The Design, Installation & Monitoring of High Capacity Antiflotation Bar Anchors to Restrain Deep Basements in Dublin

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Abstract
The construction boom in Dublin over the past decade has resulted in a demand for larger and deeper basements. These structures often extend several metres below the local ground water level and may have insufficient dead load to resist flotation.

This paper discusses four recent case histories where passive high capacity bar anchors, often-referred to as Anti-flotation Tension Minipiles (ATMs), provide the necessary restraint against hydrostatic uplift. In particular, it discusses opportunities where close coordination of the ATM designer and the basement slab designer can provide the optimum solution, both from a cost and programme perspective.

Due to the competent founding strata in much of Dublin, either the boulder clays, dense gravels or limestone bedrock, the use of ATMs is commercially attractive and offers several advantages over traditional stressed tendon anchors.

This paper concludes that full understanding of the appropriate design case and close interaction between the structural, geotechnical and ATM engineers is necessary to achieve the optimum basement solution.

Introduction
Ground water is the origin of many geotechnical problems and the understanding of the ground water regime over the life of a structure is of paramount importance. The depth of a basement below ground water level, the dead weight of the new structure and the construction method will dictate the requirement for antiflotation measures. The importance of simple uplift calculations should not be underestimated because hydrostatic forces can be considerable and the consequence of mistakes potentially catastrophic. Plates 1 & 2 show the basement car park of a large hotel in Dublin, constructed during the dry summer of 1996, with no consideration for long-term hydrostatic forces. This relatively light two-storey basement is founded within Dublin black boulder clay, which due to its low permeability appears dry when excavated, masking the actual porewater pressures present. Long-term standpipe readings in this stratum indicated the groundwater level to be 5m above the underside of the basement slab. Following construction, the gradual build up of hydrostatic forces exceeded the capacity of structural elements, resulting in failure of the columns, cracked floor slabs and

seepage into the basement. The retrofitting of antiflotation measures is difficult in such circumstances so long-term pumping to reduce uplift pressures is often the most suitable solution, albeit requiring a high ongoing operating and maintenance cost. However, this solution is only viable provided there are no adverse effects on adjacent services and structures, and that there is a suitable discharge location. Normally it is most economic to design in any antiflotation measures at an early stage.

Plate 1: Basement suffering failure due to hydrostatic uplift

Plate 2: Column failure due to hydrostatic uplift of slab

Historically a common solution to providing uplift resistance was to use stressed tendon anchors (Plate 3). This solution applies a pre-stress into the structure, which may be beneficial if there is poor ground below the basement slab, however there are several disadvantages:

Plate 3 –Stressing of Tendon Anchor

- Additional compression piles may be required to provide a reaction force as the anchors only provide tension resistance.
- Time consuming to construct due to the separate installation and stressing phases required.
- Cold joints formed where the anchor passes through the structure, with the risk of subsequent leaks if not sealed correctly.
- Maintenance is required over the life of the structure to ensure durability
- Anchor head pockets are necessary which set a restriction on the minimum slab thickness.
- Highly stressed tendons remain within the basement structure and substructure.
- Corrosion risks in a wet environment.
- Relatively high cost.

Deep basements in Dublin are often within competent strata where the alternative of passive high capacity bar anchors, also known as Antiflotation Tension Minipiles or ATMs, can be suitable (Figure 1). Compared with tendon anchors ATMs exhibit a number of advantages:

- Quicker construction as anchor head pockets are not needed and stressing is not required
- They can act both in compression (temporary) and in tension (permanent)
- The risk of leakage through cold joints does not exist as the anchors are cast integrally within and simultaneously with the basement slab
- No maintenance is required, unlike tendon anchors
- Thinner basement slabs can be employed, especially when integral anchor head plates are used (Plate 4)
- The uplift restraining forces develop passively with only a few millimetres of anchor head movement
- Basement restraint is possible upon the slab gaining its design strength, potentially allowing early decommissioning of construction dewatering systems
- Relatively low supply and installation cost

