NEW ZEALAND CONCRETE SOCIETY
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CONCRETE 2000

Better, Faster, Smarter

WAIRAKEI RESORT HOTEL, TAUPO
13 – 15 OCTOBER 2000

Technical Paper

NEW ZEALAND CONCRETE SOCIETY

CONCRETE '00
"BETTER, FASTER, SMARTER"

Technical Conference and AGM
Wairakei Resort Hotel, Taupo
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Conference Programme and Table of Contents

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Princes Wharf Development – Strengthening of a Reinforced Concrete Wharf and Superstructure

David J. Turkington[1]

Introduction

The commercial heart of Auckland City has been separated from its harbour for 100 years by its ports and wharfs. Apart from access to board ferries, the public has had very limited access to the waters edge. For too long there were only a limited number of cafés and restaurants where Aucklanders could enjoy the waterfront. This began to change with the sale of Princes Wharf to Kitchener Investments and their plans to develop.

The wharf development has now revitalized the waters edge. The public are no longer locked out. The wharf is now host to a number of cafés and restaurants including Leftfield where the TAB sports café is filmed live every Wednesday night.

The Wharf

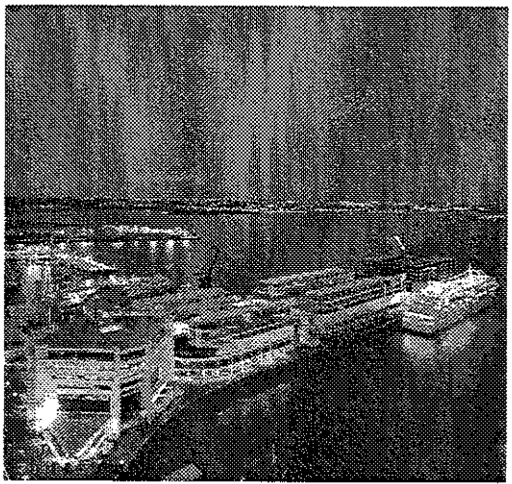

Fig.1 Princes Wharf

Princes Wharf was built in 1926 and was the "flagship" of the Ports of Auckland. A 340 metres long by 90 metres wide concrete structure, the wharf supported six two-storey concrete sheds that covered approximately 50% of the wharf area. Some 2000 No. 500 mm square reinforced concrete piles support the wharf and superstructure. A robust structure originally designed to support railway tracks and 34 kPa live loads.

The existing two levels of the 4 landward end sheds 19, 20, 22 & 23 would remain with the addition of 4 new floors constructed above. The two seaward end sheds 21 & 24 were demolished to wharf deck level to make way for 8 floors of new structure.

The redeveloped wharf now gives a "liner" appearance. The sharp end of shed 22 symbolises the bow and the seaward end the stern. Large circular portholes ventilating the car park floors, the masts on the roof, and stainless steel balustrades all provide for a nautical theme.

Five of the buildings are multi-use with restaurants and bars at the wharf level, apartments on the first floor, car parking on the second and third floors, and more apartments above.

The sixth building is dedicated to the Hilton Hotel containing 167 rooms. The hotel will also manage 80 serviced apartments in the neighbouring sheds. On the first floor is Auckland's Overseas Terminal, the Terminal will also serve as conference space for the hotel.

The entire development will contain almost 300 residential and serviced apartments, 167 Hotel rooms, 680 carparks and 14,500 square meters of commercial / retail space. 2400 Tonnes of structural steel and 9,000 m^3 of concrete has been used in the development.

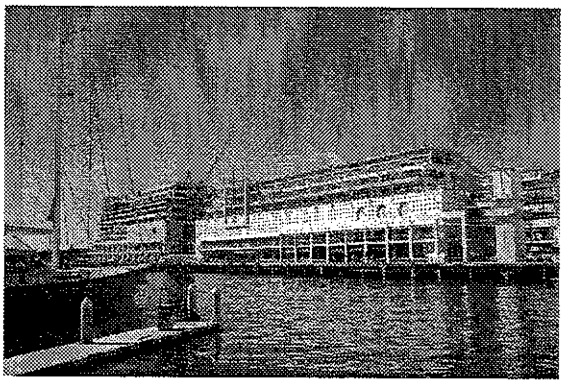

Fig.2 Shed 24 is pictured in the distance under construction. Shed 23 is in the foreground

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[1] *Buller George Engineers, Auckland*

Completion dates

Construction commenced October 1998

Sheds 19, 22 & 23 completed November 1999

Shed 20 completed July 2000

Shed 24 (Luxury apartments) late 2000

Shed 21 (Hilton Hotel) early 2001

Wharf Seismic Strengthening Philosophy

Seismic design provisions have changed considerably since the 1920's. The New Zealand Building Code requires that where a building's use is changed the building must comply with the present day Building Code as nearly as is reasonably practical to the same extent, as if it were a new structure.

