Geotechnical Aspects of Underground Construction in Soft Ground – Negro & Cecilio Jr. (Eds)

Effects of wall embedment on base heave failure arising from deep excavations in soft soils

J.P. Panchal, S.E. Stallebrass & A.M. McNamara
City, University of London, London, UK

ABSTRACT: There is an increasing demand for basement construction in congested areas and the successful execution of this relies heavily on mitigating ground movements. The three primary causes of ground movements include wall bending, wall displacement and base heave, thus parameters such as wall stiffness, embedment and excavation support systems can influence movements. Physical modelling was used to focus on preventing basal heave at the formation level and subsequently reduce movements adjacent the excavation. To investigate the significance of wall embedment in soft soils control measures were in place to isolate displacements typically observed from wall bending or inadequate prop stiffness. A 160 *g* centrifuge test was performed to observe ground movements during an excavation of a wall retaining 12 m soil and of 8.8 m embedment. Measurements from this experiment were used in the upper bound analysis of a fan mechanism and indicated that the factor of safety against basal heave was 1.25.

1 INTRODUCTION

With the rise in development projects worldwide cities are increasingly becoming more congested, thus additional space is often obtained by engineering underground spaces. Ensuring the stability of the excavation is of paramount importance to mitigate the risk of fatalities and damage to neighbouring structures and services.

Soil movements largely arise owing to lateral wall movement, wall bending and ground heave at the formation level. Physical and numerical studies previously investigated the mechanisms of ground movements in soft soils. However, owing to the fact that these studies typically include all three parameters of ground movements, it is difficult to isolate the benefits of a remedial measure to a particular one of the three factors governing ground movements.

For instance, a series of centrifuge tests were undertaken by Lam *et al*. (2014) to investigate a number of variables on ground movements in soft clay. Such variables included toe fixity, prop stiffness and wall stiffness. Unsurprisingly, the results indicated that stiff walls and props can reduce the magnitude of movements. In addition to this, the results showed that wall movements could be reduced by improving the fixity of the toe of the wall and the extent of the settlement behind the wall is a function of the depth of the clay strata. However, owing to numerous variations between tests, it is difficult to confirm which variable reduced the magnitude of basal heave.

2 BACKGROUND

In practice the retaining wall is typically driven into firm strata, however in thick deposits of soft soils this may not be feasible. Therefore, it is important to ascertain whether the embedment of a wall determined using the limit equilibrium method is sufficient for controlling ground movements to acceptable levels.

In 2004, one of the worst civil engineering disasters of the decade occurred in Singapore (Hansford, 2012) resulting in four fatalities and ground movements extending 70 m away from the site. The project involved 2 km of the Singapore Circle Line tunnel section using cut and cover construction to a depth of 35 m, adjacent the Nicoll Highway.

The ground conditions comprised Singapore Marine Clay overlying dense Old Alluvium (OA). 800 mm thick reinforced concrete diaphragm walls were founded in OA but in some areas did not sufficiently penetrate the stratum, as specified by the design. A layer of jet grouting stabilised the formation level with the upper portion being removed during the excavation works, as illustrated in Figure 1a.

Props were used as supports as the excavation progressed and Figure 1b illustrates the wall deflection recorded by inclinometers. Simpson *et al*. (2009) summarised that the failure of the excavation was owing to a number of factors, which included, but were not limited to, the inadequate connection design of the struts, the failure of the jet grouted layer and poor use of finite

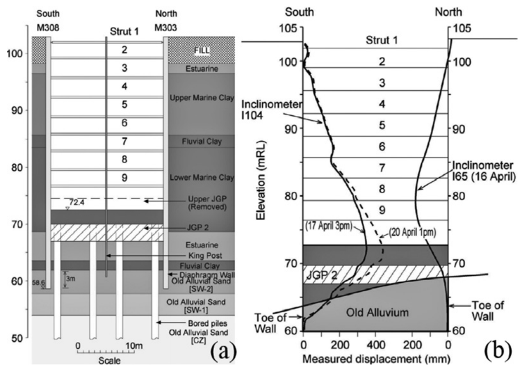

Figure 1. (a) Cross section of proposed Singapore Circle Line tunnel and (b) measured displacements during excavation (Simpson *et al.*, 2009).

element analysis software (PLAXIS). Although the toe of the wall had been embedded in stiff strata, the excavation failed, partly owing to the lack of understanding of the importance of embedment. This emphasises the complex interaction of a variety of factors affecting the stability of excavations and the requirement to isolate them to better understand the influence of each variable on the failure mechanism in soft soils.

