wall, negating the need to use a large floating or jack up barge. The wall structure comprises twin king piles and intermediate sheet piles, 12.5m apart tied with steel bars at two levels and reinforced concrete deck top.

Design was undertaken using finite difference software FLAC to predict displacement, movement, shear and anchor loads during various stages of construction. Deflection monitoring of the wharf performance was undertaken with survey and inclinometers. This paper describes the FLAC modelling for design of the twin wall and compares monitoring results from the performance of the wall both during construction and one year following completion with predicted displacements.

1 INTRODUCTION

The project comprised development of Berth 3 at Northport's wharf facility at Marsden Point. The site is located 30km south-east of Whangarei and is adjacent to the NZ Refining Company facilities. The General Cargo Wharf is Northport's principal operating wharf and prior to the recent extension was 390m long, primarily supporting log handling and woodchip exports but capable of container handling with large container cranes. This wharf and reclamation was designed and constructed in 2000 by Beca Infrastructure Ltd (Beca) and The Fletcher Construction Company Ltd (Fletcher) when Northport had to relocate from further up Whangarei Harbour. In 2005 Northport procured another design and build (D & B) contract with Fletcher who engaged Beca as designer to extend its berth operation by 180m and reclamation by 30,000m². Construction using the jack-up barge previously used for Berths 1 and 2 was not possible as the spuds were not long enough as the harbour had been dredged from around -7 mCD to -13 mCD during construction of Berths 1 and 2. The D & B team therefore developed a new design and construction methodology for the wharf extension comprising of twin sheet pile walls with two tie levels. The design methodology, construction and post construction monitoring are detailed in this paper.

2 STRUCTURAL FORM

Design requirements included a design dredge depth of -16 mCD, a 180m long berth, an integral back wall to provide retaining support to the adjacent reclamation, surcharge loadings of 50kPa for general port and container operation, and provision for supporting a future container crane. The design adopted comprised twin HZ775A "king piles" located at 1.79m centres with intermediate AZ26 sections for the front wall and lighter AZ17 sections for the rear wall. The two walls are tied together using two levels of steel rods attached to each king pile. The upper ties are 52mm in diameter and positioned at -2m CD and the lower ties are 80mm diameter steel bars positioned at -10 mCD. The wharf deck is a reinforced concrete flat slab spanning between the two sheet pile walls, creating a third top tie between the two walls. Two rows of steel ladder anchor strips 20m long are also attached to the rear sheet piles to reduce lateral deflection under load. The steel ladder anchor strips consisted of two steel reinforcing bars 230mm apart running perpendicular to the wharf with short lengths of reinforcing bars 300mm long at 300mm spacing over the end 15m connecting the two main bars. Figure 1 shows the typical wall cross-section.

