Earth Pressures on Walls of a Deep Excavation

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SYNOPSIS This paper reports recorded earthpressures acting on diaphragm walls during a deep excavation carried out in a soft ground and discusses factors affecting the readings. The main theme of the paper is on wall friction and its influence on vertical earthpressures. It can be demonstrated that the assumption normally adopted in the design of the retaining structures for braced excavations that the vertical earthpressures equal to the overburden pressures could be erroneous. As a result, the vertical pressures on the active side are often over- estimated and those on the passive side under-estimated. In conclusion, it is appropriate for soft to medium stiff sites to assume that the angle of wall friction equals to the angle of internal friction of soils in computing the limiting active and passive earth pressures for designing the retaining structures of braced excavations.

INTRODUCTION

The 13-story CPH Building is located in the central business district of Taipei City. The soil profile at the site is shown in Fig. 1. The soil strata shown are typical in the Taipei Basin with the six sublayers of the Sungshan formation clearly identifiable.

The Building has a 4-level basement constructed by using the top-down method. The excavation was carried out to a maximum depth of 17.35 m below the ground surface. After the excavation reached its final level, as shown in Fig. 2, the central portion of the base slab was cast and diaphragm walls were braced against this slab. The base slab is in fact a water storage tank with its thick partition walls serving as ground beams.

MONITORING OF TOTAL/WATER PRESSURES

A total of 14 combined total/water pressure cells were installed, 10 on the back and 4 on the front, on two diaphragm wall panels at depths shown in Fig. 1. The initial and the final readings are presented in the Fig. 3. The initial total pressures were affected by the jacking operation during the installation of the pressure cells in order to make a good contact with the side wall of the trench and do not represent the earth pressures at rest. The groundwater in the Taipei basin has been overdrawn causing serious ground subsidences in the past and therefore, as can be noted from the figure, the initial water pressures were much lower than the hydrostatic pressures.

