

# Piling for Extensions to the International Terminal Building at Sydney (Kingsford-Smith) Airport

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**Summary** The International Terminal at Sydney (Kingsford-Smith) Airport is located on a geologically complex site, with dredged sand filling overlying deep alluvial sediments. The design and construction of foundations for numerous extensions to the original terminal building has been further complicated by the need to comply with vibration, noise and settlement criteria, that satisfy existing structures and day-to-day airport operations. This paper presents a review of the site investigation, design, construction method and foundation testing undertaken on two specific extensions to the terminal building with reference to these conditions.

## 1 INTRODUCTION

The continuing development of the International Terminal at Sydney (Kingsford-Smith) Airport has provided many challenges for foundation design and construction. Extensions to existing buildings are regularly undertaken and often utilise the same foundation type as the existing structure. However, in this project the original foundation type is unsuitable for construction in close proximity to buildings, and other foundation types that satisfy vibration, noise and settlement criteria in a complex geological environment must be considered.

## 2 HISTORY OF SYDNEY AIRPORT

Sydney (Kingsford-Smith) Airport started in the early 1920s as a private aerodrome in a low lying swampy area beside the Cooks River on the northern foreshores of Botany Bay. Since conception the airport has undergone extensive changes including a diversion of the Cooks River, construction of terminal facilities, extension of the north south runway into Botany Bay and construction of the third runway.

The International Terminal is located in the northeastern sector of the airport, with parts of the terminal building situated over the former Cooks River channel. Filled with dredged sand during the 1950's, the river is now located west of the terminal building.

Prior to construction of the terminal building, this area was used for dumping of flyash from a local powerstation. This material was removed and replaced with dredged sand in the early 1960s, and surcharged with up to 2 m of sand for 2 years in an attempt to reduce the consolidation of underlying soft soils. Review of settlement data over the past 30 years

indicates that this surcharging was not successful in eliminating settlement, albeit reducing it to an extent.

From 1967 to 1970, the International Terminal and Concourse (Terminal B and Pier B) was constructed, comprising a three storey prestressed concrete structure. Construction of a southerly extension to the terminal and new concourse (Terminal C and Pier C) commenced in 1988, comprising a similar structure with the addition of two basement levels to the new terminal and associated surrounding aircraft pavements. The latest development, "Sydney Airport 2000" commenced in 1998, and incorporates a variety of new expansions. These include northerly and southerly extensions to Piers B and C respectively, westerly and easterly extensions to the terminal building, upgraded road network including elevated overpasses, extensive expansions to existing aprons and a number of smaller associated projects.

## 3 GEOLOGY OF SYDNEY AIRPORT

The regional geology of the International Terminal area consists of dredged sand overlying quaternary alluvium. Extensive investigation of the area has identified a number of distinct layers:

- sand filling dredged from Botany Bay, generally medium dense to very dense up to 4 m thick
- soft organic clay up to 10 m thick, undergoing consolidation due to filling and now firm in some locations
- dense or very dense silty sand up to 3 m thick
- stiff or very stiff sandy clay
- dense sand between 0.5 and 4 m thick
- very stiff clay possibly of residual origin, and
- Hawkesbury sandstone bedrock at depths generally ranging between 25 and 50 m, occasionally up to 80 m

The lithology at the location of the terminal building is complicated by the fact that it was constructed over the former river channel, as shown in Figure 1. Geotechnical

investigations at the site refer to the possibility of deeply incised river channels, filled with soft and loose soils in recent geological times.

