Development of an alternative concept of earth retention: The soil/caisson buttress

Le développement d'un concept alternatif de soutènement du terrain: Le contrefort du sol et du caisson

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SYNOPSIS : Intense development of steep terrain in Hong Kong often necessitates the construction of earth-retaining structures. Due to land constraints, hand-dug caissons in the form of cantilevers are sometimes constructed to retain great heights of soil. An alternative concept which utilizes the caisson construction technique and soil/portal frame structure interaction has been conceived. This form of soil structure, which has been named the "soil/caisson buttress", has been found to provide an economical alternative for medium to high retained heights. In view of the potential benefits to be gained, a series of model tests has been carried out to observe failure mechanisms for this type of structure. This Paper examines the collapse limit states involved in the soil/caisson buttress interaction. Results of the experimental investigation are then summarised.

1 INTRODUCTION

Intense development of steep terrain in Hong Kong often necessitates the construction of earth-retaining structures to provide platform space. Conventional L-shaped reinforced concrete retaining walls are often used. However, construction of this type of retaining wall involves creating a temporary cut slope behind the proposed wall. In addition to being marginally stable, the temporary cutting may not be feasible due to land constraints. For this reason, hand-dug caissons are sometimes constructed (Malone 1982, Pope 1983, Greenway et al 1986).

During an investigation into alternative types of retaining walls for an engineering project, an alternative concept which utilizes the caisson construction technique and soil/portal frame structure interaction has been conceived (Figure 1). This form of soil structure, which has been named the "soil/caisson buttress", has been found to provide an economical alternative for medium to high retained heights (Pang 1988). In view of the potential benefits to be gained, a series of model tests was arranged to observe failure mechanisms for this type of structure.

2 COLLAPSE LIMIT STATES

The following collapse limit states can be identified for a soil/caisson buttress retaining structure :

(a) sliding failure of the soil/caisson buttress mass as a monolith,
(b) overturning failure of the soil/caisson buttress mass as a monolith,
(c) foundation bearing failure,
(d) caisson buttress pull-out failure,
(e) caisson buttress overturning failure, and
(f) structural failure of the caisson portal frame.

Only the fourth and fifth limit states are discussed in detail here. For these two limit states, the stress distributions in the soil around the rear caissons are three-dimensional.

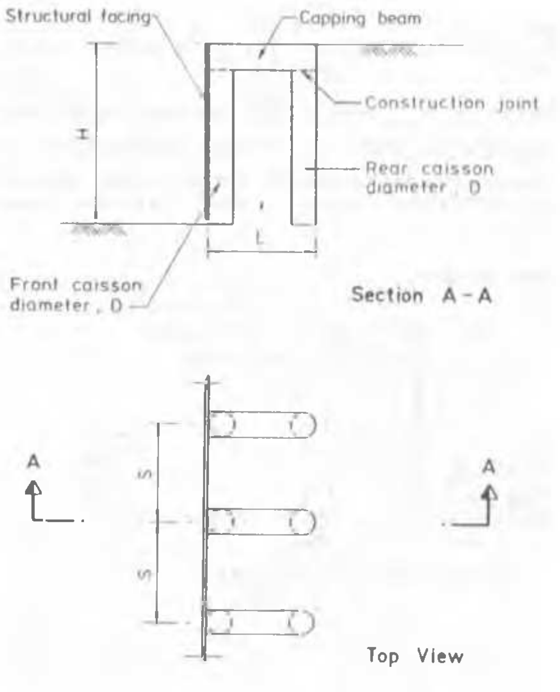


Figure 1. Geometry and components of a soil/caisson buttress.

In order to facilitate the assessment of these limit states, a simplified limit equilibrium analysis is proposed. Figure 2 depicts the forces acting on a caisson buttress.

