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Peter Boyd
AECOM Ltd., United Kingdom

Baran Ozsoy
Zirve University, Turkey

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HIGH CAPACITY ROCK SOCKETED PILES IN SCOTLAND, UK

Peter Boyd
AECOM Ltd, 63 - 77 Victoria Street,
St Albans, AL1 3ER, Hertfordshire,
United Kingdom
peter.boyd@aecom.com

Dr. Baran Ozsoy
(formerly AECOM Ltd) Zirve University,
Kizilhisar Kampusu, 27260, Gaziantep,
Turkey
baran.ozsoy@zirve.edu.tr

ABSTRACT

The New Victoria Hospital in Glasgow, Scotland provides a 30,000m² 'Ambulatory Care and Diagnostic' (ACAD) facility. The ground conditions beneath the site comprise a variable thickness of Made Ground overlying Glaciomarine Deposits and Glacial Till which in turn rests on the Carboniferous bedrock. However the solid geology is complex and interrupted by a number of faults. The Carboniferous rocks under the site include a number of coal seams, although it is thought that none of these seams has been worked at this location. Due to the relatively low strength of the glacial soils, it was necessary to support the building on piles socketed into the bed rock. However the various coal layers could not be relied upon to provide adequate end bearing capacity and therefore piles were designed to be supported solely on side resistance from the rock socket. In order to investigate pile load bearing capacity and to differentiate end bearing from shaft friction capacity, a number of preliminary and working load tests were carried out with one pile particularly using a' soft toe' system. The results of load tests revealed an ultimate shaft friction capacity value in the rock socket of approximately 1.3MPa (189psi).

THE DEVELOPMENT

The site forms part of a former battleground of the Battle of Langside which was fought on 13 May 1568. Some 300 men were killed during the battle although it is recorded that the conflict lasted just 45 minutes. The site appears to have remained as open fields for the many years thereafter and the first development on the site occurred in the late 1800s when a road was established through the site and a school, later referred to as Queens Park Secondary School, was built on the south western corner of the site. It was at this time that the Victoria Infirmary (named after Queen Victoria) was built on the land opposite. Some houses were later built on the site north of the school, but by the late 1970s these had been demolished and by 2005 the one remaining school building had fallen into disrepair.

By this time the local health authority, the NHS Greater Glasgow Health Board, had identified the need for a new Ambulatory Care and Diagnostic (ACAD) Hospital to be built on the site. The proposed new £100million development comprised a four storey building and a semi-basement to be built on the now largely derelict land. The development site extended to approximately 0.3 hectares in area.

Under a PPP/PFI finance arrangement, the ultimate client appointed the Canmore Consortium to deliver the project. Balfour Beatty, a consortium member, was appointed to design and construct the new building and in turn AECOM was appointed as Balfour Beatty's civil, structural and geotechnical designer. The subsequent piling work was carried out by Stent Foundations Ltd. The architectural design of the project was performed by HLM Architects.

The preliminary structural design of the proposed buildings called for the construction of nearly 400No 600mm and 750mm diameter bored cast in-situ piles socketed into the rock. The principles of the pile design with initial load capacities were produced by AECOM's Geotechnical Group. The pile performance criteria were established in consultation with AECOM's structural designers to ensure the permissible settlement and angular distortion limits of the proposed structure were not exceeded. The final pile design was developed by the piling contractor Stent Foundations in line with the recommendations in the geotechnical interpretative report prepared by AECOM.

This paper describes the work undertaken to investigate the site and design the pile foundations, focusing particularly on the rock socket design and pile load testing.

GEOLOGICAL HISTORY

Various phases of ground investigation were carried out at the site and these were latterly supplemented by a further investigation focused on the proposed development.

Superficial Geology

The published geological map of the area indicates the majority of the site to be underlain by Quaternary drift deposits of the late Devensian stage.