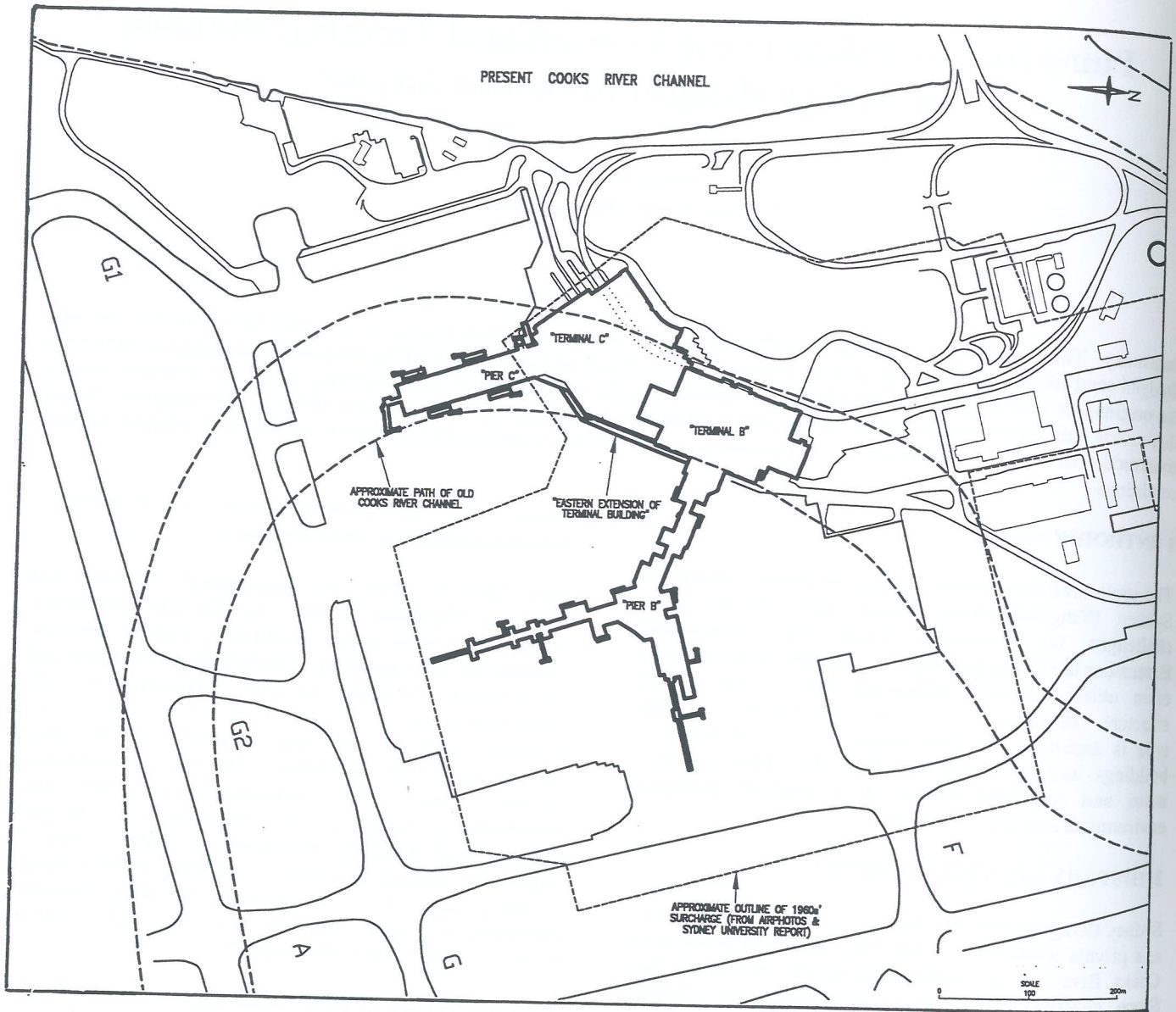


Figure 1: Location of terminal building and former river channel

#### 4 INTERNATIONAL TERMINAL AND CONCOURSE

The original terminal building and concourse (Terminal B and Pier B), was constructed following a number of small geotechnical investigations and an extensive surcharging and testing program undertaken by the University of Sydney. The failure to induce the majority of consolidation in soft clays over a 2 year period by surcharging, has led to the preclusion of shallow footings under moderate loads at this site.

As a result the original terminal building and concourse are founded on tar epoxy coated steel H-piles driven to rock.

Founding depths are in the range 24 to 36 m. Providing a simple cost effective construction method, these piles were driven by a 7 tonne hammer falling from a maximum drop height of 20 m.

A silico-iron anode buried approximately 9 m below the ground provides cathodic protection to most of these piles and buried service pipes. Subsequent monitoring of piles not protected cathodically indicates no structural deterioration after 30 years of service

## 5 CASE STUDY 1 - TERMINAL C AND PIER C

The Terminal C and Pier C expansion comprised a five storey reinforced concrete southerly extension to the terminal building, with a two level fully tanked basement and three storey steel concourse.