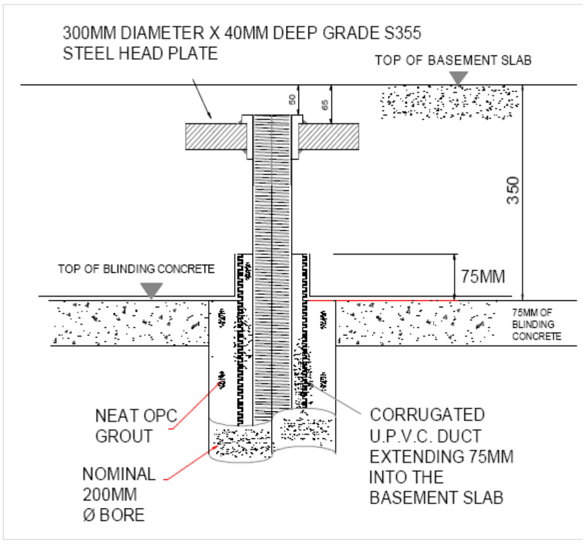


Figure 1 – Example of antiflotation passive bar anchor (ATM) head detail

Plate 4 – ATM head plate prior to casting of basement slab

This paper discusses the design and construction of four deep basement projects in Dublin where ATMs have been used during the past decade. The primary focus of this paper is on the redevelopment of Smithfield Market. This project involved one of the largest basements in Dublin (220m long x 110m wide x 10m deep) where 3No ATMs were instrumented with strain gauges and monitored over a two-year period covering both construction and building commissioning. Also discussed is the successful use of ATMs for deep basements at Spencer Dock (founding in boulder clay), Elm Park and Swords (both founding within strong limestone). Smithfield and Spencer Dock were uniform thickness raft slabs, while Elm Park and Swords were raft slabs with pad thickenings.

Typical Dublin Ground Conditions

The ground conditions in the city of Dublin are particularly suited to passive high capacity antiflotation minipiles (ATMs). Typically, in much of the city centre, made ground overlies glacial tills that overlies strong limestone bedrock. The glacial till consists of weathered upper brown boulder clay overlying a very stiff to hard black boulder clay. Alluvial gravels are present in the down stream port area where they overlie the glacial deposits. A buried preglacial gorge of the River Liffey, filled with dense water bearing gravels, lies to the north of the present course of the river. The Smithfield site discussed later is located on this buried gorge.

Design Philosophy

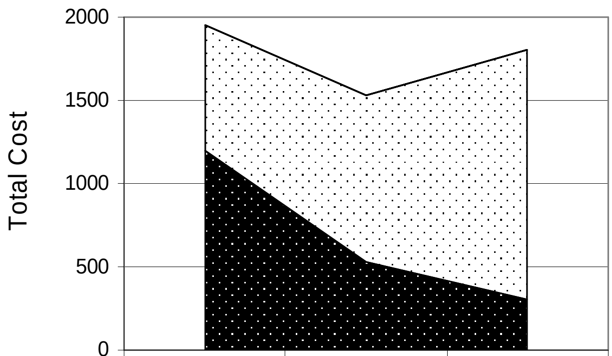
The design of antiflotation measures cannot be effective or economic unless the long-term ground water regime is fully understood. This can be done through site-specific ground investigation and groundwater monitoring over an appropriate period of time to understand tidal and seasonal effects.

Once this information is available, it is necessary to understand whether the basement is subject to either long term and/or short-term uplift. A simple check comparing the dead weight of the structure relative to the uplift force developed during short and long-term loading cases can be carried out to determine whether the basement requires uplift restraint. In this calculation, it is common practice to apply a partial factor to the dead weight of the structure (typically taking 90% of the building dead weight although this is dependent on the code of practice adopted). The pressure acting on the underside of the basement slab is then calculated from the depth of the underside of the slab relative to the standing groundwater level, or *worst credible* water level likely to occur for the specific loading case.

The total nett ‘working’ hydrostatic uplift force is therefore:

$$\textit{Nett Uplift Force} = \textit{Basement Area} \times 9.81\,kN/m^2 \times \textit{head of water} - (0.9 \times \textit{dead weight})$$

This is the force (if positive) which is to be resisted by the ATMs. The restraining effect of the basement walls is usually conservatively ignored in this calculation. Whilst this check is carried out for the structure as a whole it may also be necessary to check local areas of the structure, particularly where wide column spacing’s are employed, as antiflotation measures can lead to savings in member (slab) thicknesses even if only applied locally.