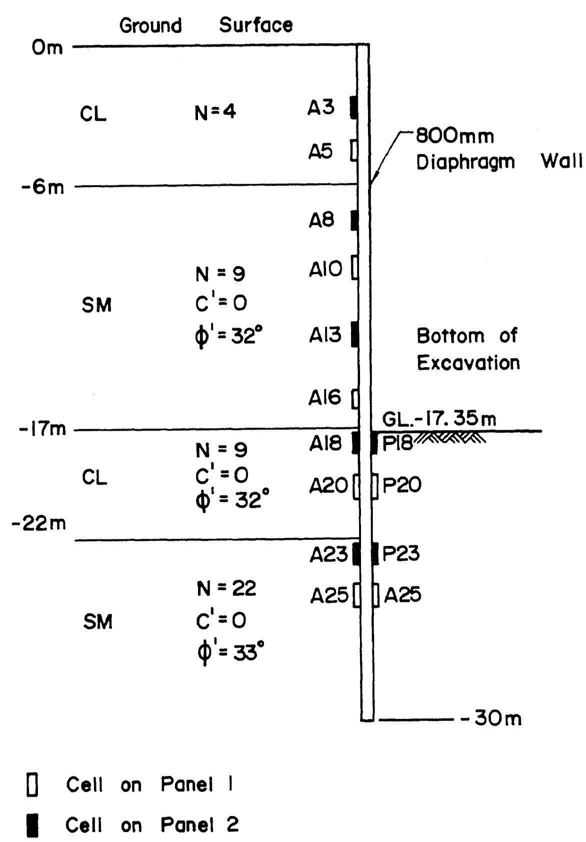


Figure 1 Soil Profile and Depths of Pressure Cells

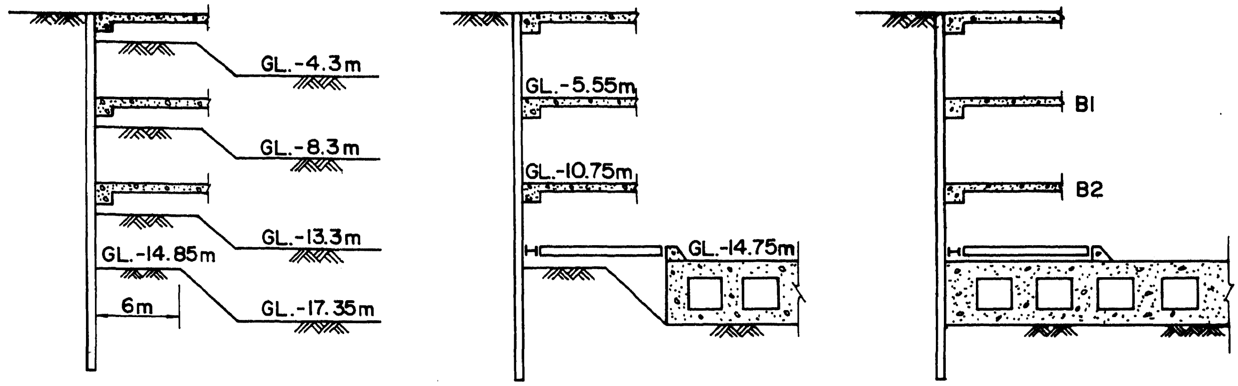


Figure 2 Sequence of Excavation

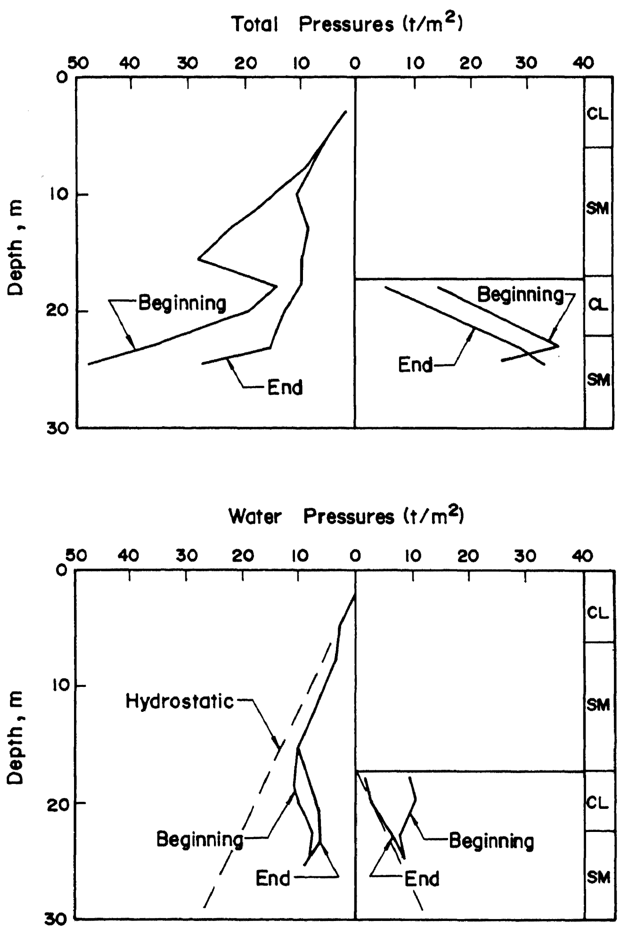


Figure 3 Distribution of Total and Water Pressures

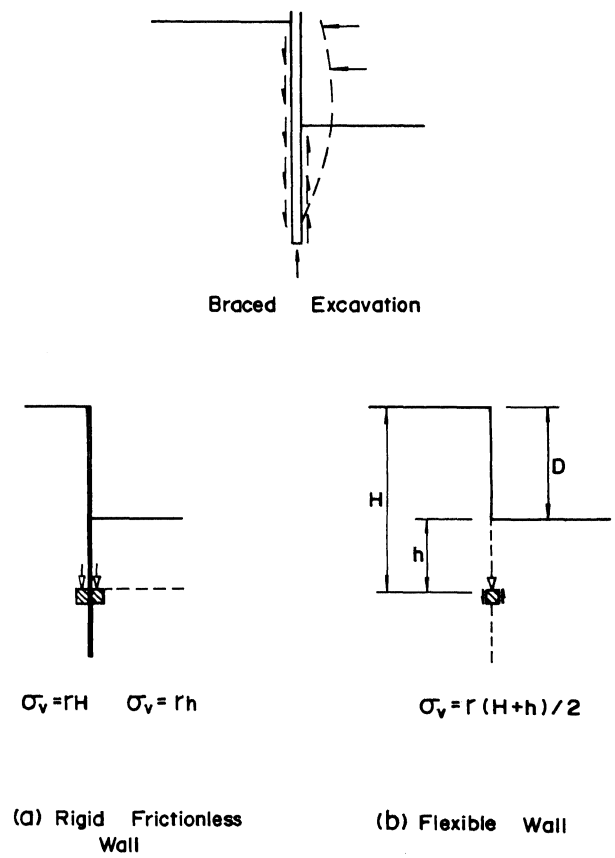


Figure 4 Vertical Earth Pressures as Affected by Characteristics of Wall

VERTICAL PRESSURES AND WALL FRICTION

In designing retaining structures for braced excavations using beam models, which still is the most popular method of analysis nowadays, it is almost a standard practice to assume that the vertical pressure, on either the excavation (passive) side or the back (active) side, acting on a soil element equals to the weight of soil column on top of the element. This assumption, strictly speaking, would have been correct, as shown in Fig. 4(a), only if the wall were perfectly rigid and perfectly smooth. Obviously, these two conditions can never be met in reality.

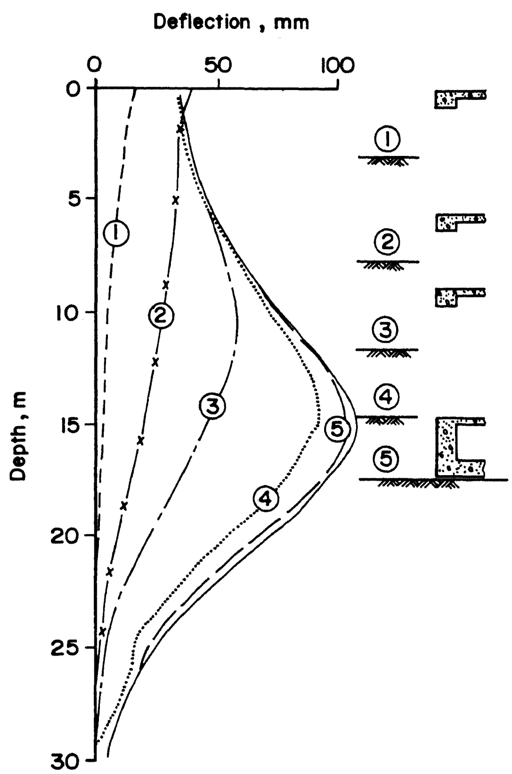


Figure 5 Wall Deflections During Excavation

On the contrary, if the wall were perfectly flexible and could be pretended to be nonexistent, the vertical pressure on both sides of the wall would have been the same and would equal to the average overburden pressures as illustrated in Fig. 4(b). The soil in this case is assumed to be an elastic continuum with an unlimited strength disallowing yielding.

The fact is, however, walls must have a certain rigidity which is required to restrain ground movements. Take CPH Building for example, the deflections of the walls, as shown in Fig. 5, were limited to a maximum of 110 mm. Conceivably, if the walls were perfectly flexible, the ground would have collapsed.

Furthermore, as the wall deforms, the negative friction on the active side drives the wall down and this friction is balanced partly by the positive friction on the passive side and partly by the bearing at the toe. Because of the difficulty in cleaning the bottom of the trench, diaphragm walls almost always have soft toes. Therefore, in usual cases the negative friction on the back of the wall, as well as the weight of the diaphragm wall, is entirely supported by the positive friction on the excavation side of the wall.

In any case, the negative friction on the back side of the wall tends to reduce the vertical pressures in soils and the positive friction on the excavation side tends to increase the vertical pressures in soils below the bottom of excavation.

ACTIVE EARTH PRESSURES

The recorded horizontal total/water pressures on the back of the wall at a depth of 20 m are shown in Fig. 6. As can be noted that both the total and water pressures responded to excavation faithfully. The effective horizontal pressures, σ_h' , on the wall can be obtained by subtracting the water pressures, u , from the total pressure, σ_h .

