

Geotechnical aspects of constructing the new Bullring, Birmingham

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Introduction

This paper presents a review of the geotechnical issues involved in the redevelopment of the famous 1960s shopping centre site in Birmingham.

The objective throughout the project was to optimise foundation construction and temporary works. The following aspects of the work are discussed: pad foundation construction; temporary works for the contiguous bored pile wall alongside brick railway tunnels; installation of rock nails to secure a basement slab against uplift; and the dewatering arrangements for the low level excavation. Design and build contractor for the scheme was Sir Robert McAlpine and the structural engineer was the Waterman Partnership.

Scope of work

The new development is on the site of the 1960s Bullring complex, which was built on market and industrial developments dating back 800 years. It will provide 110,000m² of retail space and include two flagship stores, Selfridges and Debenhams. A plan of the site is shown in Figure 1.

The development includes up to three underground car parking levels and delivery access. The ground is retained by retaining walls in the south and a new contiguous bored pile wall to the northern perimeter (Figure 2). Two railway tunnels lie immediately next to the north of the site.

Initial enabling works involved diverting part of the Northern Arm inner city ring road and construction of a bored pile retaining wall. Earthworks for the development required excavation of 204,000m³ of sandstone to achieve the required formation levels up to 16m below ground level.

The new Bullring structure is founded on pads in the Bromsgrove Sandstone. Foundation design depended on the allowable bearing capacity of the sandstone and had to make allowance for possible marl bands within the strata.

Site investigation

A number of site investigations were carried out on and around the Bullring site before the tender submission. These identified the founding strata as very weak sandstone with interbedded marl. However, it was felt that measured strengths were lower than the true strength because of sample disturbance and there was scope for finding a more economic design for walls and foundations.

Additional site investigation was carried out post-tender, with the objective of characterising the sandstone, and establishing the nature and extent of the marl bands. The site investigation comprised 26 boreholes and 10 trial pits. These were carried out between 1988 and 2000 by Soil Mechanics and Soil Consultants. All boreholes were cable percussion to rockhead and extended by rotary methods to a maximum depth of 22m. Testing tended to focus on the sandstone and marl with SPTs, UCSs, and point load testing on a substantial number of samples.

McAlpine assessed all the available data. Its interpretative report concluded that some of the exploratory work had indeed damaged or destroyed the sandstone structure and in fact the stratum was an intact

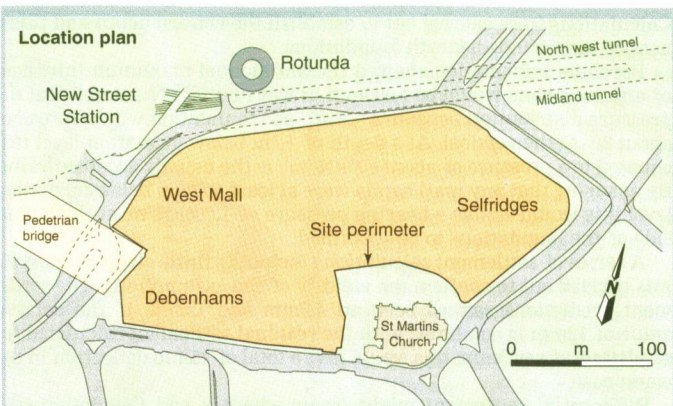


Figure 1: The new Bullring is being built on a site that has had a market and industrial developments on it for 800 years.



Figure 2: The ground is retained by retaining walls and a contiguous bored pile wall.

weak to moderately weak rock. This justified a substantially higher bearing capacity than would have been adopted normally and meant significant refinements could be made to the temporary works scheme for the contiguous piled wall.

Pad foundations

The new Bullring building comprises a reinforced concrete frame substructure and steel framed superstructure. The substructure includes up to three car-parking levels, plant rooms and delivery access and the superstructure comprises three levels of shopping malls. Typical column loads are 17,000kN.

The columns are carried by pads founded in the Bromsgrove Sandstone. Research showed that buildings next to the Bullring site had adopted 750kN/m² allowable bearing capacity but recent practice had been to limit the bearing capacity to 600kN/m² throughout Birmingham because of the presence of marl bands.