ATM Spacing (m)	2	3	4
Slab Thickness (m)	0.5	0.75	1
Slab Cost	750	1000	1500
ATM Cost	1200	533	300

Figure 2 – Schematic detail of the optimisation process showing variation in total cost with increasing ATM spacing and slab thickness

At this key stage, close cooperation between the ATM and structural engineers is required to obtain the optimum anchor layout and slab thickness. The anchor layout should fit uniformly between the column layouts, because the building dead load transmitted down the columns combines with the ATMs to resist the uplift forces. It is necessary to determine an efficient anchor spacing that reduces the basement slab thickness and reinforcement quantities to produce the optimum solution (Figure 2). Special integral head plates assist with this optimisation by locating the head plate near the top of the slab, which reduces the risk of punching failure. In the case of raft slabs with pad thickenings, this optimisation process can reduce the slab thickness considerably, which can lead to major cost and programme savings via reduced excavation (often in strong rock) and reduced quantities of reinforced concrete.

The calculated antiflotation anchor force from the long-term 'worst case' ground water level is defined as the anchor working load and can be designed for in accordance with the standard methods in the National and European Codes of Practice. In the Dublin area, the 'worst credible' flood condition is when the design water level is at ground level and this defines the 'ultimate' anchor force. This is because the water table cannot rise above the ground level without the basement flooding unless prevented from doing so, although such protection is rare in Dublin. Lower structural load factors (of close to unity) and lower geotechnical factors of safety (approximately 1.5 on the shaft friction) may be permissible with this 'ultimate' design force (which should also be the maximum test load for the project). However, this approach should ensure that a suitable factor of safety still exists under service conditions.

The main reinforcement element of the ATM anchor is typically a fully threaded rebar (GEWI or MAC500) with a yield stress of 500MPa, with diameters in the range 50mm∅ to 75mm∅. This rebar has a coarse thread and can be installed in sections if required, connected together using full strength couplers. The high capacity bar anchors are normally grouted in-situ by pumping a neat colloidally mixed (high shear) Ordinary Portland Cement grout, with a 0.4 water: cement ratio, through a small diameter tremie pipe into the base of the bore until completely full of clean grout. This grout mix typically achieves unconfined compressive strengths of 50MPa to 80MPa at 28 days. As the passive bar anchors generate their restraining forces micro cracks can form within the grout column. Therefore, an unperforated corrugated UPVC duct is installed over the full bar length, extending into the basement slab, to provide continuous corrosion protection to the bar, and protect against possible corrosion at the interface of the ATM and base of slab. As bar anchors operate at typically 30% of the stress levels of tendon anchors they are less susceptible to corrosion because of the large single rebar, rather than the numerous small diameter wires within tendon anchors.

Smithfield, Dublin

The Smithfield project is a 4-acre mixed-use development located in Dublin city centre, surrounded by public roads on all four sides. The development comprises over 350 apartments, a hotel and 139,000 ft^2 of offices, retail and leisure space. It includes a three level basement primarily used for car parking, but which also houses a 4-screen cinema, retail space, gymnasium with pool and plant room areas (Figure 3). The perimeter retaining walls for the basement were constructed using diaphragm walls. The diaphragm walls extended to bedrock, however only limited penetration into the strong limestone was possible and additional shear pins through the base of the walls were required. Penetration into the bedrock would have required costly and undesirable chiselling, resulting in excessive vibration of sensitive services. The structures found on a 900mm thick reinforced concrete raft slab bearing onto dense gravel at a depth of up to 13m.

The buildings at Smithfield are typically eight stories high with a 14-storey tower block. The existing ground water level is approximately 6m above the underside of the basement slab generating approximately 60kN/m^2 hydrostatic water pressure. Over the majority of the site, there was sufficient dead load to resist the uplift pressure; however, in some lightly loaded areas (e.g. below internal streets and atriums), there was a nett hydrostatic uplift force and in these areas ATM anchors were used. In addition, there were also areas where long spans were required in the building superstructure, for example in the cinema space and it was cost effective to utilise ATM anchors to reduce the span of the slab, rather than designing a thicker or more heavily reinforced slab to span the large column spacing.