Also shown in the same figure are the total vertical pressures computed by adopting the common practice of assuming that the vertical pressures, σ_v , equal to the overburden pressures, γh, i.e. the case shown in Fig. 4(a). Since the ground surface was maintained unchanged throughout the excavation, the total vertical pressures were constant. The effective vertical pressures, σ_v', will then simply be the differences between the total vertical pressures, σ_v , and the water pressures, u .

Coefficients of Active Earth Pressures

The ratios of σ_h' / σ_v' for the active pressures at the depth of 20 m are shown in Fig. 6. As can be noted that this ratio dropped gradually to a low of about 0.17 which can be considered as the coefficient of active pressure for the soil layer in which the cell was buried. The pattern shown in the figure is typical for cells below the excavation lines all the times. Figure 7 shows the results for Cell A5 at the depth of 5m, and as can be noted that the earthpressures behind the wall did increase after the B1 slab at the depth of 5.55 m had been cast.

The representative lower-bound ratios of σ_h'/σ_v' obtained by all the cells installed on the back of the wall are :

Depth	Soil Type	σ_h'/σ_v'
5m	CL	0.16
10m	SM	0.21
20m	CL	0.17
23m	SM	0.21

The readings taken at the depth of 25m indicate that yielding was not reached even at the end of excavation. The ratios obtained by other cells were too low, as low as zero, to be explainable. This could have been a result of other complications such as arching effects. The possibility of malfunction of either total or water pressure cells of course can not be ruled out.

The ratios of σ_h'/σ_v' listed above correspond to the set of vertical pressures estimated by assuming that the wall were perfectly rigid and perfectly smooth and can only be called "apparent coefficients of active pressures". The effective friction angles of soils obtained from laboratory tests, see Fig. 1, were about 32 or 33 degrees for all the layers. The ratio of σ_h'/σ_v' of 0.16 to 0.21 will indicate an angle of wall friction much in excess of the internal friction angle of the soils as shown in Fig. 8. This is of course illogical. The most likely reason for this to happen is that, the vertical pressures were over-estimated because of the omission of the effects of wall friction.

Assuming that the wall friction indeed equals to soil friction, i.e., $\delta' = \phi'$ then the coefficient of active pressure should be about 0.25 as indicated by Fig. 8. The effective vertical pressures can be computed by dividing the effective horizontal pressures by this coefficient and the total vertical pressures can be obtained by adding the water pressures to the results.

The total vertical pressures so estimated are (in t/m^2):

Depth	Est. From Horizontal Pressures	Rigid Smooth Wall	Flexible Wall
20m	30.3	38	21.5
23m	37.5	43.7	27.2