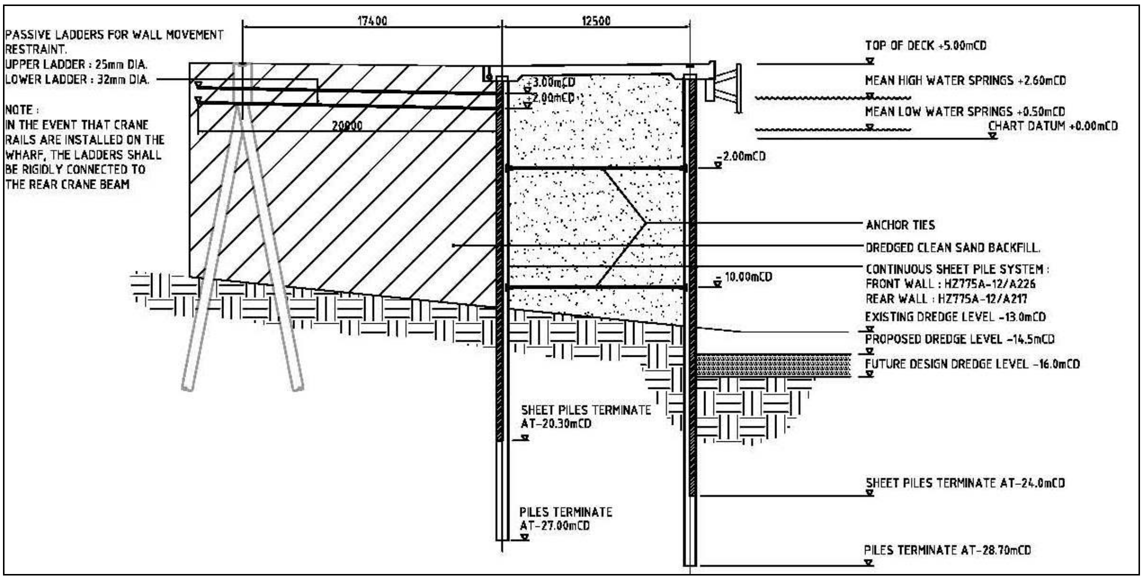

Figure 1: Berth 3 typical twin sheet pile wall cross-section

3 SOIL PROFILE

Geological Map Sheet 2A Whangarei describes geology of the Marsden Point area as comprising Quaternary aged foredunes, sedimentary deposits of undifferentiated sands, silts and clays with some peaty areas. Basement rock greywacke and argillite exist at large depths in the area and have not been encountered in investigations at this site.

A number of site investigations have been undertaken in the harbour for the proposed development. The soil profile encountered in boreholes below Berth 3 comprises interbedded layers of silty sand, sandy silt, shelly sand and silt clay layers. A very dense sand layer (SPT 'N' >50 blows/300mm) was encountered at the western end of Berth 3. Investigations indicated soil strengths to be variable immediately above this layer ranging from stiff sand silt to very dense sand with SPT values varying between 11 and 50+ blows. The very dense sand layer thins towards the east, becoming non-existent at the eastern end of Berth 3. Initially two soil profiles, a stiff profile representing the western end and a less stiff profile representing the eastern end of the site, were modelled to determine the effect of and sensitivity to the founding materials. The less stiff model was found to be critical and therefore selected for detailed design analyses.

Table 1: Soil Profile

Depth (mCD)	Layer	Description
-13 to -22	SILTY SAND/SANDY SILT	Loose; non-plastic.
-22 to -25	SANDY SILT	Very stiff, slightly to non-plastic.
-25 to 36	SILTY SAND	Medium dense; non-plastic.
-36+	SILT, some clay	Very stiff to hard; moderately to highly plastic.

4 DESIGN ANALYSIS

The sheet pile caisson structure was modelled using the numerical modelling software FLAC. FLAC incorporates both soil and structural elements, enabling the interaction effects between the sand internal fill, backfill, insitu material and the structural elements of the wharf structure to be modelled. FLAC was used to calculate bending moments and shear forces within the piles, axial loads within the tie and ladder elements, to predict the displacement of the sheet pile and expected movements within the sand fill.

The soils were modelled using an inbuilt plasticity model with Mohr Coulomb strength parameters plus elastic stiffness parameters, input in terms of Bulk Modulus and Shear Modulus. All analyses were undertaken using effective stresses. All material above the toe of the structure was modelled as non-cohesive, with increasing stiffness over depth to take account of increased confining pressure. Soil stiffness is dependent on the effective confining stress and as such the soil stiffness (and interface stiffness) was increased to model the effect of the filling as construction proceeded. The stiffness profiles adopted for all stages are given in Table 2.

Table 2: Design Soil Parameters

Layer	Density (kN/m3)	Cohesion (kPa)	Friction Angle (degrees)	Elastic Modulus (MPa) (Prior to Filling)	Elastic Modulus (MPa) (After Filling)	Poisson's Ratio
SAND FILL (Internal)	17	0	30	N/A	50 to 80	0.3
SAND FILL (Reclamation Fill)	17	0	30	N/A	17.5 to 32.5	0.3
SILTY SAND/ SANDY SILT	17	0	28	10 to 20	20 to 30	0.3
SANDY SILT	18	0	34	35	45	0.3
SILTY SAND	18	0	33	35	45	0.3
SILT, some clay (Below Pile Toe)	17	5	32	500	500	0.3

The sheet piles, ties and wharf deck were modelled as elastic beam elements with appropriate end conditions and the steel ladders modelled as cable elements. The uncorroded sheet pile section was modelled for the construction phases and a corroded sheet pile section used in the final stage.

The construction methodology and sequence was found to be critical in determining wall and tie demands. Variations on the construction sequencing were modelled and the following sequence was selected:

- Define geometry, soil properties and build structure (sheet piles and tie bars).
- Excavate to -14.5mCD in front of the sheet pile wall (Stage 1 design dredge level).
- Fill internally between sheet pile walls.
- Fill behind sheet pile walls and install steel ladders (consecutive stages fill to ladder, install ladder +2 mCD fill to next ladder, etc).
- Model tidal lag then build wharf deck.
- Apply 50kPa surcharge behind rear sheet pile wall then to wharf deck.
- Excavate to -16.0mCD in front of the sheet pile wall (Final design dredge level).