3 OBJECTIVES

The aim of this study was to establish whether retaining walls of considerable embedment mitigate the risk of basal heave failure.

The aim of the centrifuge test conducted at 160 *g* was to ascertain the undrained shear strength profile and bulk unit weight of the soil. Image analysis of subsurface deformations were also used to measure relative movements of the ground during the excavation process for a retaining wall of 8.8 m embedment at prototype scale.

The focus of this paper was to determine the upper bound solution for the wall embedment that would result in base heave failure. Results from the centrifuge experiment were used in this analysis.

4 PRINCIPLES OF CENTRIFUGE MODELLING

Typically, physical modelling involves replicating real life events of a prototype at a reduced scale (Taylor, 1995). Centrifuge modelling can achieve this as it enables the simplification of complex geotechnical problems and allows engineers to physically simulate such problems using geotechnical materials.

Similarity between the prototype and model must be achieved by applying the correct scaling laws and ensuring that the stress history of the soil model represents the prototype.

An inertial radial acceleration field (N) is induced on the model by the acceleration of the centrifuge. This radial acceleration field can be related to a gravitational acceleration field. The basic law of centrifuge modelling states that the height of the prototype h_p, is equal to Nh_m, where h_m is the height of the model. Thus, as the bulk unit weight of the soils are similar, the in situ vertical stresses within the soil model can be compared to the prototype.

The Acutronic 661 beam centrifuge used for this experiment is located at City, University of London, has a 40 *g*/tonne capacity and a radius of 1.8 m. Dimensions and measurements of this centrifuge experiment will be presented in model scale, unless stated otherwise.

5 SOIL MODEL

The experiments were conducted in a rectangular strongbox, 375 mm deep with internal plan dimensions 550 mm 200 mm. The walls of the strongbox were lubricated with waterpump grease and sheets of porous plastic and filter paper were placed at the bottom.

Speswhite kaolin was mixed to a water content of approximately 120%, twice its liquid limit, producing a workable slurry. The slurry was carefully placed in the strongbox using a scoop and palette knife to avoid air entrapment. A 300 mm high extension was bolted to the top of the strongbox to produce a sample with a minimum final height of 290 mm. A second layer of porous plastic and filter paper were placed on top of the sample to allow drainage from the top and bottom of the sample, thus accelerating the consolidation process.

The strongbox was transferred to a hydraulic press where a tightly fitting platen was lowered onto the sample. The pressure on the sample was gradually increased to a maximum vertical effective stress of 100 kPa over a period of 2 days before being left to consolidate for a total of 10 days.

Consolidating the clay to 100 kPa produced a very soft sample, so it was necessary to further consolidate the sample on the centrifuge at 160 *g*. Firstly, a scraper was used to trim the sample to a height of 290 mm before placing it on the centrifuge swing. A standpipe was connected to the sample to supply a water table 10 mm above the soil surface to ensure the sample remained saturated. A lid was also bolted to the top of the strongbox to prevent the water from evaporating during flight. An LVDT was clamped to this lid to measure the settlement of the soil and was used to confirm that

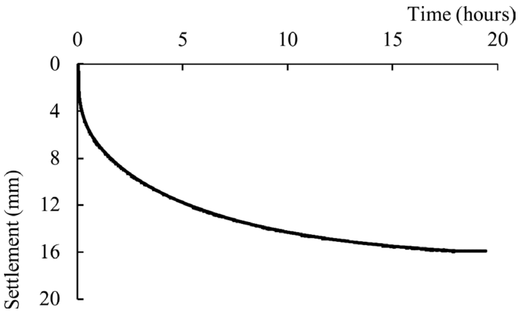

Figure 2. Settlement profile of clay surface during initial in-flight consolidation phase.

