Granular Anchors Under Vertical Loading/Axial Pull

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Abstract

Granular anchors are a relatively new concept in ground engineering with relatively little known regarding their load–displacement behaviour, failure modes, ultimate pullout capacity and also potential applications. A granular anchor consists of three main components: a base plate; tendon and compacted granular backfill. The tendon is used to transmit the applied load to the base plate which compresses the granular material to form the anchor. A study of the load–displacement response and ultimate pullout capacity of granular anchors constructed in intact lodgement till and made ground deposits is reported in this paper. Parallel tests were also performed on cast *in-situ* concrete anchors which are traditionally used for anchoring purposes. A new method of analysis for the determination of the ultimate pullout capacity of granular anchors is presented and verified experimentally, with the dominant mode of failure controlled by the column length to diameter ratio. Granular anchors with L/D > 7 principally failed on bulging whereas short granular anchors failed on shaft resistance, with the latter mobilising similar pullout capacities as conventional concrete anchors.

Key words: Ground improvement, anchors, retaining structures

INTRODUCTION

Granular columns are traditionally used for improving weak deposits, and under suitable conditions, offer a valuable means of increasing the bearing capacity of foundations and stability of embankments founded on soft ground as well as reducing total settlement and increasing the rate of consolidation. There has been some discussion in recent years as to whether granular columns could also be used to resist tension/pullout forces (Phani Kumar and Ramachandra Rao 2000, Liu *et al*. 2006, Srirama Rao *et al*. 2007, Madhav *et al*. 2008, Phanikumar *et al*. 2008). Such granular anchors consist of a horizontal base plate, a centrally-located tendon (stretched cable or metallic rod) and compacted granular backfill. The tendon is used to transmit the applied load to the column base via the circular base plate, which compresses the granular material to form the anchor. The load can be applied to the anchor immediately after its construction and drainage is also provided, via the granular column, to the soil surrounding the anchor. Granular anchors have been used, for example, to prevent uplift caused by flooding (Liu *et al.* 2006) and resist heaving of foundations in expansive clays (Srirama Rao *et al*. 2007), and in such scenarios, have many applications for lightly-loaded civil engineering structures, including residential buildings and pavements. However, granular anchors can have much wider applications in the construction industry, not only to enhance the stability of retaining structures, rock faces or sheet piles but also to act as an effective drainage system in order to prevent excessive build-up of pore water pressure, particularly in slope stabilization. However, research is required to understand the load–displacement response, failure mode(s) and ultimate pullout capacity of granular anchors, and importantly how they can be appropriately integrated into routine civil engineering construction. This is the premise that forms the basis to the research described in this paper.

EXPERIMENTAL PROGRAMME

The experimental studies reported in this paper were performed in three parts. The focus of the first part was to compare the ultimate pullout capacity of granular anchors in direct pullout against that of conventional cast *in-situ* concrete anchors. The ultimate pullout capacity is the load at which the anchor is pulled out of the ground, either by failure in shaft resistance mobilised between the granular/concrete column and surrounding soil or alternatively, in the case of granular columns, by localised end-bulging of the column itself (Hughes and Withers, 1974). These tests were performed at Queen's University Belfast (QUB), with the experimental programme considering the assessment of two variables; namely anchor lengths (L) of 0.5, 1.0 and 1.5 m, and anchor diameters (D) of 0.07 and 0.15 m. Incremental loading of the anchors in direct tension was achieved using a custom-built loading device (Fig. 1) in which a bucket supported on a loading arm of 3.0-m in overall length was progressively filled with concrete cubes, each weighing ~64 N. The safe capacity of the loading bucket was 600 kg, which with a lever-arm ratio of 5:1, generated a possible maximum tension force of ~30 kN on the anchor tendon. The 1.2 × 0.75 m supporting platform spread the reaction from the frame in order to reduce the bearing pressure on the supporting soil.

The second part of the investigation was performed at the Santry Sports Grounds of Trinity College of Dublin (TCD) and examined in greater detail the performance of granular anchors having different configurations, again considering the two variables of L = 0.5, 1.0 and 1.5 m, with D = 0.15 and 0.20 m nominally. The tension/pullout loading to the anchor tendon was applied using a hydraulic jack supported on a heavy steel reaction frame (Fig. 2). The legs of the reaction frame were sufficiently distant from the centrally-aligned tendon so as not to influence the anchor response.

The load–displacement response of the ground anchor system was measured using load cells and long-stroke displacement transducers (see Figs. 1 and 2). The vertical displacement of the ground surface was also measured at a distance of 0.3 m radially from the anchor tendon by a displacement transducer mounted on an independent reference beam (LVDT2 in Fig. 2). Load cells of 30 and 300 kN capacities were used to measure the applied anchor load for the QUB and TCD tests, respectively, with the mobilised load resistance recorded after a period of one minute following the application of each load increment.

A single test was also performed at the TCD site in order to examine the viability of using double anchor plates for the purpose of increasing the ultimate pullout capacity by inducing bulging failure at two locations along the granular column. Due to constrains, this aspect was not fully examined by means of full-scale field tests. Hence the third part of the study involved performing laboratory model studies at QUB (Fig. 3), in which soft-firm stone-fee clay (undrained shear strength c_u of ~30 kPa) was packed into a wooden box of dimensions 1.2m×0.7m×0.7m in depth. Three column configurations were examined: (a) L = 0.7 m and D = 0.035m, with a single plate located at the bottom of the column; (b) L = 0.7 m and D = 0.035 m, with a plate located at the bottom and a second plate located at mid height of the column; (c) L = 0.35 m and D = 0.035m,with a single plate located at the bottom of the column. Pull out loading was applied using a pneumatic activator attached at the top of the reaction frame (Fig. 3).

Ground conditions

The granular anchors at the QUB site were installed in made ground that had been placed about 50 years previously, and was classified as firm to stiff clayey silty sand with occasional gravel. Mean values of c_u of 55 kPa were measured for depths greater than 0.5 m below the ground surface, with slightly higher c_u determined for shallow depths. The in-situ bulk unit weight was 21kN/m^3. Hand augurs with the relevant diameters were used to bore holes in the ground in which the anchors were constructed. Further details on the 5 tests (designated QUB1–5) performed on these granular anchors are reported in Table 1. In addition, 4 tests were performed on concrete anchors.

All of the anchors at the TCD site were installed in the Upper Dublin Brown Boulder Clay (UDBrBC) formation; a heavily-weathered stiff to very stiff, brown, slightly sandy clay of low plasticity, with rare silt/gravel lenses. The geotechnical properties of the Dublin Boulder Clay have been reported by Farrell *et al.* (1995) and Long and Menkiti (2007), among others. Borehole logs for the TCD site indicated that the UDBrBC layer was ~1.8 m in thickness across the test area, with mean values of water content of 12%, bulk unit weight of 23 kN/m^3 and a relatively high stone content (> 20 mm in particle size) of between typically 5% and 10% measured over this depth. The standing groundwater table was located at ~1.8–2.0 m below the ground surface, appearing to approximately coincide with the transition boundary between the UDBrBC formation and underlying Upper Dublin Black Boulder Clay formation. Larger bores of nominally 0.15 and 0.20 m in diameter were formed at this site by professional drillers using a light cable-percussion drilling rig. Boreholes ~0.5 m in depth were formed using the clay cutter only, whereas deeper holes were formed using the clay cutter in combination with a temporary steel casing, in accordance with British Standard BS879 (BSI, 1985). Hence, with the casing removed, the actual bore diameter of the deeper holes was equivalent to the outer casing diameter; i.e. precisely D = 0.168 and 0.219 m for holes nominally 0.15 and 0.20 m in diameter. Further details on the 9 tests (designated TCD1–9) performed on these granular anchors are reported in Table 2.