### 5.1 Geotechnical Investigation

The initial investigation comprised 5 bores and CPTs installed on a 100 m square grid over the site, as a part of a larger investigation of the airport. Results of this work indicated the presence of a 2 m thick dense sand layer at approximately 11 m depth beneath some of the site. This layer was suitable for founding enlarged base piles, but it was necessary to prove the thickness and continuity of the layer. Therefore in order to confidently design piles, further investigation was suggested. The Stage I investigation comprising 70 bores and CPTs highlighted the variability of the aforementioned sand layer towards the centre of the concourse and north of the terminal, but provided reasonable data for cost estimation and design. Further investigation was recommended at all column locations where this dense layer was not present, and less frequent testing in other areas. As a result, the Stage II investigation incorporated 77 bores and CPTs, and facilitated the design of piles based on three areal classifications, as shown in Figure 2.

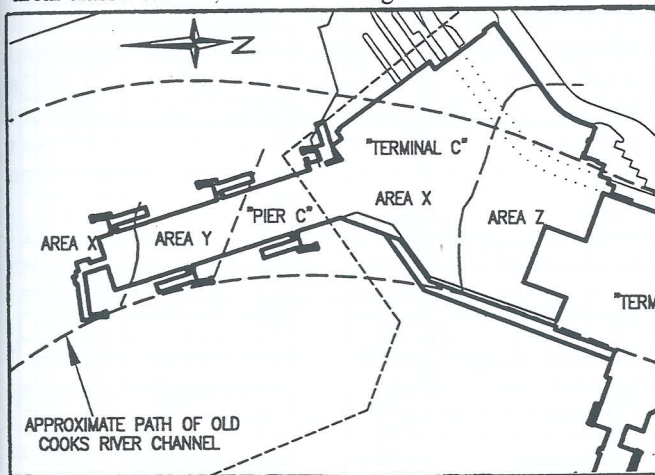


Figure 2 Areal classification derived from geotechnical investigation

### 5.2 Pile Design

The primary concern for design of piles on this project was the support of a compressive serviceability load of 1800 kN and restraint of a maximum possible tension load of 630 kN in the terminal area. The proposed foundation design for the terminal building was a beam and slab system, with piles supporting/restraining the beams at 2.5 m and 5 m intervals on longitudinal and transverse beams respectively.

For the concourse, compressive serviceability loads of 2000 kN per pile and no tension loads were expected.

Differential settlements between the existing building and new extension also had to be within a 15 mm tolerance.

### 5.3 Piling Method

Selection of pile types was based on the areal classifications (Figure 2) determined during the investigation stages of the project.

Area X, with its consistent dense sand layer at 11 m depth was considered suitable for founding of cast insitu enlarged based piles. A deeper dense sand layer at approximately 16 m depth in Area Y, was also considered suitable for founding of cast insitu enlarged based piles. However, the depth of this sand layer increased to 19 m in the area close to the terminal, requiring deep piling. Vibration effects in the existing structure precluded selection of driven piles, and as a consequence, grout injected piles or Atlas piles were suggested in this area. Area Z, with a 1.5 m very dense sand layer at depths up to 21 m was considered suitable for application of driven precast piles.

### 5.4 Pile Testing

Three tension pile tests and five trial penetration tests were undertaken to confirm the ultimate tension load capacity, penetration resistance and basing conditions for 500 mm enlarged base piles, prior to commencement of construction.

Tension pile construction was typical for a pile of this type with the exception that tension capacity was developed by replugging the tube and driving it through the first base enlargement to a lower level so a second base enlargement could be carried out. The pile shafts were then poured to level similar to the proposed basement level, so as not to contribute shaft resistance in the upper 4 m of the pile. Trial driving was undertaken by driving the pile tube to depths of 12 m where a variety of base combinations similar to the tension piles were attempted. Upon completion of basing the pile tube was withdrawn, while recording the load required to pull the tube. A summary of depth and basing details for the three tension piles is shown in Table 1.