- Remove 50kPa surcharge
- Fix back of steel ladders into crane beam and apply 50kPa surcharge.
- Apply crane loading (loads transferred in wharf deck into sheet pile sections).
- Reduce sheet pile wall stiffness to model 50-year corrosion case.

5 FLAC RESULTS

Expected wall deflections have been predicted using FLAC for the various construction stages. The deflections of the wharf deck have been analysed for both a 50kPa and 35kPa live surcharge. The 50kPa load case is considered an extreme situation. The 35kPa surcharge is the serviceability case allowing for the likely maximum surcharge load taking into account layout, access ways and stacking weights. Table 3 shows the predicted movements during construction stages and surface loadings.

Table 3: Estimated Horizontal Displacements

Stage	Description	Estimated Front Sheet Pile Horizontal Displacements (mm)			
		Serviceability Load Case		Ultimate Load Case	
		Total	Incremental	Total	Incremental
4	Fill behind structure to RL5.0m	146	-	146	-
5	Tidal lag	151	5	151	5
6	Build wharf deck	154	3	154	3
7	35kPa/50kPa surcharge	182 (35kPa)	28	198 (50kPa)	44
8	Dredge to RL-16.0m	210	28	227	29

These results indicate the structure is likely to move 30mm under the area wide application of 35kPa surcharge and up to 45mm under the ultimate 50kPa surcharge.

6 CONSTRUCTION

6.1 General

The construction of Berth 3 commenced in 2005 from the end of Berth 2. One of the challenges of the construction was access. A 200 tonne crane initially located on the end of Berth 2 was used to install the first driven piles for the wall. A platform system was developed by Fletcher using shallow steel bracing and sand fill to enable the crawler crane to advance over a 13m depth of water on the previously constructed length of wall. To enable the king piles to be driven at their correct location, a piling frame was constructed. This frame was supported by temporary locator king piles driven outside the works and the king piles were driven in sets of 7 at each set up. Intermediate sheets were driven following installation of the king piles. The temporary piles were removed and reused as the construction proceeded. The 33m long driven king piles were driven in one continuous length to minimise the requirement for onsite splicing. At the time of construction, these piles were the longest piles in Oceania to have been driven in a continuous length. Figure 2 below shows the installation of the sheet piles from the temporary platform system, dredging and reclamation process.

The construction generally proceeded in accordance with the proposed sequence outlined in Section 4. However, the dredging to -14.5mCD was carried out by a separate contractor and was completed between installing the wall and constructing the wharf deck, and not necessarily prior to infilling between the sheets or backfilling behind the wall as originally designed. The modified dredge sequence was modelled and resulted in a relatively small change in long term predicted lateral deflections.

Figure 2: Construction of twin king pile wall at Berth 3

6.2 Deflection Monitoring

The wall deflection behaviour was monitored during construction using inclinometers installed on the front sheet pile wall and surveying of the top of the front and rear sheet pile walls. Three inclinometers NP54, NP74 and NP102 were installed along Berth 3. These were positioned towards the eastern end as the soil profile was less stiff and where higher deflections were expected. Inclinometer casings were installed in steel pipes welded to the front of the king pile which were driven with the king pile. Inclinometer casing was installed in the steel pipe prior to infilling between the walls with sand. The inclinometers have been monitored during construction and re-surveyed one year following completion of the berth structure. Figure 3 below presents the measured deflections.

The measured deflections at the top of the front wall were within the predicted range; however the deflected shape is different, with less displacement measured at seabed than predicted. The inclinometer readings indicate the actual soil stiffness below -20mCD is significantly higher than that modelled with measured deflections of less than 5mm recorded compared to the predicted deflections of 20 – 50mm. Subsequent back analyses were undertaken, matching the predicted displacements with the measured displacements, to find the actual stiffness of the founding material. These analyses indicate the stiffness is in the order of 500 MPa compared with the 35MPa adopted for design. This is considered to be attributed to the piles at the being founded within the very dense sand layer encountered towards the western end of the site and actual strains being much lower corresponding to a greater stiffness.