Anchor installation

Uniformly-graded basalt gravel (nominally 10-mm in size and with an angle of shearing resistance ϕ'_g of 45^o for the density achieved in the anchor setups) was used as backfill for the QUB and TCD granular anchors and also as coarse aggregate in forming the QUB concrete anchors. In the QUB laboratory model studies), the backfill material was uniformly-graded basalt having particle sizes between 2.36 and 3.35 mm. In constructing the anchors, the steel base plate with the tendon (threaded steel rod) was inserted to the base of the borehole (Fig. 4a). The base plate diameters of 0.148 and 0.196 m used at the TCD site were marginally less than the diameters of the deeper holes since a temporary casing had been required in forming the bore, which also had the effect of producing a smooth borehole sidewall. In the case of the granular anchors, the borehole was backfilled by pouring the gravel into the bore cavity to form ~0.12 m thick layers, which were individually compacted to achieve maximum density using a special hammer, comprising an annular compaction-plate and hollow tube assembly (Fig. 4b), which fitted down around the anchor tendon. The mass of the hammer was ~2.5 kg and the gravel layers were compacted, in turn, by dropping the hammer 27 times through a free-fall distance of 0.7 m, which produced a bulk unit weight for the gravel of 22 kN/m^3. In the case of the concrete anchors, the bore cavity was backfilled with a concrete mix prepared at a water-cement ratio of 0.55 in ~0.1 m layers which were tamped using the same procedure used for the granular anchors. The concrete anchors were allowed to cure for 7 days before performing the tension/pullout load tests.

EXPERIMENTAL RESULTS

QUB Site

The experimental results of the first part of the study at the QUB test site, which compared the performance of granular and conventional concrete anchors, are reported as tension load against vertical anchor displacement in Fig. 5. The pullout capacities of the granular and concrete anchors of L x D = 0.5 x 0.07 m were 5.5 and 5.2 kN respectively (Fig. 5a). The granular anchor displaced significantly (> 40 mm upward movement of the top surface of the gravel column) during the course of loading compared with the concrete anchor, although the displacement of the latter at the time of failure was considerable (i.e. sudden pullout occurred), implying both of these anchors failed on resistance mobilised along the column shaft. The soil surrounding the concrete anchor did not undergo any significant displacement (either heave or subsidence) until the failure state was achieved. However the soil surrounding the granular anchor progressively heaved as the anchor was incrementally loaded to failure. Anchors of L x D = 0.5 x 0.15 m also failed on shaft resistance (Fig. 5b), experiencing ductile and sudden pullout behaviour for granular and concrete constructions, respectively, with mobilised pullout capacities of 6.7 and 8.0 kN respectively. The granular anchor of L x D = 1.0 x 0.07 m experiencing ductile failure, undergoing localised end-bulging (Fig. 5c), whereas the concrete anchor experienced sudden pullout, failing in shaft resistance. Pullout capacities of 16.1 and 16.3 kN were mobilised for these granular and concrete columns respectively. During the early loading stage, the surrounding ground barely moved, although ground heave started to occur as the anchors approached pullout capacity. The 1.0 and 1.5 m long anchors of 0.15 m diameter (Fig. 5d) could not be taken to true failure since this exceeded the capacity of the loading system used in performing these series of tests. Nevertheless, it would appear from the load–displacement responses in Fig. 5d that failure of both concrete and granular anchors was imminent at the time when the loading had to be terminated prematurely, particularly in the case of the 1.0 m long anchors.

TCD Site

The experimental results of the second part of the study performed at the TCD test site are shown in Fig. 6, including additional data of the vertical displacement response of the surrounding ground measured at a distance of 0.3-m radially from the anchor tendon. Short anchors of L x D = 0.45 x 0.148 m and 0.5 x 0.196 m (Fig 6a&b) failed on shaft resistance, mobilising a pullout capacity of ~12 kN, with a visual observation of the surface of the gravel backfill lifting in addition to substantial heave of the surrounding ground occurring once the applied load exceeded 10 kN. An increase in anchor length and/or diameter produced an increase in pullout capacity. Anchors having L = 0.96, 1.0 and 1.3 m with D = 0.219 m mobilised pullout capacities of 39, 42 and 44 kN respectively (Fig. 6a). In the case of 0.168 m diameter anchors, the pullout capacities were 33, 40 and 42 kN for L = 0.8, 1.47 and 1.62 m respectively (Fig. 6b). A marginal ground heave (~1 mm) was observed at failure in the case of 0.219-m diameter anchors of L = 0.96 and 1.3 m (Fig. 6c). However, vertical displacements recorded at the ground surface were insignificant (~0.2 mm) in the case of the 0.168-m diameter anchors of L = 1.47 and 1.62 m (Fig. 6d), even though the anchors themselves had been displaced by more than 100 mm.

The applied anchor load is resisted by the bulging capacity (Hughes and Withers, 1974) of the granular column in the vicinity of the base plate and by shaft resistance mobilised along the column shaft. Hence mobilisation of multiple bulging locations may contribute to enhanced loading capacity. This possibility was examined in one of the 0.219-m diameter anchors (TCD9, Table 2) for which a second anchor plate was positioned 0.7 m vertically above the base plate which was located at 1.4 m depth. The relevant load–displacement curve is shown in Fig. 6a. The anchor resistance initially plateau at ~40 kN, but a step increase in the load resistance subsequently occurred for larger displacements (> 90 mm), followed shortly afterwards by pullout failure at an anchor load of 44 kN. The fact that the pullout capacity mobilised by this 1.4-m long double-plate anchor was less than that achieved by the 1.3-m long single-plate granular anchor required further investigation and this will be covered later in the discussion section. Also note that in one of the anchor tests, the load on the anchor was temporarily removed and then re-applied (Fig. 6a), with the result that the unload–reload process substantially increased the stiffness of the composite anchoring system.

DISCUSSION

Various methods of analyses that consider different failure modes (including vertical slip, cone, circular arc) exist for the determination of the ultimate pullout capacity of ground anchors constructed in homogeneous deposits of either sand or clay (Meyerhof and Adams 1968, Ilamparuthi *et al*. 2002, Merifield and Sloan 2005). However, in the case of granular anchors, the bore is backfilled with compacted granular material that is generally significantly different from surrounding native material. Under these conditions, the failure mode can be complex and may involve localised bulging failure at the base of the granular anchor (Hughes and Withers, 1974), mobilization of shaft resistance and/or wedging failure, as illustrated in Fig. 7a.

In the full-scale studies performed at the QUB and TCD test sites, the granular anchors generally failed at anchor displacements of ~60 mm. If the bulging mechanism was the main cause of pullout failure, the enlargement in diameter occurring at the base of the granular column may be ~10% of its original diameter at this anchor displacement, assuming the length of bulging was twice that of the column diameter and no significant movement of the gravel backfill occurred above the bulging zone. This localised and marginal increase in column diameter may not be sufficient enough to trigger a wedging failure mode. Hence, as a first approximation, only shaft resistance and localised end-bulging modes are considered in the following method of analysis proposed for granular anchors.

The loading applied to the anchor tendon is simultaneously resisted by localised bulging in the vicinity of the column base and by shaft resistance developed over the column shaft (Fig. 7b), with the dominant failure mode governed by the column L/D ratio (see later). In analogue to the ultimate pullout capacity of a rigid pile, the ultimate resistance of the granular anchor in shaft resistance, including its self-weight contribution, is given by

$$T_F = \pi D L \alpha c_u + \frac{\pi D^2 L \gamma_g}{4} \qquad \text{Equation 1}$$

where L and D are anchor length and diameter respectively; c_u is the undrained shear strength of the surrounding soil; α an adhesion factor and γ_g is the unit weight of the granular backfill.

Note that vacuum cannot develop in the cavity that forms directly below the base plate during pullout on account of the open pore structure of the granular column. The local bulging capacity of the granular column itself is given by

$$T_B = \frac{\pi D^2 \sigma_v}{4} \qquad \text{Equation 2a}$$

with the bearing pressure at the column base σ_v estimated using the relationship proposed by Hughes and Withers (1974):

$$\sigma_v = \left[\frac{1+\sin\phi'_g}{1-\sin\phi'_g}\right]\left[\sigma_{vc} + N_c^* c_u\right] \qquad \text{Equation 2b}$$

where σ_{vc} is the overburden pressure caused by the surrounding soil at the point of bulging; ϕ'_g is the angle of shearing resistance of the granular column and N_c^* is a bearing capacity factor that considers local shear failure. Gibson and Anderson (1961) proposed that the value of N_c^* was given by

$$N_c^* = 1 + \log\frac{G}{c_u} \qquad \text{Equation 3}$$

where *G* is the shear modulus of the soil.

