Braced excavation – Deformation and displacement of walls

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ABSTRACT

This General Report reviews the papers in Session 4 of the 3rd International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground. The intention is to provide a brief summary and broad overview of the papers. The reader is encouraged to refer to the original papers for further details.

1 INTRODUCTION

This 1999 Tokyo symposium is the third symposium on Geotechnical Aspects of Underground Construction in Soft Ground to be organised by Technical Committee TC28. The first was held in New Delhi in 1994 and the second in London in 1996.

The London Conference attracted twenty six papers on braced excavations, eleven from Japan and nine from the United Kingdom. Forty two papers on braced excavations have been submitted to this Tokyo symposium, half dealing with deformation and displacement of walls and the remainder with excavations in general. Not surprisingly, the vast majority of these are from the host country, and many are from in South East Asia.

In his General Report to the London conference, Kusakabe (1996) expressed surprise at the lack of contributions from North America and certain European countries. Once again, these countries and, for that matter, the entire southern hemisphere are conspicuously absent from this forum.

A further point of note is the dearth of papers dealing with excavations supported by soldier piles and anchors. A possible explanation for this is that such solutions are more suited to stiffer, often residual and partially saturated, soils which do not fit into the "soft ground" category.

This session report seeks to summarise and review the information given in the twenty one papers submitted to Session 4 on Deformation and Displacement of Walls.

When reading the papers to the session, most have a dominant theme which fits under one of four main headings, namely:

- Method of analysis
- Method of construction
- Predictions and observations
- Limitation of movement

In the remaining sections of this report, these themes will be explored in some detail.

2 ANALYSIS METHODS

Two methods of analysis emerge as clear favourites, namely the finite element method and subgrade reaction analyses. In the case of the finite element method, both two dimensional and three dimensional modelling was carried out, generally making use of non-linear elasto-plastic constitutive models.

The subgrade reaction analyses include a number of approaches ranging from beam-on-elastic-foundation, beam-spring and even frame analyses. Most allow for non-linear subgrade
reactions and for cracking, and in some cases even yield, of the wall.

The paper by Onishi and Sugawara describes an 18m deep diaphragm wall excavation using top-down methods. A linear elastic subgrade reaction was assumed between the active and passive limits beyond which a perfectly plastic response was assumed.

The first stage of the excavation to a depth of 4m produced deformations three to four times higher than initially predicted. This prompted the authors to re-evaluate the properties of the soils and of the wall itself. The stiffness and shear strength of the soils in front of the diaphragm wall were reduced by 50% to take account of the disturbance of the ground resulting from pre-excavation works including the installation of piles and drains. In addition, the wall stiffness parameter was replaced with a tri-linear relationship between moment and curvature to take account of cracking and yield.

The revised predictions agreed well with the observed performance of the excavation during the early stages and accurately predicted its future performance.

Many of the excavations involved in the construction of the high speed railway line from Berlin to Hamburg in Germany are single propped diaphragm walls with sufficient embedment depth to permit excavation to final level without further propping. No de-watering takes place until the concrete base slab has been cast after which the inside of the excavation is pumped dry.

Siemer et al report on a standard design method developed by the Projektgesellschaft für Verkehrsanlagen im Zentralen Bereich Berlin GmbH known as the GBOB method. The method deals with the two stages of construction described above, namely (a) excavation under water and (b) de-watering of the excavation after casting of the base slab. Two approaches are used for each phase of the excavation. The first approach assumes point bearing at strut level and at the resultant position of the passive force. The second approach allows for “bedding” of the wall into the soils in the passive zone.

The authors illustrate the use of the method with reference to a 15.2m deep diaphragm wall at Lehrter Station. During the first excavation phase, the method over-predicted displacements near the top of the wall but under-predicted the movement of the embedded length into the passive zone. The over-prediction was ascribed to the higher stiffness of the upper sections of the wall where the bending moments were insufficient to cause cracking. The under-prediction at the base was attributed to disturbance of the soils in the passive zone by the installation of Gewi piles to resist uplift water pressures on the completed structure.

The need to take account of these two effects is in line with the experience of Onishi and Sugawara.

Kazama applied subgrade reaction theory to the analysis of braced excavations subjected to asymmetrical lateral loads. Such asymmetry can arise due to different ground conditions on either side of the excavation, surcharges, sloping ground or even differential water pressures.

