Ultimate Resistance of Vertical Square Anchors in Clay

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Laboratory model test results for the ultimate pullout resistance of square vertical anchors in saturated clay are presented. Pullout resistance can be expressed in the form of a nondimensional breakout factor. The breakout factor increases with the embedment ratio of the anchor up to a maximum value and remains constant thereafter. A tentative empirical procedure, based on the laboratory test results, for estimating anchor pullout resistance is outlined.

Vertical anchor slabs (Figure 1*a*) are used in many instances in the construction of earth-retaining structures. They are also used in the design of pipeline bends. Recently, the results of a number of experimental and theoretical studies related to the ultimate resistance of vertical anchors in sand have been published *(1–5)*. Most of the important findings of those studies have been summarized elsewhere by Das *(6)*. In contrast, few attempts have so far been made to estimate the ultimate resistance of vertical anchors of limited height and width embedded in clay soils. The purpose of this paper is to present the results of some laboratory experimental studies of the ultimate pullout resistance of square anchor plates embedded in saturated or nearly saturated clayey soils.

ULTIMATE RESISTANCE OF VERTICAL ANCHORS IN CLAY

Mackenzie *(7)* conducted a number of laboratory model tests on strip anchors in two different saturated clay soils. According to this study, the ultimate resistance of vertical anchors can be conveniently expressed in a nondimensional form as

$$F_c = \frac{Q_u}{(BL)c_u} \tag{1}$$

where

F_c = breakout factor,
Q_u = ultimate anchor pullout resistance,
B = height of anchor plate,
L = length of anchor plate, and
c_u = undrained cohesion of clay.

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It needs to be pointed out that the term "breakout factor" was not used by Mackenzie *(7)*. For convenience only, it has been introduced here.

The average variation of the breakout factor with embedment ratio H/B (where H = depth of embedment of the anchor plate as shown in Figure 1) as obtained by Mackenzie *(7)* is shown in Figure 2. This has also been shown by Tschebotarioff *(8)*. On the basis of Figure 2, the following general conclusions can be drawn:

1. The magnitude of F_c for a given anchor plate in a given soil increases with H/B.
2. There appears to be a critical value of embedment ratio, $H/B = (H/B)_{cr}$, at which the magnitude of the breakout factor approximately attains a maximum value, $F_c = \overline{F_c}$.
3. For $H/B \geq (H/B)_{cr}$, the value of F_c remains constant (i.e., equal to $\overline{F_c}$).

In Figure 2, the value of $(H/B)_{cr}$ is approximately equal to 12 or 13, and $\overline{F_c} \cong 9$. Hence, anchors with $H/B \leq (H/B)_{cr}$ may be referred to as shallow anchors for which general shear failure in soil takes place and the failure surface extends to the ground surface as shown in Figure 1*b*. For anchors with embedment ratios of $H/B > (H/B)_{cr}$, local shear failure in soil takes place, and these anchors may be referred to as deep anchors, as shown in Figure 1*c*. Tschebotarioff *(8)* has commented that in the field the unit weight of the soil (γ) should have some influence on the ultimate resistance of an anchor. The effect of unit weight on the ultimate resistance (Q_u) obtained from the laboratory tests is somewhat negligible. However, for actual design work the use of laboratory test results will provide conservative estimates.

Meyerhof *(9)* has suggested that a conservative estimate of the breakout factor (F_c) with embedment ratio may be given as follows:

For strip anchors,

$$F_c = 1.0(H/B) \quad \text{(with a maximum of 8)} \tag{2}$$

For square anchors,

$$F_c = 1.2(H/B) \quad \text{(with a maximum of 9)} \quad (^{\prime}$$

Equations 2 and 3 imply that the critical embedment ratio oi strip anchors is about 8 and that for square (or circular) anchors it is about 7.5. For comparison purposes, these equations have also been plotted in Figure 2.

