Physical modelling of lime stabilisation in soft soils around deep excavations

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Abstract

The availability of space above ground decreases as cities expand, causing a demand for very deep underground structures so developments must mitigate the risk of damaging adjacent buildings. This is especially critical in soft clays where ground movements are considerable and can extend far beyond the excavation site. This paper investigates the efficacy of a shallow lime stabilised clay layer on reducing heave and the settlement profile behind an embedded retaining wall. Centrifuge modelling at 160*g* was used to observe surface and subsurface soil movements of a 12m deep excavation (*H*) supported by a retaining wall of 8.8m embedment at prototype scale. Since this research focussed on measures used to minimise heave the model comprised a high stiffness, fully supported 'rigid wall' to eliminate ground movements attributed to wall deformation. A direct comparison between a reference test, with no improvements and a test comprising *H*/2 thick 5% lime stabilised layer indicated that the lime treatment increased the excavation stability by a factor of three.

Keywords: centrifuge modelling; ground improvement; deep excavation; soft Soils; lime stabilisation; upper bound solutions

Introduction

Developers are keen to exploit underground spaces as urbanisation of towns and cities leads to heavily congested spaces. Owing to the complex stress relief during an excavation, the process can result in significant ground movements extending far beyond the site boundary as a result of both wall bending and heave at the formation level. Material published by Peck (1969), Clough and O'Rourke (1990), O'Rourke

(1993), Hashash & Whittle (1996), Karlsrud & Andresen (2008) and Langford *et al*. (2015) highlighted that movements arising during the excavation of soft soils can be excessive and damaging. Neighbouring buildings and services are at risk from differential settlements and it is therefore essential that such movements are minimised to ensure the success and safe delivery of deep excavation projects.

Ground movements associate with deep excavations are a complex combination of both lateral movements; from wall bending and rotation and basal heave, owing to the vertical stress relief of removing the overburden in front of the wall. Both mechanisms induce vertical soil displacements behind the wall that can extend far beyond the site boundary.

Extensive research (O'Rourke, 1993; Osman and Bolton, 2005; Lam *et al.*, 2014) was conducted to identify the mechanisms of movement around deep excavations in soft soils. Such research typically modelled relatively flexible walls hence subsequent ground movements stemming from the excavation were a function of both lateral and vertical movements. Therefore, any excavation techniques or ground improvement measures implemented in each experiment did not directly quantify the change in the magnitude of heave.

Background

Significant deposits of soft soils are found alongside marine environments and in countries such as Malaysia, Singapore and Taiwan, to name a few. These areas are predominately made up of deposits of very soft clay overlying stiffer bearing stratum. Design guides of retaining structures (Broms, 1988; Gaba *et al.*, 2017; ICE SPERWall, 2016) typically recommend that sheet piled walls are embedded into stiff ground as this can increase stability and reduce heave at formation level. However, the thickness of soft soil deposits can often exceed 40m, making wall embedment into underlying strata an unfeasible and uneconomic solution.

Peck (1969) suggested a number of methods to reduce ground movements around deep excavations in soft soils. This research focusses on one method by increasing the passive resistance at the toe of a retaining wall by strengthening a layer of soil at the formation level. Ground treatment is a means of increasing soil strength (Bryhn *et al.*, 1983) by churning an additive, such as lime or cement into low strength soil to design depth. This layer of improved soil behaves as a strut below formation level and restrains the toe of the wall against active failure.

Deep soil mixed (DSM) columns can be drilled and cast in-situ or shallow trenches can be mass mixed with a dry powered additive or a wet slurry (Taylor, 2017). Columns are typically used to achieve deeper levels of treatment and rely on paddled augers to churn the soil to achieve a uniformly mixed soil. The additive is applied to the required improvement design depth and left to cure before works progress.

Ohnishi *et al*. (2000) performed three centrifuge tests modelling a 16m deep excavation propped at two levels, stabilised by a 7m deep layer of coal ash treated clay, as illustrated in Figure 1(a). The percentage of coal ash was varied so that the unconfined compressive strengths of the soil at formation were 400kPa and 100kPa, whilst the strength of the untreated soil was approximately 60kPa. Three rubber bags filled with a heavy fluid supported the retaining walls, whilst the central bag was drained to simulate the excavation. As expected, the greater strength test resulted in negligible ground movements whilst the excavation without ground improvement exhibited 500mm heave at prototype scale, shown by the displacement vectors in Figures 1(b) and (c).

Although these experiments illustrated that ground improvement reduces the magnitude of heave in deep excavations the cross sectional area of ground improvement and strengths achieved are costly and unlikely to be replicated in industry (Taylor, 2017). If ground improvement is the only option for improving excavation stability it is important to limit the percentage of additive and the area of treatment to reduce project costs.

Objectives

This paper aims to investigate whether a shallow raft of improved ground at the formation level can sufficiently improve the stability of a deep excavation in soft soils, as opposed to ground treatment extending to the toe of the retaining wall. A geotechnical centrifuge was used to model a deep excavation in very soft clay with undrained shear strengths of $7kN/m^2$ at ground level increasing to $30kN/m^2$ at 40m depth. Two centrifuge tests and unconfined compressive strength tests were carried out to observe ground movements around the excavation and determine the strength of soil samples with time.

Principles of centrifuge modelling

Three keys methods are available to engineers for the analysis of geotechnical events. The most common is numerical analysis; where soil and excavation parameters are estimated and the excavation is simulated through a series of steps. Limitations arise from the complex nature of soils and using a reliable soil analysis model that accurately represents the geotechnical event. Alternatively, field testing can be used to obtain ‘real-life’ on-site data, however the sheer cost and risk of field testing, the lack of space on site and the variability of ground conditions makes this unsuitable for parametric studies.

In this project physical modelling, another means of simulating and observing a geotechnical event, was used. The repeatability of physical modelling tests and the ability to control variables makes this an increasingly popular tool amongst researchers. In addition, the cost of performing a test is significantly less than performing full scale field trials. A geotechnical centrifuge can be used to physically model a real life (prototype) event at a reduced scale. Centrifuge testing enables the correct stresses to be developed through the depth of the model so that its behaviour is representative of the prototype event.

Physical modelling relies on stress similarity between the prototype and the model (Taylor, 1995) and is defined in Equation [1]. The scaling laws of centrifuge modelling state that Equation [2] holds true for a model scaled *N* times. Hence, assuming that the density of the two materials are comparable, the condition of stress similarity is met, as defined by Equation [2].

$$\sigma_{vp} = \sigma_{vm} \quad [1]$$

$$h_p = Nh_m \quad [2]$$

$$\sigma_{vp} = \rho g h_p \text{ and } \sigma_{vm} = \rho N g h_m \quad [3]$$

Where h_p = height of the prototype, h_m = the model height, which results in linear scaling of model dimensions, g = acceleration due to gravity (9.81m/s^2), ρ = density of material (kg/m^3) and N = dimensionless gravity scaling factor. The Acutronic 661 beam centrifuge, with a radius of 1.8m and 40g/tonne capacity located at City, University of London, was used for this series of experiments.