In his analysis, the subgrade reaction of the soil is based on the results of cyclical lateral load tests on piles. The analysis takes account of the axial stiffness of the struts and the bending stiffness of the soldier piles in either the cracked or uncracked condition. A number of case histories are presented which illustrate the use of the method.

![Analytical model for butressed diaphragm wall](image)
A variation on the subgrade reaction method is presented by Tsuzuki et al who consider the behaviour of a 5.5m deep cantilever sheet pile wall in which the passive resistance in front of the wall is enhanced by deep mixing stabilisation. The authors indicate a preference for the stabilisation of counterfort buttresses rather than of the entire passive block as this is said to be less demanding in terms of bearing pressure.

The behaviour of the wall was modelled using a frame analysis in which the passive resistance acting against the buttress or on the embedded section of the wall below the buttress, the shear resistance on the sides of the buttresses and the bearing resistance below the buttresses were all modelled as springs (Figure 1). Active earth pressure was assumed behind the wall.

For the soil parameters chosen, the method yielded acceptable results.

Turning now to finite element methods, Higgins et al present a case study in which alternative designs for underpasses on the A406 north circular road in London were assessed using the Imperial College finite element programme (ICFEP). The aim was to examine the sensitivity of the design to various inferences including upper- and lower-bound K_0 profiles, the effect of rising ground water, permeability of the wall and wall installation methods. In view of the time restrictions and reliability of available geotechnical data, a simple (linear elastic, perfectly plastic) constitutive soil model was used.

The major influencing factor on the design was found to be the possible rise in the ground water table on cessation of pumping from the chalk which underlies the London clay. The assumed K_0 values for the soil and long term stiffness of the concrete influenced the predicted deflections of the upper part of the wall but had little effect on the bending moments and forces within the structure.

This case history demonstrates the versatility of modern finite element coding.

Matsumoto et al present an intriguing case history of a 12m deep braced excavation in soft Bangkok clay using a secant pile wall propped off the partially completed raft foundation (Figure 2). The first 2.5m deep phase of the excavation resulted in less than 10mm of inward movement initially but increased with time to a maximum of 60mm over ten weeks. This forced the designers to re-evaluate the support system.

Employing a soil-water coupling finite element programme called DACSAR and an elasto / visco-plastic constitutive model, the authors were able to simulate the observed creep of the wall (Figure 3). Applying the method to the completed excavation indicated movements as large as 150mm for the original support proposals. The designers elected to include a horizontal strut at the top of the wall which succeeded in limiting deformations to an acceptable 100mm.

The reasons advanced for the excessive movements during the first phase of the excavation were disturbance of the soil by the installation of large diameter foundation piles and the softening effect of ponding water.

In contrast to the successful finite element modelling of creep movements reported by Matsumoto, Hou et al contend that continuum finite element methods have not developed to the degree where they can accurately model time dependent movement of the soil due to the lack of adequate constitutive models. In their opinion, all that is known is that a myriad of soil properties effect the magnitude and distribution of ground movements including anisotropy and non-linear stiffness.

As an alternative, the authors propose a concept known as the “Time-Space Effect”. The time effect takes account of the creep properties of the soil and any delay in the installation of support.

![Figure 2. Section through propped secant pile wall in Bangkok. (Matsumoto et al)](image)

![Figure 3. Finite element modelling of creep after first stage of excavation. (Matsumoto et al)](image)
The space effect deals with the influence of size and shape of the exposed face on excavation movement.

Whether or not the method will replace conventional finite element analyses remains to be seen.

Tan et al apply modern finite element modelling to the movement of ground during construction of diaphragm walls, a problem which attracted a lot of attention in the 1970's. The analysis was carried out using the CRISP finite element coding coupled with the Biot theory of consolidation.

The authors compare the results obtained from the 3-D finite element modelling with those from an axi-symmetric analysis and conclude that the axi-symmetric analysis under-predicts the deformations near the centre of the diaphragm wall panel.

The method was then applied to a well documented case history of slurry trenching from the early 1970's. Acceptable agreement was obtained between observed and predicted performance.

3 CONSTRUCTION METHODS

As indicated in the introduction, the majority of the case studies in this session deal with strutted walls. Approximately half the walls described are diaphragm walls, the remainder being secant piles, sheet piles or DSM (deep soil mixing) walls. This section deals with those excavations which have particularly interesting aspects to the construction methods used.