In the analysis, the following assumptions are made:

(a) the caisson portal frame and the soil within it act as a buttress (hence the name "soil/caisson buttress"),
(b) the soil in the front 'yielding zone' is

in a state of plastic equilibrium, with active earth pressures, σ'_{hf}, acting on the front facing,

(c) active earth pressures, σ'_{hb}, are developed at the rear of the soil/caisson buttress,

(d) the soil in the resisting zone 'grips' the soil/caisson buttress to provide the necessary restraint against failure,

(e) shear stresses τ_s are limiting at the sides of the buttress,

(f) shear force Q_s developed in the ground at foundation level is limiting,and

(g) shear forces Q_{cf} and Q_{cb} at the base of the front and rear caisson respectively are limiting.

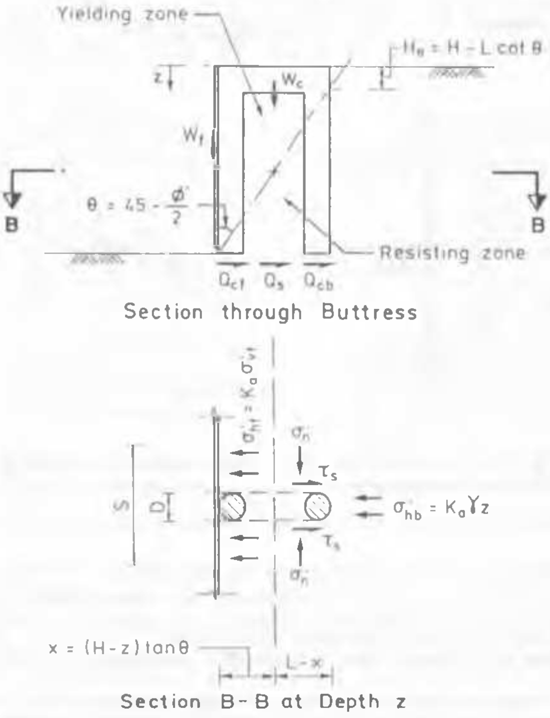
The total horizontal force activating failure, P_a, equals the sum of the forces acting on the facing and those acting on the rear caisson :

$$P_a = \int_0^H \sigma'_{hf}(S-D)dz + \int_0^H \sigma'_{hb}Ddz \quad \ldots\ldots\ldots\ldots(1)$$

The total moment about the front caisson toe activating failure, M_a, is given by :

$$M_a = \int_0^H \sigma'_{hf}(S-D)(H-z)dz + \int_0^H \sigma'_{hb}D(H-z)dz \quad ..(2)$$

The value of σ'_{hf} could fall between $K_a\gamma z$ and $K_a\gamma z(1+K_a z^2/L^2)$, where K_a is the coefficient of active earth pressive and γ is the unit weight of the retained soil. The latter case corresponds to a trapezoidal ground bearing pressure distribution (Bolton & Pang, 1982).



Section through Buttress

Section B-B at Depth z

Figure 2. Forces acting on a caisson buttress.

The total horizontal force resisting failure, P_r, equals the sum of the resisting forces acting on the two sides of the buttress and those acting along its base (Figure 2) :

$$P_r = 2\int_{H_o}^H \tau_s(L-x)dz + Q_s + Q_{cf} + Q_{cb} \quad \ldots\ldots\ldots(3)$$

where H_o = H-Lcotθ and x = (H-z)tanθ.

The total moment about the front caisson toe resisting failure, M_r, equals the sum of the moments due to τ_s and W_c, the weight of the caisson portal frame :

$$M_r = 2\int_{H_o}^H \tau_s(L-x)(H-z)dz + W_cL/2 \quad \ldots\ldots\ldots\ldots(4)$$

The limiting value of τ_s equals $\sigma'_n\tan\phi'$, where σ'_n is the normal stress acting across the buttress and ϕ' is the angle of shearing resistance of the retained soil. It is assumed in the analysis that τ_s equals $K_o\gamma z\tan\phi'$.

The limiting values of Q_s, Q_{cf} and Q_{cb} are given by :

$$Q_s = \gamma H[(L-D/2)D-1.5\ D^2/4]\tan\phi'$$
$$\doteq \gamma HD(L-1.7D)\tan\phi' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$
$$Q_{cf} + Q_{cb} = (W_c+W_f)\tan\delta \quad \ldots\ldots\ldots\ldots\ldots\ldots(6)$$

where W_f is the weight of facing having a width S and δ is the angle of shearing resistance between the caisson base and the foundation soil.