Soil model

The test specimens were prepared in a 375mm deep rectangular aluminium alloy strongbox with internal plan dimensions 550mm x 200mm. Herringbone channels cut into the base of the strongbox allow water to drain from the base of the sample.

Waterpump grease was applied to the walls of the strongbox to mitigate boundary effects and sheets of porous plastic and filter paper were placed at the bottom of the strongbox.

Speswhite kaolin clay powder was mixed with distilled water to a water content of 120%, approximately twice its liquid limit, to facilitate workability. It has a relatively low permeability allowing the sample to consolidate in a short period of time. A 300mm high extension was bolted to the top of the box to allow a 500mm deep layer of slurry to be placed in the strongbox, giving a minimum sample height of 290mm post consolidation. The slurry was carefully placed in the strongbox with a scoop and agitated with a palette knife to avoid air entrapment. Another layer of porous plastic and filter paper was used to sandwich the slurry, allowing two-way drainage which accelerated the consolidation process at 1*g*.

The sample was consolidated in a hydraulic press where a tightly fitting platen was lowered onto the sample. The pressure was gradually increased from 10kPa to 100kPa over a period of 2 days and the sample was left to consolidate at a maximum vertical effective stress of 100kPa for 10 days.

Owing to the low preconsolidation pressure of 100kPa applied at 1*g*, the sample was very soft and subsequently extremely difficult to work with. The final sample preparation stage involved consolidating the sample at 160*g* on the centrifuge. The purpose of this was to ensure that the soil was of sufficient strength that voids could later be cut to form the excavation trench. Following 1*g* consolidation, the sample was removed from the hydraulic press and trimmed to a height of 290mm. A lid was bolted to the top of the strongbox and a linear variable differential transformer (LVDT) was clamped to the lid so that the footing rested on the top of the clay surface, depicted in Figure 2(a). The model was transferred to the centrifuge platform and an overflow standpipe was connected to the base of the model providing a water feed 10mm above the sample height. This ensured that the sample remained saturated during consolidation and the lid prevented the standing water from evaporating in-flight. The model was left to consolidate overnight whilst the pressure at the base of the standpipe and the settlement of the LVDT were recorded by the centrifuge on-board computer. The sample surface typically settled 17-18mm and once the LVDT readings had plateaued, the model was removed from the centrifuge in preparation for model making.

A consolidated lime-kaolin sample was also required for this series of tests. This sample comprised 5% lime by Speswhite kaolin dry weight and was mixed to a water content of 140%. The uniform slurry was then carefully placed in a smaller drainage box before being consolidated to 150kPa over one day. This was left for a further 24 hours before being trimmed and placed in the centrifuge model.

Experiment apparatus and instrumentation

This series of experiments aimed to investigate whether a shallow raft of lime stabilised clay could improve the stability of a deep excavation in very soft clay and reduce ground movements, whilst preventing wall deformation. Bespoke apparatus described here was designed and fabricated specifically for this project. The retaining wall used in this series of tests was designed as an inherently stiff structure with an equivalent prototype comparable with a 2m thick reinforced concrete diaphragm wall. Owing to the low strength of the soil, it was impossible to cut a trench in the soft soil and it was essential to push the wall into the soil prior to forming the excavation area. Consequently, this required a sheet piled wall to be fabricated from a 10mm thick stainless steel plate 10mm wide ribs, each 1mm thick. Silicone seals were cast along the edges of the wall to create a waterproof seal and prevent water ingress into the excavation.

An aluminium stiffener, initially developed by McNamara *et al.* (2009) and a capping beam supported the full length of the exposed wall and pinned the top of the wall. This ensured that movements arising from the excavation process were a direct result of basal heave and not of wall movements. Details of the centrifuge equipment are shown in Figure 2(b).

As these tests were designed to investigate the influence of a layer of lime stabilised clay at the formation level, it was essential that variables were controlled. Hence, the wall embedment, groundwater level, equipment set up, sample preparation and testing schemes were consistent between the two tests.

During wall installation it was essential that the wall maintained verticality and the depth of embedment could be controlled. Thus, a Perspex guide was fabricated with ribs to house the retaining wall (Figure 2c), whilst an aluminium bar was used to embed the wall into the soil. An aluminium cutting shelf was design such that it could be bolted to the front of the model and support the flanges of the Perspex guide, shown in Figure 2(d). This shelf also demarked the excavation zone and ensured accurate model making in tests.

Clough *et al.* (1989) illustrated that a high system stiffness positively influenced the excavation behaviour by reducing the magnitude of lateral wall displacements (Figure 3). A very stiff wall was therefore used in this series of tests whose purpose was to limit the magnitude of movements associated with lateral bending. Movements measured during the excavation process could therefore be directly attributed to basal heave.

Owing to the use of a pressurised latex airbag to surcharge the formation level, which was in contact with the retaining wall, it was necessary to design a spacer that would protect the bag from bursting when it reacted against the sharp wall ribs. This spacer consisted of square aluminium channels screwed onto a 1mm thick steel plate. A layer of silicone rubber was cast over this spacer to create a watertight seal against the wall which prevented seepage into the excavation area during in-flight reconsolidation.

Model making stage

Following in-flight consolidation at 160*g* the sample was removed from the platform and a scraper was drawn across the clay to trim the sample to a height of 255mm. A thin layer of PlastiDip was sprayed onto the surface. PlastiDip is a flexible impermeable synthetic rubber membrane which prevented the model from drying out whilst in-flight.

The front face of the strongbox was removed and the excess waterpump grease was carefully scraped from the sample. A thin layer of silicone oil was applied to the front face to inhibit the drying out of the sample during model making. The cutting guide was bolted to the strongbox and the Perspex guide attached prior to using thin walled cutters to create voids for the wall seals. Following this, silicone grease was applied to the wall before pushing the ribbed wall into the clay.

Steel plates were used to scrape soil from the excavation area and samples were taken to establish the water contents at varying depths. The lime stabilised clay layer in test 2 was formed by removing the cutting guide and clamping an aluminium angle across the front of the model to the required height. The clay was excavated to this depth to cater for the lime stabilised clay layer. Care was taken to ensure that the lime layer was in full contact with the rest of the soil sample with minimal disturbance.

After the excavation was cut the test apparatus were placed within the excavation void, which included sheets of filter paper and porous plastic, a latex airbag secured to a brass union, the spacer between the airbag and the wall and the aluminium stiffener. Finally, the instrumentation were secured to the model which included LVDTs, pore pressure transducers (PPTs) and an air pressure transducer.

Locations of the instruments are illustrated in Figure 4(a). In plan, 11 LVDTs were positioned along the centreline of the strongbox directly behind the wall and at $H/2$ intervals. Two PPTs were positioned at the same elevation either side of the retaining wall and one was placed at $4H$ behind the wall to measure far field pore pressure changes.

