THREE-DIMENSIONAL ANALYSIS OF STRUT FAILURE FOR BRACED EXCAVATIONS IN CLAY

A T C Goh[1] and K S Wong[2]

ABSTRACT: In Singapore, cut and cover tunnels for expressway projects are commonly constructed using braced or anchored wall systems. One concern in these excavation projects is the consequence of the failure of one or two struts (due to accidental damage) in the bracing system and whether it would lead to progressive failure and eventual total collapse of the braced excavation. The focus of this paper is on the transfer of the load(s) when one or two supporting struts fail. In this paper, three-dimensional finite element analyses were carried out to assess the impact of the failure of one or two struts on the remaining struts. The numerical results show that the increase in the wall and soil movements was small for the failure of a single strut. The majority of the load from the failed strut was redistributed to the struts immediately adjacent to the failed strut. The maximum redistribution of load to the adjacent strut expressed as a percentage of the initial load of the failed strut was 25% for the failure of a single strut. There were only minimal changes in the magnitude of the maximum bending moment in the retaining wall as a result of the failure of one or two struts. The finite element study suggests that the failure of one or two struts due to an accident would not result in detrimental failure of the entire excavation system, provided the struts have been adequately designed against compression failure.

Keywords: Braced excavation; failure; finite element analysis; retaining wall; strut.

INTRODUCTION

In Singapore, cut and cover tunnels for expressway projects are commonly constructed using braced or anchored wall systems (Wong et al. 1997). The sides of the excavation are normally supported by soldier piles, sheet piles or concrete diaphragm walls with two or more levels of struts/tiebacks. The purpose of the excavation support system is to provide lateral support for the soil surrounding the excavation and to limit movement of the surrounding soil. The method presented by Peck (1969) for specifying apparent lateral earth pressures forms the basis of the design of the excavation support system and the determination of the loads on the bracing system. The limitations of Peck's method have been highlighted by Liao and Neff (1990), and various enhancements to the method have been proposed by Liao and Neff (1990), Chang and Wong (1997), and Twine and Roscoe (1997). The mechanisms controlling the development of lateral earth pressures around a braced excavation in a deep deposit of soft clay has been presented by Hashash and Whittle (2002).

One concern in these cut and cover excavation projects is the consequence of the failure of one or two struts (due to accidental damage) in the bracing system and whether it would lead to progressive failure and eventual total collapse of the braced excavation. The focus of this paper is on the transfer of the load(s) when one or two of the supporting struts fail. This may arise for example as a result of some construction activity accidentally damaging one or two of the struts.

To date, only limited studies have been carried out to examine this aspect in retaining walls supported by multi-levels of anchors/struts. One widely reported field study was carried out at four sites in Sweden and are described in Stille (1976), and Stille and Broms (1976).

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At all four sites, the anchors were inclined at 45°. "Failure" of an anchor was simulated by releasing the load in one of the anchors and measuring the resulting changes in the loads in the adjacent anchors. Most of the wall panels consisted of two or three levels (rows) of anchors. Figure 1 shows the cross-section of wall panel C of one of the sites at Manfred, Sweden. The sheet pile wall was anchored at two levels with anchor rods inclined at 45° and spaced 2.6 m apart. The upper and lower level anchors were pre-stressed to 200kN and 250kN, respectively. The depth of the excavation was approximately 6.3m. Underlying 2m of fill was a 12m thick clay stratum with an average undrained shear strength of about 25kPa. The clay was underlain by fine sand and silt. The depth to bedrock was about 25m.