The settlement limit for the pad foundations was set at 15mm based on acceptable differential settlement distortion of 1:500 for the reinforced concrete framed building.

The pad formation levels range from approximately 106mAOD to 108mAOD. At this level the site investigations showed the Bromsgrove Sandstone was a well-cemented sandstone containing bands of marl. The marl was characterised as a very stiff/hard over consolidated clay, with a tendency to soften when unloaded and exposed to water.

Foundation design

A review of data from uniaxial compression tests, point load tests, and standard penetration tests concluded that the minimum allowable bearing capacity of homogeneous Bromsgrove Sandstone was 1000kN/m² at levels below 110mAOD. The derived values are allowable capacities incorporating a factor of safety of 3.

The additional ground investigation suggested that the anticipated settlement of the rock mass would be 5mm based on the 1,000kN/m² loading. This was calculated using values of Young's Modulus of 900MN/m² and Poisson's Ratio of 0.2 proposed in the interpretive report. However, marl at or near the underside of pad foundations may have increased settlement and reduced the allowable bearing capacity.

Calculations were carried out to establish the critical depth and thickness of marl bands beneath foundations.

Borehole information showed the anticipated maximum thickness of any marl band would be 1,000mm. It was further considered that the pressure distribution beneath a "stiff" pad foundation would spread at about 30° to the vertical. At a depth of 1.1m below formation level this equated to a pressure of about 600kN/m², ie the usually accepted value. By ensuring that any marl bands were at least 600mm below formation level this would enable a bearing pressure of 1,000kN/m² at the underside of the foundations to be sustained.

A series of settlement calculations including finite element analysis was carried out to confirm the validity of the assumptions. The settlement predictions ranged between 4.6mm and 12mm. If the derived value of 12mm is combined with the residual settlement calculated for sandstone of say 3mm, this would give a total of 15mm deflection in the worst case.

Differential settlement might occur where a pad foundation was built in the "worst case location" described above, and a neighbouring pad was built in a zone of homogeneous sandstone. The differential would be 12mm, ie less than 1 in 500 for pads spaced on a 7.5m grid.

Based on these calculations it was decided that a 1m thick marl band 600mm beneath a pad foundation would be acceptable. These predictions were presented to Birmingham City Council Building Control which, after careful consideration, agreed to the bearing capacity of 1,000kN/m².

A McAlpine geotechnical engineer supervised foundation excavations full-time. Small, 600mm deep trial pits were dug within the footprint of each excavation to check for the presence of marl. Where marl was encountered it was removed to its full depth over the entire area of the pad base. If marl was not found the pad was formed at the designed formation level. During construction the method statement was relaxed to allow small amounts of marl to remain within the base footprint close to formation level.

A number of columns within Selfridges and West Mall South are being monitored for settlement using precise levelling. At the time of writing, with about 90% of dead loading applied, the maximum settlement is 4mm. This is in good agreement with the predictions.

Economics

The economies of using a higher bearing capacity are illustrated in the comparison of typical foundations shown in Figures 3 and 4. With savings of thousands of pounds per base the overall cost reduction was substantial.

Northern contiguous piled wall and temporary propping

The contiguous piled wall formed part of the Northern Arm works carried out by Balfour Beatty Civil Engineering which was later employed as subcontractor to McAlpine.

The propping of the northern contiguous wall gave rise to some interesting arrangements. There are twin Victorian masonry railway tunnels just 3m outside this wall and excavation was to take place up to 6m below tunnel founding level (Figure 5).

Analysis and monitoring of the tunnel during piling was presented at the CIRIA conference on the "Response of buildings to excavation-induced ground movements" by G Prakha and H Natrass in July 2001. The contiguous wall was initially designed by the Waterman Partnership as part of the enabling works contract. Deflection was to be limited to between 10mm and 20mm using two levels of temporary propping. McAlpine explored a range of construction sequences to find the most buildable option.