On-board centrifuge cameras facilitated the observation of subsurface movements and identification of a failure mechanism. In order to clearly define and compute these movements in a Matlab programme developed by Stanier & White (2013), a texture was applied to the front face of the model. For these experiments, 1mm diameter black glass ballotini beads were randomly scattered and rolled onto the model surface which created sufficient contrast for image analysis, see Figure 4(b). A thin layer of high viscosity silicone oil was spread across the Perspex window before bolting it to the model and transferring it to the centrifuge platform.

Testing procedure

A water table was set 5mm below the surface of the clay by means of an overflow standpipe. As the model was accelerated to 160*g*, the pressure in the airbag was gradually increased to balance the overburden stresses. Upon reaching 160*g* excess pore pressures had accumulated in the sample and were left to dissipate. The PPTs confirmed that the sample had come into equilibrium within approximately 6 hours.

The airbag applied 200kPa surcharge to the formation level. The excavation was simulated by reducing this air pressure at a rate of approximately 1kPa/sec and was typically completed within 3.5 minutes. Post excavation, the model was decelerated and shear vane readings were immediately taken at different locations behind the wall to establish the average undrained shear strength profile. Samples to determine water contents were also taken at active, passive, far field locations including the lime stabilised layer.

Test results and analysis

Test 1 denotes the reference test where no lime treatment had been used whilst test 2 refers to the lime stabilised test which comprised a 5% lime-kaolin layer that extended across the full width of the excavation and was $H/2$ in depth. The experiment geometry was given in Figure 2(b).

Surface settlement profile

Comparisons were drawn between the surface settlement profiles at discrete overburden pressures. Figure 5 illustrates that the lime stabilised clay significantly increased the system stability and delayed excavation failure. At 50kPa the vertical displacements adjacent the stabilised excavation were 65% lower than those measured in the reference test. The lime stabilised displacements at 30kPa are also comparable to the settlement trough of the reference test subjected to an overburden pressure of 50kPa.

Image analysis of the reference test revealed that the toe of the retaining wall rotated into the excavation as the overburden pressure fell below 39kPa (Figure 6a). Maximum resultant displacements in the region of 4.5mm occurred on the active side of the wall just below formation level. Image analysis of the lime stabilised test (Figure 6b) at an overburden pressure of 39kPa illustrated a triangular displacement mechanism and resultant movements of 1.5mm; a third of the magnitude seen in the reference test.

Pore pressure response

In each test three pore pressure transducers were used to monitor the development of excess pore pressures as the excavation progressed. Figure 7 compares the changes in pore pressure in the active and passive zones for both the reference test and one where improved soil was present. The far field pore pressure response was comparable in both tests and remained unaffected by the excavation process. Therefore, the model was deemed sufficiently large to be unaffected by boundary effects.

A similar pore pressure response was observed in the active zone for both tests until approximately 40kPa, at which point the test without ground improvement failed. This was characterised by a sudden drop in pore pressure. In comparison, the lime stabilised test showed little change in pore pressure until overburden pressure reached approximately 20kPa.

In both tests the passive zone pore pressures exhibited similar behaviour, however the presence of the lime stabilised soil reduced the rate of change in pore pressures directly below the excavation. This was consistent with the smaller magnitude of movements measured in the lime stabilised test compared with the reference test (see Figure 5).

Upper bound analysis

Upper bound solutions of the overburden failure pressure (P) were calculated based on results from image analysis and the best solutions are given in Figures 8(a) and (b). Image analysis suggested that owing to the extremely stiff wall there was no measured wall deformation and, at failure, the wall rotated about the lowest prop. It was therefore assumed that a solid rectangular block of soil, extending from ground level to the lowest prop, slid vertically downwards. Subsequently, a 90° fan mechanism included the lower portion of the wall and extended below the toe of the wall. Another 90° fan mechanism, rotating about the formation level was chosen for test 1 (Fig 7a). Owing to the brittle behaviour of the lime stabilised clay, the second fan in test 2 progressed to the bottom of the ground improvement zone, before shearing at 45° to the normal (Fig 7b).

Mechanism one (Fig 7a) indicated that excavation failure must occur when the overburden pressure falls below 14.5kPa, whilst mechanism two suggests the lime stabilised excavation fails at 10kPa. Both mechanisms resulted in similar overburden pressures at failure, however mechanism two is more closely representative of the expected failure pressure.

Discussion

In construction, the lime soil mix is typically cured for five days prior to excavation and is surcharged by the weight of soil above it (Taylor, 2017). The lime stabilised clay used in this parametric study had been left to cure for a period of two days, therefore the increase in strength of the lime stabilised clay could be arguably lower than strengths expected in the field.

A series of unconfined compressive strength tests were performed on 5% lime-kaolin samples consolidated under a vertical effective stress of 150kPa, comparable with the pressure applied to the centrifuge test sample. Results given in Figure 9(a) show peak strengths developing after 14 days. This becomes more pronounced over time owing to the pozzolanic reaction (Locat *et al*., 1990) which leads to an increase in soil strength with little influence on water content, shown in Figure 9(b).

After 7 days there is a 50% increase in the soil strength (Figure 9a) owing to the cementitious bonding of particles. Interestingly, the peak strength of the untreated clay was approximately 15% higher than lime stabilised clay tested at 1 day. A reduction in strength was observed as the untreated clay sheared; whereas the lime soil strength remained constant, possibly as a result of an increase in the internal angle of friction of lime treated clay (Qiang and Chen, 2015).

Unsurprisingly Figure 9(a) shows a noticeable peak strength at 28 days. Upper bound solutions were recomputed for the varying soil strengths and are plotted in Figure 9(b) and show the respective water contents. Calculations suggested that no significant benefit was gained from delaying the excavation more than a week post treatment, as a failure overburden pressure of approximately 9.5kPa is equivalent to approximately 0.5m soil depth at prototype scale.

Measured surface settlements and the typical settlement troughs outlined by Clough & O'Rourke (1990) and Hsieh & Ou (1998) were plotted in Figure 10(a). The experimental vertical displacements followed the trends in the literature, however both settlement profiles are wider than expected with relatively large displacements observed as far as 3*H* from the retaining wall. This may be owing to a combination of the very low soil strength and the high excavation support stiffness which prevents wall deformation. Typical of propped excavations, the maximum settlements occurred at 0.5*H* as the soil 'hangs' onto the retaining wall thus movements at the wall/soil interface are comparably smaller which is consistent with the Hsieh & Ou (1998) settlement profile.

Settlements have been normalised against the maximum excavation depth (Figure 10b) at an overburden pressure of 40kPa, which equates to 2.4m above final formation level at prototype scale. These movements were negligible compared with those classified as Zone I (Peck, 1969) movements. This trend was also observed by Liu, Ng & Wang (2005) owing to improvements in workmanship during construction. Hence, a higher system stiffness is achieved compared with the relatively flexible excavations included in the early published database (Peck, 1969).

Implications and limitations of research

Soil stabilisation was first introduced around 1970 and became increasingly popular in the 1990's. This technique has since been greatly improved and is now widely used to treat very soft ground. The treatment zone often extends to the toe of a retaining wall and across the full width of the excavation, which is expensive and probably unsustainable means of excavating soft soils.