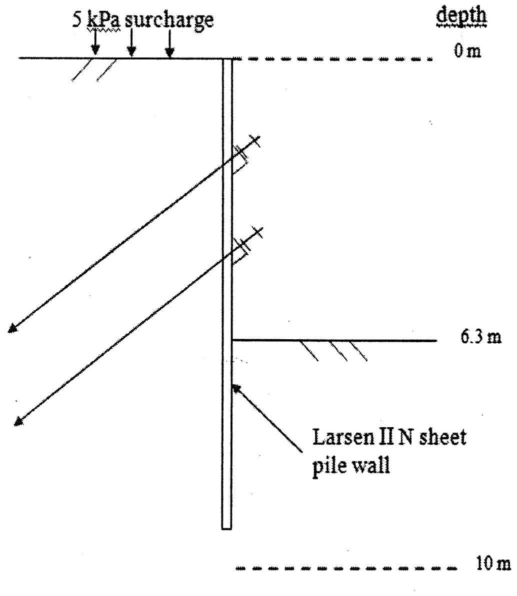

Fig. 1 Cross section of wall panel at Manfred (modified from Stille 1976)

Figure 2 shows the field measurements as a result of unloading of an anchor in the lower row. The results show the redistribution of the anchor load expressed as a percent of the initial load of the failed anchor (i.e., load at the end of excavation to formation level). Figure. 3 shows the field results when a second anchor was unloaded in the upper row to simulate the failure of two anchors. The redistribution was expressed as a percent of the load carried by the upper anchor prior to failure. Interestingly, the total load redistributed onto the anchors is less than the anchor load prior to failure, presumably because of the movement of the wall.

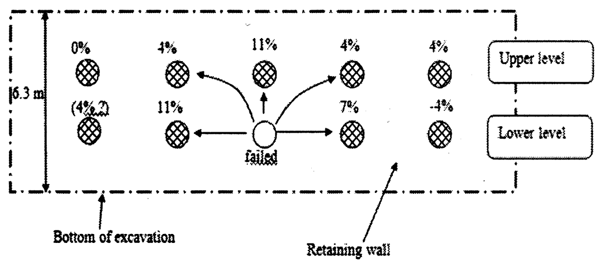

Fig. 2 Measured redistribution of anchor load from failure of a single anchor (modified from Stille 1976)

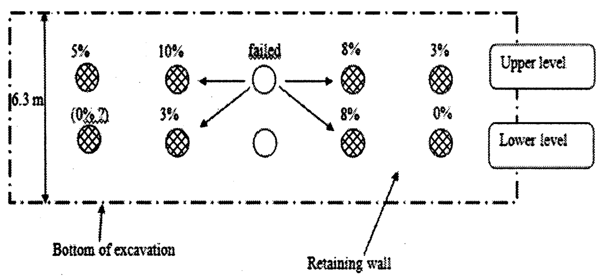

Fig. 3. Measured redistribution of anchor load from failure of a second anchor (modified from Stille 1976)

Stille (1976) reported that at the four sites, the maximum change of the anchor load expressed as a percent of the initial load of the failed anchor was 35% for walls with two or more rows of anchors. The field study indicated that the increase in the loads of the adjacent anchors was of the same order of magnitude in the horizontal and vertical directions. As the wall movements were small, it was concluded that the transfer of load to the adjacent anchors was not through the arching of the soil behind the wall, but through the wall and wale members.

However, since struts provide passive resistance to movement, while anchors rely on stresses in the ground being mobilized to retain the wall, it may not be realistic to compare the redistribution of forces for both types of retaining wall systems. This paper focuses on the effects of removal of one or two struts. The removal/failure of a strut and the resulting stress transfer cannot be modelled in plane strain. This paper describes the use of a three-dimensional finite element analysis to assess the impact of the failure of one or two of struts on the remaining struts.

While a number of three-dimensional studies have been carried out to examine the wall movements associated with braced excavations (Ou et al. 1993), the effect of diaphragm wall installation (Ng and Yan 1998; Gourvenec and Powrie 1999), and the impact on wall movements of excavating a long berm section by section (Gourvenec and Powrie 2000), this is one of the first numerical studies to investigate the impact of the failure of one or more struts.