Uchiyama et al present three case histories in which soil improvement was carried out in the passive zone at the toe of multi-propped diaphragm walls using deep soil mixing to form buttresses. In general, the buttresses were approximately 10m long (at right angles to the wall) and were spaced at approximately 6m centres. The depth of soil improvement in all three cases was of the order of 9m, occasionally extending below the depth of the perimeter wall. All three excavations were formed in very soft clays over significant portions of the excavation depth and all were supported with multiple struts.

Monitoring of the two shallower excavations showed the deformation during the initial stages of excavation to be larger in the case of the buttressed walls than was recorded in other areas of the site where no buttresses were installed.

Although not mentioned by the authors, this is probably due to the disturbance of soil in front of the wall during the deep soil mixing process. Nevertheless, comparatively smaller movements were recorded for the completed buttressed walls.

On the third site, the method succeeded in limiting the deformation of an 18.4m deep excavation in these very soft clays to between 50mm and 80mm.

Inclinometer readings show the buttressed soil behaves as a rigid mass. Negligible curvature of the wall occurs over the stabilised depth even midway between buttress positions.

Tsuzuki et al used a similar technique for a 5.5m deep pit below a power station boiler house in very soft clays with SPT values of 0 - 1 (see Figure 1 and analysis method described above). This cantilever retaining system was constructed using 14m deep sheet piles buttressed to a depth of 10m. The bulk of the 140mm recorded movement of the wall took place during installation of foundation piles from the -4m level.

Katzenbach et al present a fascinating case study of a 23m deep excavation for a 200m high tower block in the highly developed banking district of Frankfurt (Germany). The use of tied back soldier pile walls was rejected on account of the large movements which have occurred with the use of such systems in the past.

In this case, 1.5m diameter secant pile walls were installed into the Frankfurt clays to a depth of 34m. Bearing piles for the pile raft foundation

![Figure 4: Plan of Main Tower excavation in Frankfurt. (Katzenbach et al)]
system were constructed prior to commencement of excavation, backfilling the un-concreted section of the pile hole with soil.

Following de-watering of the soils within the secant pile wall, an initial pit was excavated to final level below the core of the proposed tower block. This pit was supported using timbering and a steel bracing structure.

After construction of the core back up to ground level, the top of the perimeter wall was propped from this structure. The main excavation then followed conventional top-down methods proceeding simultaneously with the above-ground construction of the tower block. By the time the excavation had reached the design depth of 21m, the tower block structure was up to the 12th floor.

The presence of piles below the raft foundation and the weight of the rising tower block, all but eliminated heave of the excavation base. The final settlement of the completed tower block was less than 30mm. The maximum inward movement of the perimeter walls was 23mm, the bulk of which occurred during the sinking of the initial pit. Good agreement was found between the observed movements and those predicted using 2-D and 3-D non-linear elasto-plastic finite element analyses.

The National Theatre in central Oslo (Norway) was originally founded on timber piles. In the early 1970’s, these were replaced with steel piles extending to bedrock. Shortly thereafter, the construction of the railway station led to horizontal movements of between 40mm and 60mm which detrimentally affected the founding of the structure.

Bye et al describe the widening of the existing railway tunnel to within a few meters of the theatre foundations. This involved the driving of steel sheet piles to bedrock. Wherever possible, jet grouting was carried out below proposed excavation level to create a “pre-formed” strut between the sheet piles and the existing diaphragm wall. (Figure 4). In areas where jet grouting could not be undertaken, conventional pre-stressed struts were installed as excavation progressed.

The sections of the wall over which jet grouting was used did not perform as well as expected. This was attributed to the lack of intimate contact between the jet grouting and the diaphragm wall of the existing station. Most of the movement (about 35mm) occurred during the first 7m stage of excavation. This appears to have been sufficient to establish contact and mobilise the full resistance of the jet grouting. Movements during subsequent stages of excavation were nominal.

No horizontal movement was recorded on the theatre building and measured settlements were less than 10mm.

Although not mentioned by the authors, disturbance of the ground above excavation level during the jet grouting above excavation level contributed to the high initial movements of the sheet pile wall. Nevertheless, the principle of minimising the movement by creating struts below excavation level is sound.