This research illustrated that movements were controlled when a shallow layer of soil was treated and a more stable excavation was achieved. This suggests that lime stabilised zones can be better designed to obtain a more sustainable solution. An optimum treatment area has not yet been determined and it is possible that a narrow zone of treated soil may bring further improvements to the behaviour of a deep excavation in soft very soil.

Conclusions

Two centrifuge tests at 160*g* were conducted to measure the variation in magnitude and extent of ground movements around a deep excavation in very soft clay. Sample preparation techniques, excavation geometry, retaining wall and support stiffness were consistent between tests. A reference test was conducted for comparison against an excavation with a layer of lime stabilised clay of thickness $H/2$ extending across the entire formation level. The results indicated that doubling the soil strength in the passive zone increased the stability of the excavation by a factor of three and significantly reduced the magnitude of surface settlements behind the wall.

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deep excavations in soft soil and lime stabilisation.

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Figures

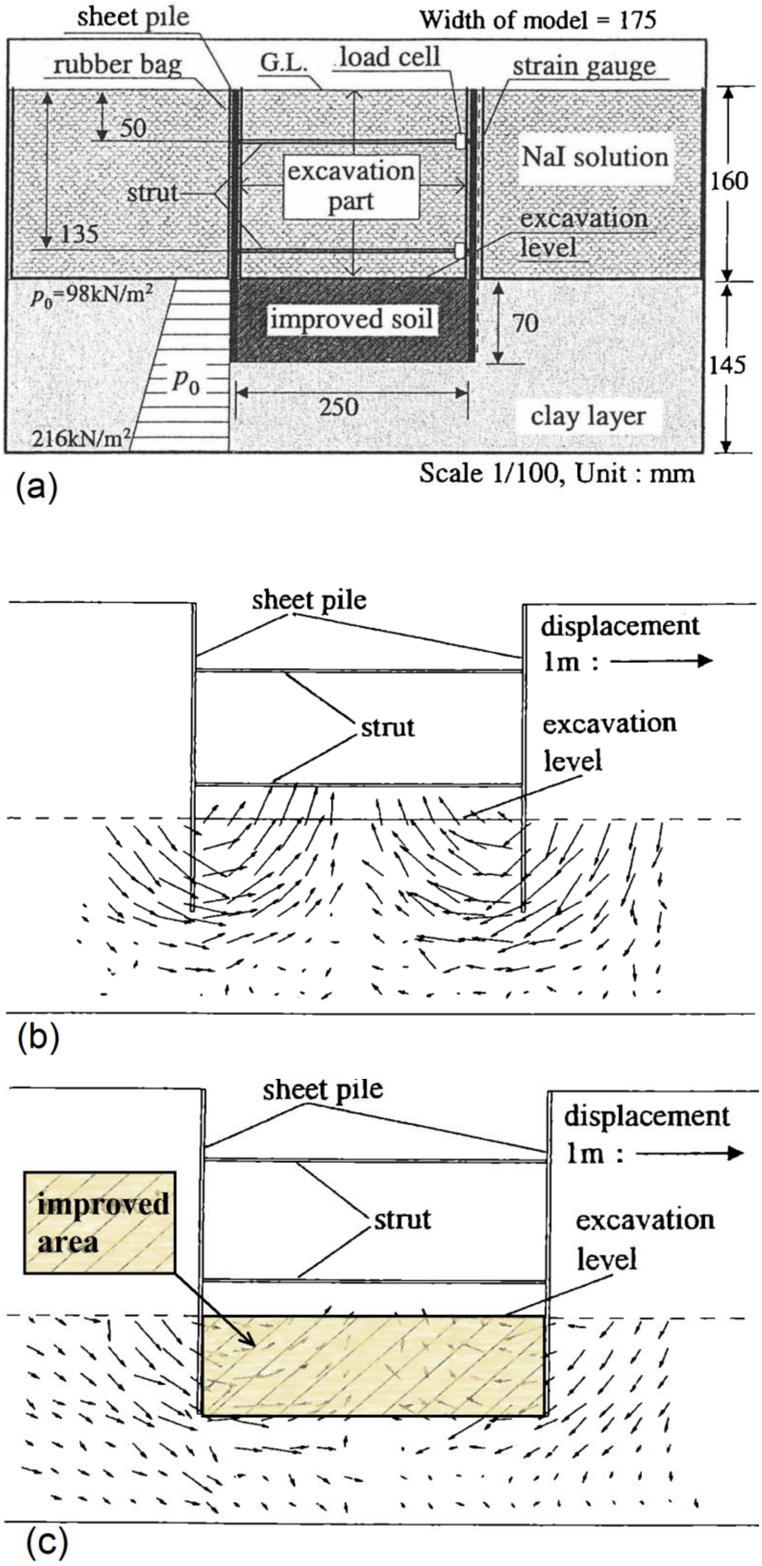


Figure 1. Ground improvement centrifuge test (a) experimental set up (Ohnishi *et al.*, 2000) and subsequent ground movement displacement vectors for tests with (b) no improvement (60kPa) and (c) 400kPa improved soil