The factors of safety against caisson buttress pull-out and overturning failure, F_{bp} and F_{bo} respectively are given by :

$$F_{bp} = P_r/P_a \quad \ldots\ldots\ldots\ldots\ldots\ldots(7)$$
$$F_{bo} = M_r/M_a \quad \ldots\ldots\ldots\ldots\ldots\ldots(8)$$

3 EXPERIMENTAL INVESTIGATION

The experimental investigation was concerned with observing collapse. The models were constructed in a perspex box 500 x 480 x 400 mm high. Sand was glued to the base of the box to

Table 1. Types of 'legs' used in model tests.

Material	Shape	Symbol	Dimensions
PVC		P	D = 18 0 mm
Wood		S	D = 20 0 mm
Wood		C	D = 12 7 mm

provide a rough foundation. The height of sand retained by the model structures was 150 mm, giving a height to width ratio of 0.31 for the models. This is sufficiently small for side-wall friction effects on active earth pressures to be neglected (Bransby & Smith 1975).

The dry sand used was a quartz (river) sand with an effective size of 0.18 mm and a uniformity coefficient of about 2.3. It was poured into the perspex box in layers using a hand scoop to give a unit weight of about 14.5 kN/m^3. This corresponds to a void ratio of 0.78 and about 20% relative density. The shear strength of the sand was determined using a 60 mm direct shear box, which gave an angle of shearing resistance ϕ' of about 44°.

Three types of 'legs' were used for constructing the model portal frames and their details are given in Table 1.

A purpose-made perspex frame was used to provide support in front of the facing during placement of sand in the model. Portions of this frame could be removed so that, after completion of the model, an unstable model could move forward slightly, but would soon be arrested by the remaining portions of the perspex frame. This enabled the failure mechanism to be 'frozen' for the purpose of description.

A total of 36 models were constructed to determine the dimension of length L at failure for 10 series of tests, viz. S1 to S10. A summary of the model test results is given in Table 2. All the models that collapsed did so by overturning about their front legs. Furthermore, the portal frames were observed to have 'pulled out' of the sand, leaving a localised surface depression that had a width of about the same magnitude as dimension D at the trailing edge of the rear legs. Therefore, it is considered that the collapse mechanism is overturning of the caisson buttresses. The analytical models put forward in the previous section also predict that this mechanism is the most critical one for S/D ratios larger than about 2.5.

As indicated in the notes to Table 2, some of the models had built-in 'defects' at the base of the legs. These defects were to model the effect of a poorly-cleaned caisson base or localised soft spot in the foundation. Models without defects were found to be more stable than those with defects. Also, models with

Table 2. Summary of model test results.

Series	Type	Material Symbol	L (mm) 50 55 60 65 70 75 80 85 90	$(\frac{L}{H})_f$
S1	I	P	8 x, S1 ●, 29 ○, 4 ○, 3 ○	0 51
S2	II	P	17 x, 32 x, S2 ●, 12 ○	0 48
S3	I	S	9 x, 30 x, S2 ●, 10 ○	0 45
S4	II	S	18 x, S4 ●, 31 ○, 21 ○	0 413
S5	I	C	23 x, S5 ●, 33 ○, 24 ○	0 483
S6	II	C	22 19 xx, S6 ●, 25 ○, 20 ○	0 49
S7	III	C	16 x, 27 x, S7 ●, 14 ○	0 46
S8	I	P	2,7 x, S8 ●, 28 ○, 11 ○, 1 ○	0 44
S9	I	S	13,15 x, 26 x, S9 ●, 6 ○, 5 ○	0 38
S10	I	C	35 x, 36 x, S10 ●, 34 ○	0 42