Test Pile No.	1	2	9
<u>First Base</u>			
Depth (m)	11.5	10.5	11.5
Volume (m <sup>3</sup> )	0.14	0.70	0.14
Location	Just in sand	In firm clay	Just in sand
<u>Second Base</u>			
Depth (m)	12.3	11.8	13
Volume (m <sup>3</sup> )	0.14	0.28	0.28
Location	In sand	Just above sand	1.5 m in sand

Table 1 Summary of depth and basing details

The results of tension testing, shown in Figure 3 indicate ultimate uplift loads between 1100 and 1400 kN. However,

it should be noted that these tests were not an actual simulation of constructed piles. With basement level approximately 4 m below surface level, the decrease in overburden pressure was considered to reduce the tension capacity of the piles by up to 30 %. Theoretical estimates used in this capacity reduction could estimate the effect of overburden pressure reduction on lateral pressure, but no reasonable account of the densification effect while driving and basing the piles could be made. Overall the reduction was estimated unlikely to be greater than 15 %, but a value of 20 % was adopted to remove all doubt.

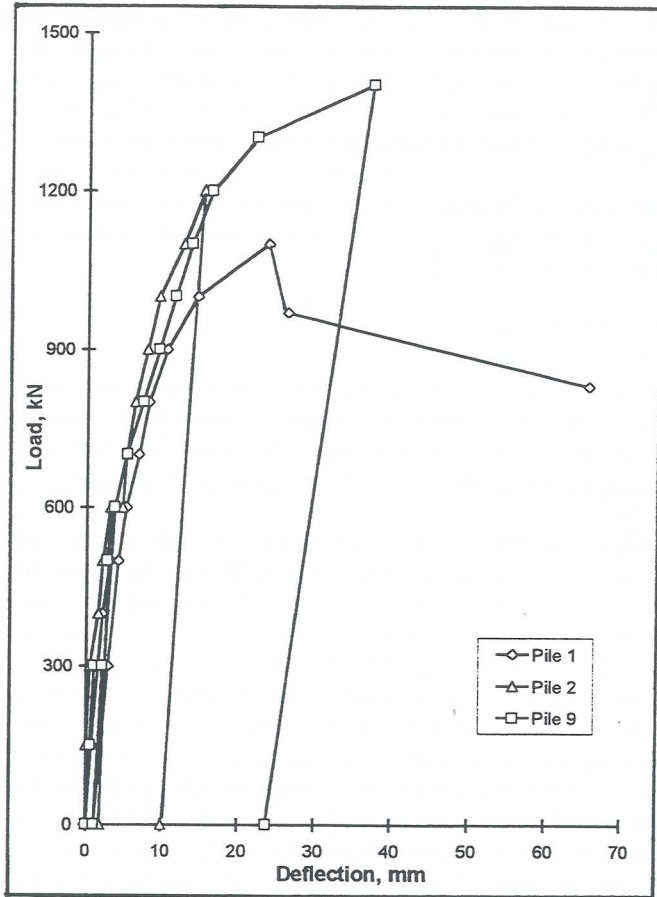


Figure 3 Load response of tension piles

From this it was concluded that test piles needed to restrain a test load of 750 kN. Given that the serviceability load was an extremely unlikely event, and should one pile fail the shedded load would be shared along the beam system and furthermore, pile testing was also to be undertaken during construction, a factor of safety of 1.5 on tension loads was considered reasonable. Hence, this required the piles to have an ultimate tension capacity of 1140 kN. With a modification of the construction procedure for the pile with an ultimate capacity of 1100 kN, the tension trials indicated that this capacity could be achieved.

## 6 CASE STUDY 2 - EASTERN EXTENSION OF TERMINAL BUILDING

The eastern extension, or "Clip On" is approximately a 10 m wide three storey structure extending along the entire eastern side of the terminal building between Piers B and C.

### 6.1 Geotechnical Investigation

Initial investigation of the site was undertaken to supplement data available from earlier investigations in the area, and determine founding conditions for piles. Investigation confirmed the presence of a dense to very dense sand layer at approximately 12 m depth at the southern end of the site. This layer was utilised in support of foundation loads in a large proportion of the Terminal C and Pier C expansion, however this layer had been eroded towards the north end of the site and ideal founding conditions for piles in this area no longer existed. The variation in soil conditions is illustrated in the CPT results for the south and north of the site shown in Figure 4.