In the present investigation, G/c_u = 100 was assumed for the TCD site, and accordingly N_c^*= 4.6. The undrained shear strength against depth profile of the surrounding soil is the crucial piece of information required for the prediction of anchor performance/mode of failure, with shaft resistance mobilised along the full length of the column shaft whereas bulging occurs locally in the vicinity of the column base.

QUB Site

The experimental programme at the QUB site considered the assessment of anchor length (0.5, 1.0 and 1.5 m) and diameter (0.07 and 0.15 m) on ultimate pullout capacity. Control tests were also performed using concrete anchors of similar dimensions. The strength against depth profile of the ground determined using a hand vane indicated an average undrained strength of 55 kPa for depths greater than 0.5-m below the ground surface (Fig. 8).

In granular column applications for ground improvement, the column can fail by one of two distinct mechanisms. As the load increases on the granular column, the shaft resistance developed along the cylindrical surface and the end bearing resistance developed at the base of the granular column are mobilised gradually. This is typical for short columns and for values of L/D ratio < ~6–7 (Black *et. al*. 2011, Sivakumar *et. al*. 2011, Wood *et. al.* 2000, Hughes and Withers, 1975). In contrast, longer columns fail in localised bulging occurs in the vicinity of the column head since the shaft resistance and end bearing capacities exceed the bulging capacity. This analogy can be extended to granular anchors, with the proviso that bulging in granular anchors occurs close to the bottom of the column.

Failure over the column length would occur due to a shear zone developing within the remoulded soil next to the bore sidewall and not along the granular/soil interface since no distinct granular surface forms, with the confined granular material intruding slightly into the adjacent soil under pullout loading. Hence $\alpha = 1$ is assumed in determining the shaft resistance. This is also supported by back-calculating the value of α from the observed performance of the concrete anchors. Table 1 lists values of predicted shaft resistance and bulging capacities together with measured pullout loads at the termination of each test. Note that loading was terminated at 30 kN load for one of the anchors (QUB5) on account of the load cell capacity being reached. Based on available information, it can be concluded that the 0.07 and 0.15-m diameter by 0.5-m long anchors failed in shaft resistance whereas the 0.07 and 0.15-m diameter by 1.0-m long anchors may have failed by localised end-bulging. This postulation is further illustrated by plotting bearing pressure acting on the column base against L/D ratio (see Fig. 9). Included in this figure and indicated by a broken line, is the mobilisation of shaft resistance for an average undrained shear strength of 55 kPa over the column length. Based on the results obtained, it can be concluded that the 0.07 and 0.15-m diameter by 0.5-m long anchors failed on shaft resistance whereas the other anchors may have failed on bulging. Furthermore the work clearly suggests that the L/D ratio which distinguishes whether pullout failure occurs in shaft resistance or localised end-bulging is about 7. The results from the QUB model studies on double-plate capacity will be discussed later in this paper for clarity reasons.

TCD Site

Figure 10 shows the undrained strength against depth profile for the TCD site. *In situ* probing and laboratory strength measurements were made using a 20-tonne CPT truck and unconsolidated-undrained triaxial compression tests performed on 100-mm diameter by 200-mm high specimens reconstituted by standard Proctor-compaction of material at its natural water content that had been recovered using the clay cutter during borehole formation. A few 'undisturbed' specimens that had been obtained from just below the base of the boreholes using a 38-mm diameter sampling tube were also tested in triaxial compression. The CPT-derived peak undrained shear strength $c_u = (q_c - \sigma_{vo})/N_{kt}$, where q_c is the CPT cone-tip resistance and σ_{vo} is the overburden pressure. However no major study of this relationship has been reported in literature for Dublin Boulder Clay, mainly because of limited penetrations achieved, and the q_c profile also tends to be 'spiky' due to the presence of stones and inherent variability of the material. This was collaborated by significantly higher gravel contents observed at certain levels within recovered borehole cores. Hence an average value of N_{kt} = 15, given by Lunne et al. (2002) for lodgement till deposits, was deemed appropriate. Unsurprisingly the CPT peak c_u was consistently greater than laboratory measurements on reconstituted specimens. However, since granular anchors are generally taken through large displacement and interaction between the granular material and surrounding soil is more intense than may prevail in the case of rigid piles, strength parameters obtained from remoulded test-specimens are considered appropriate in this analysis. An average c_u = 55 kPa was used for depths of up to 0.8 m below the ground surface, with a step increase to c_u = 80 kPa assumed for greater depths (Fig. 10).

Predicted anchor loads based on failure in shaft resistance and localised end-bulging modes (Eqs. 1 and 2 respectively) are listed in Table 2. Note that ultimate pullout capacity by failure in shaft resistance increases linearly, and is strongly sensitive to, increasing L/D ratio. Bulging capacity depends on G/c_u, ϕ'_g and L/D ratio on account of the increase in confining stress and undrained strength with depth, although for a given column diameter, the experimental pullout capacity by bulging failure was found to increase only marginally with increasing L/D ratio (see Table 2). Shaft failure is generally dominant in short columns whereas bulging failure can be expected in longer columns. In the case of the 0.168-m diameter anchors, the longer columns with L = 1.47 and 1.62 m failed on bulging (L/D = 8.8 and 9.6 respectively). Furthermore the ground heave measured for these anchors was insignificant (Fig. 6c), validating the argument for bulging failure having occurred in the vicinity of the column base. Similar diameter columns but with L = 0.45 and 0.80 m failed on mobilazation of shaft resistance, although the pullout capacity for the latter was noticeably higher than the predicted shaft resistance. This may be due to some variability in strength due to heterogeneity of the lodgement till material at the location of the testing. The occurrence of shaft failure was further substantiated in the case of the 0.45-m long column, which underwent significant ground heave from the start of loading (Fig. 6d), and also indicates good interaction between the granular column and surrounding soil. It appears that none of the 0.219-m diameter columns failed on bulging, with the measured pullout capacity in close agreement with the predicted capacity in shaft resistance.

The bearing pressure acting on the column base for the TCD anchors was determined using Eq. (2b) and is plotted against the column L/D ratio in Fig. 11. Included also is the predicted capacity in shaft resistance based on the measured remoulded c_u value of 55 kPa. Based on the results obtained, it can be concluded that the two 0.168-m diameter anchors with L/D > 7 (i.e. TCD 3 and 4) failed on bulging. However in the case of the 0.219-m diameter anchors, it is possible that failure in both bulging and shaft resistance may have occurred simultaneously, at least for the longer columns.

As reported earlier, the TCD double-plate granular anchor system (L = 1.4m with the second plate firmly located at mid height, i.e. 0.7 m depth) exhibited some complex behaviour (Fig. 6a), with its measured overall pullout capacity lower than that of the 1.3-m long anchor with a single plate located at the base. The differences in performance can be explained by considering the mode of failure developed for the two segments of the double-plate anchor system. Figure 12 illustrates single and double-plate granular anchor system configurations. The bottom plate in the single plate system

(Fig. 12a) and bottom- and mid-plates in the double-plate system (Fig. 12b) move vertically as the pullout loading is applied. In the latter case, assuming insignificant extension of the steel tendon under loading, vertical displacements of the bottom- and mid-plates are similar and cavities may also develop directly below the plates (see Fig. 12c). Note that a cavity may also develop for the single-plate anchor system. This therefore suggests that mobilisation of bulging resistance and shaft resistance are identical at the both segments of the double-plate anchor system, assuming a uniform undrained shear strength profile**.** This implies that for the double-plate configuration, the two segments of the anchor system behave as independent units, hence the responses are also practically independent and controlled by the values of L/D ratio for the respective segments. Compared with L/D = ~5.9 for the 0.219-m diameter by 1.3-m long single-plate system tested at the TCD site, the L/D ratio for the two equal segments of the double-plate anchor system was 3.2 , considerably less than the critical L/D ratio of ~ 7 required for potential bulging failure. Hence, for a uniform undrained strength against depth profile, the resistances mobilised by the equal-length segments of the experimental double-plate anchor system will be similar. Moreover, the two segments of the double-plate anchor behave as independent units. Hence their individual responses are largely controlled by the values of L/D ratio for the respective segments. Based on this postulation, a simple estimation for this shaft resistance was calculated based on the measured pullout capacity of the 0.196-m diameter by 0.5-m long anchor which failed on shaft resistance at the TCD site (see Fig. 6). The estimations involved taking account of the different diameters and lengths of these anchors. Figure 13 shows the actual performance of the double-plate anchor system and predicted performance based on these calculations. The agreement is good, though the authors agree that it is only an approximation. This intriguing response of the double-plate anchor system prompted further investigation by the authors using model studies. .