THREE-DIMENSIONAL FINITE ELEMENT ANALYSIS

The basic geometry of the wall and the layout of the struts considered in this paper are shown in Figs. 4 and 5. The horizontal strut spacing is 4m. For clarity, the wale beams supporting the struts have not been shown in the figures. The wall is assumed to be wished-in-place in a uniform deposit of clay without change to the in-situ soil stresses. The three-dimensional finite element code PLAXIS 3D tunnel (Brinkgreve and Vermeer, 2001) was used to perform a series of analyses. A typical finite element mesh is shown in Fig. 6. The mesh consisted of 2954 elements and 9486 nodes and was discretized into fourteen 2m long "panels", with each panel having the same configuration along the length of the esh. All the nodes on the vertical mesh boundaries were on rollers, preventing movement normal to the boundary. The nodes at the bottom mesh boundary were pinned to prevent movement in any direction. The left hand boundary was positioned to represent a plane of symmetry along the centreline of the excavation. The right hand boundary was positioned far enough away that it would not influence the wall behaviour. The front and back faces (boundaries) of the mesh represent planes of symmetry for a long section of wall and are sufficiently remote not to affect the behaviour of the central panels.

The soil was modelled by quadratic fifteen-node wedge elements, and the wall and wale beams by eight-node plate elements. The soil was assumed to be linearly elastic-perfectly plastic with a Mohr-Coulomb (Tresca) yield surface. The saturated undrained properties of the soil (typical of the soft marine clays in Singapore) were: elastic modulus E = 7500 kPa, saturated unit weight = 16 kN/m^3, undrained shear strength c_u = 25 kPa, Poisson's ratio = 0.49, and coefficient of at-rest earth pressure = 1.0. One limitation of the use of the Mohr-Coulomb model is the inability to capture the small strain nonlinearity below the state boundary surface (e.g., Jardine et al. 1986). However, the small strain nonlinearity below the state boundary surface are likely to be less important for normally consolidated and lightly overconsolidated clays such as the Singapore marine clay than they are for heavily overconsolidated clays (Chua 1990).

The wall and wale beams were assumed to be linear elastic. For simplicity, it has been assumed that the flexural stiffness of the wall is isotropic and that the bending moments can be transmitted laterally because of the flexural rigidity of the wale beams. Linear elastic spring elements were used to model the struts. The full shear strength of the clay can be mobilised along the soil-wall interface and was modelled by sixteen-node interface elements. The analyses were carried out for a FSPIIIA sheet pile wall with bending stiffness EI = 46512 kNm^2/m. The strut axial stiffness and wale bending stiffness properties were EA=6x10^{5}kN/m and 2.162x10^4 kNm^2/m, respectively.