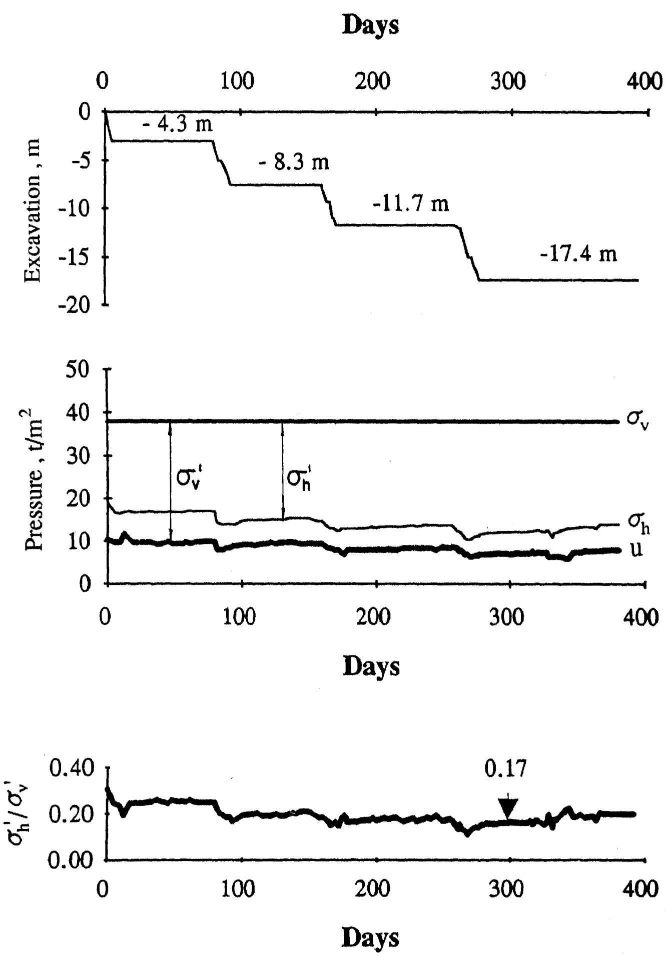


Figure 6 Active Pressures at a Depth of 20m

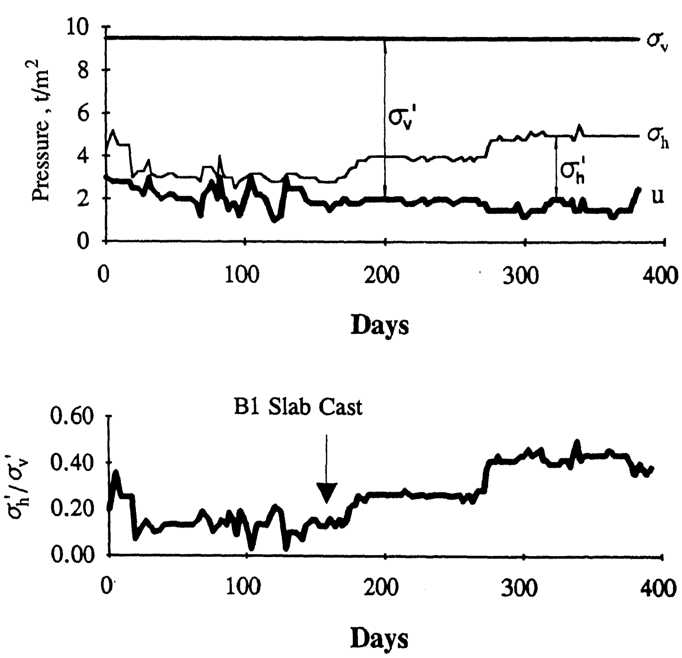


Figure 7 Active Pressures at a Depth of 5m

As shown in Fig. 9, they fall in the range for the two cases shown in Fig. 4, and, are at about halfway in-between.

The assumption of $\delta' = \phi'$ is, in the opinion of the authors, totally logical in consideration of the roughness of soil/diaphragm wall interface. It is often argued that the bentonite cake at the interface forms weakness and reduces wall friction. Recent loading tests have proved that for soft to medium stiff ground the shaft frictions of bored piles are unaffected by the presence of bentonite cake. It is believed that the dehydration as the concrete hardens will reduce the water content of the bentonite cake and effectively increase the strength of the cake to a value at least equal to the strength of soil, if not greater.

PASSIVE EARTH PRESSURES

The total/water pressures obtained on the excavation (passive) side of the wall at the depth of 20 m are shown in Fig. 10, together with the total vertical pressures obtained for the case of rigid smooth walls. Since the excavation was carried out in stages, the vertical pressures were no longer constant.

The ratios of σ_h'/σ_v' were computed in a similar manner as that for the active pressures and are shown at the bottom of Fig. 10. It is difficult to decide whether

