THE DESIGN & CONSTRUCTION OF THE CLYDE QUAY WHARF BASEMENT

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SUMMARY

In 2004 the Wellington Overseas Passenger Terminal (O.P.T) was a dilapidated, earthquake-prone, white elephant. Occupying what is arguably Wellington’s most desirable real estate, it was severely underutilised and a potential liability for its owners, the Wellington City Council (via Wellington Waterfront Ltd).

Following a development tender and a drawn out resource consent process, Willis Bond & Co developed a luxury apartment complex above wharf-level public space and a below-wharf, partially submerged carpark. Structural challenges included:

- Re-founding the existing wharf in ground that has some probability of liquefaction.
- Seismic strengthening the existing wharf.
- Designing and constructing a partially submerged, water-tight, basement carpark beneath an existing wharf deck.
- Repairing and encapsulating a 100 year old, severely spalled concrete structure.
- Designing resilience into a structure that has effectively a 12m “ground floor” storey height.
- A highly corrosive salt-laden environment.

The resulting building is a striking renewal of a prime city location, well prepared for a life in a harsh environment and achieving premium returns for the developer. This paper discusses the design and construction challenges and solutions.

BACKGROUND

Immediately prior to the redevelopment, the wharf structure consisted of three distinct parts of different constructions and conditions. The original portion of wharf dates from 1908 and was 170m long and 19.2m wide. This original wharf was constructed entirely of reinforced concrete with precast driven piles (driven to low tide level) and a cast-insitu wharf deck with a grid of beams and a 200mm (8 inch) deck slab. Somewhat unusually, there was a lattice of insitu concrete bracing linking all the piles between low tide level and the underside of the wharf deck. These lattice trusses provided lateral bracing. This portion of the wharf was designated as a heritage structure and, notwithstanding its condition, was a fine piece of engineering and early concrete construction.
In the early 1960s the wharf was widened and lengthened with the addition of a new wharf structure on both sides and a significant longitudinal extension. These additions were more typical of other Wellington wharfs with a concrete deck slab supported on a grid of timber joists and bearers which in turn are supported on driven timber piles. A number of the vertical piles were paired with raking piles to provide lateral bracing. The extended wharf dimensions were 256m long by 35.5m wide.

In the mid-1960s the terminal building was added, covering most of the original 1908 “concrete” wharf and extending along the extended “timber” wharf. It was essentially a concrete structure with a high-stud ground floor space and an upper first floor space with external access. The first floor structure was of reasonably robust concrete construction while the roof and the walls were generally lightweight. This 1960s redevelopment was intended to be Wellington’s international passenger terminal for ocean-going liners. However, its developers had failed to foresee the growth in air traffic and the accompanying demise of sea transportation. From its opening it was effectively a white elephant. Separated from the main working port, and effectively made redundant by a large reclamation which occurred around the same time as the new terminal was constructed, it became used as a small boat chandelling base and an occasional events centre.

By 2000, all parts of the structure were in poor condition. The original concrete wharf structure could be described as being in a parlous state. Nearing 100 years old, it was showing the ravages of exposure to the corrosive marine environment. The original wharf slab, deck beams and lattice bracing had been severely affected by spalling. The structure was pockmarked with previous repairs but in many areas these had since re-failed with cracking and exposed reinforcing common place. It was apparent that very little in the way of structural maintenance had occurred within the previous 20 or so years.

The original concrete piles themselves, below the tidal range, appeared to be in quite reasonable condition (because they are deprived of oxygen which is necessary for the formation of rust). The upper section of the piles were generally in better condition than the lattice bracing, but worse than the lower pile sections. Subsequent gravity (vertical) load tests, prior to the redevelopment, revealed that the concrete piles had significantly lesser load capacities than had been previously assumed. While they safely carried the wharf and
building loads, they had very limited ability to carry any additional loads. The timber wharf extension sections were generally in better condition, but also showed signs of maintenance deficit. All the piles were wasting (getting narrower) in the tidal zone, due primarily to tidal erosion. A relatively small number had also suffered severe rot and required immediate repair. The concrete deck and timber beams were generally in quite good condition, however, the connecting bolts were severely rusted. The terminal building’s primary structure was generally in reasonable condition however, the external secondary structure (roof trusses, balustrades and exposed decks) and roof cladding was showing signs of significant deterioration in the marine environment.

Analysis of the combined wharf structure in 2005, revealed that it was earthquake prone.