When considering the cross section of the southern tunnel it was apparent that the wall thickness was the same as would be required if the ground provided no lateral support. The strength of the sandstone on which the tunnels were built was considered to be such that they would theoretically stand with the adjacent excavation in open cut. The finite element models used for tunnel analysis demonstrated this and a Category III check confirmed it. The propping arrangement for the wall in terms of tunnel stability thus became a contingency and the checking requirement for this was relaxed to Category II.

Agreement of soil parameters for the analysis and design was difficult. Skilled drillers had achieved good core recovery during the additional site investigation and no real difference was seen between this rock and the sandstone at upper levels, where other investigations had recovered less intact cores.

It was therefore considered that the sandstone mass would exhibit significant cohesion. As it was not possible to carry out an effective large scale test, a nominal 20kN/m² cohesion from back-analysis of the stability of the temporary berm slope was used (Figure 6). Soil parameters used in design are shown on the strata cross section in Figure 5.

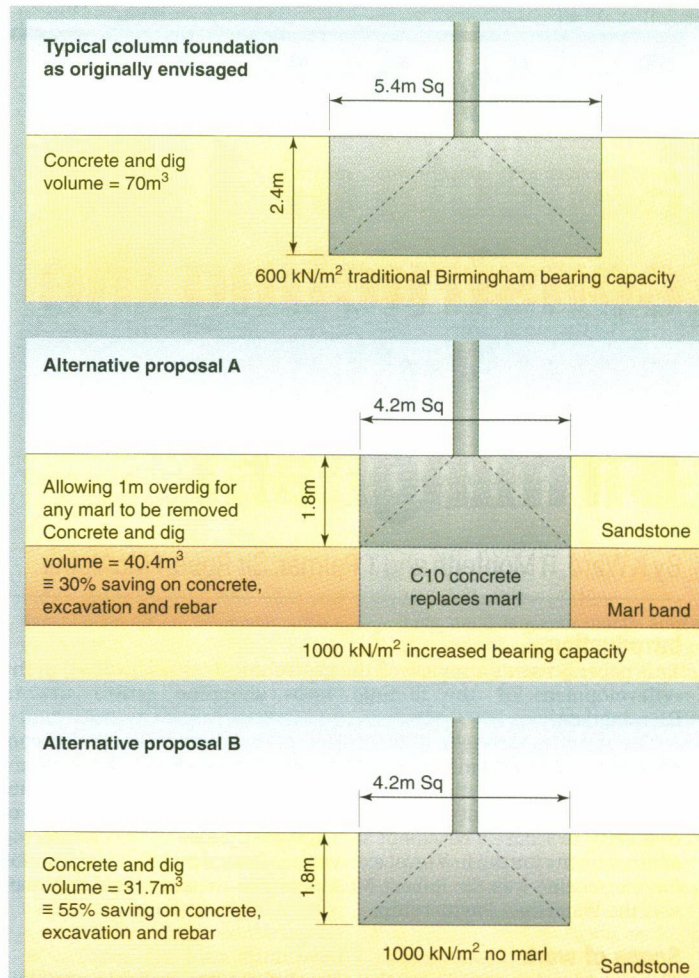


Figure 3: Comparison of typical foundation design.

Three typical sections of the propping are shown in Figure 7. Tubular propping was only used to either limit wall deflection or to allow full depth excavation for the cores. The main propping of the wall was provided by casting the level 3 slab before removing the supporting berm, ie top down. A comparison of predicted movements with recorded deflection is shown in Figure 8 using WallAP.

Piling work had already started before the McAlpine alternative proposal could be implemented, and the scheme generated bending moments and shear forces in excess of the structural capacity of the original piles. The pile reinforcement was therefore revised to accommodate the increased loading. Typically the reinforcement was increased from 15 T20 to 15 T25 bars. Where piles had already been installed the excavation to the toe of the wall had to be minimised and support generated from adjacent cores was used.

Ground anchor alternative

The western end of the retaining wall away from the tunnels was restrained using temporary ground anchors instead of props. Ground anchors may have proved an economic alternative to props throughout the length of the retaining wall but could not be installed due to the proximity of the Railtrack tunnels.