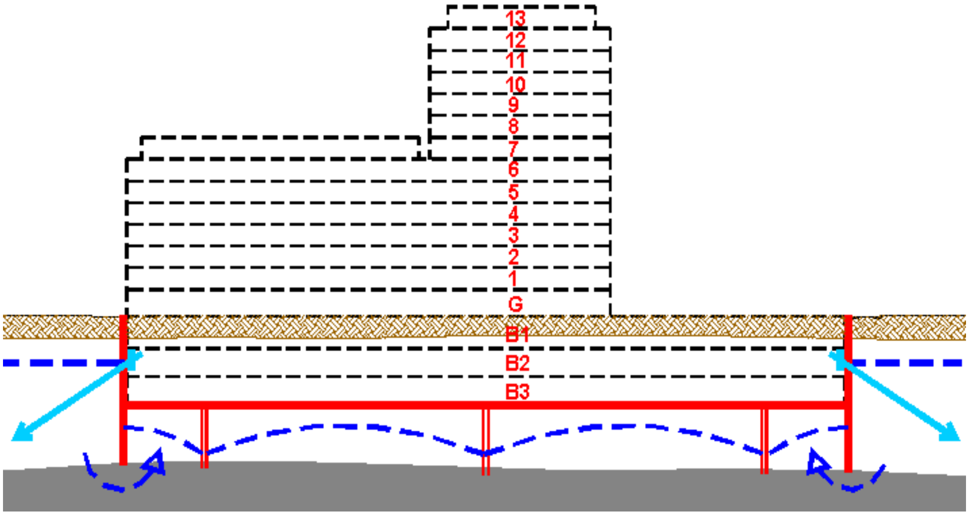


Figure 3: Schematic section of the Smithfield development

Geology & Groundwater

There was a phased site investigation for the Smithfield development, with a total of 20 rotary cable percussion boreholes and 7 rotary-cored boreholes, drilled between 1998 and 2002. These indicated the following general succession of strata:

Made Ground (1.8m to 7.5m thick)
Dense Gravel (9.0m to 13.0m thick)
Limestone Bedrock (encountered 13.8m to 21.0m below ground level)

Isolated sections of site encountered alluvium, medium dense sand and boulder clay. A significant proportion of the Standard Penetration Tests (SPT's) achieved "refusal" (penetration < 0.3m for 50 blows) on cobbles or boulders. All cable percussion boreholes recorded substantial chiselling time (typically 7 hours per location). Bedrock consisted of moderately to very strong, fresh to locally slightly weathered limestone, with Uniaxial Compressive Strength (UCS) values ranging between 11-158MPa (average of 100MPa). Solid Core Recovery (SCR) ranges from 49 to 87% and Rock Quality Designation (RQD) from 0 to 83%. The limestone bedrock in Dublin typically has a karst surface, facilitating flow paths through fissures and joints. The initial site investigation indicated standing groundwater at a depth of 4m to 6m. The second phase of site investigation provided more information on the rock profile as it varied across the site and rising and falling head permeability tests were carried out to estimate the permeability of the gravels. Ground water strikes were recorded in the high permeability gravel at between +0.23mOD and +0.99mOD. Ground level is at +4.5mOD and the underside of the basement slab is at – 6.0mOD.

A pumping test was undertaken within the bedrock to gain a more detailed understanding of the ground water regime and the permeabilities of the overlying gravel and limestone, and to optimise the design of the temporary dewatering system. Standpipes were monitored at varying distances from the pumping test to approximate a drawdown curve applicable to the gravel and limestone water tables.

ATM Design

The ATMs were generally designed in accordance with principles of BS8081:1989. During the design development, early consultation between the structural engineer and the foundation (ATM) contractor determined that a characteristic anchor-working load of 1,100kN was achievable. The engineer used this to determine the optimum anchor layout for the base slab taking into account slab spans. The bar anchor comprised of a 63mm diameter MAC500 (f_y = 500MPa) fully threaded bar with a full-length 160mm diameter semi-rigid unperforated corrugated sheath to provide corrosion protection. Generally, the anchor fixed length was designed upon a minimum 4m socket into strong limestone, but some contribution from the dense gravels was utilised where the bedrock dipped lower and ignoring the gravels was over conservative. An ultimate grout to ground bond stress of 2MPa was adopted for the rock socket design, although pullout tests by the bar manufacturers report ultimate bond stresses of up to 5MPa for ribbed bars of this type.

Cone pullout calculations gave higher factors of safety than an ATM ground to grout failure mechanism meaning the latter was the critical case. The anchors ranged from 10m to 16m below slab invert level. Each ATM was fitted with a 50mm thick by 500mm diameter steel head plate, secured above and below by full strength nuts. This was located within the upper 1/3 of the basement slab, below the top reinforcement mat.