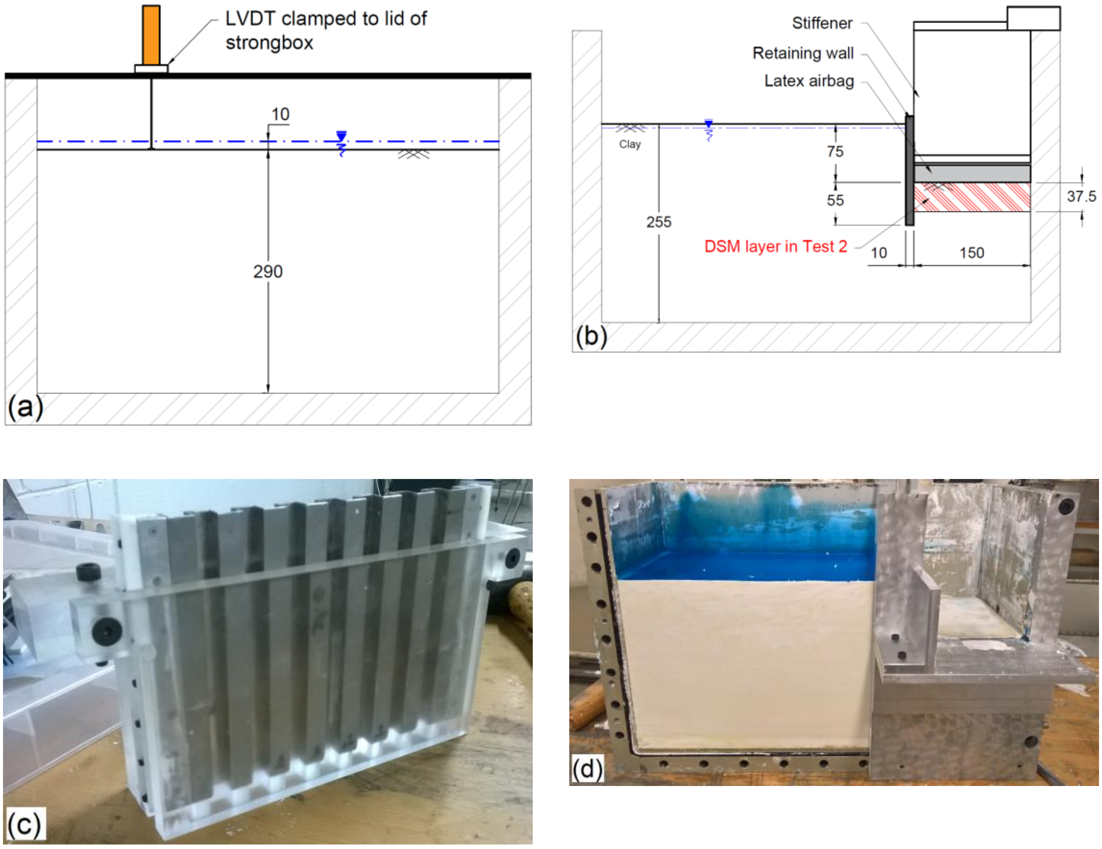


Figure 2. (a) In-flight consolidation set up (b) excavation set up (c) retaining wall supported by Perspex guide and (d) Cutting shelf attached to model to form excavation void (Note; all dimensions in millimetres at model scale)

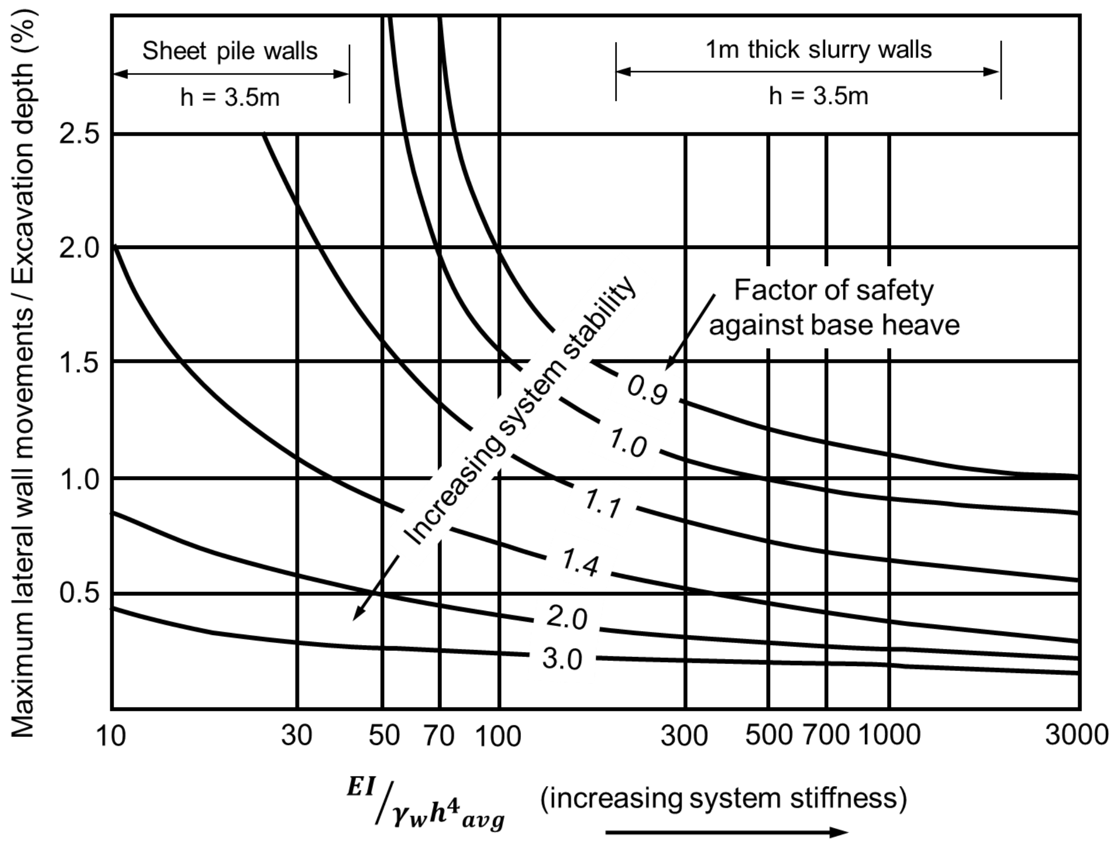


Figure 3. Influence of system stiffness on lateral wall movements (Clough & O'Rourke, 1990)