Table 1. Published Superficial Geology of the Area

Formation	Provenance	Age (years bp)
Paisley Formation	Marine	11,500 -13,500
Wilderness Till	Glacial	13,500-27,500

The Paisley Formation comprises glaciomarine deposits typically manifest as sands, silts and clays. Its thickness in the Clyde valley is typically around 5m but is locally absent. The Wilderness Till is a glacial till comprising boulders and stones in a hard to stiff sandy silty clay matrix. Its thickness is highly variable and significant thicknesses may be found in infilled glacial valleys.

Solid Geology

The published geological map shows the solid strata in the area to form the following succession:

Table 2. The Published Solid Geology of the Area

Formation	Description	Significant Seams
Middle Coal Measures	Sandstones, siltstones and mudstones with numerous coal seams	Glasgow Main Coal. Humph Coal, Glasgow Splint Coal, Virgin Coal
Passage Group	Mainly sandstones with fireclays and thin mudstones and coals	No named seams
Upper Limestone Formation	Sandstones with mudstones, thin limestones and coals	Lyoncross Limestone, Lyoncross Coal, Index Limestone
Limestone Coal Formation	Sandstones, siltstones and mudstones with numerous coals and ironstones	Ashfield Coking Coal, King Coal, Possil Main Coal, Pollock Stone Coal. Glasgow Shale Coal, Jubilee Coal

The published geological map of the area shows the Dechmont fault running through site close to the northern boundary. The strata either side of the fault are quite distinct with the Middle (Productive) Coal Measures subcropping north of the fault and the Upper Limestone Formation subcropping to the south on the downthrow side.

However drilling and interpretation of the mining geology during one of the early phases of ground investigation cast doubt on the accuracy of the published map and at least two further faults were inferred from the new data. The re-interpretation of the mapped geology indicated that the 'Limestone Coal Formation' (which includes numerous workable coal seams) may underlie much of the northern half of the site. South of the conjectured fault 'The Passage Group' may be present and beyond a further fault near the southern end of the site the 'Upper Limestone' is conjectured lie at subcrop. Whilst both the 'Upper Limestone' and 'Passage Group' include some potentially workable coal seams, they are not thought to have been worked. Furthermore there are no recorded workings in the 'Middle Coal Measures' immediately underlying the site.

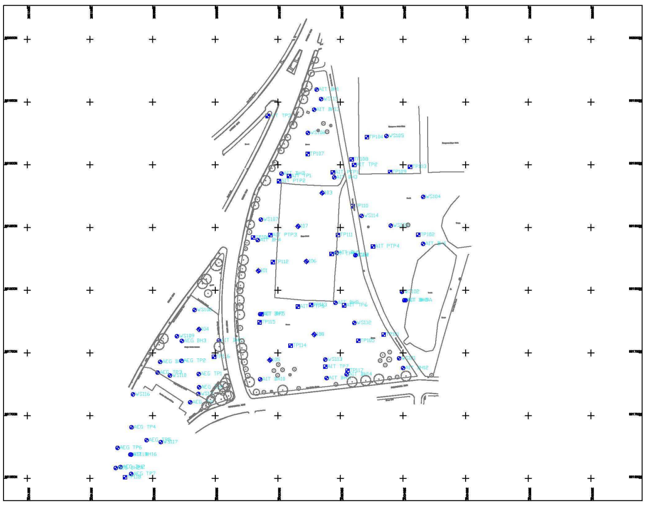

Fig.1 Site Plan and Borehole Locations

GROUND INVESTIGATION DATA

Stratigraphy

The detailed ground investigation revealed a variable thickness of superficial deposits overlying rockhead, the depth to rockhead from surface varying from 6 m to 20.4m. The majority of the site was found to be underlain by made ground and this is turn was found to overlie a highly variable succession of Glaciomarine deposits comprising loose sand and silt overlying soft and occasionally laminated clays. The Glaciomarine deposits were often found to rest on the Glacial Till (stiff boulder clay) but the thickness of the Till was generally limited and in some cases appeared to be absent. The Glaciomarine deposits were deemed unsuitable for support of the proposed building and therefore the design solution was to support the new hospital on piles socketed into the bedrock.