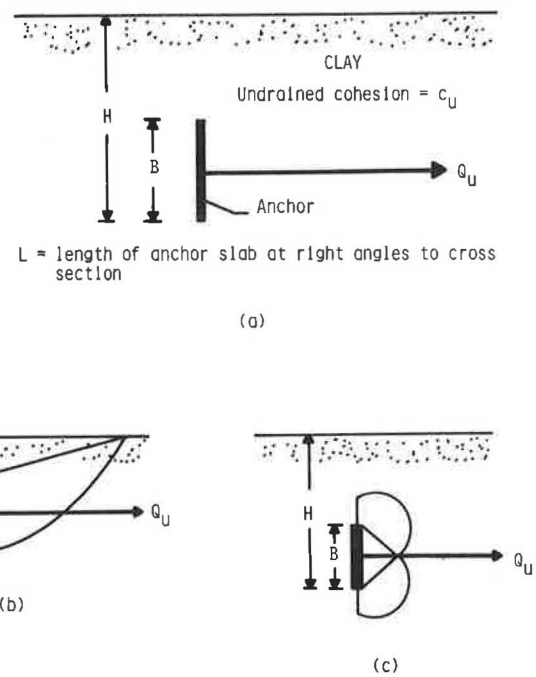

FIGURE 1 Parameters of a vertical anchor slab (a); definition of a shallow anchor (b); and definition of a deep anchor (c).

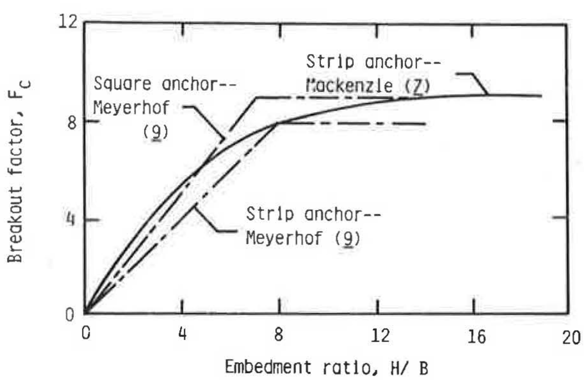

FIGURE 2 Variation of breakout factor with embedment ratio as given in the studies of Mackenzie *(7)* and Meyerhof *(9)*.

The present study relates to the ultimate resistance of square anchors only, because there are practically no experimental results available in the literature. Also, it needs to be realized that the magnitude of $(H/B)_{cr}$ may change with the consistency of clay [i.e., the undrained shear strength (c_u)]. This is true in a closely related problem—the uplift capacity of horizontal anchors in clay *(10)*.

LABORATORY INVESTIGATION

To estimate the ultimate resistance of horizontal anchors in saturated or nearly saturated clay, a number of model tests were conducted in the laboratory with two model anchors measuring 38.1 × 38.1 mm and 50.8 × 50.8 mm ($B \times L$). The anchors were made from a steel plate 9.5 mm thick.

Two different clay soils were used for the testing program. The grain-size distributions of the two soils were determined by sieve and hydrometer analyses in the laboratory, along with their Atterberg limits. A summary of the preliminary tests is given in Table 1.

TABLE 1 PROPERTIES OF SOILS USED FOR LABORATORY TESTS

Item	Quantity	
	Soil A	Soil B
Percent passing No. 200 U.S. sieve	78	68
Percent finer than 0.002 mm	29	26
Liquid limit	32	39
Plastic limit	19	14
Plasticity index	13	25
Unified soil classification	CL	CL

The soils were initially pulverized in the laboratory, and desired amounts of water were added to them. After thorough mixing the moist soils were transferred to several plastic bags. The bags were then sealed and kept in a moist curing room for about a week before use.

All model tests were conducted in a box with inside dimensions of 0.915 × 0.508 × 0.915 m (height). A schematic diagram of the laboratory test arrangement is shown in Figure 3. To conduct a test, the anchor was rigidly attached to a steel rod 7.94 mm in diameter. The rod, in turn, was attached to a steel cable with a diameter of 4.76 mm. The cable passed over a pulley, and the other side of the cable was attached to a load hanger.