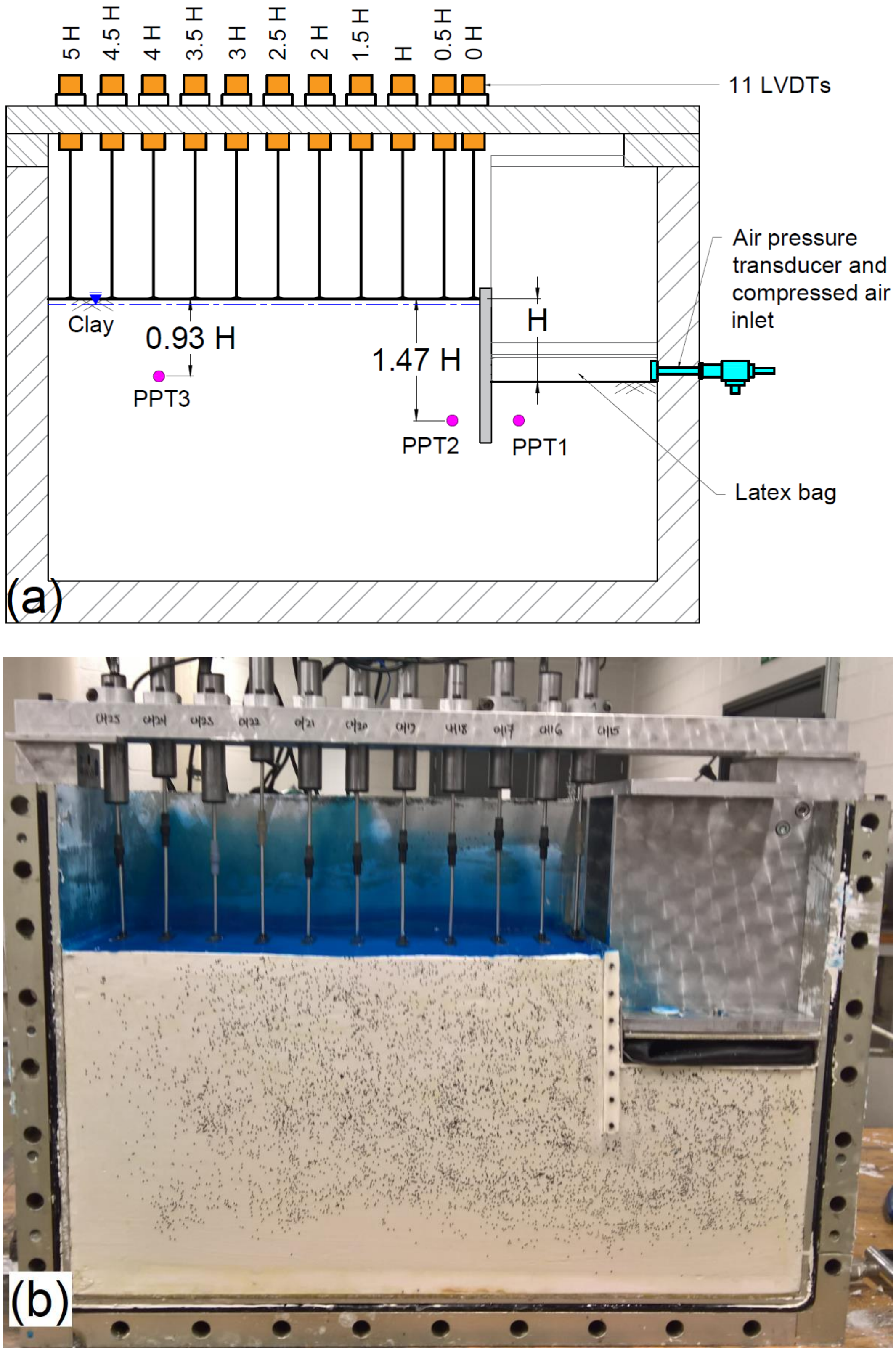


Figure 4. (a) Location of instrumentation and (b) centrifuge model immediately prior testing

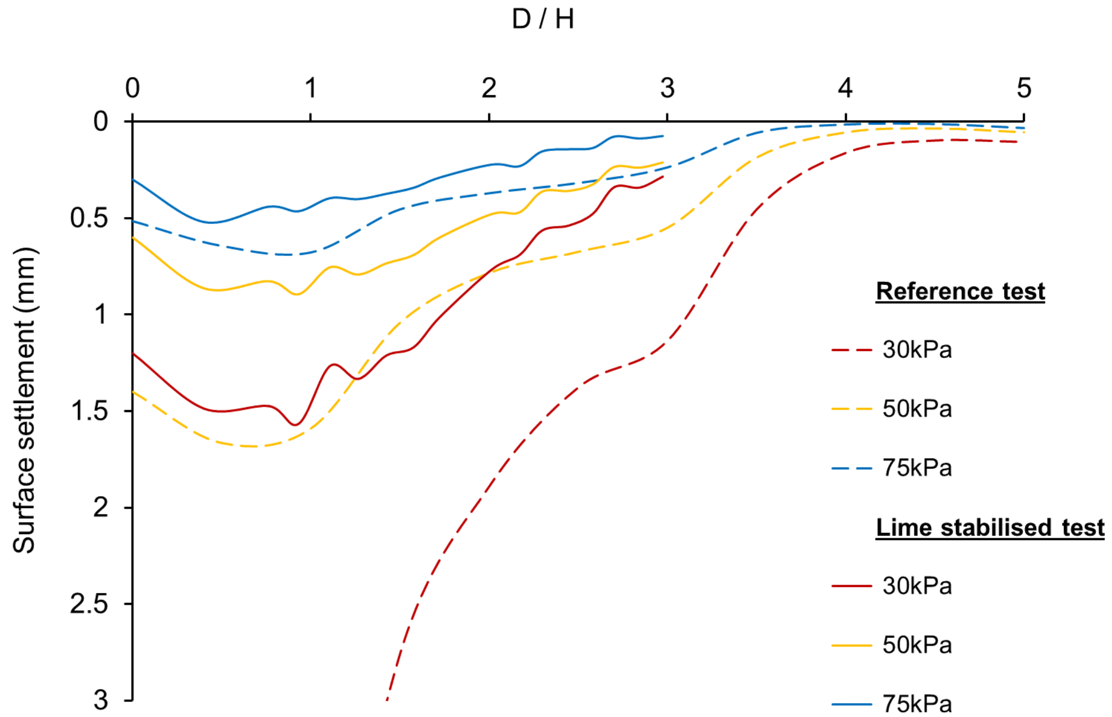
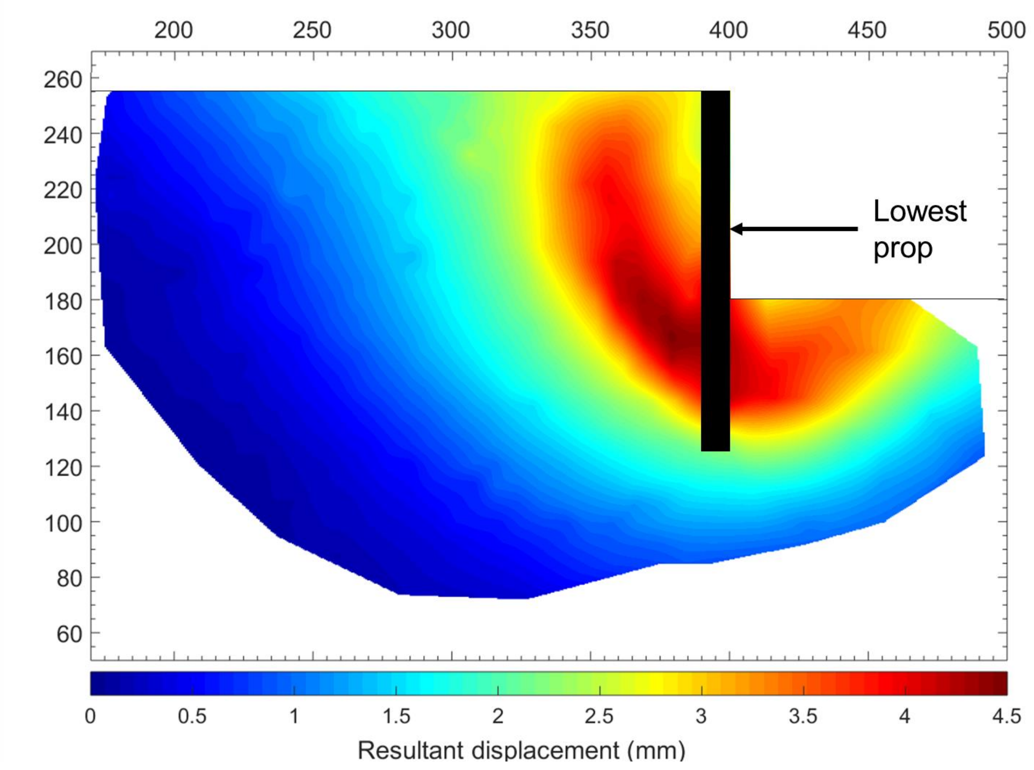
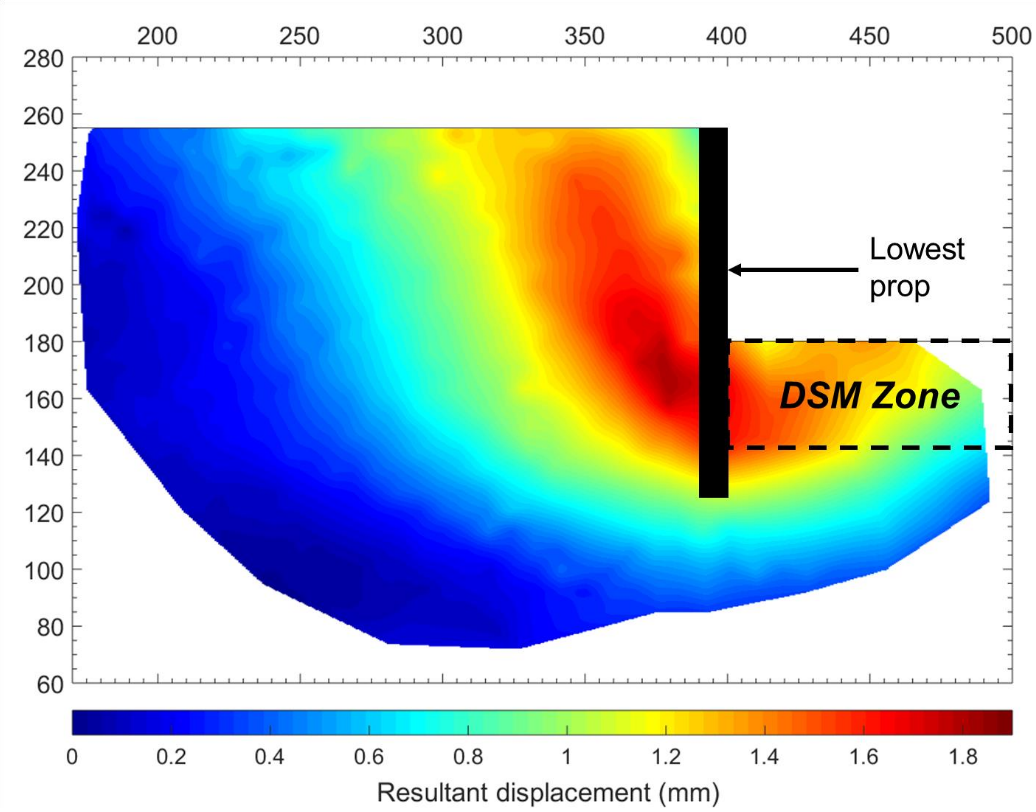