Two case histories in this session deal with buttressed diaphragm walls. Sei and Miyazaki describe the performance of a 12m deep excavation supported by a cantilevered diaphragm wall with strengthening buttresses. The buttresses, which were constructed on the inner face of the diaphragm wall, were 7,5m wide at 6m spacings. They extended to a dense gravel founding stratum at a depth of 35m providing bearing for the final structure.

In the analysis of the structure, the authors compared the results from 3-D finite element modelling with those from a beam-spring analysis. The finite element model correctly predicted the deformation but under-estimated the bending moments within the embedded zone. The beam-spring analysis was affected by the assumed fixity at the base of the wall. The best fit appeared to be

![Figure 5: Subway widening adjacent to existing building. (Bye et al)](image)
achieved by assuming the fixity to lie between the free-base and pinned-base situations.

To cater for this, the authors developed a new analysis model in which the wall was treated as a rigid unit and the subgrade reaction in both vertical and horizontal directions was replaced by a combination of linear and rotational springs. The stiffness of these springs was determined using finite element modelling. Good agreement was obtained between observations and predictions.

The second case history of a buttressed wall is reported by Poh et al and deals with the construction of an 11m deep cultural and art centre in Singapore. The excavation had a semi-circular shape, 180m long and 110m wide.

The support system consisted of a conventional diaphragm wall around the perimeter of the site extending to a dense alluvial layer at depths of up to 45m. A sheet piled wall was then driven along a line parallel to the diaphragm wall, 18m inside the excavation. After the installation of load bearing piles, the space between the two retaining walls was excavated and supported using struts. Once the excavation between the two walls had reached its design depth of 11m, the base slab was cast and the buttresses were constructed to strengthen the upper portion of the diaphragm wall and render it free-standing.

The maximum recorded movement was of the order of 50mm, the bulk of which occurred during excavation to founding level between the two walls. Minimal additional movement occurred during stripping of the props. These movements were considerably smaller than those predicted using finite element methods. The difference between observed and predicted movements was attributed by the authors to the arching effect of the semi-circular excavation.

Azuma et al describe one way of disposing the vast quantity of coal ash produced by Japan’s thermal power stations. By a process of deep soil mixing, they stabilised some 60 000 cubic metres of soil on either side of a 12m deep excavation for a cooling water channel below a power station. A blend of fly ash, gypsum and cement was used in the stabilisation process applied at a rate of between 150kg and 225 kg per cubic metre at a water/stabiliser ratio of 1:0. This stabilised material formed 15m wide gravity retaining walls extending to bedrock at depths of between 20m and 25m. Excavation of the soil between the gravity retaining walls resulted in maximum displacements of 80mm.

The braced excavation of Matsumoto et al described in the previous section deserves further mention under the heading of construction methods. The provision of horizontal struts across a 100m wide excavation is no mean feat!

4 MOVEMENT PREDICTIONS AND THE OBSERVATIONAL METHOD

Numerous authors refer to their use of the observational method. Onishi and Sugawara used it to justify their omission of a temporary strut above final excavation level. Matsumoto et al recognised the need for additional horizontal bracing in the Bangkok excavation. The observational method was used by Katzenbach et al to control the movements of surrounding high rise buildings in their excavation in Frankfurt.

The paper by Hanakura et al is an excellent example of the application of the observational method. It describes the performance of 4.5m diameter cable duct at depth of 19m in close proximity to a 16m deep cut and cover transportation tunnel in central Hiroshima (Figure 6).

The tunnel excavation was supported using 600mm diameter tube piles with steel cross bracing. Chemical grout was injected to seal the excavation against water inflow from an underlying confined aquifer. Prior to commencement of excavation, a 2-D finite
element analysis was carried out to estimate the likely displacement of the cable duct and the stresses which would be developed in the duct’s primary and secondary linings. Based on this analysis, three levels of “control standard values” were set for settlement, horizontal displacement, rotation and cavity displacement (convergence) of the duct section. Actions to be instituted when each of these control standard values were exceeded were spelt out prior to commencement of work. The limiting (highest) control standard value in each case was based on the maximum allowable tensile stresses in the concrete of the secondary lining or 50% of the maximum allowable tensile stress in the primary lining.

Performance of the duct was monitored continuously during execution of the work using settlement gauges, inclinometers, displacement gauges and transit measurements. The limiting control standard value was exceeded only in the case of inclination of the cable duct prompting crack injection grouting of the secondary lining and protection of the cables.