<u>Design parameters</u>

The strength of the bedrock was determined from uniaxial unconfined compression and point load index testing. The point load index tests were carried out on cores in either an axial or diametrical orientation. Generally it is the axial tests that are correlated with unconfined compressive strength test data. The point load index was converted to unconfined compressive strength using the relationship proposed by Broch and Franklin (1972) who established that a reasonable correlation exists between the uniaxial compressive strength (q_{uc}) and the point load strength index ($I_{s(50)}$), where;

$$q_{uc}=24\ I_{s(50)} \qquad (1)$$

Other researchers such as Rusnak and Mark (2000) have derived strata-specific correlations for Carboniferous rocks and they propose a similar correlation coefficient for these rocks of around 21.

The data obtained for the ACAD site are plotted on Fig.2 and the design line is drawn preferentially through the q_{uc} data.

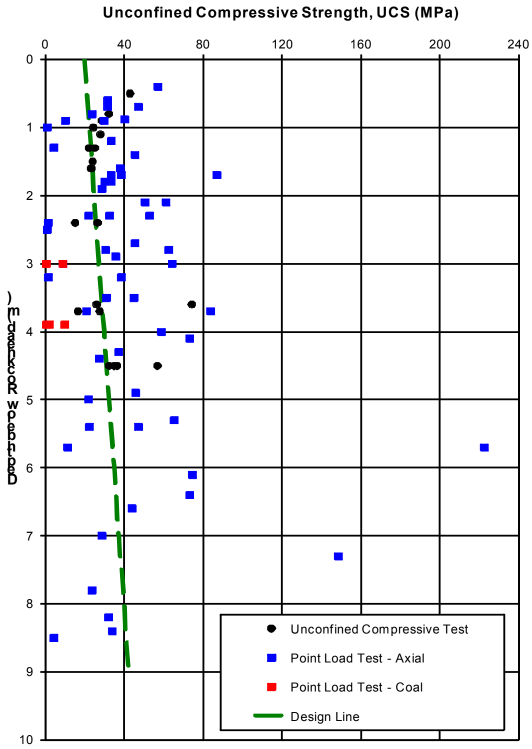

Fig.2 Unconfined Compressive Strengths Derived from UCS tests and Point Load Tests

As discussed in the next section, the RQD is also a significant factor in calculating the pile rock socket capacity when direct measurement of rock mass factor is not available. As can be seen the majority of results are below 50% (Fig. 3).