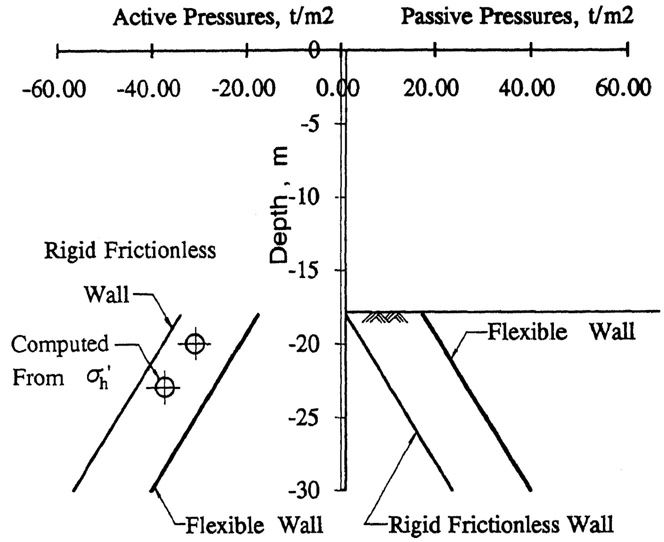
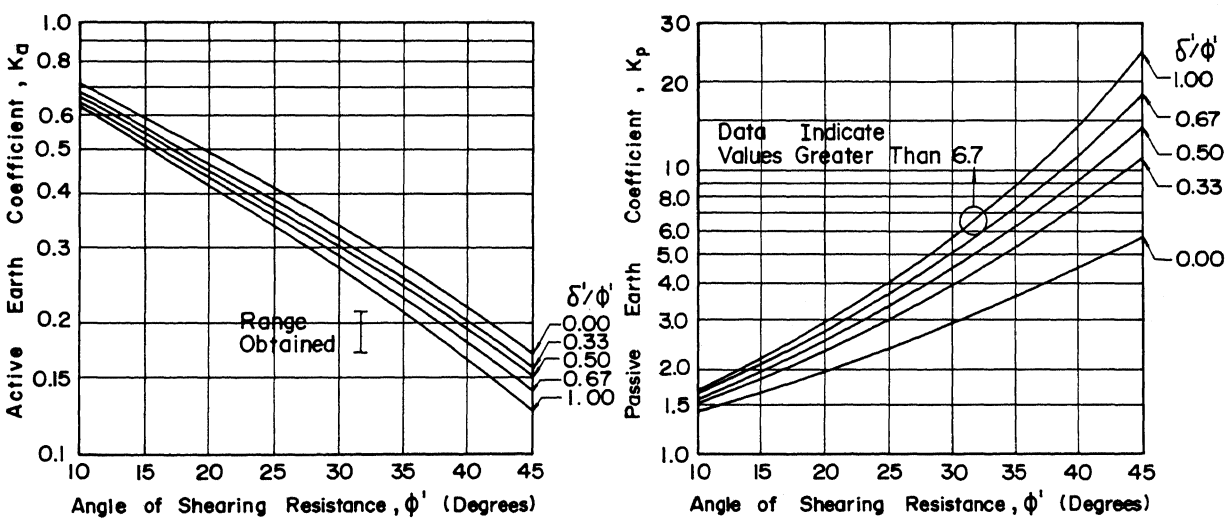


Figure 9 Computed Vertical Pressures

yielding was indeed reached . The decrease in the ratios after Day 353 was due to the increase of vertical pressures as the base slab was cast, not necessarily because of yielding. Normally, stress path will be useful in determining if yielding did occur, however, without accurate vertical pressure and accurate wall friction, any such attempt would only lead to misleading results.



Coefficients of active earth pressure (horizontal component) for horizontal retained surface

Coefficients of passive earth pressure (horizontal component) for horizontal retained surface

Figure 8 Coefficient of Earth Pressure (after Caquot and Kerisel, 1948)

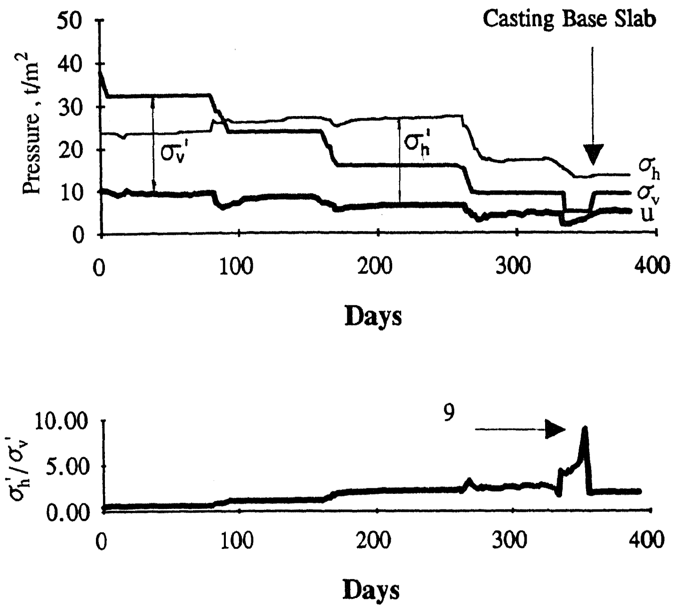


Figure 10 Passive Pressure at a Depth of 20m

The ratios for all the four cells at the last stage are shown in Fig. 11. The water pressures were "unsteady" and the ratios of σ_h'/σ_v' fluctuated in a wide range. A value exceeding 100 was computed for the cell installed at the depth of 18 m. In any case, it is conceivable that the apparent coefficient of earth pressures exceeded 6.7 which corresponds to $\delta' = \phi'$ as indicated in Fig. 8, leading to the suspicion that the vertical pressures were under-estimated due to the omission of the effects of wall friction.