The required ground anchor loading of 700kN was based on the tie-force values from the WallAP analyses. The ground anchor design was carried out by Fondedile. Anchors were installed from +119mAOD and inclined down to be fixed into the upper sandstone. Strand anchors were used and, because these were only temporary, only basic corrosion protection was needed.

Third party consent was required for the use of temporary anchors as these pass beneath the adjacent road. Third parties included the utility services and Birmingham City Council Highways Department.

Tunnel monitoring

Instrumentation was installed to monitor movements due to the installation and excavation of the Northern Arm contiguous bored piled wall. Instrumentation was installed both internally and externally to

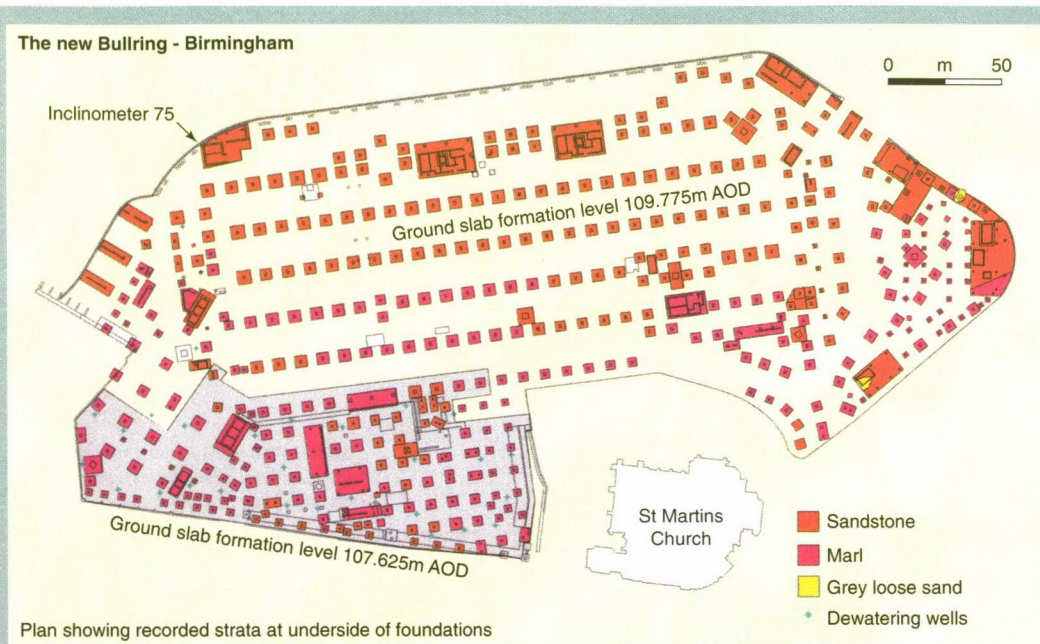


Figure 4: Geology of foundation levels.

the Railtrack tunnels. The internal instrumentation comprised arrays of electrolevels and tape extensometers, combined with precise level monitoring of the rails.

The instrumentation was set up to provide an "early warning" system of any structural movements resulting from construction activities and changes in tunnel loading configurations.

All results were collated remotely by a PC that automatically downloaded electrolevel information at set intervals. The information was linked to an automated alarm system that warned of "alert level" breaches. The early warning of structural movements provided by this system required an element of engineering judgement to interpret the results because the equipment was influenced by temperature and train movements and was prone to knocks and jumps as it became soiled with soot.

Movements recorded in the tunnels during construction varied from 3mm to 7mm excluding daily variations due to thermal effects. This resulted in tensile strains in the brickwork of around 0.05% which was regarded as acceptable.

The external monitoring comprised inclinometers installed in the piles, and electronic distance measurement to prisms and targets fixed to the capping beam. Demec gauges were installed on a number of props to measure strain and therefore check prop loads.

Back-analysis of the contiguous piled wall

Following the review of actual wall movement it was concluded that the design predictions made in the WallAP analyses were conservative. To replicate the small displacements recorded, the soil stiffnesses or cohesion had to be increased threefold. This confirmed the view that the initial design had been over-conservative.