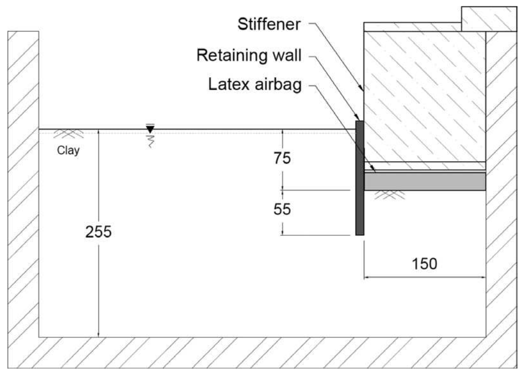

Figure 3. Centrifuge apparatus set up.

the sample had consolidated. The settlement profile is illustrated in Figure 2.

Following the in-flight consolidation phase the sample was removed from the swing and model making commenced.

6 TESTING APPARATUS

The apparatus used in this centrifuge test is illustrated in Figure 3. The model consisted of a 255 mm high clay sample and a 75 mm deep excavation, which translate to a 40.8 m thick soft deposit and 12 m deep excavation at prototype scale.

In order to limit the number of variables influencing the ground movements arising from the excavation, a high stiffness sheet piled wall was designed. The model wall comprised a ribbed profile such that each rib was 1 mm thick and the overall thickness of the model wall was 10 mm. This particular wall profile was adopted as it enabled the wall to be pushed into the clay prior to removing any soil. At prototype scale this retaining wall had the equivalent stiffness of a 2.4 m thick reinforced concrete wall. To prevent water seepage around the wall, bespoke silicone seals were cast on the sides of the wall.

The wall embedment used in the centrifuge experiment was designed using the limit equilibrium method, based on a preliminary test. The bulk unit weight was 16.7 kN/m^3 and the undrained shear strength profile of the soil was $7 + 0.1z$ kN/m^2, where z is the depth below ground level in mm.

An aluminium cutting shelf was machined to be secured to the front of the strongbox and indicated the areas of soil to be removed. A ribbed Perspex guide was used to guide the wall; maintaining verticality during the installation of the wall. It was bolted directly onto the strongbox back wall and was supported by the cutting shelf.

In addition to this an aluminium stiffener, developed by McNamara *et al*. (2009), was used to support the upper length of the wall simulating a fully propped wall with an overall high system stiffness. Owing to the fact that this stiffener was supported by the side wall of the strongbox and was also 150 mm   200 mm in plan, any possible lateral movements of the top of the wall into the excavation were eliminated.

A pressurised latex airbag surcharged the formation and consequently reacted against the aluminium stiffener directly above it, the strongbox wall and the retaining wall. Owing to the ribbed profile of the retaining wall it was necessary to design a spacer that prevented the bag from bursting under high pressure. This spacer consisted of a 200 mm   50 mm plate to which 8 mm square solid aluminium channels were fixed. A silicone seal was cast between the spacer and wall which prevented seepage into the excavation during reconsolidation.

7 MODEL MAKING PROCEDURE

Following in-flight consolidation the sample was removed from the centrifuge swing and the standing water removed. A scraper was used to trim the sample to a height of 255 mm.

To seal the top surface of the clay a synthetic impermeable rubber, PlastiDip, was sprayed onto the clay and left to cure. The front face of the strongbox was then removed and the exposed face of clay was sealed with a thin layer of silicone oil to prevent the sample from drying out.

The cutting shelf and Perspex guide were attached to the strongbox in preparation for installing the wall. However, owing to the silicone seals along the edges of the wall it was necessary to create voids in the soil to cater for these larger solid sections. Circular and square thin walled brass cutters, 10 mm in diameter and width respectively, were guided into the clay to a depth of 130 mm to remove the soil from these areas. At this point, the silicone seal on the wall was lubricated with silicone grease prior to pushing the wall into the soil such that it stood 5 mm proud of ground level. A bead of silicone grease was applied at the wall and soil interface behind the retaining wall.

Steel plates were used to scrape away the excavation area to a depth of 75 mm, whilst care was taken to ensure that neither the wall nor the formation level were disturbed. Following the removal of the bulk of the soil, a smaller scraper was used to extract the soil in between the ribs in the wall.

Upon completion of the excavation, sheets of filter paper and 0.75 mm thick porous plastic were placed on the formation level and were slotted in between the ribs of the wall. The spacer was secured to the wall and the latex bag was attached to a brass union through the side wall of the strongbox. Another sheet of porous plastic was placed above the bag before securing the aluminium stiffener to the model.

