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<b>Authors(s)</b>	Laefer, Debra F.
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# Solutions for Model-scale, Tied-back Anchors and Sheet Pile Walls

D.F. Laefer

School of Civil, Structural and Environmental Engineering, University College Dublin, Newstead, Dublin, Ireland  
email: [debra.laefer@ucd.ie](mailto:debra.laefer@ucd.ie)

**ABSTRACT:** Scaled laboratory testing is attractive, because it is faster and less expensive than large-scale or full-scale experiments and does not rely upon possible uncertainties in the constitutive models necessary for numerical modelling. However, for reduced-scale experiments to be reliable, a model must respond like its prototype. To achieve this, geometric, kinematic, and dynamic relationships must be upheld. For 1-gravity (1g), pseudo-static testing of subsurface construction systems, the main problem is obtaining geometrically-appropriate materials whose strength and stiffness are both sufficiently reduced compared to those of the prototype. There are also the usual constraints of constructability, which often present competing factors and can pose major challenges. This paper introduces new solutions for manufacturing, assembly, monitoring, and testing for tied-back anchors with a sheet pile wall for 1g soil-structure experiments and presents some of the experimental results for 1/10th scale testing.

**KEY WORDS:** Scaling, tied-back anchor, sheet pile wall

## 1 INTRODUCTION

Use of model-scale facilities is often attractive for soil-structure interaction problems as the researcher has total control of the system. This is advantageous in several areas: (1) obtaining a highly homogeneous soil of known attributes; (2) introducing constructed elements into the relationships that are of known conditions – often elements that have been in the field have been subjected to environmental degradation and/or overloading; (3) testing the system repeatedly changing only one variable in each experiment; (4) bringing the system to failure; and (5) selecting the failure mechanism. This paper will describe scaled tiedback anchors and a sheet pile wall.

## 2 BACKGROUND

Testing chambers (e.g. fig. 1) have been used to explore a wide range of geotechnical problems: fundamental soil properties (e.g. [1,2]), soil reinforcement (e.g. [3,4]), deep foundations (e.g. [5,6]), shallow foundations (e.g. [7,8]), and earthquake modelling [9,10]. This vast topic range emphasizes the critical role such facilities play in data generation often to verify numerical analysis and to further understand field data (each approach providing unique insights). One major, American facility is the Schnabel Laboratory at the Univ. of Illinois at Urbana-Champaign, an above ground testing reconfigurable chamber that can hold up to 36.29 metric tons of soil. That facility has been used to investigate soil-nailed walls [11], earth reinforcement [12], and soldier piles and lagging [13], as well as tied-back anchors [14], which is portrayed in fig. 1 and will be the subject of this paper.

## 3 SCALING

### 3.1 General

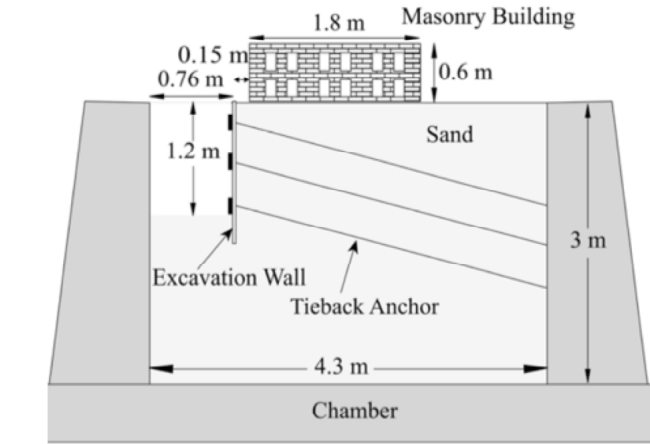
While laboratory experiments offer some unique advantages over field sites and numerical modelling, conducting

experiments at less than full-size may generate incorrect responses. Langhaar [15] named these negative repercussions “scale-effects”. Scale-effects may emerge in terms of geometric, kinematic, and/or dynamic factors. To minimize (and eliminate, where possible) scale-effects when conducting model-scale work, the concept of dimensional homogeneity or similitude was pioneered by Buckingham [16] and Rayleigh [17] and furthered by Langhaar [15]. Specifically, prototype behaviour must be understood so that variables exhibiting a significant influence on system performance can be identified as the input components of dimensionless products to describe the behaviour of both the model and prototype [18]. Significant variables are then considered in relation to components of mass, time, and length. A series of linear equations (one each for mass, time, and length) are established using a series of unknown constants for each performance variable of significance (e.g. density, velocity).