![Concrete wharf spalling](image1)

![Concrete wharf spalling](image2)

In 2004, faced with a potentially significant upgrade cost for an asset that provided only nominal income, the Wellington City Council, through its agency Wellington Waterfront Ltd, sought expressions of interest from development companies. Willis Bond & Co’s proposal, with a design by Athfield Architects was deemed to provide the best value and a development agreement was signed, subject to the gaining of a Resource Consent. Following extensive investigations and preliminary design, Resource Consent was applied for and granted in 2008. It was immediately appealed by local opponents but following a drawn-out Environment Court hearing the appeal was declined. A detailed design process, incorporating extensive geotechnical investigations, pile testing and prototype sub-wharf construction was then undertaken, occupying the best part of two years. The project was then consented and subjected to two structural peer reviews, one for structural stability and one for basement durability and water-tightness. Demolition and piling plans were issued for construction in February 2012. Final Practical Completion was granted in July 2014.

**DESIGN**

**Existing Concrete Wharf Structure**

The existing concrete wharf structure consists of bents (grids) at 6.1m (20ft) centres. A typical transverse section (see Figure 4) consists of a row of 7 driven concrete piles spaced at 2.74m (9ft) centres. Diagonal lattice bracing and beams at mean tide level in each direction braced the structure. Typically the O.P.T building was supported on the central and two outer piles of each bent.
New Apartment Superstructure

Like the O.PT building before it, the new superstructure is built on top of the wharf, matching the 1908 concrete wharf bent set out. The superstructure is steel framed with an interspan flooring system spanning 6.1m longitudinally between grids, and the overall length of the continuous superstructure is 230m.

The superstructure portion built over the basement is accessed by 7 cores spaced at every 4th grid (or 24.4m). Each core provides access for up to 3 apartments per level, and provides lift access from the basement car park below. Beyond the basement, the superstructure for the northern portion maintains a similar configuration but is instead built over the 60’s timber wharf extension. The superstructure is entirely supported on a grid of new piles and beams slotted through, and fixed to, the existing wharf.

Longitudinally, the superstructure is braced by a central spine running almost the entire length. This spine consists of a concrete wall from wharf level to level 1, then a steel moment resisting frame above. Transversely, it is braced by pairs of concentrically braced frames, every 4th grid.

Basement Structure

The basement not only provides a secure and convenient place for the apartment owners to park their cars, but it was also key to providing the solutions to 3 critical structural problems:

- Seismic bracing: The basement forms a deep rigid box that provides fixity to the tops of the piles. This reduces the piles’ effective length, essential when the distance to sea bed below is up to 9m.
- Maintaining the historic bent set out: The basement structure could accommodate the transferring of column loads from the superstructure to the new offset piles. This allowed the new building set out to match the historic grid.
- Concrete wharf durability and repair: The basement encapsulates the original spalling concrete wharf and effectively isolates it from the harsh marine environment, dramatically slowing the rate of further decay. Once constructed, it also provided access to carry out the necessary repair to the wharf soffit.

The new concrete basement has been constructed under the existing concrete wharf and around the existing piles. It forms a U-shape that is capped off on top by the existing wharf slab. All of the existing lattice bracing and beams at water level were removed. Some of the braces were salvaged and reinstated back into the finished basement, as an indicator of the wharfs heritage.

New driven and bored piles have been placed alongside the existing piles to carry the weight of the new superstructure and to take lateral loads. The existing piles have been stitched to the new piles via a pile cap the full depth of the basement. This effectively re-supports the wharf allowing transfer of superstructure column loads to the new pile.

The basement slab is formed by a new 300mm thick one-way precast slab sections spanning between bents. This slab is supported by a cast insitu ‘stitch’ beam, supported on new piles. The stitch beam also surrounds the existing concrete piles as they pass through the basement floor. The perimeter side walls are 250mm thick and were cast insitu over a 600mm high precast kicker wall, built integral to the precast slab sections. At each lift shaft, a precast lift pit projects a further 1.5m below the basement slab level into the sea.

A central aisle for the basement car park has been created by the removal of the existing central pile within the basement space. The existing transverse beam was strengthened to span the new opening.
Transverse shear walls, each side of the central aisle, are spaced at every 4\textsuperscript{th} bent. These coincide with the concentrically braced frames in the superstructure above. Each bent has 2 cars parks each side of the central aisle, giving 100 in total.

**Seismic Design**

The wharf is laterally braced by the flexural action of the piles. The base of the piles achieve partial fixity into the ground below and the tops are fixed into the rigid basement box. Energy dissipation is by way of flexural yielding at the top of the piles, with only minor yielding expected at the base.

The seismic strengthening design was carried out with the aim of achieving at least 100\% NBS. New 1050mm and 900mm diameter bored piles, with sacrificial steel casing through the sea, take the majority of the seismic load. New 500mm diameter, bottom-driven steel tube piles, and the existing piles, provide additional damping.