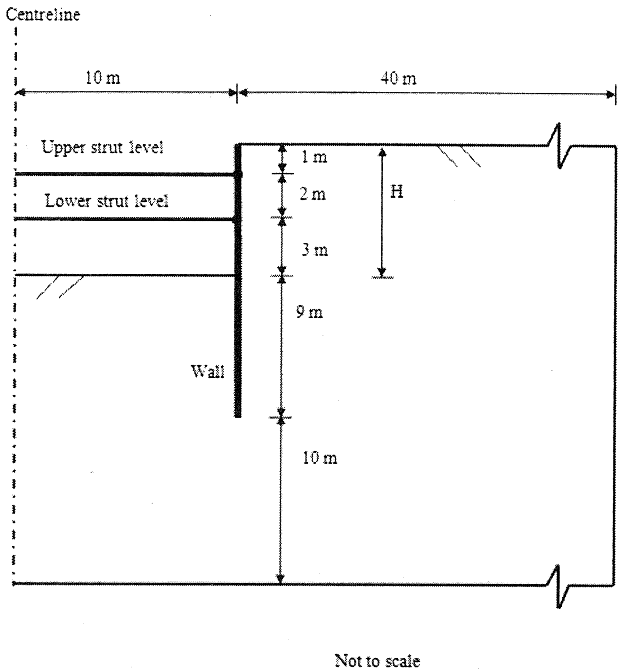

Fig. 4 Cross section of the excavation

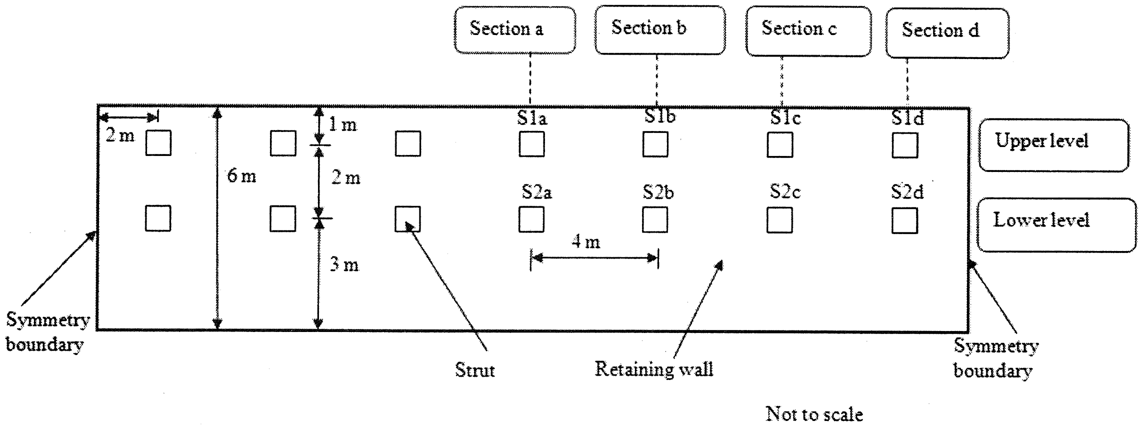

Fig. 5 View of the wall panel and strut locations

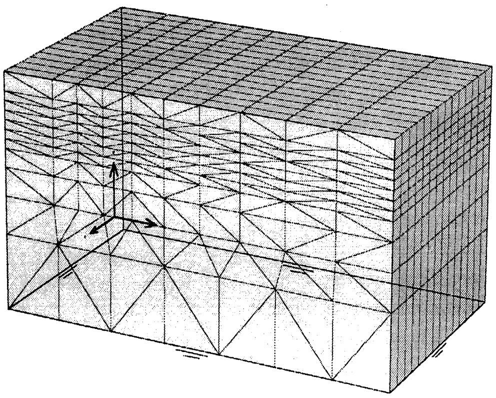

Fig. 6 Three-dimensional finite element mesh

The staged construction sequence simulated in each finite element analysis was as follows:

a) Excavate 1m of soil (i.e., formation level is 1 m below the original ground surface).
b) Excavate 1m of soil (i.e., formation level 2m below the original ground surface) and install upper level of struts at depth of 1m below the original ground surface.
c) Excavate 1m of soil (i.e., formation level 3m below the original ground surface).
d) Excavate 1m of soil (i.e., formation level is 4m below the original ground surface) and install lower level of struts at depth of 3m below the original ground surface.
e) Excavate 1m of soil (i.e., formation level 5m below the original ground surface).
f) Excavate 1m of soil (i.e., formation level is 6m below the original ground surface).
g) Removal (failure) of one strut (either S2a or S1a as shown in Fig. 5) by de-activating the previously installed strut.

RESULTS OF NUMERICAL STUDY

Throughout this paper, the redistribution of the strut load is expressed in the same manner as that of Stille (1976), i.e. as a percent of the load carried by the strut prior to "failure". The redistribution of the load as a result of the failure (removal) of the lower level strut S2a is shown in Fig. 7. The load on the strut (prior to removal) was 759 kN at the end of excavation. The majority of the load from the failed strut was redistributed to the struts immediately adjacent to the failed strut and ranged from 9-19%. The redistribution in the loads to the adjacent struts was of the same order of magnitude in the horizontal and vertical directions. For the struts located further away (Section d in Fig. 5), the changes have not been shown as there were minimal changes in the loads acting on these struts.