Rock nails

The general groundwater level on site was recorded at approximately 108.5m. As it was considered prudent to allow for a future rise in water level, the design water level was taken as the car park entrance level of 111.5m.

The basement slab at 109.925m typically spans 15m between columns and would have to be about 600mm thick with substantial reinforcement to resist the hydrostatic uplift pressure. By introducing rock nails into the spans at third points it was possible to reduce the slab thickness to 275mm. A typical detail is shown in Figure 9.

The level 0 slab in Debenhams and West Mall south was also affected by uplift pressure with formation level of these slabs at 107.7m and 108.2m respectively. The proposal for Debenhams was to use a slab with sufficient reinforcement and thickness to span between column pads to counteract uplift. The proposal for the West Mall South slab was to use rock nails but with increased capacity.

Twelve preliminary pull-out tests were conducted in two zones within the Selfridges area of the site. Rock nails were installed into the Bromsgrove Sandstone and marl formation to depths varying from 1m to 3.325m. The nails were grouted into 115mm diameter boreholes using a Pozament Ductfill 307 mix. The nail comprised a 40mm diameter Gawi bar with an ultimate tensile strength of 756kN and yield strength of 630kN. The nails were loaded to failure using a hydraulic jack and beam. Deflection was measured by dial gauges mounted on reference beams. Bar extension and ground heave were both measured.

The rock nails were either fully bonded with grout over the full depth of the bore, or were part grouted at the base of the bar. The test results are summarised in Table 1 and Figure 10.

The working load for rock nails was 250kN. Using an ultimate bond stress of 0.70N/mm² and a factor of safety of 3 from BS8081:1989, a minimum length of just under 3m was calculated. The load/deflection results for six working pull-out tests are shown in Table 2.

Typical elastic extension for a 4m long 40mm diameter Gawi bar loaded to 250kN and 375kN would be 3.88mm and 5.82mm respectively. The deflection observed in the working pull-out tests is largely elastic and a function of bar extension.

From the above analysis the rock nails could have been installed slightly shorter, ie 3.5m instead of 4m. However, the Gawi steel was available in 12m stock lengths and there would have been no material saving.

The value of grout/rock bond strength obtained from this analysis is 0.7N/mm². Test grout/bond strength values published in BS8081:1989 range from 0.84N/mm² to 1.58N/mm². The lower value obtained from the Bullring test could be due to relatively short anchors, whereby the bond strength is reduced by weathering and disturbance from drilling. There were also layers of marl within the sandstone at the level of anchor installation, this would lead to reduced overall bond strength.

Dewatering

The design formation level for the pads in Debenhams and the West Mall South area of the site was typically at 106.3m. A number of isolated pads and cores had formation levels lower than this. There was potential for a significant number of bases to be formed below the 106m level, particularly because of the risk of encountering marl. As the groundwater level was recorded at 108.5m there was a requirement to control the groundwater.

This was achieved in part of the site by the use of sump pumps which were lowered into each pad excavation to pump out the groundwater as the base was constructed. This method however began to be unsatisfactory where the pad excavations extended below 106mOD.