During the initial in-flight consolidation stage the sample had settled up to 16 mm, thus the PPTs could only be installed during the model making phase. Cores were taken from the back face of the strongbox before placing de-aired PPTs in the soil. The PPTs were backfilled with kaolin slurry, mixed to a water content of 120%.

In order to track subsurface movements, 1 mm diameter black glass ballotini beads were scattered across the surface of the clay to create a texture suitable for image analysis using Particle Image Velocimetry (PIV), details of the technique are given by Stanier and White (2013). In addition to this, 3 mm diameter black acetal bullet shaped targets were also inserted into the clay at 10 mm centres. The movements of the ballotini and targets were tracked with on board cameras. Once the targets had been rolled into the clay a Perspex window was bolted to the face of the model.

The model was weighed before being transferred to the centrifuge swing. A standpipe was connected to the base drain to provide a water table 5 mm below the clay surface of the model.

8 INSTRUMENTATION OF CENTRIFUGE MODEL

This research focusses on ground movements arising from deep excavations in soft soils. As well as observing subsurface movements, surface movements were obtained from LVDT measurements. These were clamped to a gantry which was supported across the top of the strongbox and aligned along the centreline of the strongbox. The LVDTs were spaced at $^{H}/_{2}$ centres from the wall, with an additional LVDT placed directly behind the wall (0H).

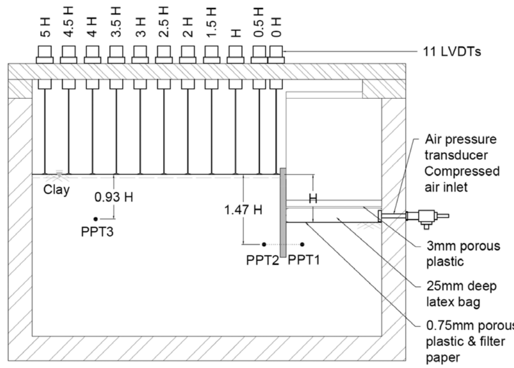

Figure 4. Location of instrumentation on centrifuge model.

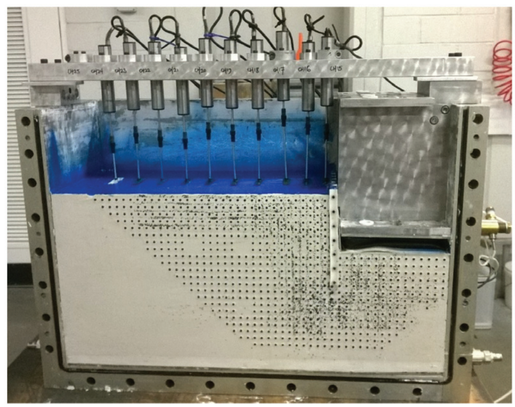

Figure 5. Completed centrifuge model prior to testing.

Two PPTs were located either side of the retaining wall at a depth of 1.47 H from ground level. Another PPT was placed at a distance 4 H from the wall and was used to measure any far field pore pressure changes arising from the excavation. Each of the PPTs were installed to the centreline of the strongbox.

The air pressure supplied to the latex bag to surcharge the formation level was measured using a pressure transducer housed within a brass union. The locations of all instrumentation are given in Figure 4 and the completed model is shown in Figure 5.

9 TESTING PROCEDURE

The model was reconsolidated on the centrifuge at 160 g and the PPTs readings were monitored

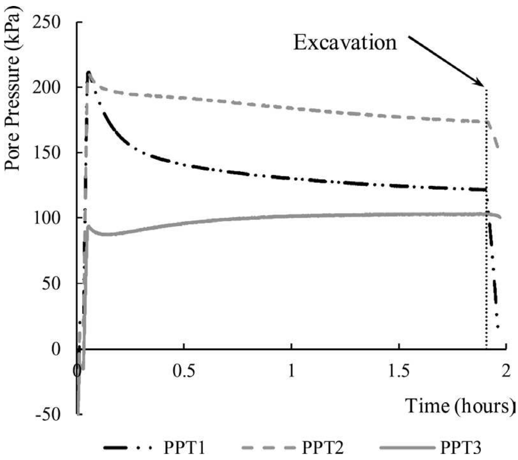

Figure 6. Dissipation of excess pore pressure during reconsolidation stage.

to confirm that the excess pore pressures had dissipated. Having consolidated the sample at 160 *g* prior to model making, the sample came into equilibrium within 2 hours, as illustrated in Figure 6.