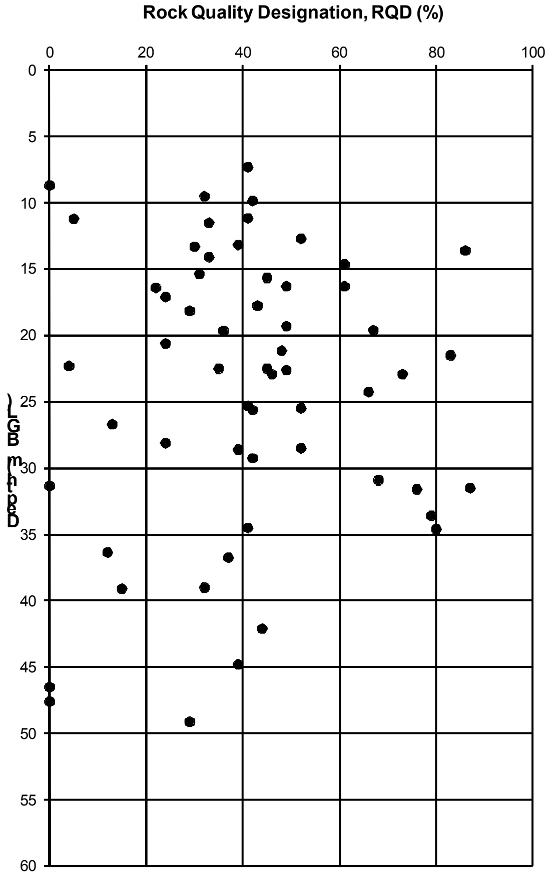

Fig.3 RQD Measurements on Rock Core

APPROACH TO PILE DESIGN

The rock strata underlying the site comprised a succession of mudstones, siltstones, sandstones as well as a number of coal seams. Whilst it is known that the coals seams have not been worked (generally they are too thin and too deep to be of economic value) the coal seams themselves are significantly weaker than the surrounding rock.

Point load tests on coal indicated unconfined compressive strengths as low as 0.5MPa (73psi), whereas the surrounding rock has a characteristic unconfined compressive strength of at least 20MPa (2900psi).

Given that the occurrence and depth of coal layers beneath the site has been shown by the ground investigation to be unpredictable (largely as a the result of significant local faulting), there was considered to be a significant risk of a pile tip bearing on, or just above, a weak coal seam. End bearing resulting from bearing onto coal is only a small proportion of that resulting from bearing on competent rock. Therefore it was felt prudent, for preliminary design purposes, to ignore the contribution to pile capacity from end bearing.

It was important therefore to establish a reliable and not unduly conservative value for the load capacity which could be derived from the rock socket side friction alone. Various methods of calculating the capacity of the rock socket were considered, the majority of which relate rock socket capacity to unconfined compressive strength. However other studies have established that further factors such as joint spacing and roughness of the socket may have a significant influence on the mobilised rock socket resistance - see Haberfield and Collingwood (2006).

Given the wide range of values which could be derived from theoretical calculations plus the inherent uncertainty in determining an accurate value for the unconfined compressive strength of the rock, it was felt important to undertake pile load testing to establish the operational in situ strength of the rock socket.

PILE PERFORMANCE CRITERIA

The piling work and load testing performance criteria were set out in accordance with the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls (SPERW) dated 1996.

The acceptance criteria for the test piles selected for the project were as follows:

a) Maximum first cycle settlement at safe working load (SWL) not more than 10mm.
b) Maximum settlement at 150% of SWL not more than 20mm.
c) Residual settlement after second cycle of loading to 150% of working load, to be not more than 50% of permitted settlements at SWL.

It was also stipulated that the pile foundations should be designed so that the differential movement between adjacent pile caps would not exceed 1 in 500.

A factor of safety of at least 2.0 on skin friction for compression loads and a factor of safety of 3.0 on tension loads was required – the latter principally due to temporary loading combinations during the construction stages of the structural work.

CALCULATION OF ROCK SOCKET CAPACITY FROM GROUND INVESTIGATION DATA

In the UK the British Standard BS 8004 'Foundations' (1986) (now withdrawn and superseded by Eurocodes) provides little guidance on the design of rock sockets for piles and common UK normal practice is to resort to well-established guidance such as 'Pile Design and Construction Practice' by Tomlinson and Woodward (2008) and previous editions of the same text. In turn Tomlinson and Woodward (2008) and previously Tomlinson (1994) cite a number of methods of calculating pile rock socket capacity, namely Horvarth (1978), Rosenberg and Journeaux (1976) and Williams and Pells (1981).

All the above, as reported by Tomlinson and Wood (2008), relate the ultimate rock socket bond strength ($f_{s)}$ to the unconfined compressive strength using the following equation:

$$f_s = \alpha.\beta.q_{uc} \qquad (2)$$

where:

α = Reduction factor relating to q_{uc}
β = Correction factor related to discontinuity spacing in the rock mass.
q_{uc} = Average unconfined compressive strength of the rock over the length of the rock socket

Whilst the Williams and Pells (1981) method gives the highest value of β the other two methods cited by Tomlinson assume an α value of unity. Therefore the approach by Williams and Pells (1981) is likely to be more conservative than the two other methods when considering highly fractured rocks.