The 1050mm diameter bored piles are arranged around the exterior of the basement at every second bent (12.2m), which coincide with the exterior side of the transverse shear walls. 900mm diameter piles are placed at the internal ends of the transverse shear walls. Bored piles were belled up 1.8m to provide axial tension and compression capacity. A geotechnical strength reduction factor of 0.55 was used (rather than the 0.85 cited in the building code) following interim advice as a result of the Canterbury earthquakes.

The existing concrete piles were found to be driven to fairly shallow depths, as such, yielding of these at the base is not expected. The top hinge zone of the piles has been confined by two methods. One, by a grout filled FRP carbon jacket, provided a slim line solution when adjacent space was tight. The other was with a concrete ‘doughnut’. This method was developed as a cost saving initiative compared with the use of FRP throughout. A reinforced circular concrete section was cast around the top of the pile. It provides confinement but does not make contact with the beam above, so cannot not significantly increase the flexural capacity. The small remaining section of pile, between the doughnut and the beam, was wrapped with FRP at low tide while it was out of the water.

Various earthquake modelling strategies were employed to model the building’s earthquake response. Initially modal and static analyses were performed on a typical bay of structure to ‘ball park’ the concept. These types of analyses were seen to have deficiencies which would mean they may not adequately represent the actual buildings response, particularly with respect to the ‘parts’ interaction between the wharf and superstructure.

Extensive soil / structure analysis was carried out using Winkler Springs models on each type of pile to obtain a graph of the nonlinear behaviour of the wharf. Using this data a Time History Analysis of a ‘simplified lumped properties stick model’ was performed to check displacements, the interaction between the wharf and superstructure, and to extract a ‘parts’ spectra to be used for a full 3D modal analysis and design of the superstructure.
Finally, a Time History analysis of a full 3D stick model representing the entire wharf was created, and subjected to 7 earthquake records. This was used to verify drifts, check torsional response, diaphragm flexibility, and sensitivity to pile length and soil types.

**Water Tightness Design**

Due to the difficulty of constructing between the water and the existing wharf, the chosen water tightness solution had to be highly compatible with the construction solution. Collaboration between the designers and constructors was essential. It was established early on that an externally tanked system would not be suitable in the harsh marine environment, leaving a water tight (inside out reservoir) concrete solution as the most viable option.

Post tensioned solutions where discarded due to the length of the basement, and restraint offered by the large diameter piles. A conventionally reinforced slab with high steel ratios to limit the crack spacing and width to acceptable levels was seen as the best way to allow some strain relief along the basement length.

17 x 5m sections of the basement slab with kicker walls were cast between bents in between the water line and the wharf deck. This on-site pre-casting process allowed a high level of quality control. A high degree of accuracy with respect to reinforcement and water stop placement was possible and the environment was sheltered, stable and moist – ideal curing conditions. These precast slabs were then lowered into the sea and connected via the stitched beam, which was cast at low tide.

The design was carried out to NZS3106:2009 (Design of Concrete Structures for the Storage of Liquids), for a Liquid Tightness Class of 2 (leakage to be minimal), and NZS3101:2006 for an exposure classification of ‘C’. Site monitoring was carried out to a CM4 level.

In addition to complying with the relevant New Zealand standards, the design was supplemented by using the CIRIA document C660 (Early Age Thermal Crack Control in
Concrete), as a more detailed guide on crack control. Generally, early age thermal cracking was mitigated by casting with low restraint to the concrete (precasting where possible), using 30% fly ash binder or 8% micro silica to reduce the heat of hydration, and the use of high reinforcement ratios.

Initially, a 28-day compressive strength of 50MPa with 30% fly ash was specified for the slabs to meet the durability requirements of NZS1303 table 3.1. This was later reduced to 40MPa at 28 days (or 50Mpa at 56 days). This reduction in Portland cement content and the apparent departure from the code was justified on the basis that it would reduce the heat of hydration and shrinkage. The slower strength gain of fly ash concrete also suggests that a 56 day strength, rather than a 28 day strength, specification is more appropriate.

Long term crack mitigation is also aided by the reduction of shrinkage strains due to immersion in the water, the relatively stable thermal environment provided by the sea, and solar shading provided by the outer wharf and the building over.

For the serviceability design case, crack widths were limited to 0.2mm for both durability and for leakage. A head of 1.2m of water was used in the design, including provision for king tides and sea level rise over the next 50 years.