Three model anchors having the same diameter of 0.035 m but (a) 0.7 m long with single base plate (i.e. L/D = 20); (b) 0.7 m long with double-plate (L/D = 10 for each segment) and (c) 0.35 m long with single base plate (i.e. L/D = 10) were constructed in a soft-firm stone-free clay bed (Fig. 3). The relevant load–displacement characteristics are shown in Fig. 14. The 0.35 and 0.70-m long anchors having a single plate located at the bottom of the column failed at pullout capacities of about 575 and 650 N respectively. These two observations are generally similar. However in the case of double-anchor system, the failure load of 1350 N was significantly greater, at least double that measured for the 0.7-m long anchor having a single plate at the bottom. In all three cases, values of L/D ratio were greater than 10 suggesting a potential bulging failure. This therefore confirms that the pullout capacity can be enhanced by employing double or multiple plate anchor systems provided that the L/D ratio of each segment is higher than the critical L/D ratio of ~ 7.

Finally, granular anchors performed to similar pullout capacities as concrete anchors tested at the QUB site, suggesting that granular anchors might provide an alternative option to concrete anchors in future engineering construction but does this not only apply for columns with L/D < 7. However it is important to note that granular anchors undergo significant displacements in mobilising ultimate pullout capacity whereas concrete anchors generally failed at very low displacements (Fig. 5). While displacement is not a favourable outcome of any geotechnical or engineering application, progressive displacement of the granular anchor under loading (ultimately resulting in ductile failure) can be considered as an early warning of possible failure of the anchoring system occurring, as opposed to more sudden/brittle failure in the case of concrete anchors.

Bulging failure can be restricted by enclosing the granular column in geotextile (Sivakumar *et al.*,2000), and in this case, the column will also partially utilise potential shaft resistance available under pullout loading. However it should be noted that such an inclusion of geotextile may hinder the interaction between granular column and surrounding soil, thereby potentially mobilising reducing shaft resistance, although further research is necessary.

CONCLUSIONS

This paper has presented the construction, testing and performance of granular anchors in old filled deposits (QUB site) and an intact lodgement till deposit (TCD site). Granular anchors of different L/D ratio were loaded to failure in direct tension/pullout.

Granular anchors of larger surface area, achieved by increasing the anchor length and/or diameter, mobilised greater ultimate pullout capacity of up to ~45 kN at TCD site.

A new method of analysis for the determination of the ultimate pullout capacity has been presented and verified experimentally. The applied anchor load is simultaneously resisted by localised bulging in the vicinity of the column base and by shaft resistance mobilised along the length of the column, with the dominant failure mode governed by the column L/D ratio. Granular anchors having $L/D > 7$ principally fail on bulging and are particularly effective in transferring applied loads to strata at depth. The study has also demonstrated that the pullout capacity can be increased significantly using a multiple-plate anchor system, provided the L/D ratio of individual column segments is greater than the critical value.

Granular anchors are a good alternative to traditional anchoring methods. Short granular anchors principally fail on shaft resistance and were found to mobilise similar pullout capacity compared with conventional cast *in-situ* concrete anchors. Other advantages of granular anchors include short construction time, lower costs as well as the ability of resist applied loading immediately after construction. Granular anchors displace significantly under increasing applied load (with pullout failure generally occurring for anchor displacements of ~60 mm in the present study) compared with the more rigid–perfectly plastic (i.e. sudden pullout) response of concrete anchors. However, a significantly stiffer response can be achieved for granular anchors by simply performing a single unload–reload cycle.

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Table 1 Predicted shaft resistance and bulging capacities and measured pull out loads of granular anchors at QUB test site. Note: F and B, failure in shaft resistance and end-bulging respectively.

Test no.	Bore diameter (m)	. Diameter base plate (m)	Column length (m)	L/D ratio	Shaft capacity kN	Bulging capacity kN	Measured pullout capacity kN	Mode of failure
QUB1	0.07	0.07	0.5	7.0	6.1	**5.9**	5.2	F
QUB2	0.07	0.07	1.0	14.0	12.2	6.1	16.5	B
QUB3	0.15	0.15	0.5	3.3	13.2	27.0	7.5	F
QUB4	0.15	0.15	1.0	6.7	26.3	28.1	30.7	F/B
QUB5	0.15	0.15	1.5	10.0	39.4	29.1	30.8*	B

*Test terminated without mobilising ultimate pullout capacity

Table 2 Predicted shaft resistance and bulging capacities and measured pull out loads of granular anchors at TCD test site. Note: F and B, failure in shaft resistance and end-bulging respectively.

Test no.	Bore diameter (m)	Diameter base plate (m)	Column length (m)	L/D ratio	Shaft capacity kN	Bulging capacity kN	Measured pullout capacity kN	Mode of failure
TCD1	0.148	0.148	0.45	3.0	12	26	12	F
TCD2	0.168	0.148	0.80	4.8	23	27	33	F
TCD3	0.168	0.148	1.47	8.8	43	40	40	B
TCD4	0.168	0.148	1.62	9.6	47	40	42	B
TCD5	0.196	0.196	0.50	2.6	17	46	12	F
TCD6	0.219	0.196	0.96	4.4	36	68	39	F
TCD7	0.219	0.196	1.00	4.6	37	68	42	F
TCD8	0.219	0.196	1.30	5.9	49	70	45	F
CD9*	0.219	0.196	1.40	6.4	53	69	44	F

* Double-plate anchor with mid-height plate located at 0.7 m below the ground surface.

Figure Captions

Figure 1. Schematic of loading frame; QUB study (not to scale)

Figure 2. Schematic of loading frame; TCD study (not to scale)

Figure 3. Testing set-up (model study at QUB)

Figure 4. Granular anchor

Figure 5. Load-displacement characteristics of concrete and granular anchors (QUB Site)

Figure 6. Load-displacement characteristics of granular anchors (TCD Site)

Figure 7. Failure mechanisms

Figure 8. Undrained shear strength profile (QUB site)

Figure 9. Bearing pressure vs L/D ratio QUB site

Figure 10. Strength profile (TCD Site)

Figure 11. Bearing pressure vs L/d ratio TCD site

Figure 12. Failure mechanisms (double plate)

Figure 13. Load-displacement characteristics, single plate (0.5m and double plate 1.4m)