Figure 5. Comparison of surface settlement profiles at varying stages throughout excavation





(b)

Figure 6. Resultant movements in model scale at 39kPa overburden pressure for (a) reference test 1 and (b) lime stabilised test 2

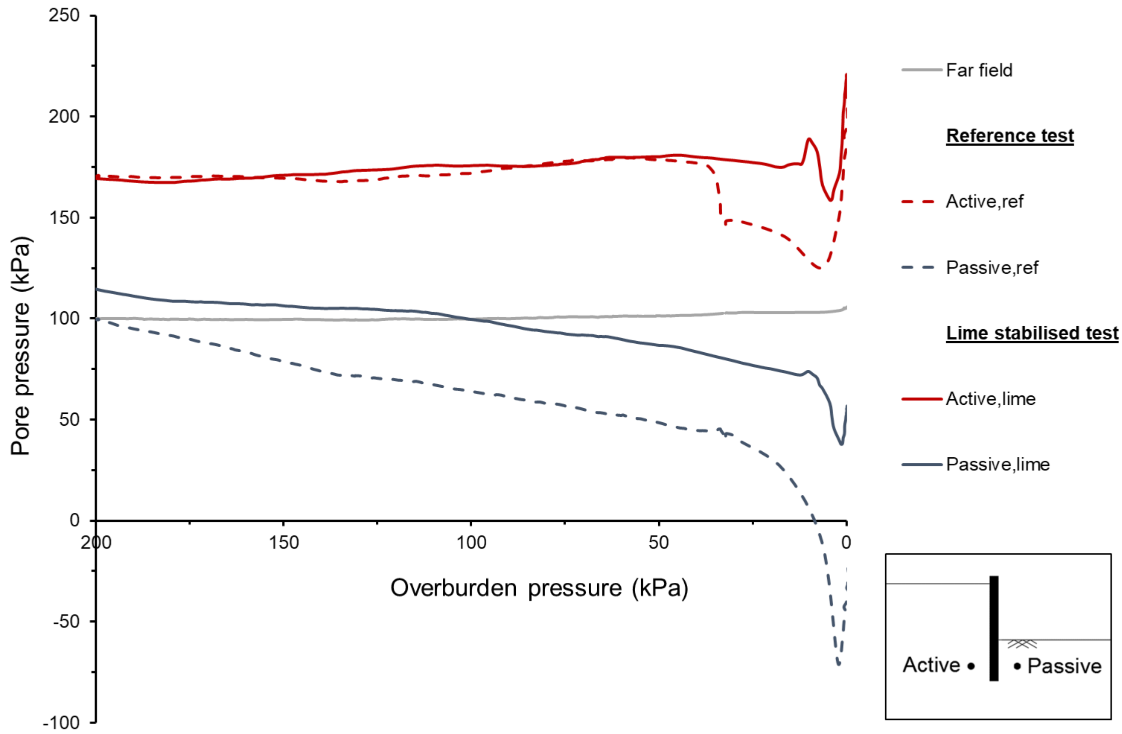


Figure 7. Pore pressure response during excavation for both reference and ground improvement tests and indication of location of PPTs.

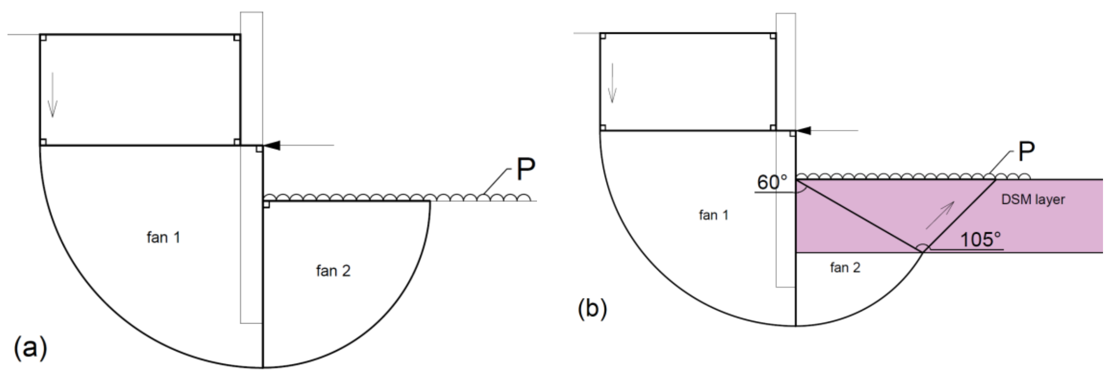


Figure 8. Proposed upper bound solutions for (a) reference and (b) lime stabilised tests