Due to the tight construction programme there were no working ATM tests, however three preliminary tests were undertaken prior to construction of the working anchors. These were installed prior to general excavation due to site and programme constraints. To compensate for additional friction the upper section of the anchor was debonded from the ground. The ATMs were successfully proof tested to 1,552kN or 70% of the guaranteed ultimate tensile capacity of the bar.

ATM Installation

Three hundred and sixteen ATM anchors were installed in two phases spanning 2002 and 2003 (Plates 5 and 6). The bar anchors were installed from the slab blinding concrete cast onto the formation. A 220mm diameter Symmetrix drilling system was used to install temporary casing though the gravel. Once this casing was sealed into bedrock, the drill bit (191mm diameter) was advanced to the design depth. The centralised corrugated sheath and rebar were then installed and grouted insitu as detailed earlier.

Plate 5: Air Flush DTHH Drilling (showing ground water blown out of bore)

Plate 6: Bar anchors with head plates fitted and slab reinforcement being fixed

ATM Instrumentation & Monitoring Results

To examine the long-term performance of the ATMs, three permanent anchors were installed with strain gauges at 2m centres over their full depth and monitored for approximately two years, i.e. during the construction, commissioning and early operational phase of the development. Figure 4 is a typical graph of the forces inferred from the strain gauges for one of the instrumented ATM anchors. From October 2003 to February 2004, the readings are erratic due to the casting of the basement slab and nearby construction activities. Then, from February 2004 to September 2004, the graph shows eight months of gradual build up in compression force as the dead load from the superstructure increases. In September 2004, the dewatering system is decommissioned and hydrostatic uplift forces develop under the basement slab. Tensile forces then develop in the ATMs to resist these flotation forces. It takes approximately seven months for the ATM (hydrostatic) forces to stabilise. The graph shows that the hydrostatic forces are greater than the self-weight of the structure because all strain gauges on the ATM rebar indicate tension forces of between 25 and 75kN. This is considerably less than the 1,100kN design loading and is probably due to inevitable load sharing between the basement structural members.