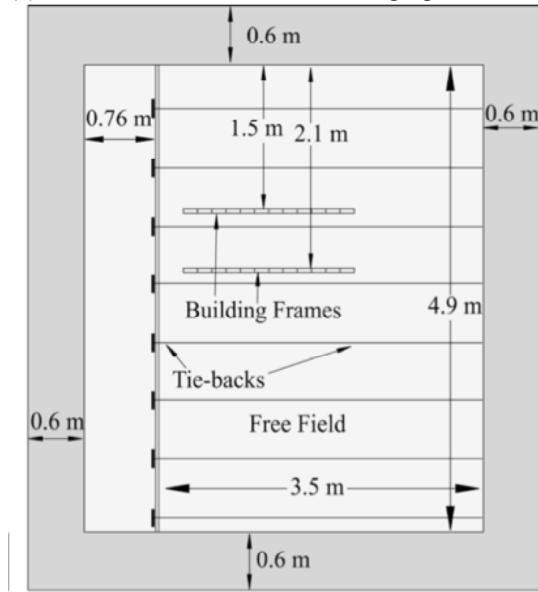
Various arrangements are tried, in order to establish a determinant for a 3x3 group. The determinant is considered the rank. The difference between the rank and number of performance variables establishes the required number of independent, dimensionless products. According to Langhaar [15], dimensionless products are independent, “If every other dimensionless product of the variables is a product of powers of dimensionless products in the set.” Independent, dimensionless products are achieved through solving the resulting linear equations by setting all excess variables equal to zero, except the variable of interest; usually the one over which there is greatest control [19].

### 3.2 Scaling for tied back anchors

Strain equivalency was determined to be a fundamental component of the scaling equivalences (eqn 1); strain parity is often recommended [20, 21] for representing equivalency between the model and prototype.



(a) Section of test excavated to design grade



(b) Plan of test with freefield and buildings

Fig. 1. Schematic of testing chamber

For the case of tiedback anchors, this was manifested in the relationship of the change in anchor length at full load, with respect to final excavation depth, as developed in equations 1 to 8:

$$\varepsilon_p = \varepsilon_m \quad (1)$$

where  $\varepsilon_p$  = prototype strain and  $\varepsilon_m$  = model strain.

$$(\delta / L')_p = (\delta / L')_m \quad (2)$$

where  $\delta$  is the deformation of the anchor under full loading and is defined in equation 3 and  $L'$  is the extensible length of the anchor

$$\delta = PL' / A_t E \quad (3)$$

where  $A_t$  is the area of the tieback,  $E$  is Young's Modulus,  $P$  the tieback anchor's load, and  $L'$  is the portion of the tieback anchor as defined by eqn 4,

$$L' = L_u + 0.5L_b \quad (4)$$

where for the prototype the unbonded length of the anchor is  $L_u$  and the bonded section represented as  $L_b$ . For the model  $L' = L_b + L_u$ , representing the entire length (bonded and

unbonded portion) of the anchor which was largely dictated by the geometry of the testing pit and

$$P = IIDLb\tau_{ult} \quad (5)$$

where  $D$  is the diameter of the reinforcing steel and  $\tau_{ult}$  the ultimate shear capacity of the anchor based on the grouting pressure and soil type. The load per anchor,  $P$ , is based on the amount of lateral pressure exerted on a predetermined tributary area,  $A_b$  (eqn 6)

$$P = \sigma / A_b \quad (6)$$

$$\text{where } \sigma = 0.65K_a\gamma H \quad (7)$$

where  $\sigma$  is the apparent earth pressure (AEP) on the excavation wall,  $\gamma$  the soil density, and  $K_a$  the coefficient of active earth pressure, as defined by eqn 8

$$K_a = \tan^2(45 - \phi/2) \quad (8)$$

where  $\phi$  is the soil's friction angle.

#### 4 EXPERIMENTAL DESIGN

Choice of the model test components and systems began with prototype selection or geometries, physical properties, and anticipated loading, followed by design of the full-scale structures. A similar set of activities were applied to the model elements, but constricted by geometrical requirements, material availability limitations, and scaling criteria. Based upon the scaling requirements, the various equations were satisfied by either modifying the geometry or selecting constituent materials of relevant properties. This occurred within the confines of the configuration of the testing facility.