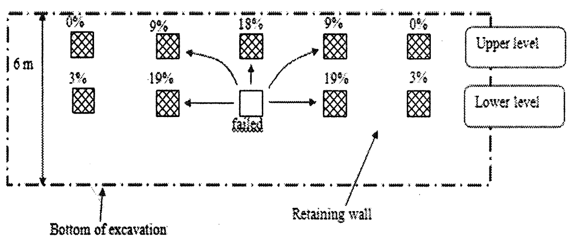

Fig. 7 Load redistribution after failure of lower level strut

Figure 8 compares the increase in lateral wall movements Δu_x at Section a and Section b (as defined in Fig. 5) as a result of the failure of the lower strut at *Section a*. The wall movements were small, with the maximum deflection of 5.2mm occurring close to the level of the failed strut. As expected, the lateral wall movement was smaller at *Section b* than at *Section a*, due to the lower level restraint at strut S2b. The increases in lateral wall movements Δu_x at the same horizontal elevation for the upper (wall depth of 1m) and lower (wall depth of 3m) struts are shown in Fig. 9. They again indicate that the increases in lateral wall movements as a result of the failure of a single strut were small (less than 5.2mm). For brevity, the increase in lateral and vertical movements for the soil on the retained side of the excavation has not been shown. The magnitude of these soil movements was slightly smaller than the wall movements.

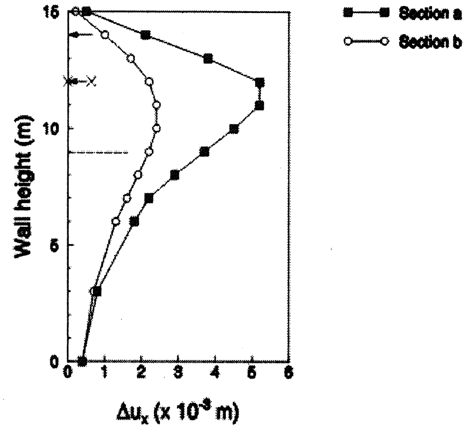

Fig. 8 Increase in lateral wall displacements after failure of lower level strut

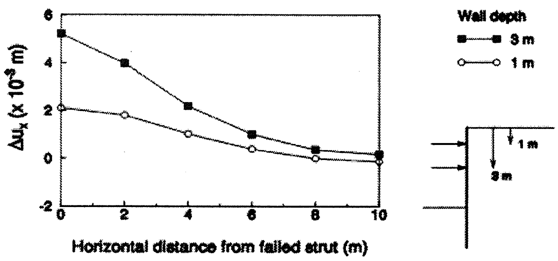

Fig. 9 Increase in lateral wall displacements at two elevations

Figure 10 shows the bending moments in the wall at the end of excavation (i.e., prior to the strut failure). The bending moments at Section a and Section b are the same, prior to strut failure. Also shown in Fig. 10 are the bending moments at Section a and Section b after the failure of the lower strut. The changes in the bending moments in *Section b* were minimal and reflect the small change in the lateral wall deflection/curvature at Section b. Most of the changes in the bending moments at Section a occurred in the vicinity of the two strut levels. With the increase in load on the upper strut, there was an increase in the magnitude of the bending moment at that position. At the position of the failed lower strut there was a reversal of the bending moment from positive to negative. The maximum wall bending moments occurred close to the formation level of the excavation. There were only minimal increases in the magnitude of the maximum bending moment as a result of the failure of the strut.

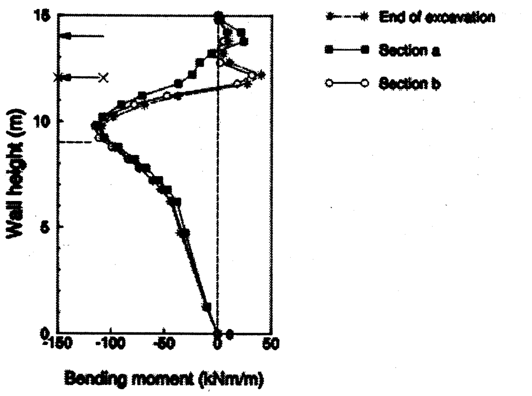

Fig. 10 Wall bending moments at the end of excavation and after failure of lower level strut

Figure 11 shows the plot of the horizontal stress σ_x for two elements of soil along *Section a* close to the wall on the retained side of the wall as depth of excavation increased and after the lower strut has failed. Element A is at the same elevation as the lower level strut that had failed, while Element B is at the same elevation as the upper level strut. Initially σ_x decreased as the excavation depth increased. After the installation of the upper strut at a depth of 1m below the ground surface, with the corresponding increases in the load on the strut, there was an increase in σ_x for Element B. The σ_x for Element A continued to decrease with the increase in depth of excavation as the lateral wall deflection increased.