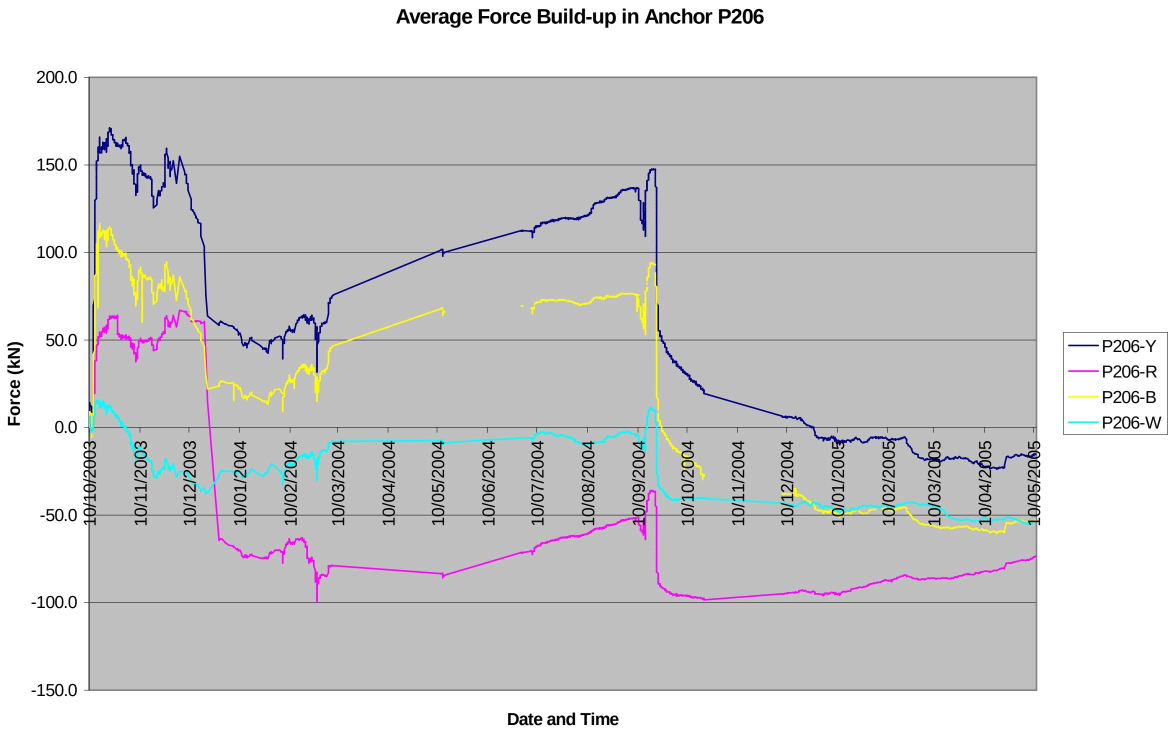


Figure 4: Load variation monitored in anchor P206

Discussion

Programme certainty was a key driver for the Smithfield development. The use of ATM anchors at Smithfield resulted in substantial programme and cost savings compared to conventional post-tensioned tendon anchors. In addition to their use where there was an overall nett uplift force they were also found to offer economic benefits by reducing long spans and hence providing reduced slab thickness or reinforcement requirement. The results of long term monitoring confirmed the build-up of uplift pressure once the temporary dewatering system had been decommissioned. This increase in water pressure occurred rapidly at Smithfield, however, this may not be the case elsewhere in Dublin where lower permeability soils exist below the basement.

Spencer Dock, Dublin

The site is located to the east of Dublin City Centre, immediately north of the River Liffey within the Spencer Dock Redevelopment and comprises a two-level 8m deep basement of approximate plan dimensions 135m x 60m. Ground water level is approximately 3m below street level. The site was for the construction of Price Waterhouse Coopers headquarters building, comprising three tower blocks interspaced with atrium areas and open spaces. Below the atrium and car park areas, a 6m hydrostatic head acts on the underside of the basement slab resulting in significant nett uplift areas. High capacity ATMs founded within the very stiff to hard boulder clay were used to resist these flotation forces.

Geology & Groundwater

The overlying made ground, alluvium and gravels were removed during the basement excavation and the antiflotation bar anchors were installed from the slab blinding concrete level, cast directly on top of the boulder clay. This low plasticity glacial deposit (PI 10-15%) extends down to limestone bedrock, a further 12m below. Cable percussion boreholes normally only penetrate 3 to 4m into the black Dublin boulder clay, even with significant chiselling. SPT 'N' values are between 40 and refusal with mean values of 100 to 200, which classifies the material as very stiff to hard with an undrained shear strength of 500 to 1,000kN/m^2 (Skipper et al., 2005 & Mentiki et al., 2004). Core drilling through this stratum was of limited use, because the water flush washed out the clay matrix, leaving only granular material, meaning good quality undisturbed samples of Dublin boulder clay were difficult to obtain (although the use of triple core barrel sampling has become more prominent in recent times). The groundwater level fluctuates from - 0.7m AOD to +0.3m AOD, relative to a ground level of +3mAOD, and is therefore loosely related to the tidal effects of the River Liffey.

Bar Anchor Design & Installation

Plate 7: Drilling for bar anchors at Spencer Dock using DTHH techniques

Plate 8: Tension test reaction system at Spencer Dock

Six hundred and eighty two x 600kN safe working load ATM anchors were specified by the engineer with a maximum allowable anchor head movement in service of 5mm. Design calculations required a 9.0m penetration into the boulder clay for a 220mm ∅ anchor, which equates to a working grout to ground bond stress of approximately 100kN/m^2. This 9m penetration is within the 10m maximum bond length limit recommended in clause 6.2.3.4 of BS8081:1989. The reference design, based on anchors founding in the underlying limestone bedrock, would have required significant 'free' lengths, but this was deemed unnecessary due to the competent nature of the overlying Dublin boulder clay. The 5mm allowable anchor head movement at SWL is demanding and therefore a 63mm ∅ rebar was used, which ensured a stiff reinforcing element and a high structural load factor (>3.0). The ATMs were drilled using an air flushed down-the-hole hammer (DTHH) technique in order to penetrate the ground quickly and efficiently (Plate 7). The vast majority of ATMs could be drilled using open-hole techniques, but a small number of holes encountered pockets of gravel, which caused the bore to collapse and in such instances a Symmetrix drilling system was used to install temporary 220mm Ø drill casing to the full depth. The ATMs were then grouted in-situ as detailed previously. As the top level of the bar was a critical dimension, the bores were over drilled by 0.5m and the rebar was suspended centrally in the fluid grout at the specified level. A corrugated duct was installed over the full depth of the hole, thus ensuring the rebar had 500mm of grout cover within a corrugated UPVC duct at its lower end. A 300mm diameter x 35mm thick steel head plate, with full strength nuts above and below was provided at the head of each ATM. The column-punching shear onto the slab and the underlying boulder clay dictated the use of a 1m thick basement slab.