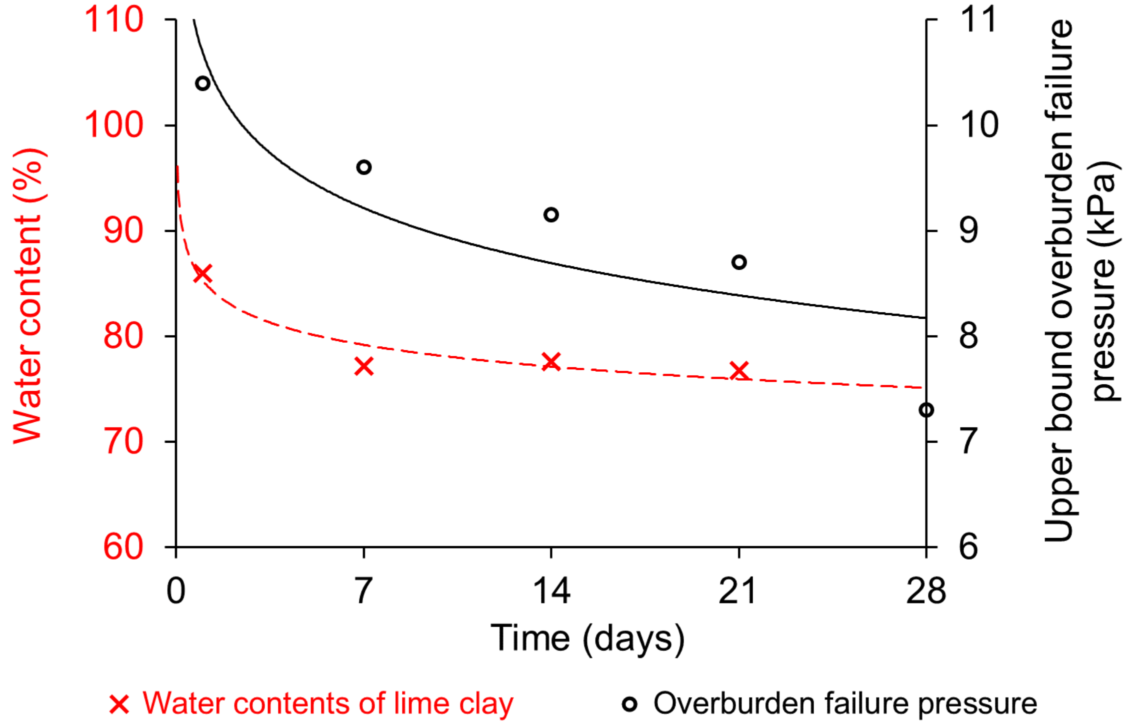


Figure 9. (a) Comparison of unconfined compressive strengths of lime-kaolin clay over time and (b) influence of time on water content and upper bound overburden failure pressure

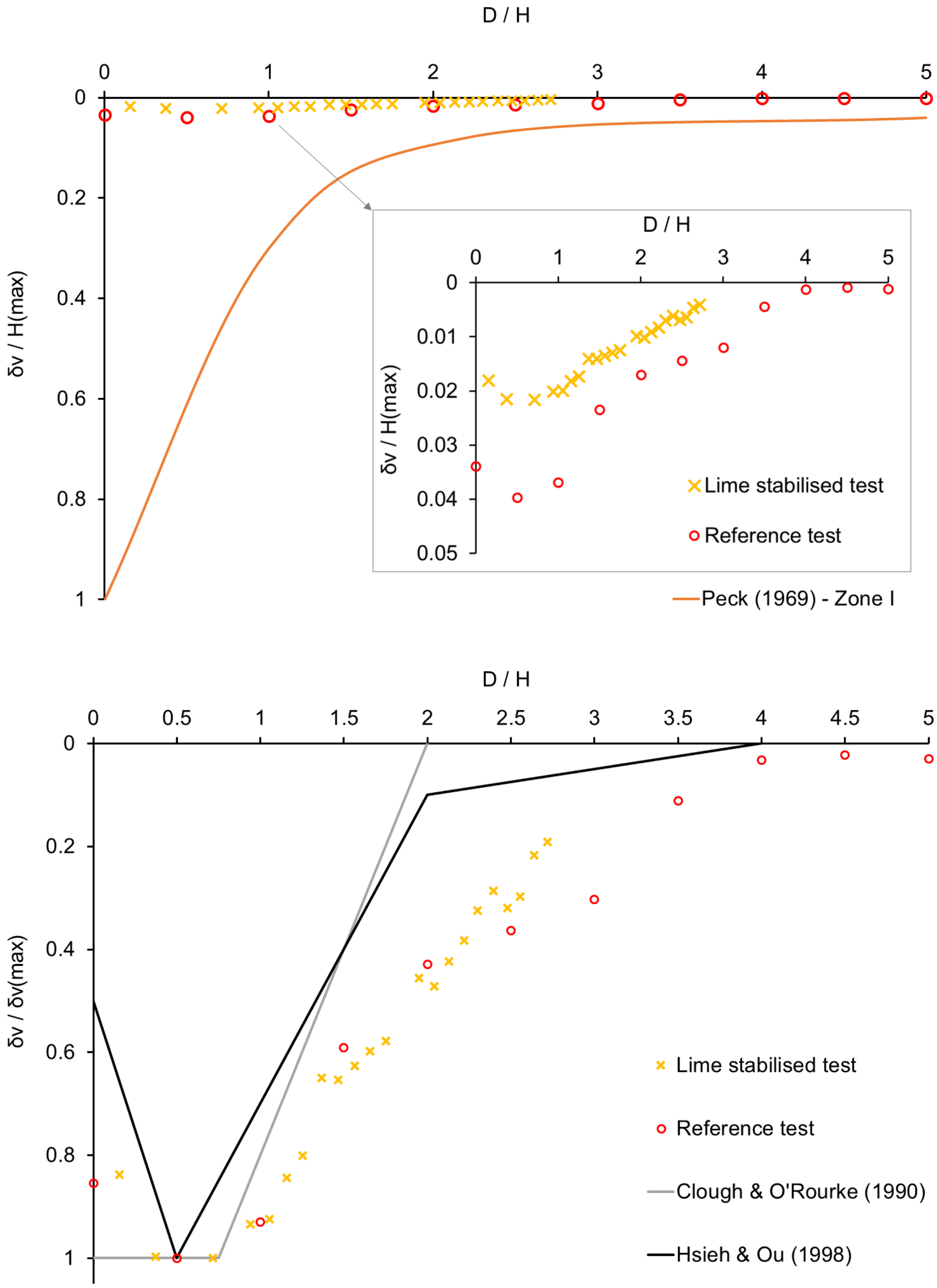


Figure 10. (a) Surface settlement profiles measured in tests compared with published literature and (b) normalised settlements from experiments within Zone I of Peck (1969)