##### 4.1 Overall testing chamber set up

The testing for which this was devised involved comparing a freefield condition with one that had building structures incorporated into it. A 6.01m by 18.03m footprint was selected as typical of a late 19th century urban building. To accommodate this, 1/10<sup>th</sup> scale was selected as the largest scale that could include both a free field area and a constructed zone without generating boundary condition problems within the test chamber. With this, a 12m prototype deep excavation could be replicated.

The excavation walls incorporated 3 levels of tiebacks and were excavated to a depth of 1.22m. Soil was removed in depths of 10.2cm. Measurements were taken with each excavation level and before and after the post-tensioning of the tieback anchors (fig. 2) [22].

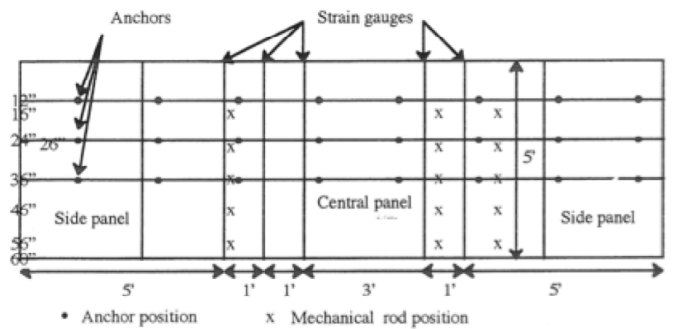


Fig. 2. Schematic of excavation wall showing 3 panels, and locations of the anchors and instrumentation (see section 4).

#### 4.2 Distribution of lateral forces

For design of the excavation wall system, a full-scale design was conducted, which started with the anticipated lateral earth pressures. Initial design of the tiedback anchors was based on the trapezoidal shape of AEP diagrams of lateral forces on braced excavations in sand as proposed by Terzaghi and Peck [23]. Looking at a range of scenarios, where the coefficient of friction for the sand ranged from 35°-40° and the set back varied from the depth of excavation plus 1/3<sup>rd</sup> of that excavation to 1/5th of it (Table 1), a tributary area of 3.05m x 3.05m was presumed. The exercise was repeated for a 1/10th scale height but with a tributary area that was twice as wide. For the model-scale anchors, instead of being on a 0.31m x 0.31m grid (thus directly reflecting a typical 3.05m x 3.05m prototype), anchors were proposed to be placed 0.61m, horizontally. This achieved the placement of the anchors in a symmetric manner in relation to the lines of the strain gauges (fig. 2) and an increase in load per anchor, thus resulting in a larger displacement, from which it would be easier to generate the target model displacement (of only 2.54 mm, even with the larger spacing). This increased distancing also assisted in generating more available space in which to place instrumentation and to the general constructability of the project. The anchor stiffness was also scaled to be 1/10th of the prototype with respect to anchor length.

Table 1. Components for the anchor design

$\Phi$	Ka	Stress kg/cm <sup>2</sup>	Unbonded anchor m		Bonded anchor m	Area mm <sup>2</sup>
Proto- type		9.29m <sup>2</sup>	Lu (H/3)	Lu (H/5)		
35	.271	5,946	13.2	11.6	3.35	
40	.217	4,761	13.2	11.6	2.74	
Model		0.19m <sup>2</sup>				Steel (mm)
35	.271	11.9	3.58	3.58	-*	D=.41
40	.217	9.52	3.58	3.58	-*	D=.36

\*Modelling was done with no bonded zone but with comparatively infinite strength; D = diameter

Using the relationships previously described and the assumption that the anchor installation was set at 15° from horizontal, sizing for the prototype was completed. The deflection under 448.22 N was calculated and considered as a percentage of the excavation height. From this, the deflection characteristics of the model anchor were established, and initial sizing of the components was made. Anchor design was based upon procedures summarized in Peck et al. [24]. The toe penetration was neglected as it was not needed to create the model. Grouting pressures were based on the experimental work by Ostermayer and Scheele [25]. This was correlated to a grouted anchor capacity in non-cohesive soils.