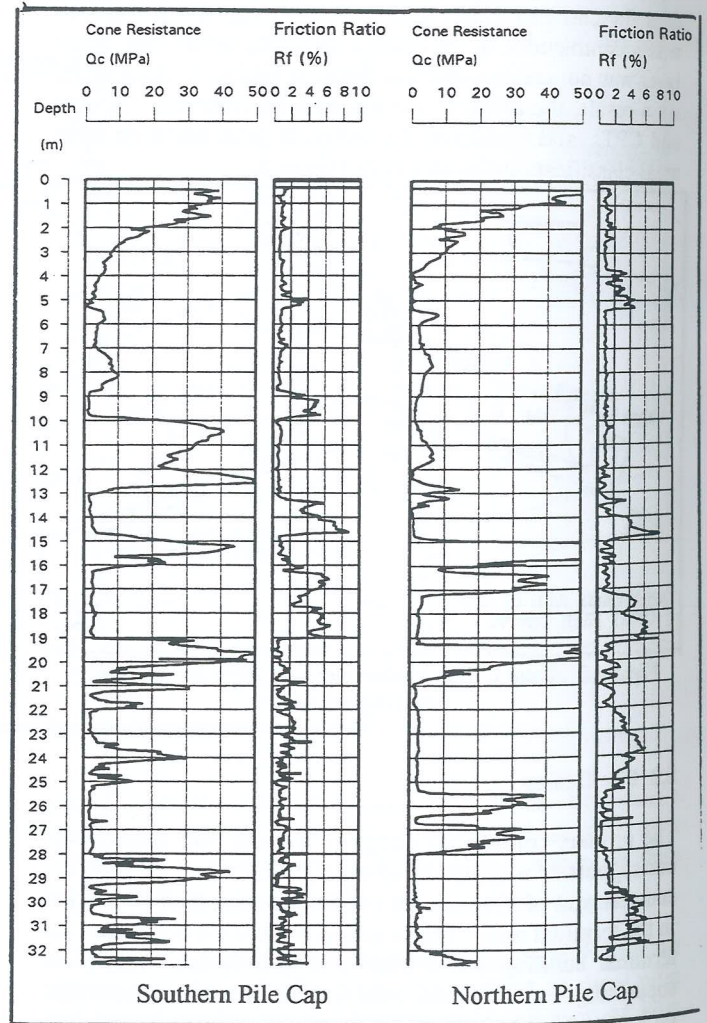


Figure 4: Typical CPT results for southern and northern pile caps

As a result, further testing was undertaken at each pile cap location prior to the commencement of piling.

### 6.2 Pile Design

Maintaining differential settlement between the existing building and new extension at tolerable limits was the primary concern for this project. Experience gained from earlier extensions suggested piles founded in soil could achieve acceptable differential settlements, when compared to the steel H piles beneath the existing terminal building. A serviceability load of 675 kN was adopted for each pile in a three pile group.

Driven piles were considered to involve unacceptable noise and vibration given the proximity to the existing building, and subsequently only drilled and jacked piles were considered. The final design adopted for construction was a jacked in precast 230 mm square concrete pile.

### 6.3 Piling Method

The "Grip-Pile", or "G-Pile" hydraulic jacking pile installation system utilises a Chinese developed technology where an electric hydraulic rig, ballasted to provide 100 tonnes thrust, pushes jointed 6 m pile segments into the soil until no visible movement of the pile is observed after 15 seconds at 100 tonnes load. This load represents 1.1 times the design action effect ( $S^*$ ) of 90 tonnes and simulates a pile test for every pile installed. Pile loads during installation were calculated from hydraulic pressures recorded by a geotechnical engineer from a calibrated gauge on the driving rig.

CPTs undertaken at pile locations indicated pile driving pressures consistently higher than cone resistance, as illustrated by a typical comparison in Figure 5

It has been suggested that this difference could be due to the increased frictional resistance of the pile, when its rough concrete surface is compared with the smooth metal CPT probe.

Variations between pile termination depths at the same pile cap were thought to be caused by the densification of soil by the first pile, leading to driving difficulties for subsequent piles. Piles were generally spaced at 350 to 400 mm centres, and installed as in-line groups of 3 or 4. Termination depths for the final pile were up to 30 % higher than the first pile installed. During construction, driving difficulties lead to the failure of only one pile, the third pile to be driven in a four pile group. This failure could not be directly attributed to the densification of soil around the existing piles because the toe of the pile was in loose sand at the time of failure. The pile was removed and later reinstalled