This case history is a textbook example of the observational method.

The design of deep excavations is often based on limited information and subjective selection of design parameters. The observational method is then used to assess the performance of the excavation during the early stages of construction and to revise the design parameters accordingly.

Chi et al describe the application of numerical optimisation techniques for the deriving of “best fit” soil parameters from back analyses. The method makes use of the standard computer codes for non-linear function minimisation introducing the finite element analysis of stresses and deformations as a sub-routine. These “best fit” parameters reflect all the factors that influenced the performance of the excavation including soil parameters, construction methods, etc. The analysis can either be based on all previous stages of the excavation or only on the most recently completed stage.

The authors illustrate the method by applying it to the 20m deep excavation for the Taipei National Enterprise centre and produce credible results.

A different form of “observational method” is presented by Allersma and Toyosawa. Their paper describes the use of a small geotechnical centrifuge to simulate the collapse of an 11m deep sheet pile excavation for a bridge pier. The excavation was designed to be supported with three rows of struts of which only two were installed. The behaviour of the excavation was modelled in a 2.5m diameter centrifuge at accelerations ranging from 80 to 150g. Deformations were monitored by image processing using a video camera equipped with a “frame grabber” facility. By subtracting the images from two different stages, deformations become clearly visible. Five model configurations were tested with struts being removed and additional loads applied “in flight” to simulate the actual course of events.

The tests show that both sets of struts must have buckled simultaneously to produce the form of failure observed in the field. The authors conclude that the model centrifuge is a useful forensic tool for understanding mechanisms of failure and apportioning liability.

A fundamental requirement of the observational method is a Class A (before the event) prediction against which the field observations can be adjudicated. With few exceptions, Class A predictions were made for all the excavations described in this session. Kort et al describe a field trial designed to test the analytical and prediction skills of practising engineers and academics. They hope to obtain as many as 50 Class A predictions of the performance of two trial sheet pile walls constructed in the Rotterdam harbour area.

The test set up consists of two facing, single-strutted walls. The north wall is constructed using Arbed AZ13 sheet piles with an unusually low yield stress to encourage the formation of plastic hinges both above and below the dredge level.

The opposing wall is formed from pairs of double U-piles with the interlocks between each pair of piles welded prior to driving. These interlocks lie on the centreline of the wall, this gives rise to the possibility of rotation of the neutral axis causing oblique blending of the piles (Figure 7). The Netherlands is one of the few countries which has design rules to cater for such an eventuality.

![Figure 7. Oblique bending of sheet piles. (Kort et al)](image_url)
The paper describes the many ingenuous measures built into the test set-up to eliminate end effects in the sheet pile wall, bracing system and retained ground. It describes the instrumentation installed to monitor strut loads, in- and out-of-plane deformations and earth pressures. A carefully planned sequence of excavation and de-watering has also been specified.

It is hoped that the test will make geotechnical engineers more aware of the effect of the formation of plastic hinges and oblique bending in sheet piles in addition to generating a great deal of valuable discussion. The progress of the test can be monitored by reference to the authors’ web page on the Internet.

5 CONTROL OF DEFORMATIONS

Figure 8 shows the range of deformations expressed in terms of a percentage of the depth of excavation. With one exception, the deformations reported in Session 4 range from 0.03% to 0.73% of excavation depth averaging 0.35%.

Three excavations were particularly successful in controlling deformations, two of them involving the construction of a circular “cofferdam” type excavation.

Powderham describes the construction of a 30m deep, 60m diameter excavation in disturbed ground following the collapse of the station tunnels at Heathrow Airport during construction in October 1994. The circular cofferdam was constructed using 182 large diameter, stepped, secant / contiguous piles. A further 255 large diameter bored piles were provided to a depth of 15m below the base slab to limit heave of the underlying London clay and control uplift due to water pressures.

The predicted maximum displacement of the contiguous pile wall under most adverse conditions was 75mm. The observational method was an integral part of the planning of this excavation. A comprehensive monitoring system was implemented and contingency measures were laid down in the event of adverse performance. These included thickening of the reinforced concrete lining and limiting the removal of soil to the perimeter of the excavation only.

