Carrying capacity of anchor slab

Capacité portante de la plaque d'ancrage

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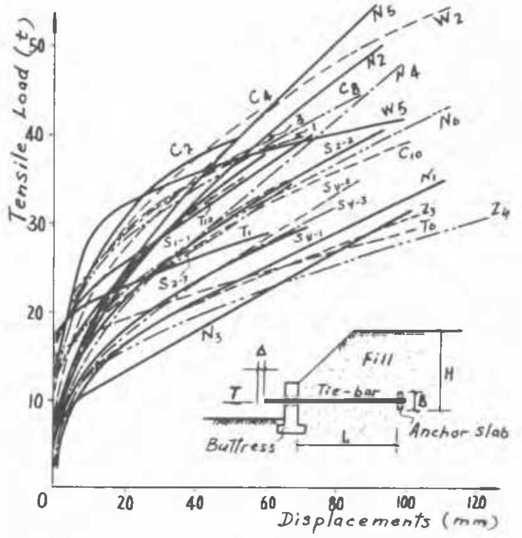
SYNOPSIS In China, anchor slabs are used frequently nowadays in various new type of earth retaining structures. (Lu, 1981) In order to find a rational method for the determination of allowable carrying capacity of anchor slab, six groups of full scale tests were carried out on the sites of anchor slab structure in different part of China. A review and study of the result of these tests are presented in this paper. It is pointed out that neither the formulae based on classical theory nor the formulae so far developed are valid for the practical calculation of anchor slab. There are so many complex influencing factors involved in this problem that a semi-empirical solution is the only way out for the time being. Much more research works are still needed.

IN-SITU TESTS ON ANCHOR SLAB

In order to obtain reliable data regarding the ultimate carrying capacity of anchor slabs, several groups of full scale field tests were carried out under guidance of the authors at six different engineering sites in the period of 1977-1980. The sites of these tests were at Wuchang, Tai-Lan, Zhangchow, Nanping, Siping and Changchun respectively.

The anchor slabs were made of reinforced concrete with bearing areas ranging from 0.5*0.5 to 1.4*1.4 square meter, and the depth of embeddment ranged from 2 to 9 meter. The test arrangements are as shown in Fig.1. Tensile loads were applied to the tie-bar in increments, and every increment of loads was held constant until the displacement reached a stabilized value. The soil properties are as shown in Table 1, and the load-deformation curves are shown in Figures 1 to 3.



H (M)	2	2.3	3			3.3
Test Number	C -2 C -4	S 4-1 S 4-2 S 4-3	T -7 T -12	Z -3 Z -4	N -1 N -2	S 2-1 S 2-2 S 2-3

4	4.9	5.1	5.48	6	6.7	9.06
C -8 C -10	W -2	S 1-1 S 1-2 S 1-3	N -3 N -4	T -1 T -6	W -5	N -5 N -6

Fig.1 Load-deformation Curves of 1m*1m Slabs

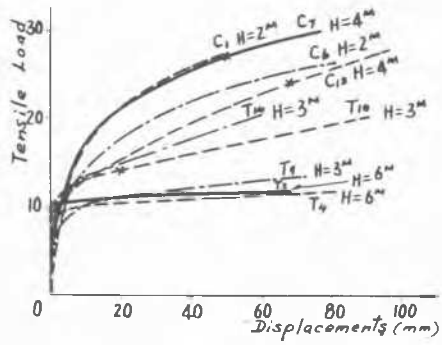


Fig.2 Load-deformation Curves of 0.5m Slabs

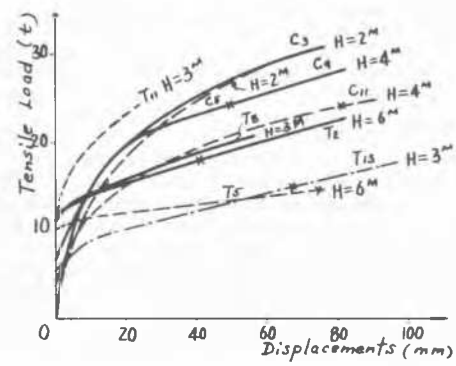


Fig.3 Load-deformation Curves of 0.75m Slabs

DEFINITION AND CRITERIA OF CARRYING CAPACITY

The difinition and criteria of the term "ultimate carrying capacity " may belong to either one of the three different categories, as following:

(1) <u>The absolute ultimate load T_{u1}</u>: It is defined as the load beyond which the soils arround the anchor slab will turn into plastic state and the position of slab will become unstable. This is a logical definition, but the criterion between stable and unstable is not easy to be determined in a field test, because creep pheonomena will enter the test before failure. In order to complete the field test within a reasonable short time, it was decided to consider an increment of load to be "stable" when the rate of displacement slows down to 0.1mm per half hour. Even though with this assumption, the creep deformation had become so large that most of the tests were compelled to stop due to limitation of equipment before the absolute ultimate load could be reached. Apparently, it is not practical for most cases to adopt this kind of definition and criterion.

(2) <u>The customary ultimate load T_{u2}</u>: It is a customary conception that the soil support has failed as soon as the load-deformation curve passes into a fairly straight tangent. Therefore, the customary ultimate load is specified as the load corresponding to the starting point of the final straight line portion of the load-deformation curve. To the authors' knowledge, this definition has been adopted in plate bearing tests and it gives good results in practice.

(3) <u>The ultimate deformation load T_{u3}</u>: The deformation of any structure must have a limit beyond which the structure will start to break or will be unsuitable for its purpose. Therefore, the anchor slab that acts as a part of the structure must also has a limit for its deformation. The ultimate deformation load is defined by the authors as the load that corrsponds to the point of limiting deformation on the load-deformation curve. In current practice, the limiting deformation value for ordinary anchor slabs is specified to be 100mm. To the authors' opinion, all the three foregoing definitions should be considered simultaneously in a carrying capacity analysis, and the one that gives the minimum value should be chosen for design in each case.

(4) <u>The allowable carrying capacities</u> Allowable carrying capacities are obtained from dividing the ultimate capacities by a factor of safety. As can be seen from the load-deformation curves, only 2 tests (T_3, T_4) had reached the absolute ultimate load T_{u1}. It is possible for all of these curves to obtain the customary ultimate loads T_{u2}. And, since most of these tests were carried to deformations quite near to 100mm, it is also possible to obtain the ultimate deformation load T_{u3} by allowing a slight extension of these load-deformation curves. Table 1 gives the result of these analysis. It is interesting to note that if T_{u2} is divided by a factor of safety 2.5 and T_{u3} by a factor of safety 3, the allowable carrying capacities obtained by these two approaches are nearly the same, and most of them are in the range between 10-15t/m^2.