The mass factor j is defined as the ratio of the elastic modulus of the rock mass to that of the intact rock.

Ideally this is measured using geophysical techniques or loading tests, but can be estimated from a knowledge of the discontinuity spacing and/or RQD from the recovered rock cores.

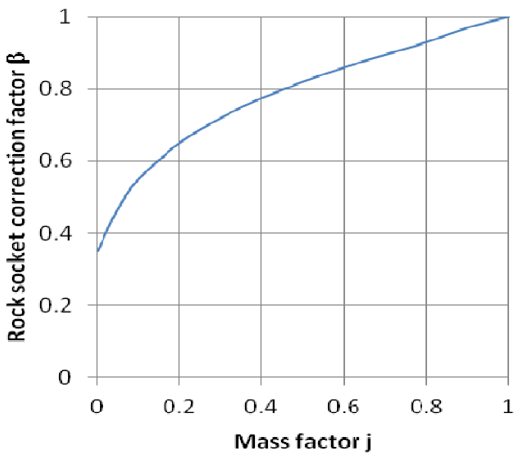

Fig.4 Reduction factors for discontinuities in rock mass (after Williams and Pells, 1981)

A relationship between RQD and mass factor j was proposed by Hobbs (1975) as follows:

Table 3. Relationship between RQD and Rock Mass factor j

RQD (%)	Fracture Frequency/m	Mass factor j
0-25	15	0.2
25-50	15-8	0.2
50-75	8-5	0.2-0.5
75-90	5-1	0.5-0.8
90-100	1	0.8-1.0

On the basis of the foregoing the following parameters were derived:

$\alpha = 0.10$

q_{uc} = 20MPa (2900psi)

$\beta =$ 0.65 (RQD assumed to be 50% or less)

Hence f_s = 1.3MPa (189psi)

A useful historical review of the various factors derived by various researchers, as well as their own recommendations, has been presented by Kulhawy et al. (2005). Various others methods of calculating the capacity of rock sockets are available. By and large these take the form as follows:

$$f_s/P_a = c.(q_{uc}/P_a)^n \quad (3)$$

Where P_a = atmospheric pressure and C and n are empirical factors.

A particular relationship of this type proposed by Fleming et al. (1992) is as follows

$$f_s/P_a = 1.3(q_{uc}/Pa)^{0.5} \quad (4)$$

Additionally Fleming at a.(1992) stipulate that the above relationship is only acceptable for use in rock sockets where the shaft is sufficiently rough to ensure full keying of the concrete and the host rock. Furthermore they caution that f_s should not exceed 5% of the concrete strength.

Since f_s for the pile designs derived using the approach of Williams and Pells (1981) does not exceed either of these values, it was considered that the unit skin friction value adopted was appropriate

A comparison of the values derived from various published methods is provided in Table 4.

Table 4. Published Relationships between Unconfined Compressive Strength and Rock Socket Bond Strength

Reference	**C**	**n**	**fs**	
			(MPa)	**(psi)**
Rosenberg and Journeaux (1976)	1.09	0.52	1.71	248
Horvath (1978)	1.04	0.50	1.47	213
Horvath & Kenney (1979)	0.65-0.78	0.50	0.92	133
Williams et al. (1980)	1.84	0.37	1.31	190
Rowe & Armitage (1984)	1.42	0.50	2.01	292
Carter & Kulhawy (1988)	0.63	0.50	0.89	129
Fleming et al. (1992)*	1.30	0.50	1.84	267
Zhang & Einstein (1999)	1.26	0.50	1.78	258
Prakoso (2002)	0.98	0.50	1.39	202
Kulhawy et al (2005)	1.00	0.50	1.41	205

(*) Not included in Kulhawy et al. data

For the ACAD site (assuming q_{uc}= 20MPa), the above methods yield shaft friction values in the range of 0.89MPa (129psi) to 2.01MPa (292psi)., the highest value being given by Rowe and Armitage (1984).