TABLE 2 LABORATORY TEST PARAMETERS

Test Series	Soil Used	Width of Plate (B) Used (mm)	Average Moist Unit Weight of Compacted Soil (kN/m^2)	Average Moisture Content of Compacted Soil (%)	Average Degree of Saturation of Compacted Soil (%)	Average Undrained Shear Strength (c_u) (kN/m^2)	Range of Embedment Ratio (H/B) Tested
1	A	50.8	19.65	24.5	97	20.3[a]	1–8
2	A	38.1	20.76	17.6	94	42.4[b]	1–9
3	B	50.8	19.03	28.5	98	12.5[a]	1–7
4	B	38.1	20.29	18.5	92	28.1[b]	1–8
5	B	38.1	20.65	16.2	93	52.0[b]	1–9

[a]Determined from laboratory vane shear tests.
[b]Determined from unconsolidated, undrained triaxial tests.

The moist soil from the plastic bags was poured into the box and compacted in 50.8-mm-thick layers to the desired height. The compaction was done in sections using a flat-bottomed rammer. Proper compaction around the anchor plate was a difficult task. A small flat-bottomed rammer with sides measuring 101.6 × 101.6 mm was used to compact the clay around the anchor.

After compaction, step loads were placed on the load hanger, and the corresponding horizontal movements of the anchor were observed by a dial gauge. A time lapse of from 5 to 8 min was allowed between the placement of each step load, and the loading continued until failure occurred. The time lapse was allowed because a steady movement of the anchor was noted immediately after load application (primary creep). After the time lapse of from 5 to 8 min, the horizontal movement of the anchor plate was practically zero.

A total of five series of tests were conducted in the laboratory. Details of the tests are given in Tables 2 and 3. It needs to be pointed out that the values of the unit weight and moisture content given in Table 2 are the arithmetic average values. For each test, five or six samples were taken from various depths at random for determination of the unit weight of compaction and moisture content. This was done at the end of the pullout tests using a 76.2-mm-diameter, thin-wall tube 152.4 mm long. The difference between the minimum and the maximum values of the unit weight for any given series was no more than about 6 percent, which was to be expected when working with moist clay soil. Some of the thin-wall tube samples for Test Series 2, 4, and 5 were trimmed to prepare triaxial test specimens (35.56 mm in diameter and 76.2 mm in height). The triaxial test specimens were tested with a chamber confining pressure of 70 kN/m^2. In Test Series 1 and 3, which involved softer clays, the trimmed specimens were not particularly good for triaxial tests.

TABLE 3 DETAILS OF LABORATORY TESTS

Test Series	Soil Used	Embedment Ratios of Tests
1	A	1, 2, 3, 4, 5, 6, 7, 8
2	A	1, 2, 3, 4, 5, 6, 7, 8, 9
3	B	1, 2, 3, 4, 5, 6, 7
4	B	1, 2, 3, 4, 5, 6, 7, 8
5	B	1, 2, 3, 4, 5, 6, 7, 8, 9

For that reason, laboratory vane shear tests were used to determine c_u, although it is well known that laboratory vane shear tests are not extremely reliable *(11)*. The stress-strain curves of triaxial tests were somewhat similar in shape to the load-displacement plots obtained in the anchor pullout tests. However, it is difficult to compare the peak axial strain levels of triaxial tests with the horizontal displacement levels of the anchors at ultimate load.

MODEL TEST RESULTS

Typical net load versus displacement plots obtained from the laboratory model tests are shown in Figure 4. For all tests, the ultimate loads were defined as the points at which sudden pullout occurred or the load-displacement plots took an almost linear shape.

The ultimate loads for all tests determined in this manner are shown in Figures 5 and 6. Figure 5 shows the values of Q_u for tests conducted with Soil A (i.e., Test Series 1 and 2); and, similarly, results of the tests conducted using Soil B (i.e., Test Series 3, 4, and 5) are shown in Figure 6. It needs to be pointed

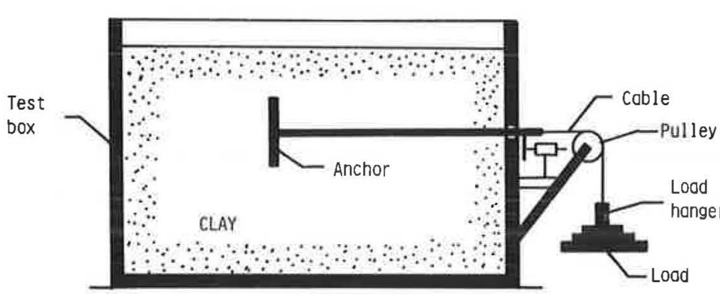

FIGURE 3 Schematic diagram of the model test arrangement in the laboratory.