Figure 14. Load-displacement characteristics, single plate and double plate

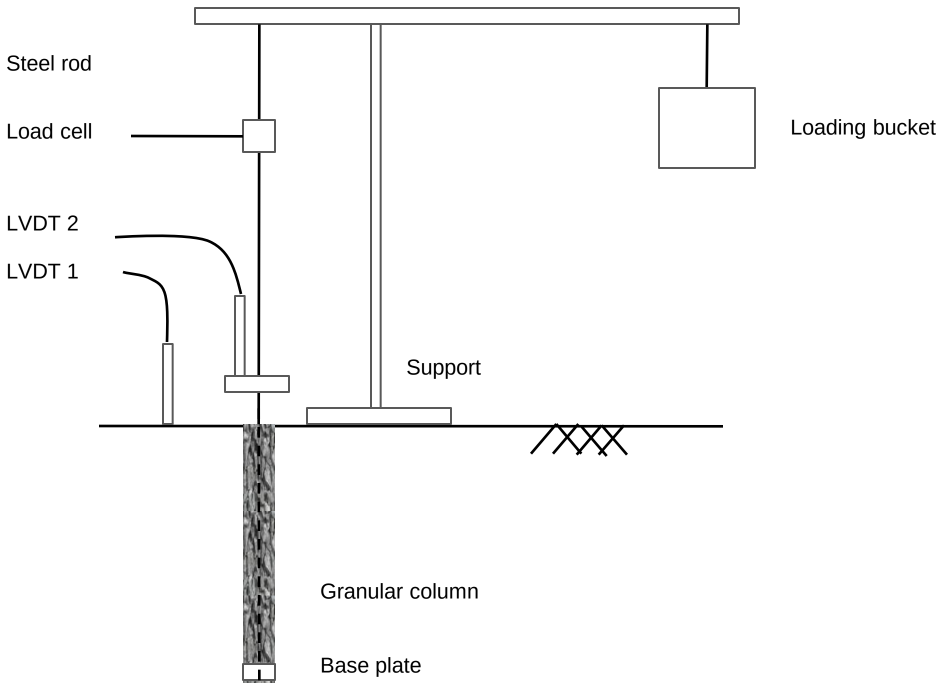


Figure 1 Schematic of loading frame; QUB study (not to scale)

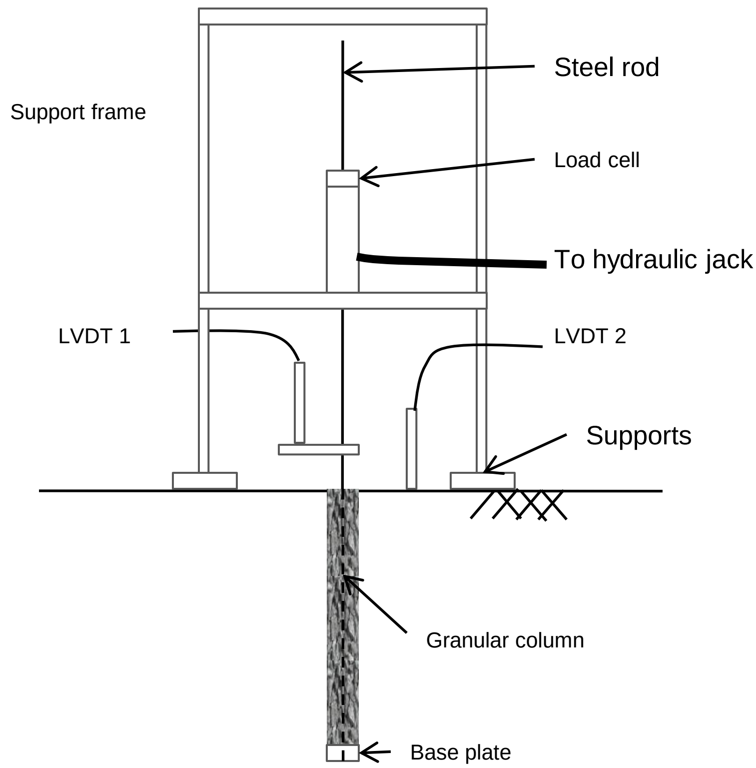


Figure 2 Schematic of loading frame; TCD study (not to scale)

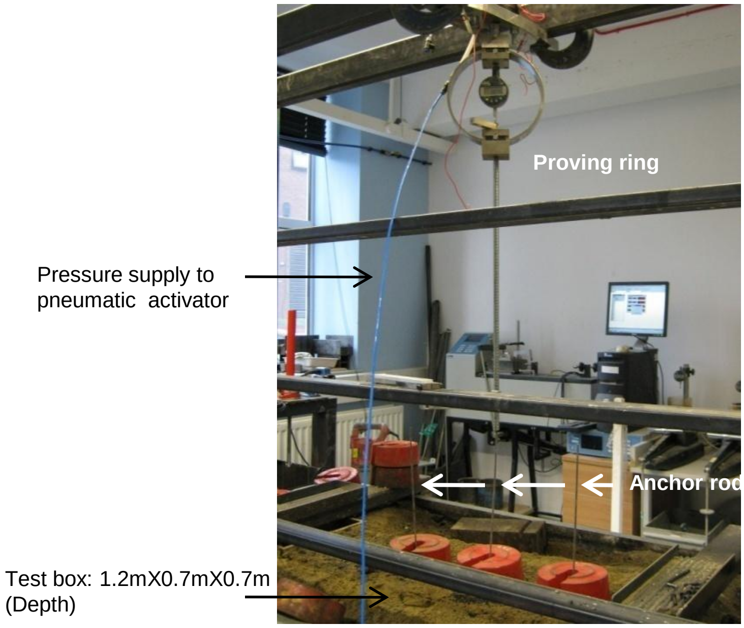


Figure 3 Testing set-up (model study at QUB)