A comprehensive review of the performance of the excavation was scheduled to take place at a depth of 7m. The favourable performance of the excavation at this depth enabled the excavation lifts to be increased from 1.0m to 1.2m and early tunnel breakthroughs to be achieved. The maximum observed deformation of the cofferdam was 25mm which occurred 5m above the base of the excavation.

A similar approach, but on a larger scale, was adopted by Ariizumi et al. Their paper describes a 29m deep, 144m diameter sub-station excavation in Tokyo. The lateral support consisted of a 2.4m thick circular diaphragm wall extending to a depth of 44m followed by a 1.2m thick cut off key to a depth of 70m. No internal bracing was provided.

Over 1,100 monitoring stations were used to assess the performance of the retaining wall. In addition to monitoring deflection of the wall, the reinforcement stresses and temperatures were monitored on both the inside and the outside faces at various points. The displacement of the wall

Figure 8. Observed movements of excavations

Figure 9. Section through 144m diameter substation excavation. (Ariizumi et al)
was found to be more dependent on temperature than on earth pressure. The maximum recorded displacement amounted to a mere 17mm or 0.06% of the total excavation depth. For a clear (unstrutted) excavation of this magnitude, this is truly a remarkable achievement.

The authors note that the initial proposals for a rectangular excavation were rejected on account of the cost and time required for construction. Undoubtedly, deformations of a rectangular excavation would have been considerably larger. Surely, we ought to be influencing our city planners to move away from rectangular city blocks to round ones which could serve as giant traffic circles!

The final example of a deep excavation which performed well is the case history presented by Okada et al. They describe a 38m deep braced excavation for the Higashi-Nakano Station on the metropolitan subway in Tokyo. The support comprised a 1.2m thick diaphragm wall to a depth of 50m. Conventional strutted excavation took place over the upper 23m followed by the casting of a 2.3m thick middle slab (Figure 10). The underlying narrower section of the excavation to a depth of approximately 38m was supported with a secant pile wall, strutted as excavation progressed. The movement record of this excavation is particularly interesting. From the outset, movements were smaller than predicted using elasto-plastic methods in which the struts and the soil were modelled as a series of springs. Initially, the top of the wall moved into the excavation as expected. However, at approximately half depth, the top of the wall appeared to move back into the supported ground. The authors attempted to explain this by examining the behaviour of the soil between the two walls below the middle slab. Despite the anomalous behaviour, it is evident that the deformations were particularly small with a maximum movement of 13mm being observed on the diaphragm wall and 10mm on the secant pile wall.

The observed behaviour of the top of the wall during the latter stages of excavation is inconsistent with the positive strut loads recorded throughout the excavation process. It is suspected that this may be due to unrecorded movement of the toe of the inclinometer tubes which extended to a depth of 50m.

6 CONCLUDING REMARKS

In soils as soft as those described in many of the papers in this session, the deformation of an excavation is critically dependent on the stiffness of the installed support and of the soils in the passive zone in front of the retaining wall. This is in contrast to the performance of the stiff, partially saturated, residual soils of the author's home country where flexible support systems (slender soldier piles or thin gunite facings) are used in conjunction with anchors or soil nails.

A number of the papers presented at this conference describe methods used to improve the stiffness of the support. Jet grouting and deep soil mixing were used by a number of authors to improve the stiffness of the passive zone. The stiffness of the wall and the supporting structure was increased by the use of buttresses, top down construction or the adoption of a circular shaped excavation.

In a number of instances, the performance of the excavations was adversely affected by the disturbance of the soil in the passive zone. Ironically, some of this disturbance was associated with measures used to stabilise the ground near the toe of the wall including jet grouting, installation of drains and deep soil mixing. The installation of foundation piles also appears to have caused significant movement in certain cases. This is either due to the loosely backfilled, "open bore" of piles installed from ground level or due to disturbance of soft clays in the case of piles installed from final excavation level.
A further effect which led to a reduction in stiffness of many structures was cracking of the concrete. The adoption of uncracked sections properties for the concrete led to under-prediction of displacements in some instances. On the other hand, where cracking was taken into account from the outset, it was occasionally necessary to increase the assumed stiffness of the wall over the uncracked section of the pile to match the observed deflections.

On face value, it would appear that designers are making equal use of finite element methods and subgrade reaction methods to estimate wall displacements. Both methods appear to be producing acceptable results. The observational method remains an important part of excavation design as it should.

The authors are congratulated on the high standard of the papers presented in this session.

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