The existing wharf piles contained no shear or confining reinforcement apart from spirally bound No. 8 wire for a length of 1200 mm at the head and toe. Additionally they were lightly reinforced with the shorter pile lengths only having four 22mm diameter plain bars, one in each corner of the pile. In assessing the displacement capacity of the piles, the lack of confinement, the behavior of plain bars and the low reinforcement ratio were all considered in the calculations.

It was recognised at the preliminary stage of design that limiting the lateral deflection of the existing vertical piles would be critical in any solution. A number of schemes to strengthen the wharf were considered.

Scheme 1.0 – Large shear wall structures. This scheme was excluded at an early stage due to the inherent construction difficulties and cost.

Scheme 2.0 – The addition of elastically responding groups of raking piles fastened directly to the underside of the wharf.

Scheme 3.0 – The addition of raking piles with each of the pile groups fastened to the wharf through an energy dissipator.

It was recognised that the third scheme had many advantages to that of the second scheme. These advantages included a 75% reduction in the raked pile loads, resulting in a more economic pile size and rock anchor configuration., It was also recognised that the elastically responding scheme had inherent problems when detailing for the high load levels.

Each raking pile group formed a rigid tetra pod. Pictured in figure 3 is a schematic drawing showing the four piles extending in to the seabed. These piles were constructed by bottom driving steel tubes into the seabed. A rock anchor was then installed through the center of the pile and embedded into the underlying sandstone. This provided a tension and compression pile. The steel piles were then filled with concrete. A pile cap was constructed supporting the energy dissipater and in turn is connected to the underside of the wharf.

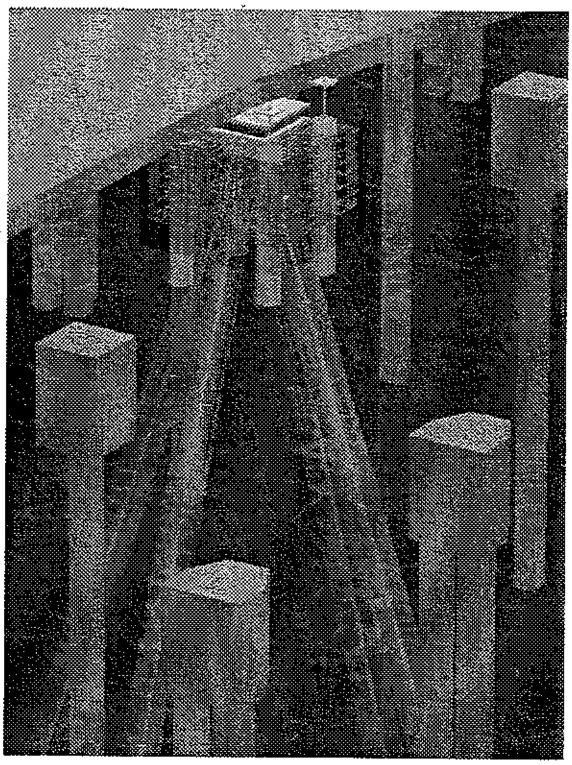


Fig.3 Raked pile arrangement

Existing lateral stability is provided by seawalls and raking piles at the northern end. The wharf would remain attached to these existing lateral support structures. This provided additional stiffness during serviceability loading.

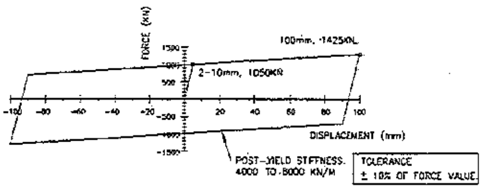
The seismic mass of the wharf is approximately 110,000 tonnes.

The resistance to ship berthing was also considered as a load case.

Energy Dissipators

Energy dissipators were sandwiched between the underside of the wharf and the top of the pile cap. These energy dissipators work like shock absorbers during an earthquake. When the wharf displaces 2-10 mm restoring lateral load of 1050 kN is applied through the bearing. If the wharf continues to move up to 100 mm the restoring force increases to 1425 kN. The same happens in the reverse cycle. The resulting "fat" hysteretic curve is an ideal mechanism for absorbing earthquake loads.

Traditional laminated lead-rubber bearings were chosen. These bearings comprised laminated steel-rubber layers with 4 lead plugs. A force-displacement curve was finalised with the supplier, providing acceptable wharf displacements as shown in figure 4.



PRINCES WHARF ENERGY DISSIPATOR
FORCE-DISPLACEMENT RELATIONSHIP.