For the ultimate strength case, the basement structure was designed to resist two tsunami load cases. One, where the basement is submerged and must exclude a 2.3m head of water, and the other in case the basement becomes flooded and is left holding 1.5m head of water. All construction joints are fully tied through with reinforcement. Submerged joints are doubly protected, both by an internal P.V.C water stop, and a hydrophilic water stop placed on the outside of the reinforcement.

Where the existing piles penetrate the basement stitch beam, double hydrophilic water stops protect the joint. To prevent potential leaks from water tracking up the inside of the existing piles through an unknown defect, the piles were sealed by wrapping them in FRP up to the high tide level.

CONSTRUCTION

A Challenging Environment

With the basement carpark structure suspended under the existing 1908 concrete wharf, the workface was essentially positioned in and out of the changing tide. The distance to the seafloor ranged from 6m to 9m, deepening towards the outer end. To create a safe working environment, some kilometres of suspended scaffolding was erected below the wharf. The dark workspace was then illuminated with water tight LED lights.

As the basement construction could be halted by heavy seas, wave screens were trialled early on, but these had only a limited effect. Instead certain activities were limited to particular wind directions and tides.

With the existing wharf being in a very dilapidated state at the commencement of construction, there was a requirement to concrete encase some of the existing timber piles
on the flanking timber wharf, which were to be used as a roadway throughout the project. Coupled with this, the existing 1908 concrete piles also only had limited axial capacities which necessitated extensive staging, involving steel beams and swamp mats, to allow the large and heavy piling plant to operate.

The most critical challenge facing the construction team, was the health and safety aspect inherent with working in the sea over such a large site. All workers had to be able to swim and lifejackets were compulsory. A buddy system was also developed along with a specialist under wharf safety induction. Rescue teams were trained and a rescue boat was launched every day to be at the ready if required.

**Demolition**

Extensive, selected, demolition had to be carried out to the existing concrete wharf structure to facilitate the construction of the new carpark structure. All of the original concrete cross bracing, and the grid of beams in the intertidal zone, had to be removed.

The demolition work required adherence to strict environmental measures as the majority of this work was carried out over, or in, the water. Special silt booms and rubble catchment methods were utilised. The most common demolition method utilized was diamond wire sawing of the existing cross braces and beams. These pieces were then floated to a position where they could be lifted by a mobile crane.

At the original wharf deck, cut outs were required at each pile location and also the location of the 7 lifts located along the length of the carpark. These openings also acted as personnel access points and removal points for the demolition waste.

With the impracticality of heavy plant operations below the wharf, there was a heavy reliance on manual jackhammer works. This was challenging as operations had to be phased with the tides. Waders and wetsuits were worn so work could continue as the water level rose.

**Piling & Strengthening to Existing Piles**

The new carpark scheme required 210 new piles (both driven and bored). The longer bored piles extended to a depth of 35m. The piling operation was carried out on staging beams and swamp mats due to the limited capacity of the existing wharf deck. To further complicate the piling, tolerances were extremely tight, with new piles wedged between the existing concrete and timber piles with millimetres to spare. Obstructions in and around the sea floor were also common, so divers with probes were sent down to the sea floor to survey before the pile casings were placed.

As the existing 1908 concrete piles were incorporated into the new carpark structure, they required an increase in ductility and resilience. The confining FRP jackets and concrete doughnuts were both positioned below the water line, and required divers to install. Specialist tremie concrete mixes and underwater grouts were used in their construction.
Initial prototyping and the trialling of methodologies established a ‘best practice’ for these works. To form the doughnuts around the piles, plastic formwork was utilised to great effect due to their durability in the seawater and ease of man-handling.

Precast Basement Slabs

Once the piling and demolition had been completed, the process of precasting the 17m x 5m basement slabs commenced. Each slab represented a bay between the existing pile grid lines. With the carpark nearly 200m long, there were a total of 32 slabs to cast. An earlier concept involved floating in thin precast slab soffit units and then pouring a thick insitu topping. However, realisation that there would be insufficient time to pour and finish a fly-ash mix, lead to the revised concept. The complexity to this solution was escalated by the limited headroom available below the original wharf deck. Once a table height had been established for the slab forms (above high tide level and clear of average wave swells), a distance of 1.2m was all that was on offer. In this limited space, workers set up the forms, tied a double mat of reinforcing, installed the PVC water stops, then placed and finished the concrete.

The formwork system comprised a series of steel framed and ply clad forms that were seated on a PFC ledger and bolted to the existing concrete piles. Once the slab had been cast and raised, the forms were pulled by a winch to the next bay, ready to repeat the casting process. To maintain the projects critical path programme, 2 sets of formwork were used.