In addition, the twin wall system above seabed is less stiff than predicted, this is probably due to the combination of the degree of compaction of the backfill between the sheet pile walls, lower degree of confinement provided by the walls and the ladder anchors creeping, allowing more movement than expected. It is noted ultimately the ladder anchors are to be connected to the rear crane beam to provide additional support and reduce differential movement between the two crane beams.

Measurements taken 1 year following construction indicate the wharf has shown about 20 - 30mm of further displacement. This is considered to be due to the result of the live surcharge from port operations and the additional loading during pavement construction.

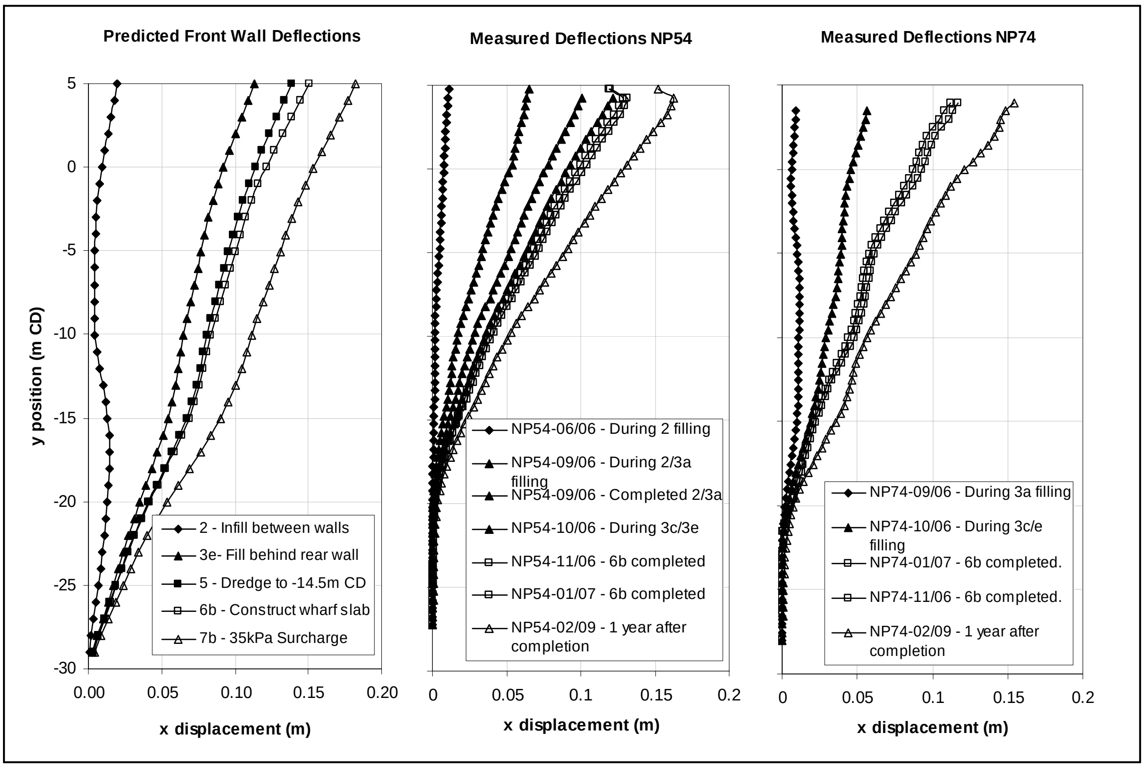

Figure 3: Predicted and measured wall deflections

7 CONCLUSIONS

The Berth 3 extension to Northport's wharf facility at Marsden point was successfully developed by Beca and Fletcher. The wharf structural form, comprising twin king pile walls tied together with two levels of steel ties and sand infill, allowed construction access by walking the crane over the constructed wall.

The design analyses were undertaken using FLAC to model the soil structure interaction. FLAC provided a tool to model the twin wall interaction and friction anchor ties within the sand backfill, which is typically outside the capabilities of standard retaining wall software. Analyses output included design loads for structural elements, deflected profiles for the walls and soil for each stage of construction. The model was sensitive to the soil stiffness properties.

Deflections during construction were successfully monitored using inclinometers attached to the king piles. The survey monitoring indicated total displacements were in the order of that predicted, however the overall deflected shape differed indicating sand on the harbour or passive side to be stiffer than that modelled, and twin wall system with ladder anchors to be less stiff than that anticipated.

8 ACKNOWLEDGEMENTS

The writers would like to thank The Fletcher Construction Company Ltd and Northport Ltd for permission to publish this paper.