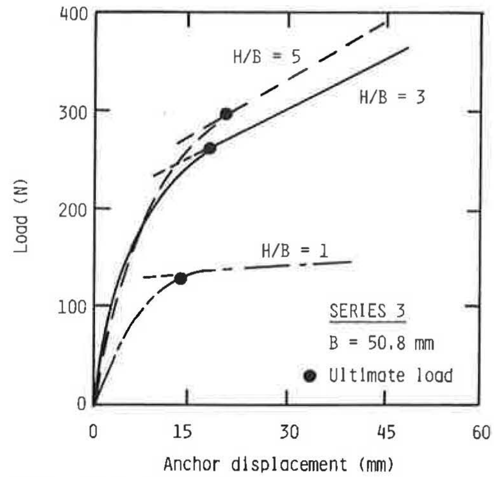

FIGURE 4 Typical load versus displacement plots obtained from the laboratory tests.

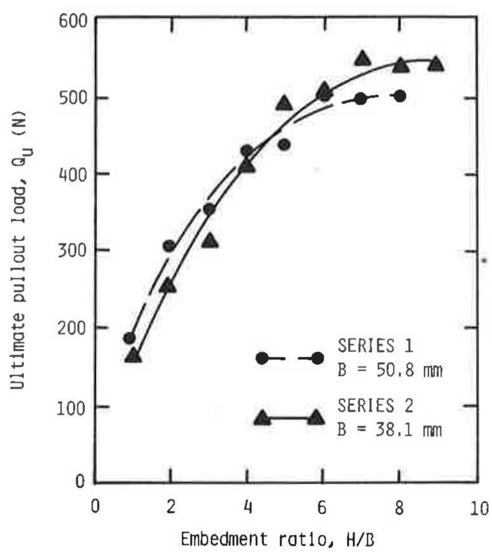

FIGURE 5 Plot of ultimate pullout load versus embedment ratio—Series 1 and 2.

out that the horizontal rod attached to the center of the anchor plate had a large diameter compared with the anchor plate dimensions. For that reason, before the start of the actual model test, the adhesion between the rod and the clay was separately determined. In plotting the variation of the load Q and the anchor displacement, the adhesive force between the soil and the rod at corresponding displacement levels was subtracted from the observed load. The maximum adhesive forces between the rod and the clay determined from the laboratory were 36, 59, 29, 55, and 66 N for Test Series 1, 2, 3, 4, and 5, respectively.

Using these net ultimate load values and Equation 1, the breakout factors at various embedment ratios have been calculated and are shown in Figures 7 and 8. It needs to be pointed out that, for square anchors, $B = L$; hence, Equation 1 takes the form

$$F_c = \frac{Q_u}{B^2 c_u} \tag{4}$$

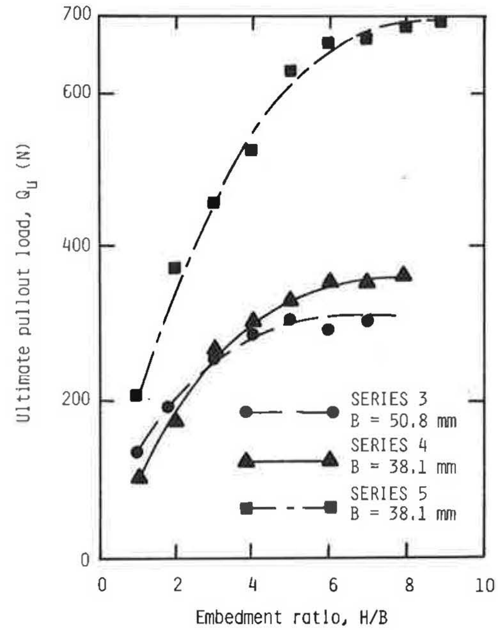

FIGURE 6 Plot of ultimate pullout load versus embedment ratio—Series 3, 4, and 5.