Fig. 4 Force-displacement curve

The position of the raked pile groups (between the sheds in the central carriage ways) meant that minimal axial loads were applied to the bearings. Traditionally lead rubber bearings are vertically confined by high column loads.

A testing regime was developed simulating this unconfined condition. Prototype testing was less than ideal. The area under the force-displacement curve was less than the design value. The recorded shear forces were also greater than the design values.

This resulted in a 27% increase in the wharf displacement and higher loads in the raked piles. This was not acceptable.

To assist in overcoming the problems with the bearing performance at zero axial loads and high shear forces a number of modifications were made. This included testing the bearing under a vertically confined condition and reducing the stiffness of the bearing rubber to help reduce the shear strengths. To achieve this constrained case on site stressing bars were detailed in the pile cap to provide this vertical constraint as shown in Figure 5.

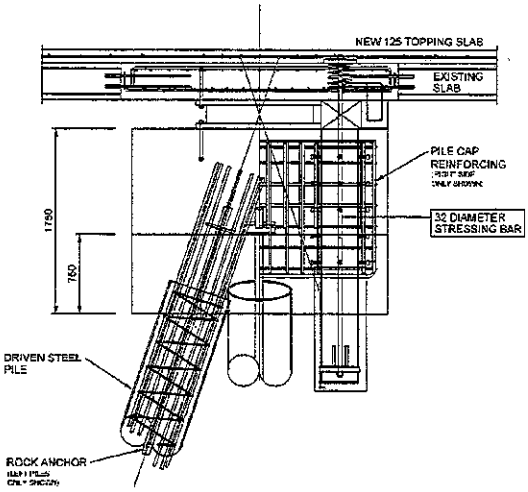


Fig.5 Pile cap showing stressing bars

The design displacement of the wharf was 90 mm. The influence of the indeterminate fixity provided by the sea walls and existing raker piles was considered. This resulted in significant torsional behaviour of the wharf and increased the maximum wharf displacement by 30%.

Existing Pile Vertical Load Capacities

The assessment of the vertical load capacity of the piles involved a physical and empirical study. Initially the vertical load capacity was based on the weight of the existing wharf and original live load capacities of up to 34kPa. These live loads were no longer required which allowed for the addition of new floors.

Existing pile records and Geotechnical investigative work confirmed initial load assessments.

Empirical studies of the lightly reinforced piles were carried out to confirm the pile cross-sectional capacity. Moment magnification methods were used to verify buckling capacities. The lack of confining steel was considered with regard to the axial stress and potential creep of the concrete.

On consideration of the above an ultimate load limit of 1060 kN (4 MPa) per pile was set. This equated to approximately 75% of the original ultimate design loading of the piles.

Finally on site load tests were carried out with up to 180 tonnes of mass placed on individual pile groups with time dependent deflection monitoring. This confirmed the above empirical studies.

Shed Structures

Shed analysis

Seismic base shears for the shed structures were derived from an upper bond condition. This considered the case of the wharf being rigidly fixed to the seabed by way of the existing raked piles and the seawall structures. The wharf deck was considered to be seismic ground. The sheds were analysed as nominally elastic responding structures.

Landward End Sheds

The existing two levels of insitu concrete shed structure required seismic strengthening. 200mm thick insitu concrete shear walls were proposed. These walls provided flexibility in position and were economical and easy constructed.

The number of these shear walls were determined by their reactions onto the existing piles. A number of these walls were detailed with openings to meet architectural constraints.

The existing two levels of concrete columns were deficient of transverse reinforcing, so a composite fiberglass wrap was used to retrofit the columns. This increased the confinement of the columns, which enabled an increase in axial load carrying capacity.

The additional four levels on top of the existing required a lightweight scheme to minimise the additional seismic mass being introduced to the wharf system and the vertical load capacity of the existing piles. A two-way moment resisting frame was adopted, which comprises circular hollow column sections and universal beams. The CHS columns were filled with concrete to eliminate the requirement for passive fire protection. The concrete core also dissipated forces generated in the joint zone.

Traditional steel braced or K-brace structures were considered but resulted in higher than allowable pile reactions.

A capacity designed joint was detailed as shown in Figure 6.

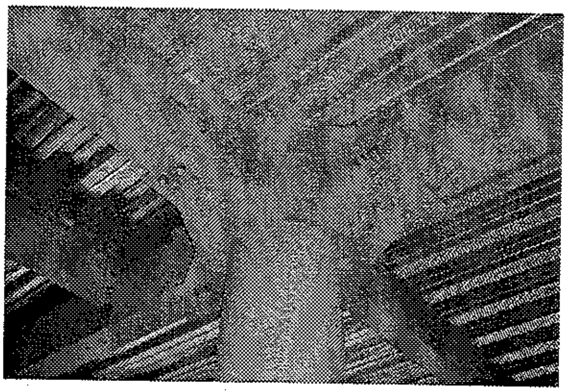


Fig.6 Moment resisting joint

Seaward End Sheds

The two seaward end sheds introduced a different set of constraints due to the extra two floors (eight floors per shed) and the fact that the existing vertical piles were the longest beneath these two sheds.