## 5 EXPERIMENTAL DESIGN

### 5.1 Excavation wall design

The impact of the relative soil/wall stiffness was dominant in many aspects of wall deformation, namely cantilevering and lateral bulging as pioneered by O'Rourke and Cording [26]. Consequently, the excavation wall thickness was selected

based on differing flexibility ratios as defined by equation (9) by Cording [27].

$$\rho = E_s L^3 / E_w I_w \quad (9)$$

where  $\rho$  is the flexibility ratio,  $L$  is the unsupported span of wall,  $E_s$  the Young's Modulus of the soil,  $E_w$  the Young's Modulus of the excavation wall, and  $I_w$  its moment of inertial. To achieve this, a single rolled sheet of 1.52m x 3.05m x 0.093" was used. This was flanked on each side by a similar panel of 1.52 m x 0.91 m x 2.36 mm. To modify the stiffness performance of the excavation wall system, for 2 of the 6 tests, the final sand elevation at the beginning of the test was lowered by 12.7 cm. This dramatically decreased the tendency of the excavation wall to cantilever along the top 30.5 cm.

There was another design component, which had to be considered with respect to wall design. Namely, the downward component caused by the tiebacks had to be sufficiently resisted to prevent excavation wall plunge during normal excavation procedures. Given the extremely thin sheet piling used in the experiments, no end contribution was considered. Side resistance was based on frictional resistance. The angle of friction was found to be approximately 16° using small-scale samples of the excavation wall material against the sand. This did not take into account the additional roughness caused by the presence of over 300 strain gauges and their respective guards and wiring. Even based on the highly conservative estimate of 16°, adequate resistance against plunging was found by a toe penetration of 30.5 cm for the continuous excavation wall. This was in part selected based on input from industrial partners.

### 5.2 Anchor design

Even by doubling the tributary area of the model anchor, proper stiffness and deformation characteristics could not easily be achieved across a nearly 3.66 m length with a single rod. Consequently, the anchors were selected to be small diameter rods with the scaling accommodated in a loading plate (with a stiffness of 156,250 lb/in), which spanned 15.3 cm to either side of the anchor and was mounted on the front of the wale. The deformation of the anchorage system occurred in the loading plates. The chosen anchor's deflection characteristics were more than an order of magnitude less than the plates to preclude unanticipated movement in other parts of the anchorage system (fig. 3).

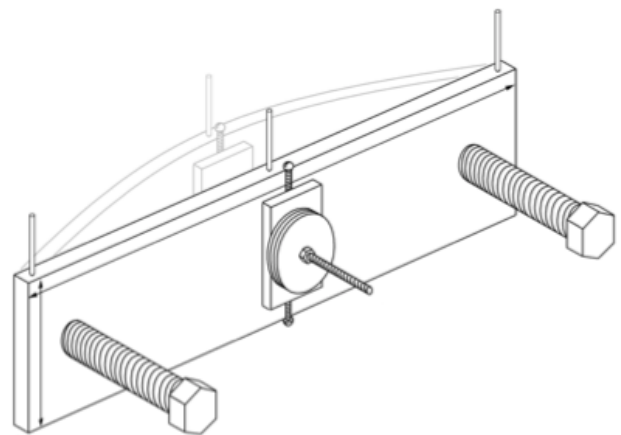


Fig. 3. Anchor plate schematic.

To ensure line loading across the depth of the plate, two pins, a series of washers, and a load distribution plate were set between the anchor plate and the nut that attached the anchor rod (centre of fig. 3). The anchors were loaded by turning the bolts. These ended in a rounded nut assembly that rested in a specially machined receptacle, which were designed to ensure that the anchor was held at the 15° design angle (fig. 4). The receptacles were welded to the continuous steel tubes that served as wales and helped ensure continuity between the three panels shown in figure 2.

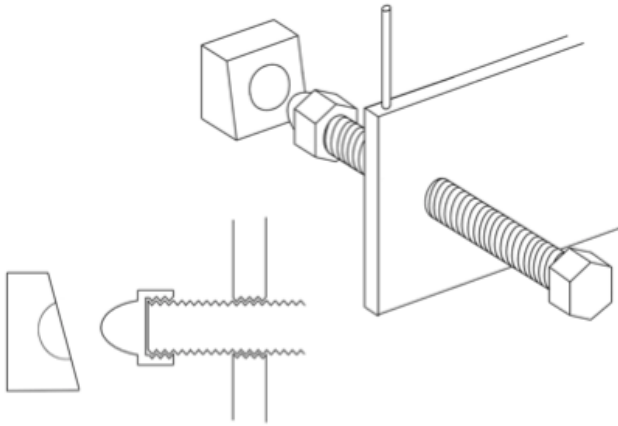


Fig. 4. Anchor bolt/nut schematic.