In order to replicate the excavation phase, the air pressure in the latex bag was reduced at a rate of 1 kPa/sec and the whole excavation process was completed within 3.5 minutes.

10 CENTRIFUGE TEST RESULTS

This paper presents the results from one centrifuge test carried out at 160 *g*. The plane strain test comprised a 75 mm deep excavation, 55 mm wall embedment and 150 mm half width. This modelled a retained soil height of 12 m, an embedment of 8.8 m and an excavation half width of 24 m at prototype scale. The overburden pressure was relieved at a constant rate and the excavation was subsequently completed in two months at prototype scale.

The extent and magnitude of surface settlement, measured using the LVDTs, whilst on board cameras were used to capture subsurface movements. Such movements are shown in Figure 7 and Figure 8 respectively.

Figure 8 illustrates that the largest ground movements are concentrated around the wall and do not extend below the toe of the wall. Smaller movements are observed below the toe of the wall, but equate to less than a third of the maximum resultant movement.

Shear vane readings, taken at 1 *g* immediately after the excavation, confirmed that the undrained shear strength profile of the soil was constant at 16 kN/m^2 to the formation level. Below this depth, the profile can be described as $16 + 0.26z$ kN/m^2,

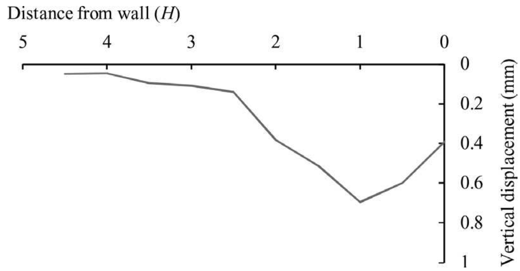

Figure 7. Surface settlement profile post excavation.

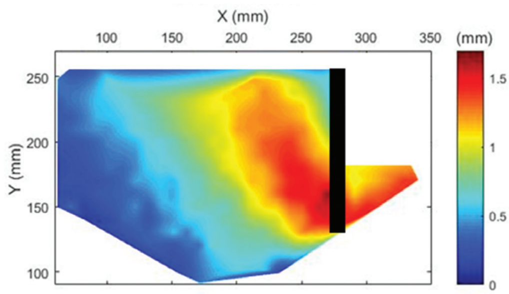

Figure 8. Resultant movement of ground during excavation.

where z is the depth below the formation level in mm at model scale. The bulk unit weight of the clay was calculated as 16.16 kN/m^3.

11 UPPER BOUND ANALYSIS AT MODEL SCALE

Atkinson (1981) stated that engineering structures cannot be allowed to collapse, therefore determining the load at which a structure will fail is important in engineering design. Atkinson explains that by neglecting equilibrium conditions and primarily focussing on compatibility conditions, an upper bound can be calculated; a condition at which a structure will collapse.

An upper bound solution for the excavation geometry and ground conditions used in the centrifuge test was computed. The aim of this was to identify a reasonable failure mechanism and embedment at which the wall would fail.

Two compatible mechanisms were investigated, a wedge mechanism (Figure 9a) and a fan mechanism (Figure 9b). The width of each mechanism (B) was varied to obtain the optimum upper bound value for the depth of embedment of the wall (d) at which the base of the excavation fails at model scale.

It was assumed that the full undrained shear strength was mobilised on all slip planes and that the retaining wall moved down with the soil behind it. Owing to the ribbed wall profile, 18% of the 10 mm thick wall was in fact stainless steel, whilst the rest of the area was filled with clay. Owing to this, the equivalent unit weight for the wall and soil was calculated as 27.45 kN/m^3.

Figure 10 graphically demonstrates how the model embedment depth varies for different widths of the mechanism. The results indicated that the wedge mechanism predicts significantly lower values of embedment at which the wall fails compared with the fan mechanism. It predicts that the optimum upper bound solution for a wedge mechanism is a 16 mm wall embedment with a *B* width of 50 mm. On the other hand, the fan mechanism suggested that at a wall embedment of 32 mm, the excavation will fail when the mechanism is 85 mm wide. Consequently, the best estimate of the required depth of embedment is provided by the fan mechanism.