Three proposals were considered.

- The shear wall option similar to that used in the previous shed strengthening resulted in an unacceptability large number of walls. This was not desirable because of the requirement for large open areas and views over the water.
- A proposal comprising construction of new insitu concrete frames cut into the existing structure to provide lateral support to the lower two existing levels. This option was considered too expensive and impacted on the program.
- An option to demolish the existing two levels of existing structure and construct 8 new floors of structural steel moment frame. This option provided many

advantages and was considered to be the best solution.

The moment frame provided maximum flexibility in the open restaurant and commercial space. And also provided acceptable column reactions which were limited by the existing pile capacity, thus also excluding K-braced type structures.

Construction of the Hilton Hotel (Shed 21) continued overhead while the Passenger Terminal on the ground and first floor operated during the 1999-2000 passenger liner season, disembarking and boarding passengers. The Passenger Terminal was completed well before the structure above was completed.

Flooring systems

The existing 6.1m square wharf grid enabled a number of flooring systems to be investigated. The design required a lightweight system.

After consultation with the builder it was proposed to use a proprietary hi-bond floor system. This comprised a secondary 250UB beam mid-grid. 0.95 gauge tray was specified enabling an unpropped construction.

Lightweight topping slabs & rib-raft

The existing Wharf deck encompassed by the sheds required leveling. Build-ups of up to 500mm were required. A proprietary rib raft solution was sought and proved economic and ideally suited for this application. The large polystyrene void formers reduced the total weight considerably.

The first floor apartment level also required a leveling floor screed of up to 120mm. A lightweight un-bonded concrete screed was used.

Two floor high colonnades simply sit on the wharf deck. Void formers have been used to reduce the colonnade weight.

Claddings

Durability and weight was paramount when considering the cladding type. GRC (glass reinforced concrete) was proposed. A ribbed panel design was developed which eliminating the need for the traditional DuraGal backing frame. This eliminated any potential durability concerns and ongoing maintenance costs. 250mm square "apertures" and portholes were formed in the GRC to clad the car parking floors. The rib type construction is pictured in Figure 7.

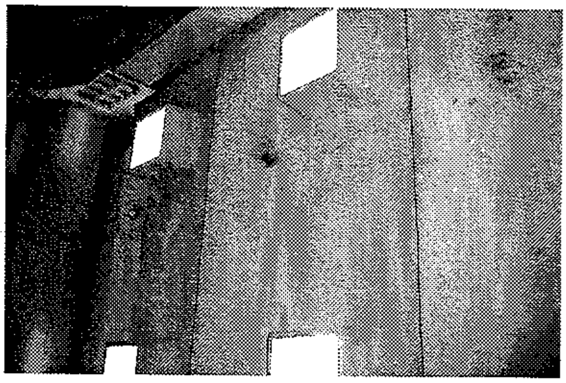


Fig.7 GRC – Rib type construction

The ribs were formed using polystyrene void formers.

Traditional DuraGal backing frame type construction was also utilised where it was protected from the environment.

Bridges

Seven bridges link all six sheds together. Traffic enters shed 23, circulates through all the sheds via the link bridges, and exits through shed 20.

The bridges are supported by elastomeric bearings fixed to the second floor of the sheds. These bearings provide for thermal and seismic separation.

Figure 8 shows one of these bridge structures linking sheds 22 & 23.

Fig 8. The link bridge connecting sheds 23 & 22.

Conclusions

Existing well constructed concrete wharf structures can be successfully strengthened to a level where they comply with the current building codes. This demonstrates the durability and integrity of reinforced concrete.

The use of a new raker pile system fixed to the underside of the wharf via energy dissipaters proved to be an economic solution in the strengthening of the Wharf structure.

The solution to minimize the lateral displacement meant that no structural strengthening was required to the 2000 No. existing piles. Positioning the raked piles in the carriageway between the sheds enabled the adjacent shed construction to commence without delay.

The choice of strengthening the existing two levels with concrete insitu shear walls also proved to be an economic and practical solution. These walls provided for enough flexibility in position to meet the architectural requirements. These walls were also able to be detailed with openings to offer further flexibility to the Architect.

Concrete filled steel hollow steel sections reduced the requirement for passive fire rating and the two way moment frame provided acceptable reaction loads in the existing piles.

The choice of materials was paramount given the marine environment of the wharf.

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2. NZNSEE (1996), *The assessment and Improvement of the Structural Performance of Earthquake Risk Buildings',* Draft June 1996