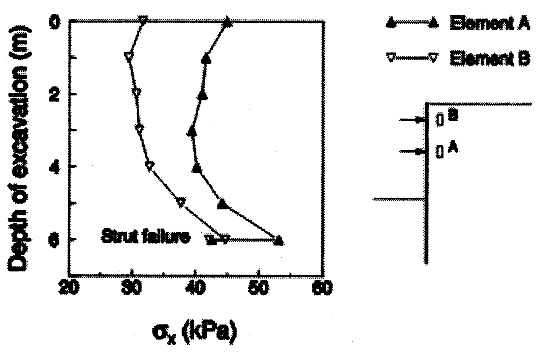

Fig. 11 Changes in lateral earth pressure with excavation depth and after failure of lower level strut

It was only after the installation of the lower strut that σ_x increased for Element A. Subsequently, the failure of the lower strut resulted in a decrease in horizontal stress of approximately 10kPa for Element A. Even though a significant proportion of the load from the failed strut was redistributed to the upper strut S2a, there was only a small decrease in horizontal stress for soil Element B of approximately 3kPa.

These small wall and soil movements and small decrease in horizontal stress in Element B presented above suggest that the redistribution of load to the adjacent struts was not through the arching of the soil behind the wall, but through the wall and wale members. Field studies similar to those of Stille (1976) would be invaluable to confirm these numerical findings.

An additional analysis was carried out to compare the impact of the failure of a strut at the lower level with the failure of the strut at the upper level which has a load of 226kN at the end of excavation. Figure 12 shows the redistribution of the load when the upper strut S1a failed. A comparison with Fig. 7 indicates that the maximum percent redistribution of 25% was slightly larger with the failure of the upper strut.

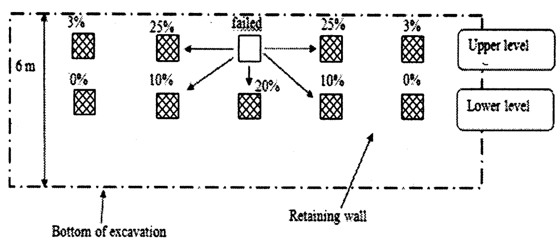

Fig. 12 Load redistribution after failure of upper level strut

A further analysis was also carried out to investigate the impact of the failure of two struts for the sheet pile wall. This was simulated in the finite element analysis through the failure of the lower strut in one loading stage, followed by the failure of the upper strut in the subsequent loading stage. Fig. 13 compares the increase in lateral wall movements Δu_x at Section a and Section b.

The increase in wall movement here refers to the difference in the wall movement between the failure of the lower strut and the failure of the upper strut. A comparison with Fig. 8 shows that the shape of the incremental wall movements differed. With the failure of two struts and the removal of any restraint on this section of the wall, the largest maximum Δu_x of 4.5 mm occurred at the top of the wall. The redistribution of the load from the failure of the upper level strut is shown in Fig. 14. The percent redistribution is with reference to the load after the failure of the first strut and before the second strut failed. The larger percent redistribution to the adjacent struts is likely to be due to the larger wall movements for this case where both struts failed. Fig. 15 compares the bending moments in the wall at the end of excavation (i.e., prior to the strut failure), and at *Section a* after the failure of the lower level strut followed by the failure of the upper level strut. The maximum wall bending moments occurred close to the formation level of the excavation. There were only minimal changes in the magnitude of the maximum bending moment as a result of the failure of the two struts.