CONCLUSIONS

The commonly adopted assumption that the vertical pressures on the two sides of the retaining walls for braced excavations equal to the overburden pressures is theoretically incorrect because the rigidity of the wall and the effects of wall friction have been neglected. As a result, the vertical pressures on the active side were over-estimated and those on the passive side were under-estimated. The corresponding "apparent" angle of wall friction is much greater than the angle of internal friction of soils.

It is thus concluded that for soft to medium stiff sites, it will be appropriate to assume that the angle of wall friction equals to the angle of internal friction of the soils in computing the limiting earthpressures to be used in the design of the retaining structures of braced excavations using beam models.

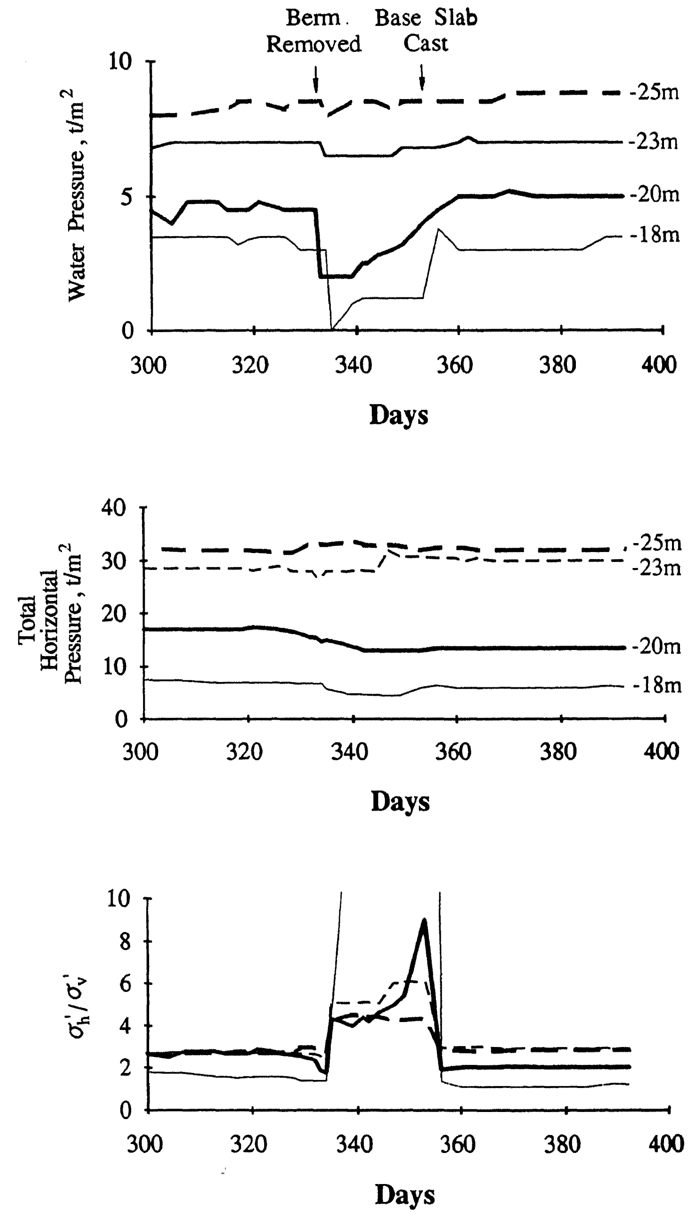


Figure 11 Results for All Cells on the Passive Side

However, it should be noted that earthpressures are a function of wall deflection and limiting active and passive pressures will develop only when wall deflection is sufficiently large. Furthermore, the earthpressures on the active side of the wall may increase due to the outward movement of the wall and such a mechanism shall be properly accounted for in the analyses.