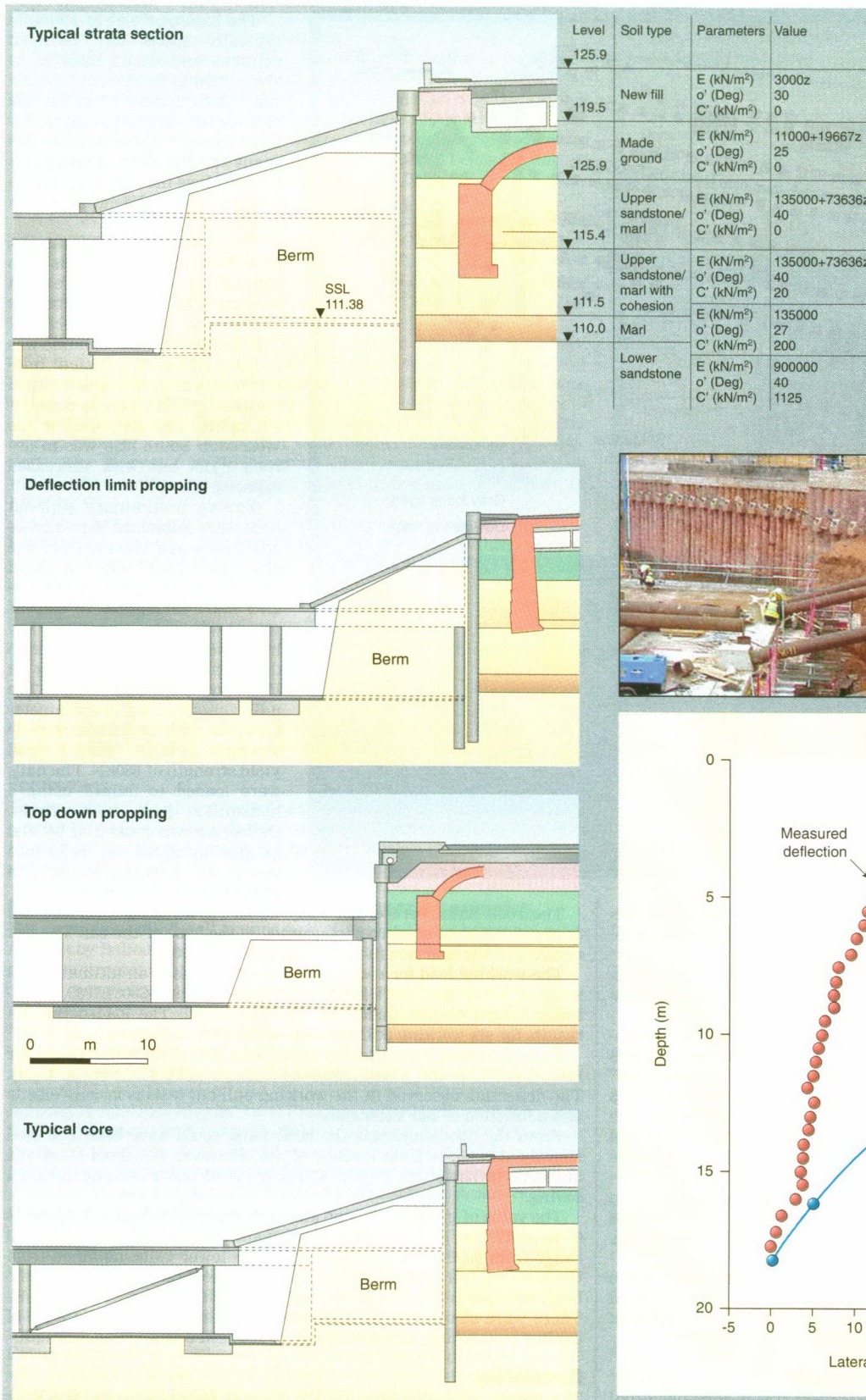
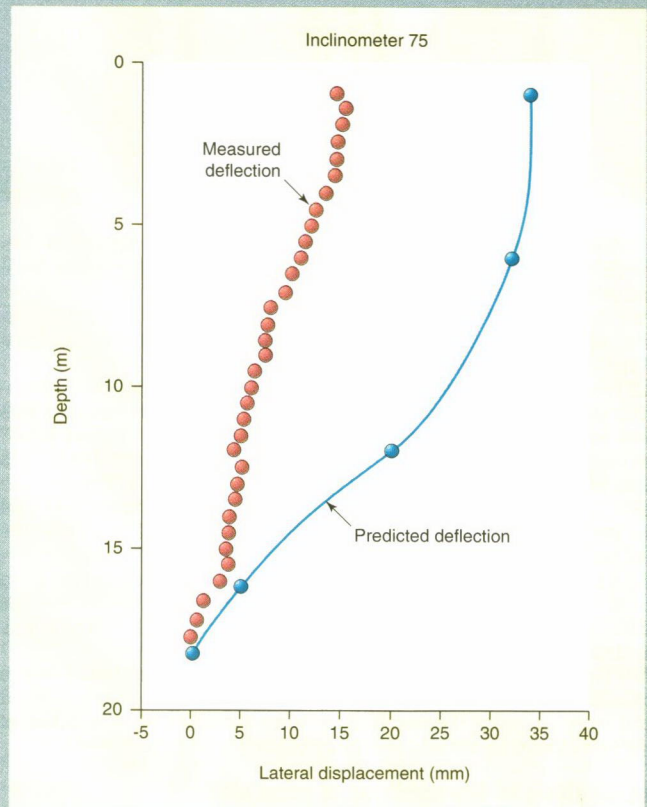