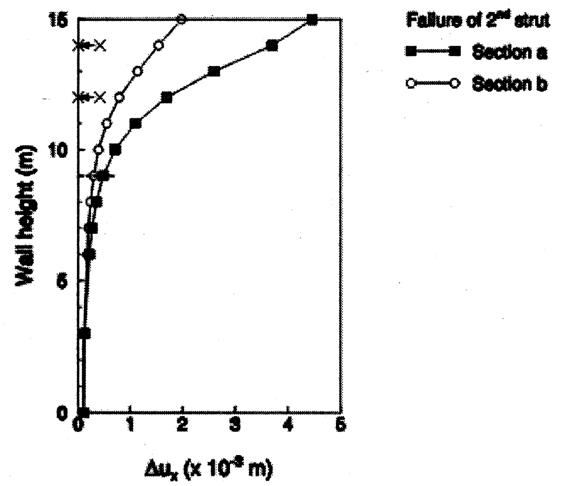

Fig. 13 Increase in lateral wall displacements after failure of second strut

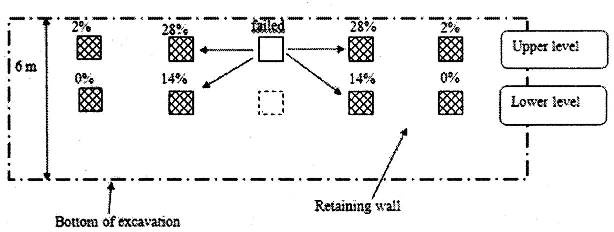

Fig. 14 Load redistribution after failure of second strut

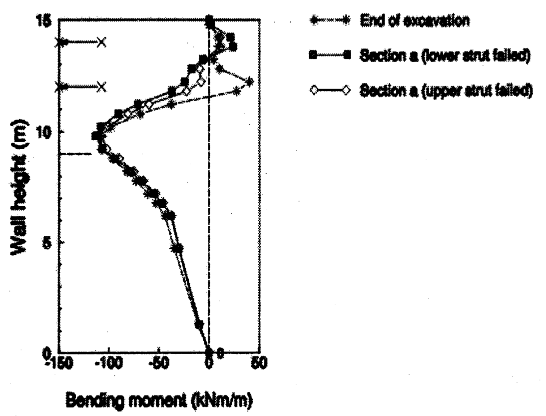

Fig. 15 Wall bending moments at the end of excavation and after failure of two struts

CONCLUSIONS

Three-dimensional finite element studies have been carried out to assess the impact of the failure of one or two struts. The failure of one or two struts caused the majority of the load(s) to be redistributed to the struts immediately adjacent to the failed strut. For the failure of a single strut, the maximum redistribution of load to the adjacent strut expressed as a percentage of the initial load of the failed strut was 25%. Since the numerical results show that the increase in the wall and soil movements were small, it is likely that the transfer of load to the adjacent struts was not through the arching of the soil behind the wall, but through the wall and wale members. The failure of two struts resulted in a maximum percent load redistribution of 28%. There were only minimal changes in the magnitude of the maximum bending moment as a result of the failure of either one or two struts. This study suggests that the failure of one or two struts would not result in detrimental failure of the entire excavation system, provided the struts have been adequately designed against compression failure. Field studies to validate these findings would be invaluable. It should be emphasized that the behaviour of a braced retaining wall system is very complicated and depends on various factors such as the excavation depth, the soil and wall properties, the wall depth, the strut properties and the horizontal and vertical spacing of the struts. Hence the findings from this paper may not be applicable for situations which differ from those considered here.

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Volume 39, Number 1, March 2008

Paper: *Issues in Evaluating Capacity of Rock Socket Foundations* by F. H. Kulhawy and W. A. Prakoso, pp. 51-62.

The following corrections need to be made in the referenced paper.

Equation 12b should read $f / p_a = 10^A (q_u / p_a)^{(1-B)}$

Equation 13 should read $f / p_a = 2.00 (q_u / p_a)^{0.31}$

Equation 14 should read $f / p_a = 1.74 (q_u / p_a)^{0.33}$

Equation 17 should read $f / p_a = 1.91 (q_u / p_a)^{0.54}$