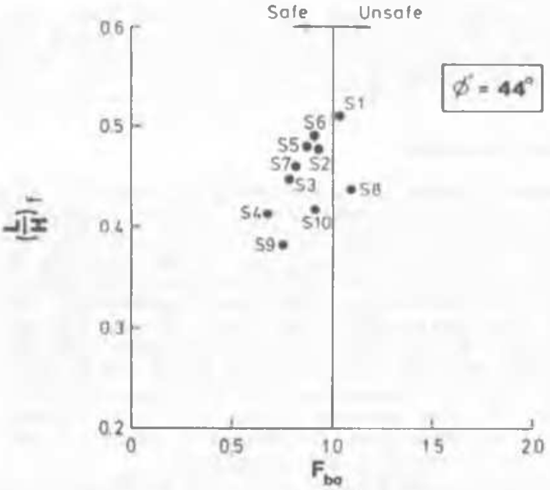
Legend

29 ○ Stable model no 29

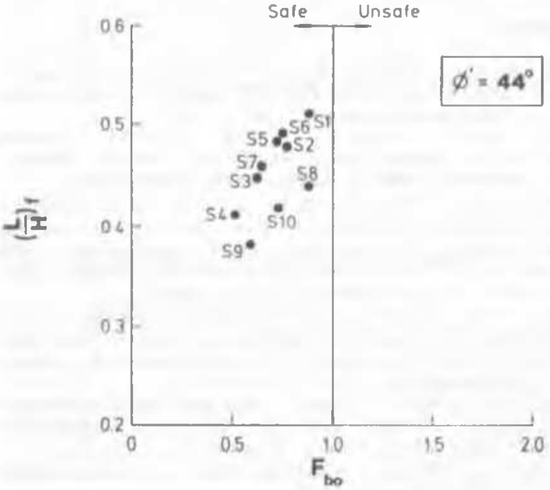
8 x Collapsed model no 8

S1 ● Interpolated value of L at limit equilibrium for Series S1

Notes
(1) All models retained 150 mm of sand
(2) The front and rear legs of Type I and II models respectively were 5mm shorter. Both front and rear legs of Type III models touched the foundation base
(3) The portal frames of the models in Series S1 to S7 were spaced at 160 mm apart, while those in Series S8 to S10 were spaced at 120 mm



(a) Based on $\sigma'_{hf} = K_a \gamma z$



(b) Based on $\sigma'_{hf} = K_a \gamma z (1 + K_a z^2/L^2)$

Figure 3. Prediction of caisson buttress overturning Failure.

slightly shorter rear legs were generally more stable than those with shorter front legs. The failure data have been back-analysed using the two assumptions of vertical stress distribution stated in Figure 3. From Figure 3(a), it can be seen that the prediction using the assumption of $\sigma'_{hf} = K_a \gamma z$ is unsafe for test series S1 and S8, which had shorter front legs. If the more conservative trapezoidal stress distribution is assumed, then the calculation model for caisson buttress overturning gives predictions on the safe side.

4 CONCLUSIONS

An alternative concept of earth retention, named the "soil/caisson buttress", is proposed. By virtue of the efficient construction technique and soil/portal frame interaction, this form of earth-retaining structure is likely to provide an economical alternative for medium to high retained heights. The experimental investigation carried out confirms that the proposed limit equilibrium analysis is conservative and could be used as the basis of a design calculation model.

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REFERENCES

Bolton, M.D. & Pang, P.L.R. (1982). Collapse limit state of reinforced earth retaining walls. Géotechnique 32, 349 - 367.

Bransby, P.L. & Smith, I.A.A. (1975). Side friction in model retaining-wall experiments. Jour. Geotech. Eng., ASCE, 101, 615 - 632.

Greenway, D.R., Powell, G.E. & Bell, G.S.(1986). Rock-socketed caissons for retention of an urban road. Proc. Int. Conf. Rock Eng. and Excavation in an Urban Environment, Hong Kong, 173 - 180. (Discussion, 450 - 456).

Malone, A.W. (1982). Moderator's Report on retaining walls and basements. Proc. 7th S.E. Asian Geotech. Conf., Hong Kong, 2, 249-271. (Discussion, 2, 275 - 280).

Pang, P.L.R. (1988). Model testing of retaining structures. Proc. Seminar on Soil Structure Interaction, Hong Kong, in press.

Pope, R.G. (1983). Caisson walls. Hong Kong Engr 11, no. 10, 37 - 38.