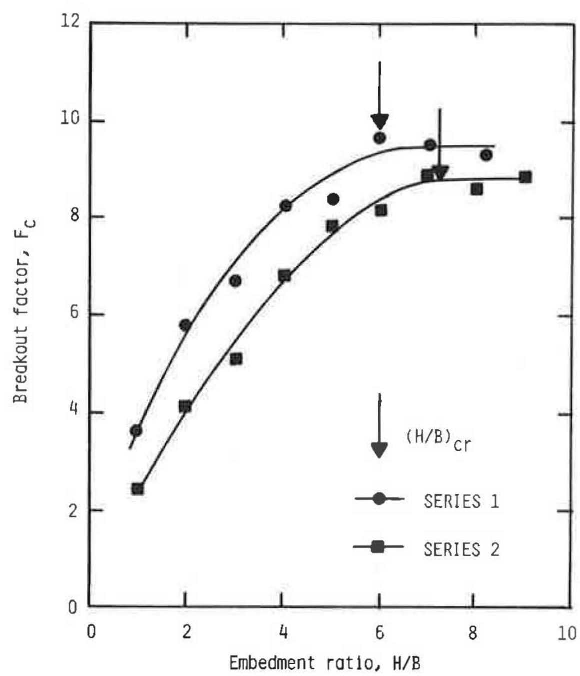

FIGURE 7 Plot of breakout factor (F_c) against embedment ratio—Series 1 and 2.

In evaluating the values of F_c shown in Figures 7 and 8, the average undrained shear strength values (c_u) given in Table 2 have been used. It can be clearly seen from Figures 7 and 8 that the general trend of the variation of F_c with H/B is similar to that obtained by Mackenzie *(7)* as shown in Figure 2. Some scattering of experimental results can well be expected in tests of this type.

It is also of interest to note that the plots of F_c versus H/B for 50.8-mm plates (Test Series 1 and 3) as shown in Figures 7 and 8 are higher than those for 38.1-mm plates. This is because

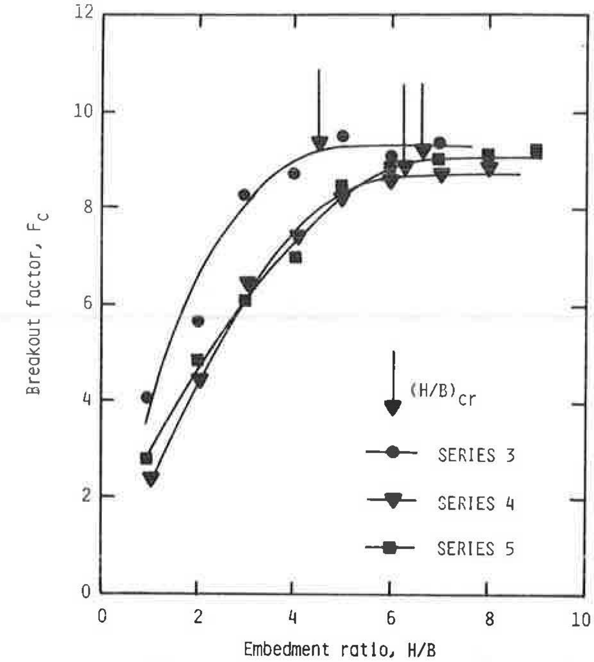

FIGURE 8 Plot of breakout factor (F_c) against embedment ratio—Series 3, 4, and 5.

tests in softer clays were conducted with 50.8-mm plates. The nondimensional breakout factor for softer clays increases more rapidly with H/B up to a maximum value than do those for stronger clays. In other words, for shallow anchors at similar embedment ratios, $\Delta F_c/\Delta(H/B)$ increases with the decrease of c_u.

The critical embedment ratio $[(H/B)_{cr}]$ as estimated from the average plots of all test series is also shown in Figures 7 and 8. These values of $(H/B)_{cr}$ have been plotted against the undrained shear strength of clay in Figure 9. The average plot can be approximated as

$$(H/B)_{cr} = 4.33 + 0.067c_u \leq 7 \tag{5}$$

where c_u is in kN/m^2.