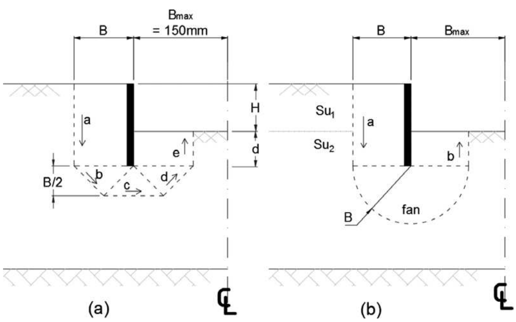

Figure 9. Plastic failure for (a) wedge mechanism and (b) fan mechanism.

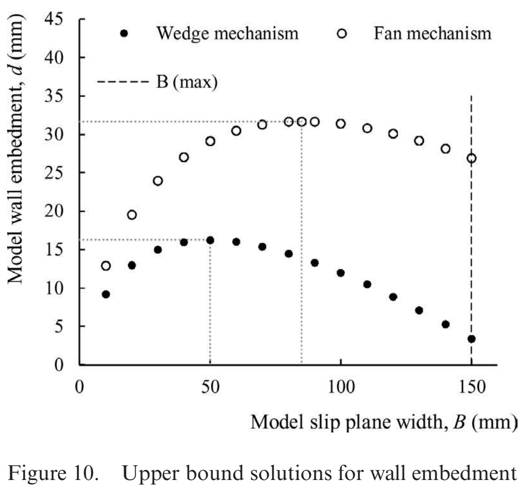

Figure 10. Upper bound solutions for wall embedment at model scale assuming the full undrained shear strength is mobilised.

Further analysis was carried out to estimate the proportion of undrained shear strength mobilised in the centrifuge test. Plotting the minimum wall embedment for varying factors of safety, the following equation (1) was determined, where *d* is the model embedment depth in mm.

$$FS_{heave} = 0.0106d + 0.6644 \quad (1)$$

Using this equation, it was calculated that the factor of safety on a 55 mm embedded wall is 1.25 and 80% of the undrained shear strength was mobilised in this experiment.

12 DISCUSSION

The displacements observed in the centrifuge test, illustrated in Figure 8, show that the largest movements extend 100 mm behind the wall. However, the centrifuge test did not result in failure of the excavation. Therefore, the mechanism illustrated in Figure 8 may represent pre-failure displacements driven by a mechanism, which does not correspond with the plastic failure mechanism of the excavation.

The upper bound solution for the fan mechanism indicated that base heave failure occurs when a wall embedment depth is shallower than 32 mm. Owing to the embedment of the model wall in the centrifuge test being 55 mm, it was expected that the excavation would not fail as a result of basal heave.

13 CONCLUSIONS

A single centrifuge test at 160 *g* was conducted to model the ground movements associated with a deep excavation in soft soil. The experiment modelled a 150 mm half width excavation, retaining 75 mm of soil with an embedment of 55 mm. The initial wall embedment was determined using limit equilibrium analysis based on ground conditions from a preliminary test.

To investigate the minimum wall embedment at which base heave failure occurs, upper bound solutions were computed to establish the wall embedment at which base heave failure would occur. The undrained shear strength profile and bulk unit weight of the soil used in the analysis were taken from the centrifuge experiment. It was assumed that the full undrained shear strength was mobilised.

The results from the centrifuge test showed that the largest movements occur in close proximity to the wall and are limited to the toe of the wall. Movements equal to approximately a third of the maximum resultant movement are noticeable below the toe of the wall.

Upper bound analysis using the fan mechanism suggests that for a model wall embedment of 32 mm base heave failure occurs, this translates to 5.12 m at prototype scale. The suggested embedment from a limit equilibrium analysis was 55 mm, corresponding to 8.8 m at prototype scale. For a wall of 55 mm embedment, the factor of safety against heave is 1.25. A wall of deeper embedment is unlikely to offer significant benefit against basal heave failure.

14 FURTHER WORK

Conducting another centrifuge test with 32 mm model embedment will confirm the validity of this upper bound mechanism. Following basal heave failure image analysis of subsurface deformations can be used to identify the failure mechanism for a deep excavation in soft soils.

Further tests can be conducted with different wall embedment depths to establish whether the extent and magnitude of ground movements in soft soils are sensitive to changes in embedment depth.

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