The above data contrasts with local practice in the Glasgow area was to assume much lower values for shaft friction – presumed values being typically 0.25MPa (36psi) for mudstone, 0.5MPa (73psi) for siltstone and 0.75MPa (109psi) for sandstone and limestone – with values being associated with rock type rather than directly correlated with rock strength.

In view of this disparity, it was therefore felt to be imperative to verify any higher values by means of pile load testing.

PILE LOAD COMPRESSION TESTS

1st Preliminary Test Pile with the Soft Toe Feature (TP1)

The pile design called for a 750mm (2' 6") diameter pile with a 3m (9') long rock socket. The nominal safe working load (SWL) of the test pile was calculated to be 4200kN (944kipf). The achieved peak test load was 9450kN (2124kipf).

The test pile location was deliberately selected to be close to a borehole location in order to facilitate a correlation the soil parameters to the observed pile behaviour. The ground conditions at the test pile location were recorded as 1.2m of

Made Ground overlaying 7.5m of Glaciomarine Deposits and Glacial Till. A 0.5m (20”) thick coal layer was recorded by the piling contractor and the top of the bedrock was observed to be 9.2m below the ground surface.

A 300mm (12”) thick soft toe made of polystyrene was placed beneath the reinforcement cage..

The load test was a maintained load test carried out in accordance with ICE ‘Specification for Piling’ which includes three loading-unloading loops and a number of load holding stages.

The load settlement relationship for the pile test undertaken with a ‘soft toe’ is shown below.

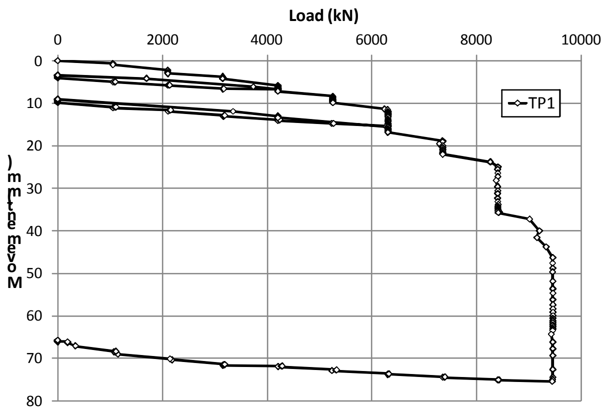

Fig.5 Pile Load Compression Test Results for TP1 (Soft Toe)

The test was taken to a maximum load of 9450kN (2125kip-f) which equates to 225% SWL. It was not possible to maintain the load beyond this point and in view of the soft toe it was felt advisable for health and safety reasons to terminate the test when settlement reached 75mm (3”). This settlement corresponds to 10% of the pile diameter and this is in itself an arbitrary definition of pile failure.

A back analysis of the load-settlement data was undertaken using the CEMSOLVE program using the method derived by Fleming (1992). This analysis suggested that higher ultimate load might have been achievable albeit at very large settlements. However this could not be confirmed with the method of testing adopted.

The predicted elastic shortening obtained from CEMSOLVE using E=30GN/m^2 gave values close to the settlement recovery at the end of the test of around 10mm. However the actual-load settlement response was much ‘softer’ than that might have been predicted at the outset using CEMSET. Specifically the shaft flexibility factor M_s was back calculated by curve fitting and this yielded a value of around 0.005 – a value associated with soft soils. M_s is in fact the tangent slope at the original of the hyberbolic function representing shaft friction. The reasons for this disparity are not clear but it is possible that rock discontinuities and overall roughness of the socket will have increased the load bearing capacity of the socket but such discontinuities may have also contributed to an overall reduction in the vertical rock stiffness measured.