Figure 5: Propping arrangements were complex to protect twin Victorian railway tunnels running next to the site.

BELOW: Figure 6: Back-analysis of the stability of a temporary berm was used for propping design.



To establish the permeability of the underlying Bromsgrove Sandstone aquifer a specialist dewatering company, Dewatering Services, was contracted to undertake a pumping test. The initial trial comprised two 15m deep wells together with five 8m standpipe piezometers to measure the draw-down of the groundwater during the tests. A flowmeter was used to record the volume of water discharged. The pumping test results gave values of permeability of between

$8.04 \times 10^{-4} \text{ m/s}$ and $3.3 \times 10^{-3} \text{ m/s}$. From previous experience of the Bromsgrove Sandstone the lower permeability value of $8.04 \times 10^{-4} \text{ m/s}$ was adopted for determining well spacing and numbers.

The pad formation levels required the groundwater level to be reduced by a minimum of 3m. The calculated volume of water to be pumped for the site area was 1,500 gallons/min ($0.114 \text{ m}^3/\text{s}$). The anticipated capacity of the wells operating at the envisaged radii was 40 gal-

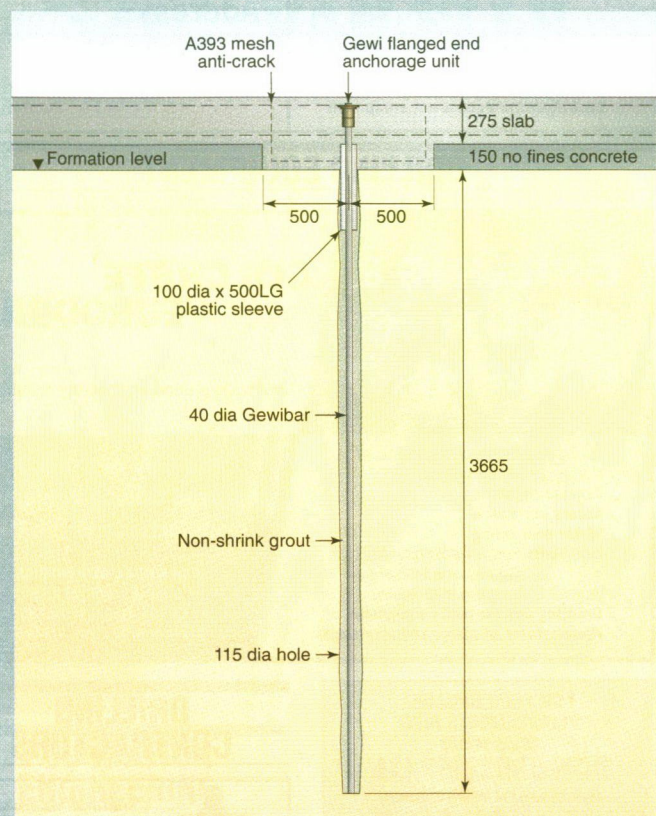


Figure 9: Typical rock nail design.