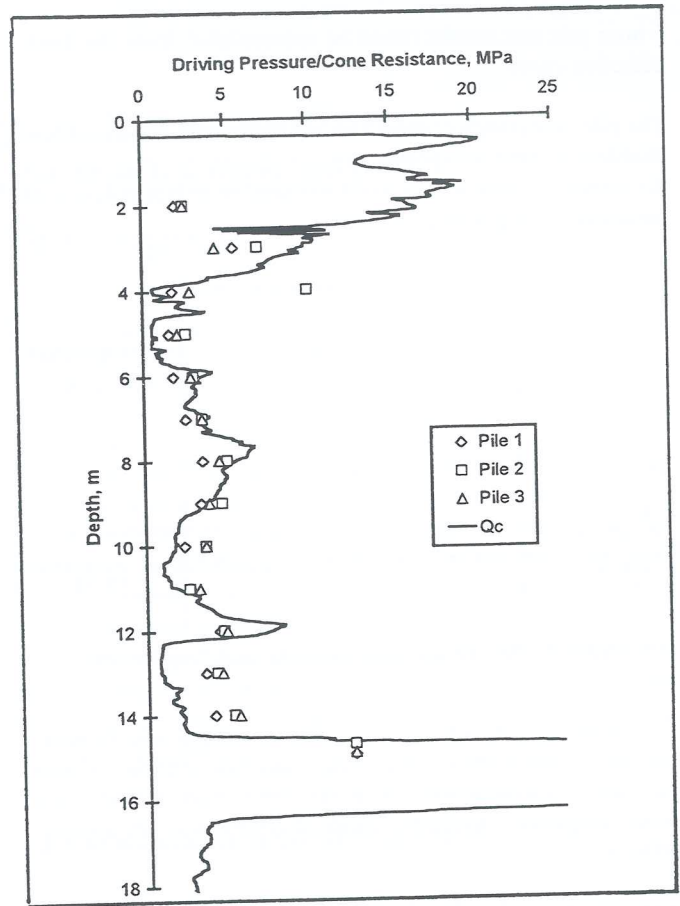


Figure 5: Comparison of Pile Driving Pressure and Cone Resistance

### 6.4 Pile Testing

A static load pile test was undertaken during pile installation to compare the response of a typical pile with the Australian Standard AS 2159.8.3.5.6. The pile was tested according to the following site specific procedure, created to accommodate curfew and rig limitations:

- install pile to desired depth
- record load and deflection at the top of the pile at 1 minute intervals as the pile is forced into the ground at a constant penetration rate of 0.5 mm/min
- when the design serviceability load (approximately 75 %  $S^*$ ) is reached, sustain the load for as long as possible (preferably 6 hours) while measuring deflections at the top of the pile.
- unload pile and record nett deflection after 10 minutes
- record load and deflection as the pile is reloaded to 100 tonnes at constant penetration rate (maximum test load normally 150 %  $S^*$  = 135 tonnes however this was beyond rig capacity)
- sustain load at 100 tonnes for 10 minutes and record nett deflections
- unload pile and record nett deflection after 10 minutes

Time restrictions due to the airport curfew restricted the maximum test load interval to 110 minutes. At the end of this period, deflections were considered to be uniform and

6 hour pile test results could be extrapolated from the load-deflection curve.

The pile acceptance criterion as outlined by the Standard was modified to meet acceptance criteria adopted in other areas of the airport. These values are compared with pile deflections measured during testing in Table 2.

Load	Deflection (mm)		
	Criteria		Extrapolated 6 hour test
	AS 2159	DP	
75 % S*	15		8.95
S*		10	
0 kN	7	3	0.93
150 % S*	50	15	
110 % S*			16.68
0 kN	30	5	2.5

Table 2 Pile Acceptance Criteria and Test Results

The measured maximum deflections exceeded site criteria, but were significantly lower than Standard criteria. Creep deflections extrapolated for a six hour load period were below suggested Australian Standard criteria and site specific criteria.

Analysis of creep performance results and observations made on site during pile installation suggest that this type of pile satisfied the design objectives.

## 7 CONCLUSIONS

The soil lithology at the International Terminal at Sydney Airport is complex and highly variable. The two case studies have demonstrated that conditions cannot be reliably predicted without detailed site investigation, prior to design and construction of foundations. However, with this data accurate estimates of pile performance can be made, and supplementary testing of specific piles during construction confirms that the new pile types satisfy their design objective, and comply with construction restrictions at this location.

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