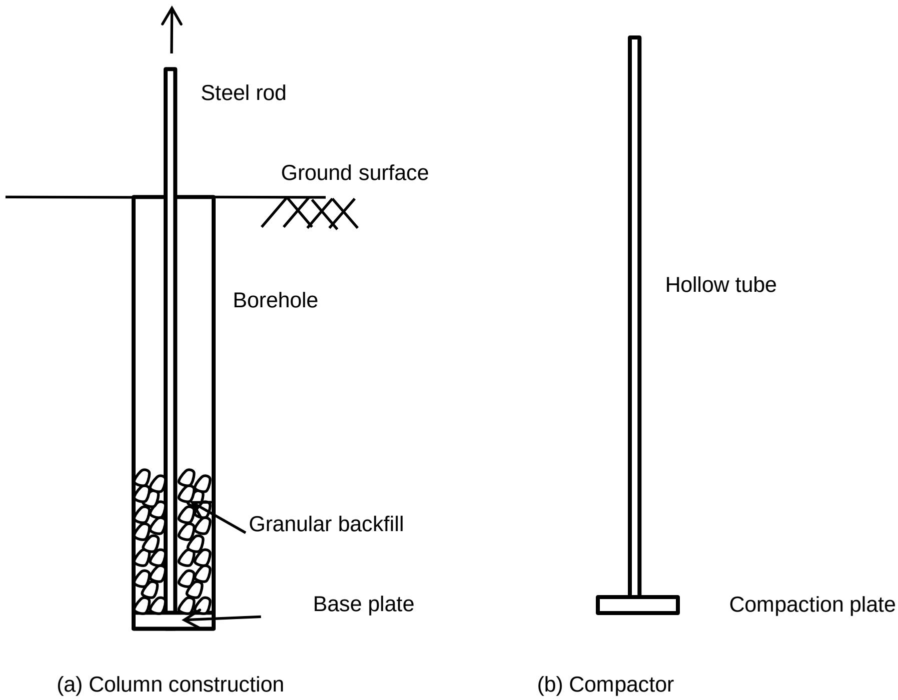


Figure 4 Granular anchor

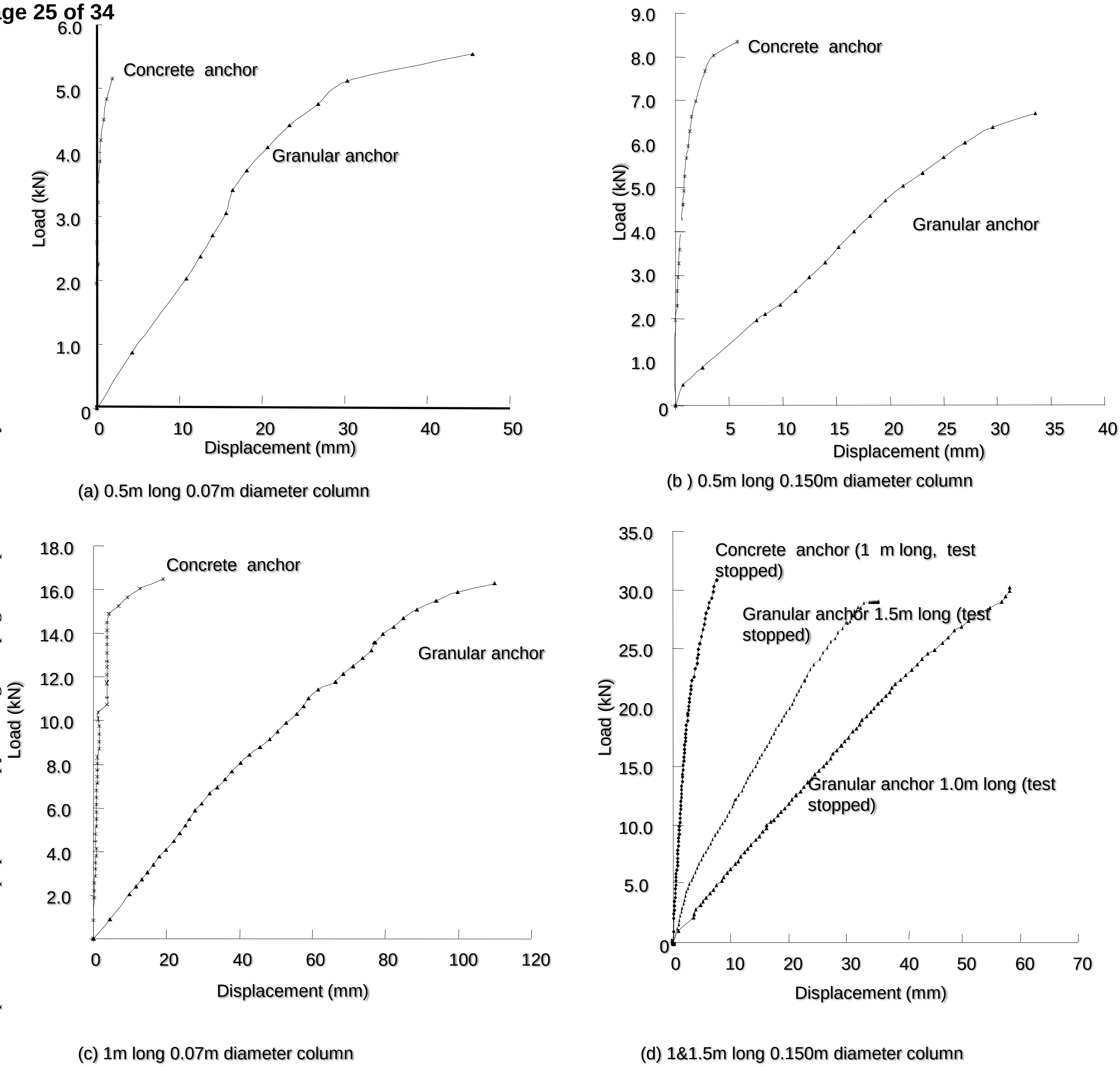
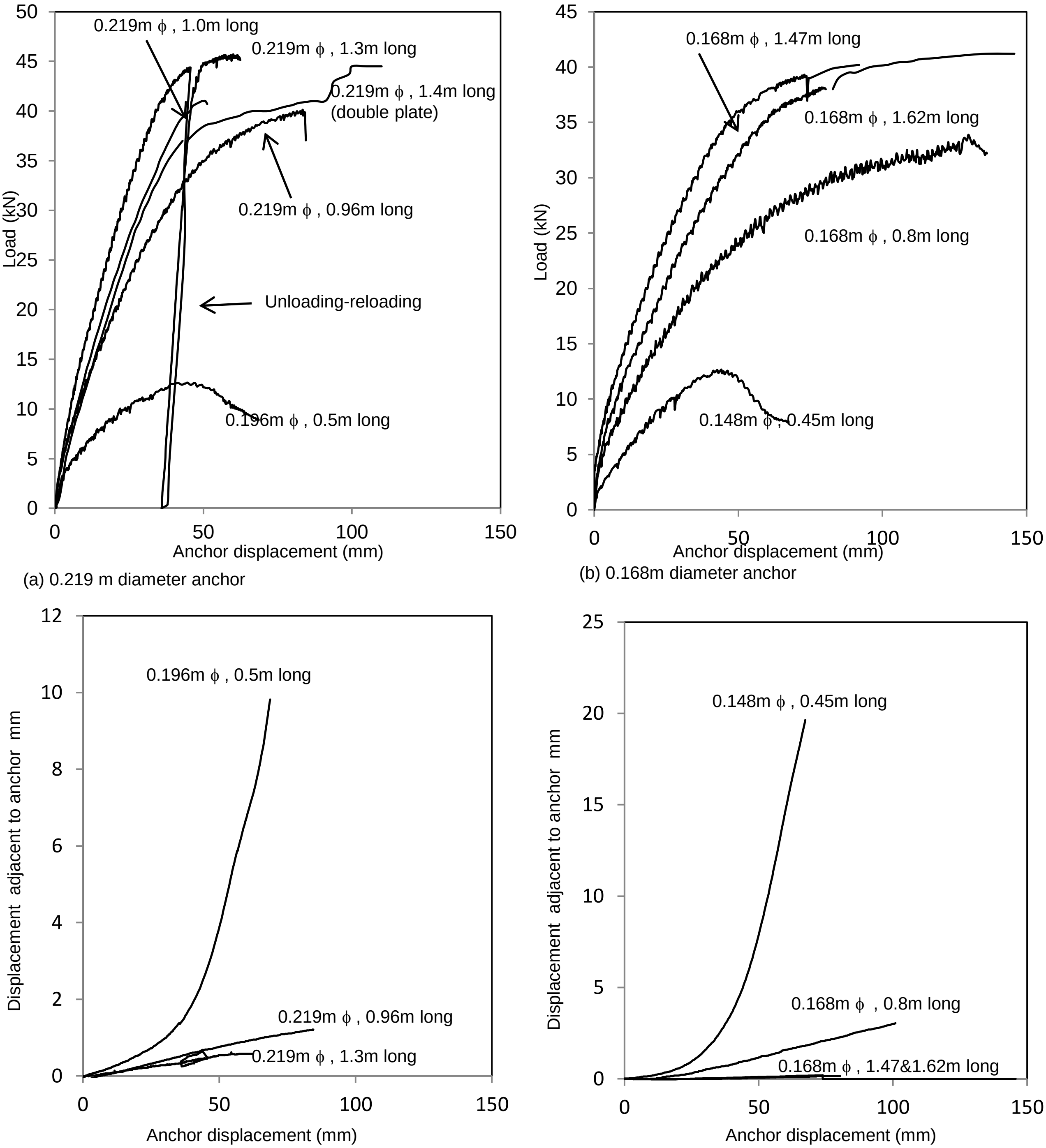
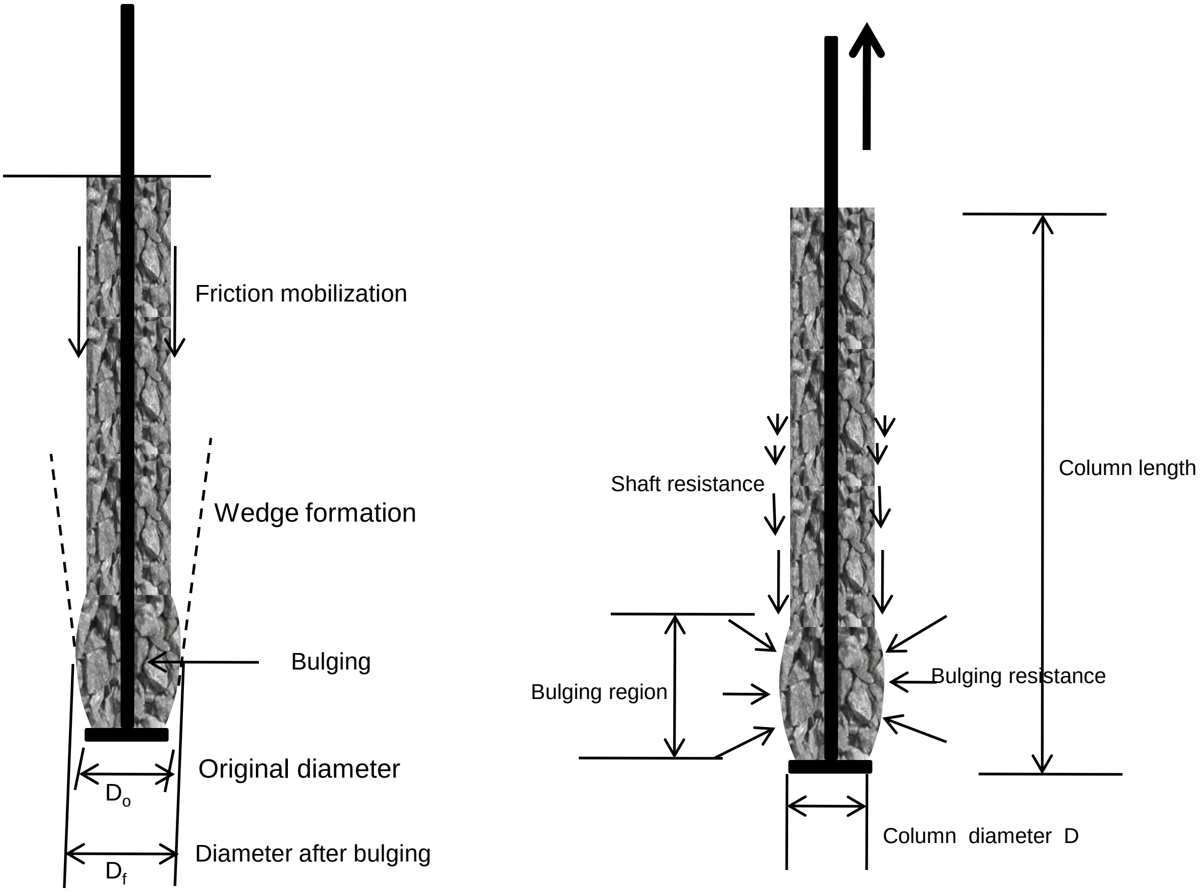