Load Test Results & Discussion

Four preliminary and six working (acceptance) tests were undertaken in accordance with Tables 13 and 18 from BS8081:1989. The reaction beams (Plate 8) were located a minimum of 1.5m from the anchor to prevent any influence on the anchor performance. The preliminary tests of up to 1,330kN showed no indication of failure, equating to a proven geotechnical factor of safety of 2.2. The ‘working’ acceptance test results are summarised in Table 1 and these show average pile head movement of 2.95mm at SWL, comfortably within the 5mm specification.

Table 1 Summary of Static Test Pile Results at Spencer Dock

Minipile Detail	Elastic movement (mm)	Permanent movement (mm)	Total pile head movement (mm)	Apparent free tendon length (mm)	Free tendon length (mm)	Tendon bond length (mm)	Free length + 50% of tendon bond length mm	Notes
Pile 67	1.88	0.00	1.88	1,360	1,500	9,000	6,000	OK
Pile 16	2.19	0.44	2.63	1,581		9,500	6,000	OK
Pile 76	3.70	0.65	4.36	2,676		9,500	6,250	OK
Pile 84	3.93	0.74	4.67	2,839		9,000	6,000	OK
“OK” BS 8081, section 11.2.12 states that the apparent free tendon length should not be less than 90% of the free intended in the design nor more than the intended free length plus 50% of the tendon bond length intended in the design								

Elm Park & Swords, Dublin

These two projects had several similarities, which allowed full optimisation of the antiflotation schemes. The Elm Park Development is a multi-functional development (residential, commercial, medical and hotel & conference units) located 4km southeast of Dublin City Centre. It covers an area of 75,000m^2 with an 8m deep basement over large areas. The Swords Primark Development, 8km north of Dublin City Centre is a residential and retail development, covering 22,500 m^2 with an 8m deep basement over the majority of the site.

Geology & Groundwater

The basement slabs on both sites found directly onto strong limestone bedrock, with the ground water level approximately 2m below ground level i.e. an uplift pressure of 60kN/m^2. The limestone bedrock is strong to very strong limestone, slightly weathered over the top metre, with UCS values of 100 to 200 MPa, locally referred to as 'Calp' limestone.

Bar Anchor Design & Installation

These most recent projects allowed the full use of the optimisation techniques described above due to the strong limestone at basement slab level and early discussions between structural and geotechnical engineers. ATM working loads of 1,000 to 1,350kN were adopted, in conjunction with integral head plates within the slabs, which allowed reduced slab thicknesses as low as 400mm to be used. These thin slabs were possible because of the strong limestone immediately below preventing the columns punching through the slab. The 6m deep anchor bores were all formed using air flushed open hole DTHH drilling techniques. This quick and efficient drilling system forms holes with good verticality tolerances and a rough bore, resulting in a good grout to ground bond values. The design of the anchor length was governed by global stability and potential interaction, rather than ground to grout bond stresses. These works were carried out on a rolling programme working off blinding concrete using relatively small 12 tonne Casagrande C6 Drill Rigs. This is an efficient method of working with the Main Contractor, using only small rigs in localised areas of the site. The final scheme at Swords used 566No x 1,000kN SWL ATMs on a 4m x 4m grid spacing (Plate 9).

Test Results & Discussion

The anchor test results at both sites were excellent with head movements being primarily elastic extension, moving ~ 2.2mm at 1,700kN (Figure 5).