The difficulty in accurately predicting the behaviour of rock sockets under load was also highlighted by Pells (1999) who found that the Young’s modulus was often highly variable even when tests were carried out in the same rock mass,

Fig.6 The Soft Toe Detail for the First Test Pile (TP1)

2nd Preliminary Test Pile without the Soft Toe Feature (TP2)

The second test pile was constructed without the soft toe feature in order to make a comparison with TP1. The designated working load for TP2 was 4200kN (944kipf) and the target peak test load was 10500kN (2360kipf). The pile settled 8.2mm (0.32”) at the peak test load and a residual settlement of 3.2mm (0.12”) was observed at the end of the test.

The most significant difference between TP1 and TP2 was the stiffer pile settlement response observed during the second preliminary test. It is interpreted that due to the relatively short rock socket length in both cases, the contribution of pile end-bearing capacity to the overall pile capacity was more evident in the second test and some significant end-bearing capacity might have mobilized before the pile test mobilized the full shaft capacity.

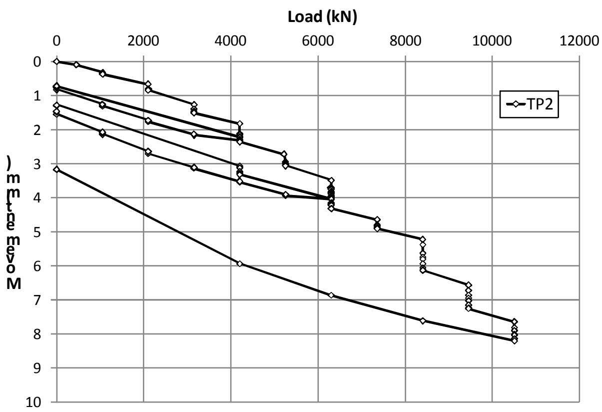

Fig.7 Pile Load Compression Test Results for TP2

In summary both tests were concluded to be satisfactory and the piling contractor commenced the site works with no change in the pile design philosophy.

The Working Load Tests for QA/QC Purposes (TP3 and TP4)

Two other test piles were selected for QA/QC purposes and maintained load tests were performed on these contract piles up to 150% of their SWL.. The test pile diameters for TP3 and TP4 were 750mm and 600mm; respectively. The performance of these two contract piles were also considered satisfactory as they satisfied the structural performance criteria set out at the piling specification.

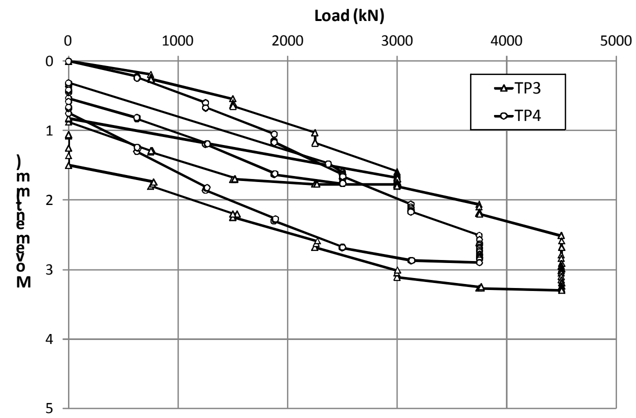

Fig.8. Pile Load Compression Test Results for TP3 and TP4

Summary of Pile Compression Load Test Results

In total, four compression load tests were performed in this project. The test pile diameters were 750mm for TP1, TP2 and TP3. The remaining load test was performed on a 600mm (2') diameter pile..

Table 5. Summary of Pile Loads and Settlements during the Compression Load Testing Programme

Pile No	WL (kN)	PTL (kN)	Settlement @ WL (mm)	Settlement @ PTL (mm)
TP1	4200	9450	7.17	75.39
TP2	4200	10500	2.36	8.20
TP3	3000	4500	1.77	3.30
TP4	2500	3750	1.76	2.90

(*) WL: Working Load / PTL: Peak Test Load

Note: The pile settlements were measured at the pile heads by averaging the values recorded electronically on four dial-gauges.