The anchors themselves consisted of 6.35 mm diameter, stainless steel rods. Approximately 15.24 cm on each end were threaded. These were placed through the excavation wall as part of the chamber preparation, as shown in the assembly in figure 5. The far end of the anchor was bolted into a metal frame that was mounted onto the back of the testing chamber. The bonded portion of the rod consisted wholly of the metal rod, which was left untreated in its “off-the-shelf” condition.

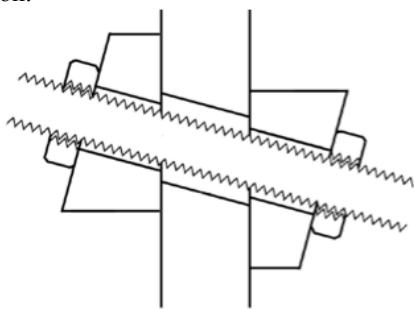


Fig. 5. Anchor connection through the excavation wall.

The connection shown in fig. 5 helped keep the anchor at the designed 15° angle from horizontal. A similar arrangement was constructed at the back of the testing chamber, as shown in fig. 6 where the anchor was attached to the back of the chamber wall. The unbonded portions of the anchors were prevented from binding in the sand by being encased in a series of PVC tubes, the ends of which were stuffed with terry cloth to preclude sand intrusion. The length of the sleeve was based on a 60° degree failure wedge. Along the side of the chamber walls, caution tape was attached to designate the pre-selected heights for horizontal displacement gauge placement, as will be described in section 6.1.2. To further ensure that no premature loading of the anchor occurred, during the filling of

the chamber the anchor plates were set rotated from their seating receptacles (fig 7.).



Fig. 6. Back corner of testing chamber shown with anchor attachment assembly and backs of anchors attached during chamber emptying.



Fig. 7. Top layer of disengaged anchor plates shown during the excavation. Excavation levels shown on the excavation wall as white lines.

## 6 MEASUREMENT MEANS

During the testing three types of measurements were made: wall displacements, anchor displacements, and anchor loads.

### 6.1 Wall displacement

#### 6.1.1 Vertical

Vertical wall displacement was obtained from several manual displacement gauges that were attached to the top of the excavation wall and fixed to an immovable reference frame. These same type of gauges were used to measure the vertical soil displacement at the soil surface. The lateral displacements were measured from both the inside and outside of the excavation wall. The interior measurements were obtained from a series of electronic displacement devices (fig. 8).

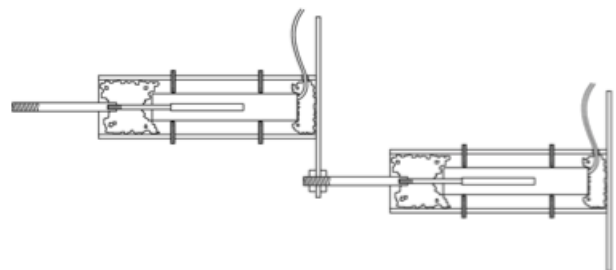


Figure 8. Pair of LVDT's housed in PVC tubes with Plexiglas bottoms.

### 6.1.2 Horizontal

Horizontal wall displacement was measured from behind (in the sand) using a series of electronic displacement devices that were daisy chained. As shown in figure 8, the linear voltage displacement transducers (LVDTs) were daisy-chained by threading the displacement needle of one to the back plate of the one in front. At the back of the testing chamber they were screwed into the wall through their anchor plates. The foremost instrument was screwed directly into the excavation wall. The housing (stuffed at both ends with cotton) were necessary to prevent sand from entering into the instruments. The instruments were kept centred within their housings by means of four setscrews per instrument. The vertical ones for soil measurement had the same detail, except the topmost instrument had a small Plexiglas plate to which it was affixed and then buried just below the soil surface. The instruments were placed according to a pre-specified layout at 5 depths within the chamber and across its width. On the excavation side of the wall, analog mechanical extensometers were installed at 5 levels. This was dictated by access constraints and the apertures in the concrete blocks (fig. 9). The details of these are showing in figure 10.