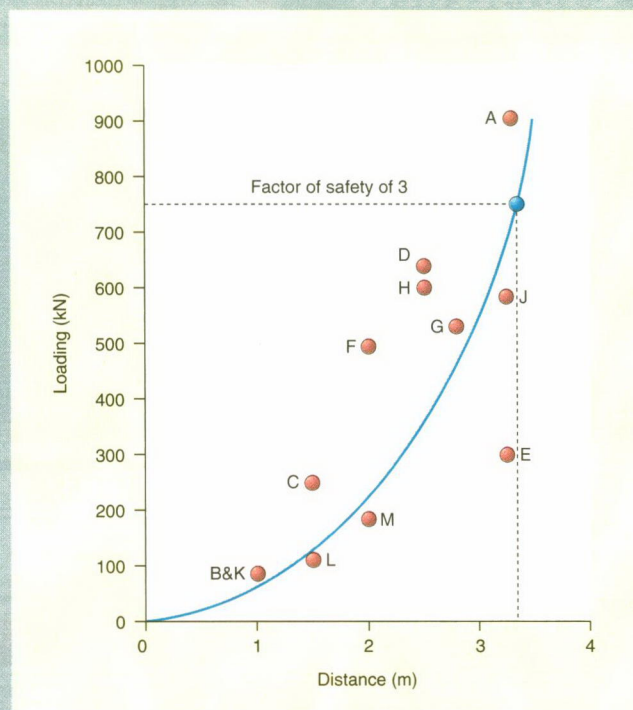


Figure 10: A summary of the rock nail pull-out tests.

lons/min ($0.003\text{m}^3/\text{s}$). This gave a required number of wells of 37.5. Normally this would have meant installing around 40 wells to increase the factor of safety but the contractor's previous experience indicated that 28 would be sufficient.

The wells were distributed across the site area as shown on Figure 3. A further three wells were added later because of the inclusion of a site access ramp and increase in the dewatering boundary.

The dewatering system was installed in March 2001 and was fully operational by early April 2001. By this stage excavation of pads adjacent to the temporary footbridge had already started.

These excavations suffered significant groundwater ingress below the 108.5m level and had to be dewatered using sump pumps. In addition, the groundwater caused both the sandstone and the marl to deteriorate. Further excavation was needed to achieve acceptable formations.

The dewatering produced a groundwater draw-down of at least 3m. The benefits to excavation were immediate in terms of construction time for a base. It also reduced the deterioration of the exposed surface allowing acceptance of some more marginal bases at a higher level.

Where formation levels were particularly deep, such as in

Debenhams, the excavation of bases would have required 150mm diameter capacity sump pumps, as used in West Mall North on a number of isolated deep bases. Formation surfaces at this depth below the at-rest groundwater level would also be very unstable.

The construction benefit of dewatering was obvious but this must be judged against the installation and running costs. The total cost for well installation plus 12 weeks hire of pumps and ancillaries was just over £100,000. This was considered cost effective when offset by the savings arising from increased speed of construction and reduced overdig.

Conclusions

Time and money were saved by rigorously reviewing all available data to derive relevant site specific parameters, and by developing alternative engineering solutions for the Bullring project. Monitoring and contingency measures were used, together with appropriate checking procedures, to provide controls and ensure the success of the works.

References

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Table 1: Rock nail pull-out tests

	Depth bgl (m)	Grout depth	Max load (kN)	Max extension (mm)	Max ground heave (mm)	Failure mode
A	3.325	fully bonded	900	15.74	5.14	none
B	1.000	fully bonded	75	6.93	0.38	ground
C	1.500	fully bonded	250	16.47	9.60	ground
D	2.575	fully bonded	625	39.93	18.09	ground
E	3.325	part bonded 750mm	300	14.78	1.56	ground
F	2.000	fully bonded	500	31.90	16.75	bar pulled out
G	2.575	part bonded 750mm	525	19.67	5.54	ground
H	2.575	part bonded 750mm	600	19.92	2.77	ground
J	3.325	part bonded 750mm	575	39.72	0.20	bar pulled out
K	1.000	fully bonded	75	35.47	6.76	bar pulled out
L	1.500	fully bonded	125	17.23	7.18	ground
M	2.000	fully bonded	175	16.62	1.89	ground

Table 2: Working pull-out tests

Test reference	Extension at 1.0 WL (mm)	Extension at 1.5 WL (mm)
S28	2.75	4.12
S29	3.55	5.37
EM19	4.15	5.38
EM22	3.00	4.21
WMN-1	2.25	3.80
WMN-2	3.80	5.56