BOND STRENGTH – COMPARISON WITH PUBLISHED DATA

The measured average bond strength from the compressive pile load with the soft toe test was 1.3MN/m^2 (1.89psi). This matches closely with the predicted value derived using the William and Pells (1981) approach which takes account of both unconfined compressive strength and the rock mass factor.

Taking an assumed unconfined compressive strength of 20MPa (2900psi), the measured bond stress was equivalent 0.065q_u. This lies close to, but slightly below the trend line produced by Long (2000) for Carboniferous rocks in Ireland .

When compared to the various predictive methods cited by Kulhawy et al (2005) the observed strength is given by the following relationship:

$$f_s/P_a = 0.92\ (q_{uc}/P_a)^{0.5} \qquad (5)$$

Fig.9 A View of the Piling Works Performed by Stent

PILE LATERAL LOAD TESTING

In the long term the lateral loads to be resisted by the piles were estimated to be very small as the pile caps will be restrained by means of stiff ground slabs spanning in both directions. During the design process of the superstructure it was revealed that the overall construction time could be significantly reduced if the erection of the structural frames could take place, without waiting the casting of the ground slabs. This would require the individual piles to be designed to resist significantly higher lateral loads, as they would be subject to lateral wind loads acting on the structural frame during the temporary construction stages.

In order to measure the lateral behavior of the contract piles two lateral load tests were performed on 600mm (LTP1) and 750mm (LTP2) piles. The working lateral load was determined to be 275kN whereas the peak test load was 330kN in both cases.

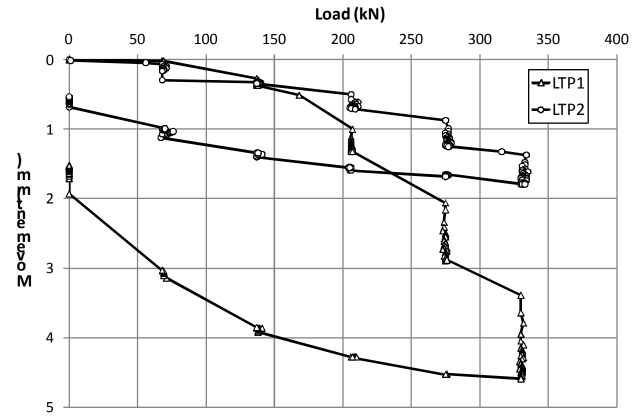

Fig.10 Pile Lateral Load Test Results

A summary of the lateral load test results is as follows:

Table 6. Summary of Pile Behaviour during the Lateral Load Testing Programme

Pile No	WL (kN)	PTL (kN)	Deflection @ WL (mm)	Deflection @ PTL (mm)
LTP1	275	330	2.88	4.59
LTP2	275	330	1.25	1.79

(*) WL: Working Load / PTL: Peak Test Load

The resultant deformations were considered satisfactory by the structural engineers and the piling works were commenced accordingly. The lateral load tests resulted in significant savings in terms of overall construction time and in turn project budget.

CONCLUSIONS

The predicted bond strength in a rock socket is usually estimated from the unconfined compressive strength. However the accuracy of such an approach is dependent upon obtaining a representative value of the rock strength. At the Victoria ACAD site the relationship proposed by Williams et al. (1980) and also Williams and Pell (1981) gave the closest approximation to the value which was later verified by the soft toe pile testing. The method proposed by Kulhawy et al. (2005) also gave a good approximation.

Although a number of load tests were performed for this project, it will be necessary to carry out further research and more rock socket load tests in Carboniferous rocks in the Glasgow area before more widely applicable design guidance can be formulated. It is certainly vital to carry out preliminary load tests particularly as major uncertainties exist in the prediction of rock socket behaviour.

The preliminary design of the rock sockets was based on a conservative assessment of the rock strength and so the soft toe pile testing allowed a higher rock socket bond strength and hence a more economic pile design to be adopted. The lateral load test also helped the structural engineer and the client to shorten the overall construction period.

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