Fig. 9. Chamber exterior with extensometer measurement plates shown

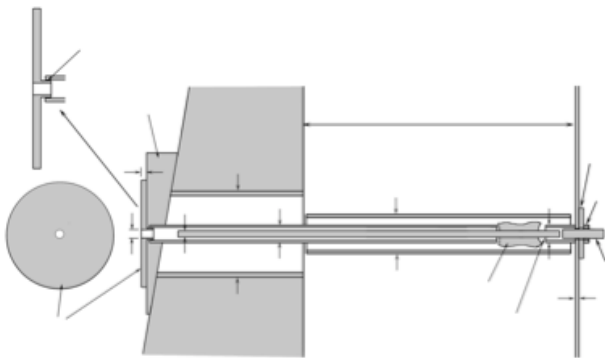


Fig. 10. Specialty devised mechanical extensometers. The chamber wall is shown shaded. The protective outer tube between the chamber and wall had a small flexible piece to prevent and reinforcing of the system.

### 6.1.3 Anchor loads

Anchor loads were determined both manually and electronically using a dual system of manual, caliper-determined deformations and of electronic, full-bridge strain gauges. The strain gauges were configured with two on the front of the plate and two on the back to determine the extent of strain during bending of the loaded plate. The manual readings were taken by measuring the relative deflection between a pin inserted along the central axis of the anchor plate and a similar pin mounted on an independent reference tool (fig.s 11-12).

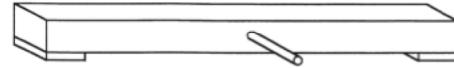


Figure 11. Measuring reference tool to determine anchor plate deformations.



Figure 12. Tool in fig. 11 shown during data collection

## 7 VALIDATION RESULTS

To show the effectiveness of the scaled anchors and excavation wall solutions and the related instrumentation devised to read them limited tests results will be presented herein. These tests were conducted in the chamber shown in fig. 1. The chamber holds approximately 40 tons of sand and is comprised of 4 levels of concrete blocks that use Dywidag bars and couplers are post-tensioned horizontally and vertically into a specialty foundation system to create a rigid reference frame. The sand is transported from a storage bin through a series of conveyors and into 1.36 metric ton concrete buckets, from which it is pluviated into the testing chamber from an overhead crane.

Sand from a local quarry was selected to match sand used with previous research by Mueller [17]. Categorized by the quarry as FA9, the clean, medium grained, uniform sand had mostly rounded to sub-rounded particles. It can be described as SP under the Unified Classification systems (see Appendix b). Based on cone penetration soundings and direct soil density measurements, the relative soil density was 47-57%. A medium dense deposit was used in the model tests to provide a material with shear strength, stress-strain, and volumetric characteristics consistent with a dense sand for the range of stresses normally encountered in engineering practices as previously determined in studies by Mueller [17]. Because of the low confining pressures in the test pit, the sand behaves more dilatantly than would be expected for its

physical characteristics at full scale. Consequently, the test sand is considered to emulate a medium dense to dense sand, despite the fact that the in-situ relative density would typically be categorized as a medium dense material, with an average total unit weight of only  $1,617 \text{ Kg/m}^3$ .

The range of experimental results for the soil surface settlement are depicted in fig. 12 as compared to the framework proposed by Peck [24] and further informed by the work of Hsieh and Ou [28]. These results were selected for comparison as equivalent displacement measurements for a sheet pile wall with tiedback anchors was not available. The results are as expected for sand excavations of average workmanship. Information is available for other tests in [29].

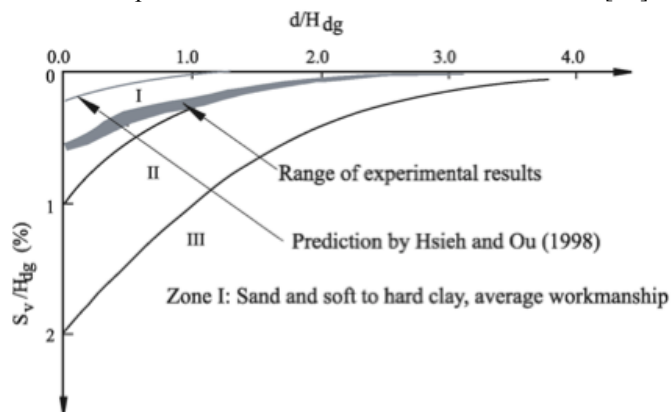


Figure 12. Experimental results plotted with respect to work reported by Peck [24] and Hsieh and Ou [28].

## 8 SUMMARY AND CONCLUSIONS

This paper outlines the fundamental of scaled solutions for tiedback anchors and a sheet pile excavation wall. Additionally the construction details and related measurement devices needed for their documentation are described. The resulting surface ground settlement verifies the general usability of this approach as the soil movement matches that which is expected for the sand used.

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