Figure 5 Load-displacement characteristics of concrete and granular anchors (QUB Site)



(c) 0.219 m diameter (displacement away from anchor) (c) 0.168m diameter (displacement away from anchor)

Figure 6 Load-displacement characteristics of granular anchors (TCD Site)



(a) Possible failure mechanisms (b) Bulging and shaft resistance

Figure 7 Failure mechanisms

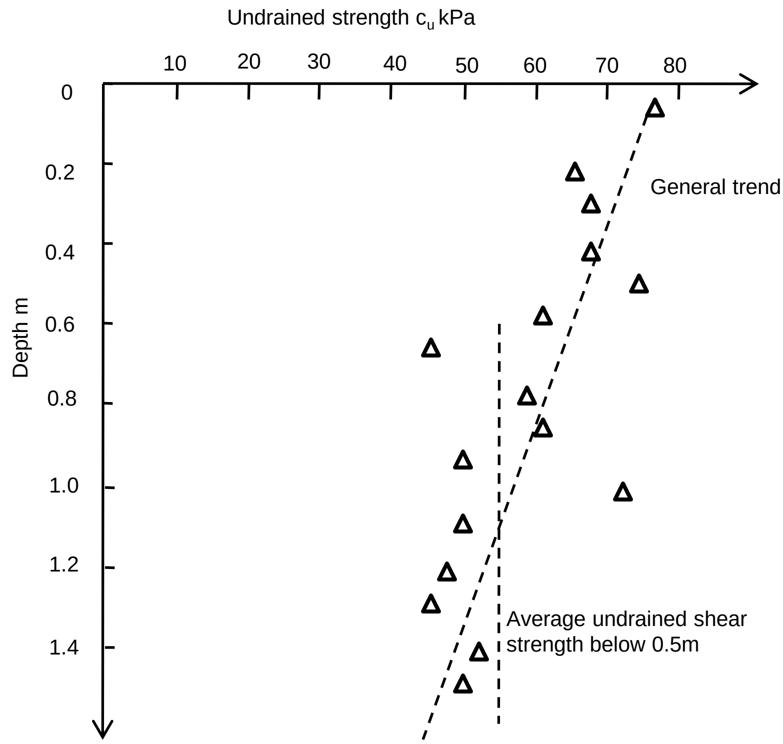


Figure 8 Undrained shear strength profile (QUB site)

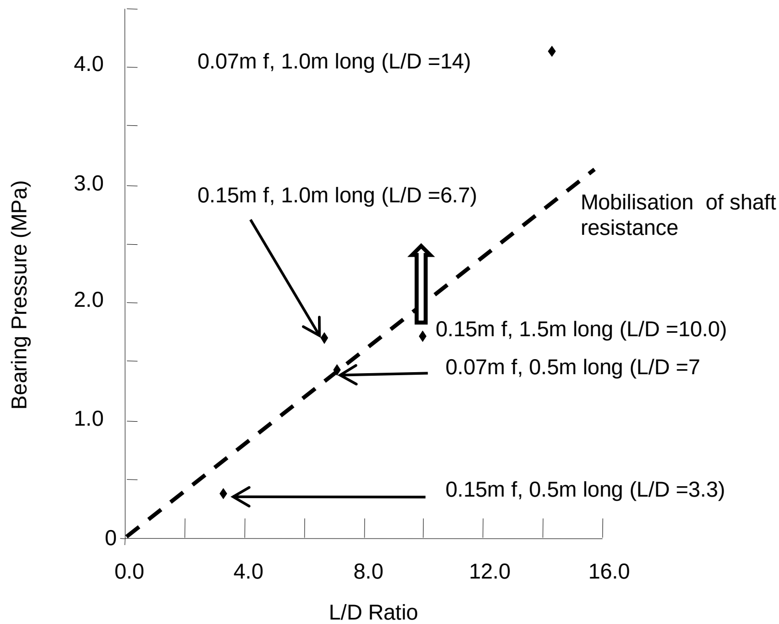


Figure 9 Bearing pressure vs L/D ratio QUB site

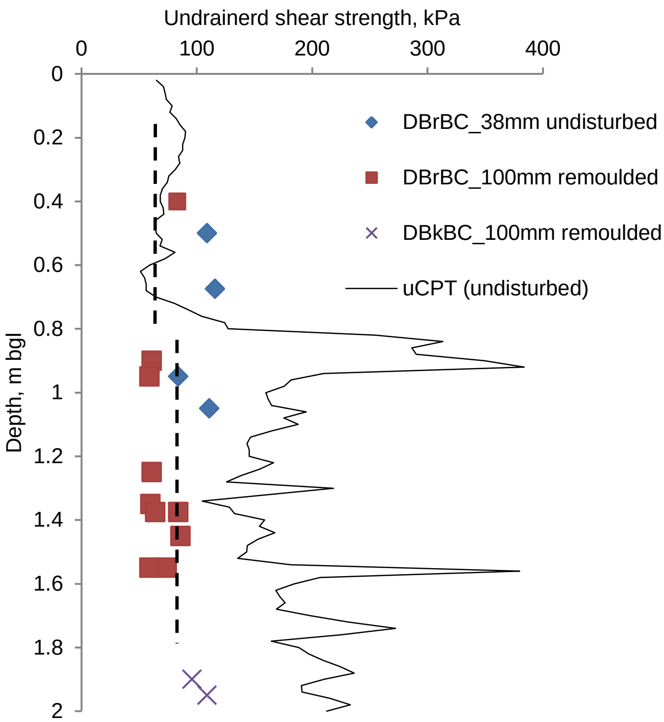


Figure 10 Strength profile (TCD Site)