The 300mm thick slabs also had a kicker wall cast along their two longitudinal sides. These walls proved challenging as they were cast integral with the slab, requiring perfect slump and vibration when casting in with the slab. Another challenging aspect was at the 7 locations along the carpark where lift pits were positioned. The lift pits were precast off site, transported to site, craned into position, and cast in with the basement slabs. As these pits hung below the slab tables, their bases were constantly submerged in the sea and were subject to rough weather requiring maximum restraint when pouring the slabs.
Lowering Slabs & Stitch Beams

Each precast slab had 8no. 32mm diameter VSL stress bars and anchorages cast in to suspend the slabs. The bars projected up through the existing wharf deck and through a series of strong back steel beams, which transferred the loads to the existing and new piles. The raising and lowering of the slab was performed with a synchronised hydraulic system, attached to barrel jacks at the 8 VSL bar locations.

This system was extremely sophisticated and could register and equalize load at each jack position. The first process after casting and curing of the slabs, was the raising of the slab 150mm to allow the formwork to be winched through to the next bay. After this the slab could be lowered into its final position.

The lowering of the slabs was subject to suitable weather (no sea swell) and had to coincide with low tide. The lowering needed to be exact, as the slab needed to align with the stitch beam formwork perfectly. Other issues to deal with were the projecting starter bars from the slab catching on existing piles, and new pile starter bars when lowering. This required a lot of pre-planning and a number of spotters were required during the lowering process.

Once each slab was lowered, it was essentially a race against the incoming tide to get the 100ton slab restrained ahead of the incoming sea. In a 2-hour window, a gang of 12 people installed up to 30 heavy duty props fixed off to the wharf deck to resist lateral loads from the waves and hydrostatic uplift from the incoming tide.

Once two adjacent slabs were restrained, the stitch beam between them could be poured. Incorporated into a typical stich beam were 2 new driven piles, 2 bored piles at every second grid, and the existing 7 concrete piles which were keyed in with drilled and grouted stainless steel dowels.
The formwork to the stitch beam was a 3mm thick sacrificial steel former, supported off the new and existing piles and installed prior to the lowering of the precast slabs. Techniques for its construction included riveting and underwater welding. The reinforcing steel in the beams was considerable and could only be positioned in the low tide window. The length of the main 32mm diameter primary bars also required them to be brought in with the aid of a raft, and be threaded from one end, stitching the two precast slabs together.

The stitch beam, like the slabs, had a kicker pile cap at each end which needed to be poured integrally. Pouring and finishing the stitch beam needed to occur during the low tide window of approximately 2-3 hours. The flow of concrete needed to be seamless to avoid missing this window, and there was always a lot of preplanning with the concrete supplier and pumping contractor. A microsilica mix rather than fly-ash was utilised here because of the greater spread of cementation.

Once the stitch beam was poured and cured, a 1.2m temporary cofferdam was installed across the slab to stop the incoming tide from entering the finished section of the carpark. This permitted the upper wall construction and ceiling remedial work to be carried out in a semi dry environment.

**Side Walls & Remedial Works to the Existing Wharf Soffits**

To enclose the sides of the basement, an infill wall had to be completed up to the underside of the original wharf deck. Extra special attention was required to these zones in regards to the water stop detailing and covers to the reinforcing, as historically this zone had represented the most degraded portion of the original concrete wharf.

Once the side walls were complete, the mammoth task of concrete remediation could begin to the underside of the 1908 wharf. The 3500m² basement ceiling required each area of spoiling to be broken out, sandblasted, and a heavy coat of zinc enriched paint applied immediately after.
Any areas on primary beams requiring repair were boxed and poured with 50MPa self-compacting concrete. Secondary beams and the underside of slab areas were sprayed with 50MPa sprayed concrete. A similar remedial process was used on the existing concrete piles.

Figure 11. The finished basement

Quality Assurance

As a condition of the building consent, a full and robust QA system was employed for the construction of the under wharf carpark. The system required full sign off for each individual task, full photographic documentation, as-builts of every PVC water bar weld, and locations and heights of all hydrophilic placements. The QA also required the tracking of strengths for all concrete poured in the basements construction.

Accompanying this QA system, a huge amount of prototyping took place prior to the contract commencing, with full scale mock-ups of slab edge detailing and stitch beam formwork being constructed at an off-site yard. Extensive research was also conducted on the PVC water bars, and on-site testing of the hydrophilic water stops was performed.

CONCLUSION

The successful completion of the Clyde Quay Wharf basement ahead of schedule, and with no leaking to date, was a result of meticulous pre-planning and collaboration between its Engineers and Constructors. Hidden from view, the carpark structure plays a vital role in the seismic performance of the wharf and the remediation of the existing 1908 concrete wharf.

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