The maximum value of $(H/B)_{cr} = 7$ is consistent with Meyerhof's *(9)* recommendation of 7.5. These values need to be confirmed by large-scale field tests during which extreme

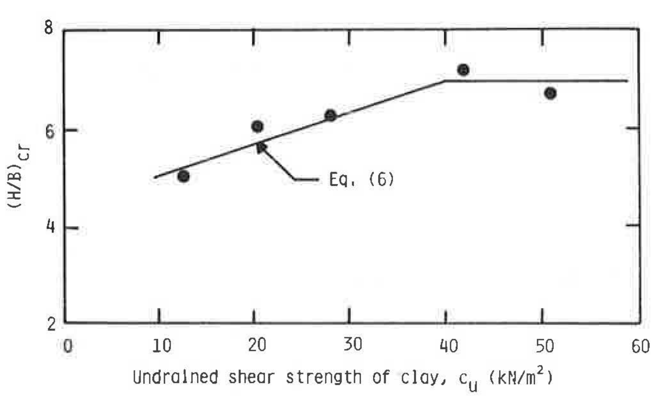

FIGURE 9 Plot of $(H/B)_{cr}$ versus undrained shear strength of clay.

care is taken to assure consistency of soil parameters, which are not available in open literature at the present time.

Figure 10 shows the plot of the maximum value of the breakout factor $F_c = \overline{F_c}$ as obtained from the average plots given in Figures 7 and 8. The magnitude of $\overline{F_c}$ varies between 8.8 and 9.5 with an average value of 9.1. This is similar to the magnitude of the bearing capacity factor $N_c = 9$ as obtained for deep square and circular foundations on saturated clay under compressive loading conditions. Considering the errors involved in laboratory tests of the present type, the value of $\overline{F_c}$ may be conservatively assumed to be about 9.

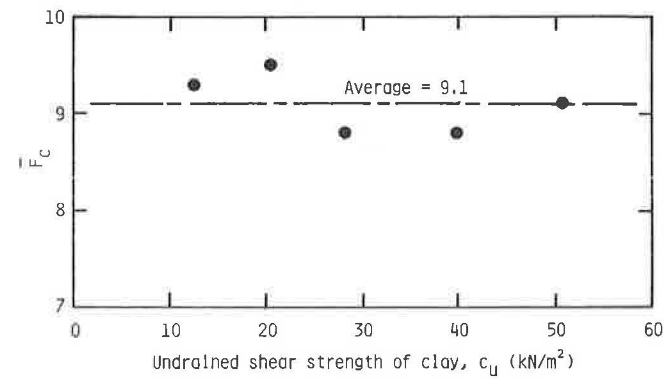

FIGURE 10 Variation of $\overline{F_c}$ with undrained shear strength of clay.

EMPIRICAL PARAMETRIC EVALUATION OF ULTIMATE PULLOUT RESISTANCE

Das *(12)* has used two nondimensional parameters to propose a design procedure for the ultimate pullout resistance of horizontal anchors in saturated clays subjected to vertical pullout forces. A similar procedure can also be adopted for the problem under consideration by defining the two nondimensional parameters as follows:

$$\alpha = F_c/\overline{F_c} \tag{6}$$

and

$$\beta = \frac{H/B}{(H/B)_{cr}} \tag{7}$$

Using the average plots of F_c versus H/B shown in Figures 7 and 8, the magnitudes of several β/α versus β have been calculated, and these have been plotted in Figure 11. Although there is some scattering, the points fall within a narrow range. The average plot can be represented as