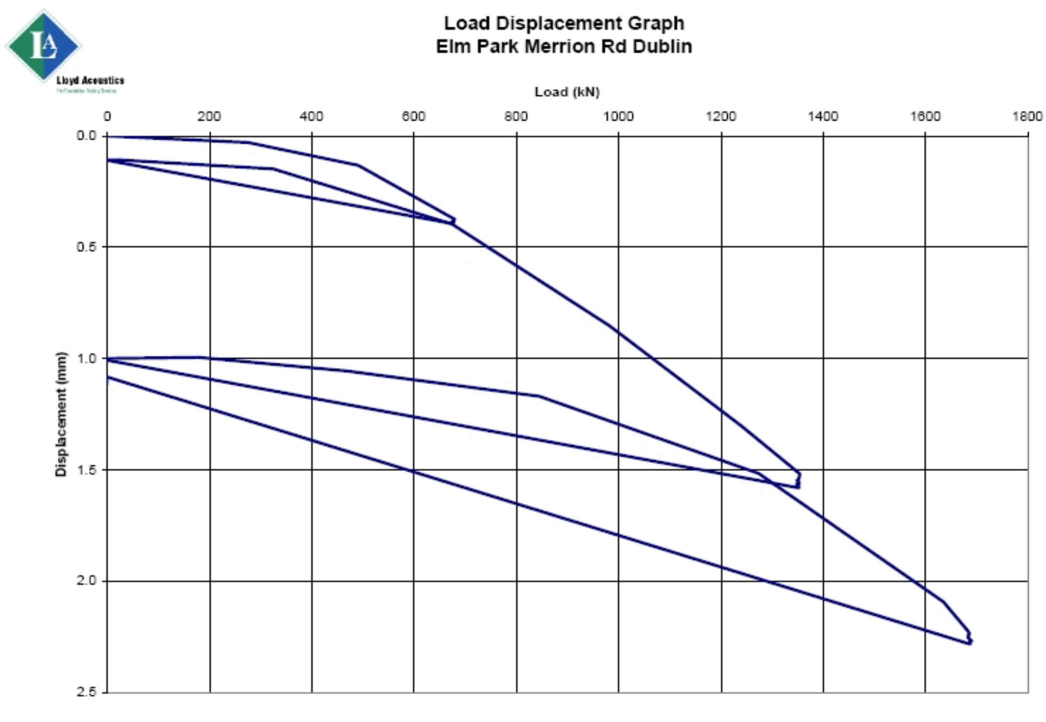


Figure 5: Static load test result at Elm Park

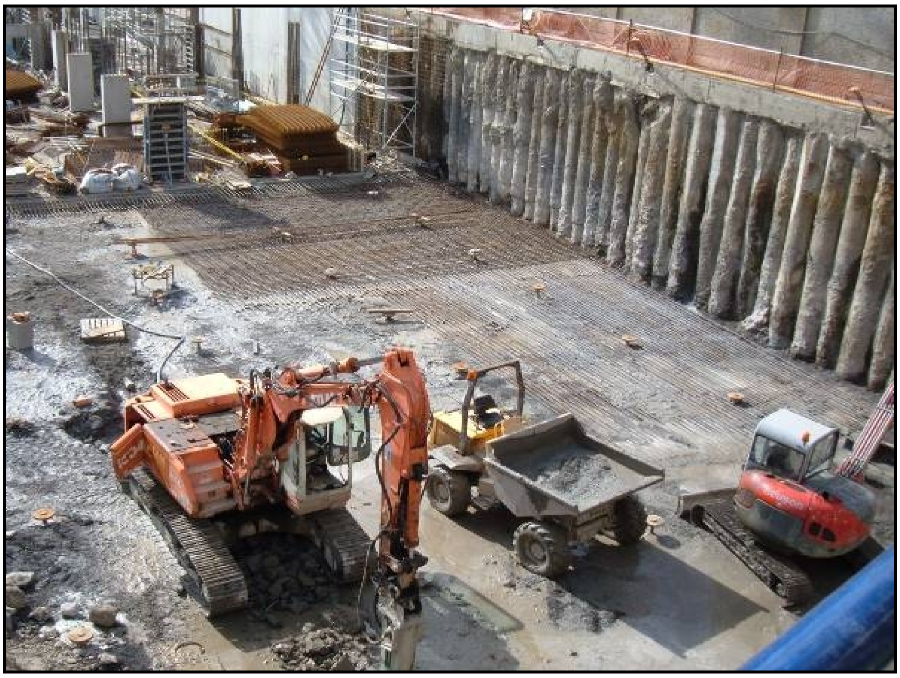


Plate 9: Basement construction at Swords

Conclusions

This paper has presented a number of applications where high capacity bar anchors (ATMs) have been used to resist flotation forces acting on basements in the Dublin area. The ground conditions in the Dublin area make the use of passive high capacity bar anchors particularly appropriate. This type of antiflotation solution offers a number of engineering, cost and construction programme advantages over more traditional forms (such as tendon anchors or permanent dewatering). However, to ensure that optimum benefit is achieved from their use early cooperation and discussion is necessary between the structural engineer and geotechnical designer (specialist contractor). In addition, an understanding of the groundwater conditions during short and long-term load cases is essential.

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