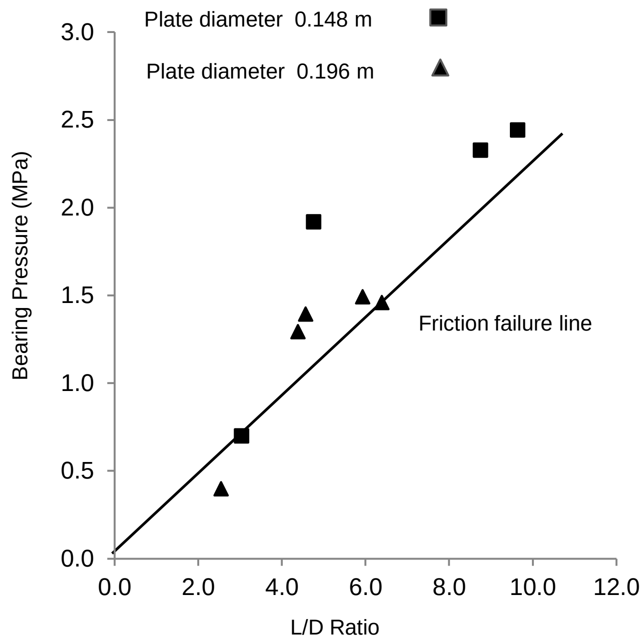
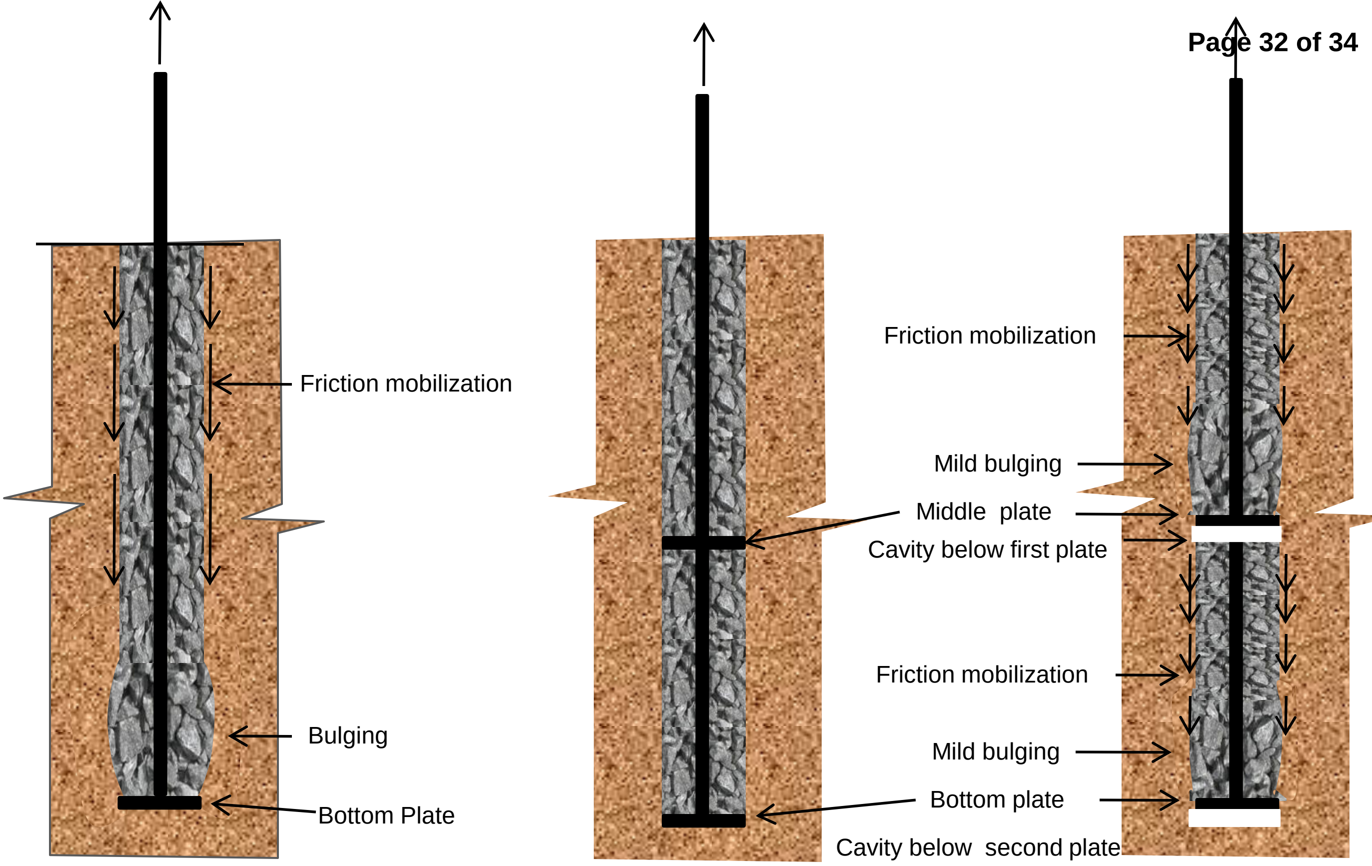


Figure 11 Bearing pressure vs L/d ratio TCD site



(a) Single plate, at failure (1.0m long)

(b) Double plate, initial conditions
(First plate at 0.7m and second plate at 1.4m)

(c) Double plate, failure conditions
(First plate at 0.7m and second plate at 1.4m)

Figure 12 Failure mechanisms (double plate)

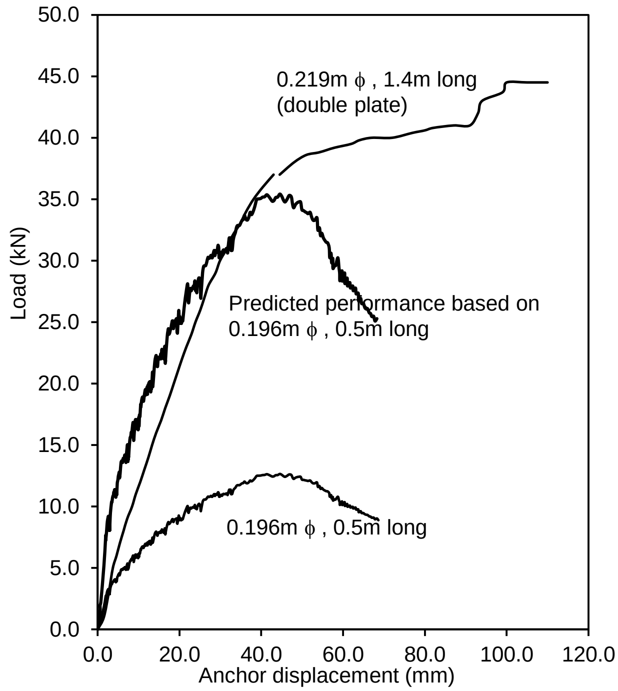


Figure 13 Load-displacement characteristics, single plate (0.5m and double plate 1.4m)

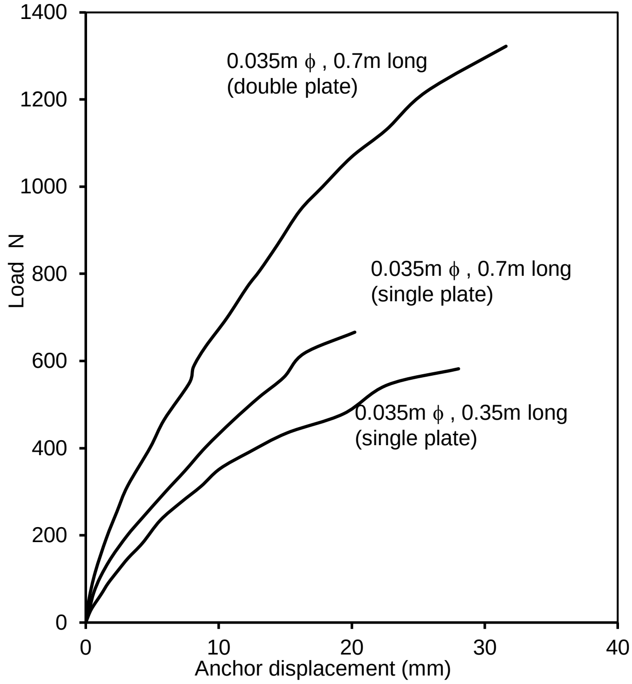


Figure 14 Load-displacement characteristics, single plate and double plate