$$\frac{\beta}{\alpha} = 0.4 + 0.6\beta$$

or

$$\alpha = \frac{\beta}{0.4 + 0.6\beta} \tag{8}$$

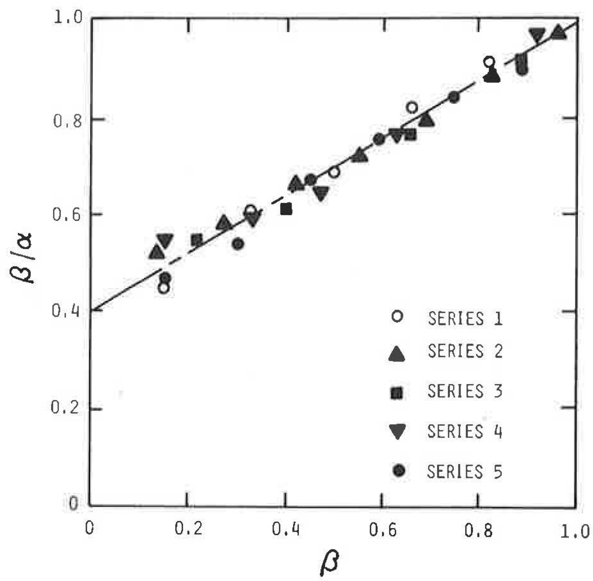

FIGURE 11 Plot of β/α versus β obtained from the average plots shown in Figures 7 and 8.

The preceding equations can now be used to propose a tentative design procedure for estimation of the ultimate pullout resistance of square vertical anchors in clay. The procedure can be modified in the future when the results of more laboratory and field tests are available. A step-by-step approach to the proposed procedure follows:

1. Obtain B, H, and c_u
2. Calculate H/B
3. Using Equation 5, estimate $(H/B)_{cr}$
4. If the actual H/B (Step 2) is greater than $(H/B)_{cr}$ (Step 3), it is a deep anchor so

$$Q_u = \overline{F_c}B^2c_u \cong 9B^2c_u \tag{9}$$

5. If $H/B < (H/B)_{cr}$, obtain β by using Equation 7. With an estimated value of β, use Equation 8 to obtain α. Now

$$Q_u = (\alpha\overline{F_c})(B^2c_u) \cong 9\alpha B^2c_u \tag{10}$$

The ultimate loads obtained by using this method may appear large compared with the present conventional approach for field design. However, many of the conventional procedures have been developed using a simplistic approach without full-scale field tests. Similarly, analogy can be made to the ultimate load determination of vertical anchors embedded in sand. In many design problems, the determination of ultimate load of shallow anchors in sand is made by using the simplistic procedure outlined by Teng *(13)*. However, recent centrifugal tests in the laboratory by Dickin and Leung *(14)* have shown that good agreement exists between their tests and the semiempirical procedure developed by Ovesen and Stromann *(5)* based on laboratory model tests. The ultimate load calculated by using Ovesen and Stromann's procedure gives higher values of Q_u than those obtained by using Teng's conventional procedure. However, with a factor of safety of about 2 the difference between the conventional approach and the present findings should not be too great.

CONCLUSIONS

A number of small-scale laboratory model test results on square vertical anchors in clay have been presented. The following conclusions, based on the present results, can be drawn:

1. The ultimate pullout resistance of an anchor can be expressed in the form of a nondimensional breakout factor (F_c).
2. The magnitude of the maximum breakout factor $(\overline{F_c})$ is approximately equal to 9 for square anchors.
3. The critical embedment ratio increases with the undrained shear strength of clay (Equation 5). However, for stiff clays the magnitude of $(H/B)_{cr}$ is about 7.
4. The variation of the breakout factor with embedment ratio in clays of various consistencies can be expressed by a single nondimensional parametric equation (Equation 8).
5. A tentative empirical procedure for estimation of the ultimate pullout resistance of square vertical anchors has been suggested.

The major contribution of this study is the development of Equations 5 and 8, which show the effect of the undrained shear strength of clay on the critical embedment ratio $[(H/B)_{cr}]$ and the hyperbolic relationship for the normalized parameters of $\alpha = F_c/\overline{F_c}$ versus $\beta = (H/B)/(H/B)_{cr}$. However, the study has some limitations:

- Only two soils have been tested under remolded conditions.
- The effect of gravity has not been modeled. Large-scale field tests are expensive. However, this needs to be done to verify the proposed relationships.
- For each test, the profile of c_u has been assumed constant with depth. In the field this may not be the case; hence, care needs to be taken in using the proposed laboratory-obtained relationships in the field.
- During the present model tests, a delay time of from 5 to 8 min was used after each step load application. This was done to take into account primary creep. However, the test results do not account for possible secondary creep at allowable load levels. In the field a factor of safety of 2 or more will probably be used. At that loading, secondary creep will be substantially minimized.

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