Table 1 Result of Pulling Tests

Location and Backfill Properties	No. of Test	Slab Width B (m)	Depth H(m)	The Customary Ultimate Load			The Ultimate Deformation Load		
				T_{u2}	$\frac{T_{u2}}{B^2}$	$\frac{T_{u2}}{2.5B^2}$	T_{u3}	$\frac{T_{u3}}{B^2}$	$\frac{T_{u3}}{3B^2}$
Changchun	1	0.5	2	24.8	49.6	19.8	34.0	68.0	22.6
γ=2.01t/m^3	3	0.75	2	29.6	52.6	21.0	37.2	66.2	22.0
c=2.0 t/m^2	2	1.0	2	38.0	38.0	15.2	48.0	48.0	16.0
ϕ=29.5^0	7	0.5	4	27.6	55.2	22.1	32.8	65.6	21.8
	9	0.75	4	21.6	38.4	15.4	30.5	54.2	18.1
	8	1.0	4	34.0	34.0	13.6	47.2	47.2	15.7
Tai-Lan	9	0.5	3	12.4	49.6	19.8	15.6	62.4	20.8
γ=1.70t/m^3	8	0.75	3	14.2	25.2	10.1	37.6	66.8	22.3
c=2.0 t/m^2	7	1.0	3	26.0	26.0	10.4	52.0	52.0	17.3
ϕ=26.5^0	3	0.5	6	11.6	46.4	18.6	12.4	49.6	16.5
	2	0.75	6	16.0	28.4	11.4	25.6	45.5	15.1
	6	1.0	6	26.0	26.0	10.4	29.0	29.0	9.6
Zhangchow	5	0.6	3	23.6	39.3	15.7	31.2	52.0	17.3
γ=1.91t/m^3	3	1.0	3	26.0	26.0	10.4	30.8	30.8	10.3
c=2.3 t/m^2 ϕ=29^0	2	1.2	3	24.6	18.5	7.4	36.8	25.5	8.5
Wuchang	2	1.0	4.9	48.0	48.0	19.2	51.5	51.5	17.1
γ=1.98t/m^3	1	1.4	4.9	54.8	28.0	11.2	58.8	30.0	10.0
c=2.3 t/m^2	5	1.0	6.7	37.2	37.2	14.9	42.0	42.0	14.0
ϕ=22.6^0	4	1.4	6.7	50.0	25.5	10.2	69.6	35.5	11.8
Nanping γ=1.8 t/m^3	2	1.0	3.0	38.0	38.0	15.2	51.2	51.2	17.0
c=4.0 t/m^2	3	1.0	5.5	28.0	28.0	11.2	31.6	31.6	10.5
ϕ=22^0	5	1.0	9.1	36.0	36.0	14.4	57.6	57.6	19.2
Siping	4	1.0	2.3	24.8	24.8	9.9	34.0	34.0	11.3
γ=1.98t/m^3	3	0.8	3.3	20.0	31.3	12.5	34.0	53.2	17.7
c=2.0 t/m^2	2	1.0	3.3	30.0	30.0	12.0	43.2	43.2	14.4
ϕ=26.5^0	1	1.0	5.1	36.0	36.0	14.4	44.5	44.5	14.8

ON THE THEORETICAL COMPUTATIONS OF ANCHOR SLAB

(1) <u>Classical formula</u> In the design of anchored bulkheads, the classical formula for computing the carrying capacity of an anchor slab is based on the theory of passive earth pressure

$$T_u=\frac{1}{2}\gamma BH^2(K_p-K_a)\left[1+0.5\,\frac{H\tan\phi}{B}\right] \qquad (1)$$

when H<4.5B; and

$$T_u=\frac{1}{2}\gamma BH^2(K_p-K_a)\left[H^2-(H-B^2)\right](1+0.5\,\frac{H\tan\phi}{B}) \qquad (2)$$

when H>4.5B

in which T_u represents the ultimate carrying capacity of a square anchor slab, B the length of one side of the slab, H the depth of embeddment from the surface of soil to the bottom of the slab, K_p, K_a the coefficients of passive and active earth pressure, γ the unit weight of soil and ϕ the angle of internal friction of soil.

Assuming ϕ to be 25^0 and 30^0, the values of T_u computed from formula (1) and (2) are plotted in Fig. 4 where it can be compared with the results of field tests. It can be seen from Fig.4 that when the depth of embeddment is greater than 3 meters, the computed values from classical formulae are much too high and should not be used.

(2) Formula based on Terzaghi's assumption

Terzaghi(1942) suggested an assumption that the force required to pull a deep embedded anchor slab is approximately equal to the bearing capacity of a footing slab located at the depth (H-B/2). Thus, the formula for T_u may be written as follows

$$T_u = B^2\left[1.3c\,N_c + \gamma(H-B/2)N_q + 0.4\gamma BN_\gamma\right] \qquad (3)$$

in which c represents the cohesion of soil and N_c, N_q, N_γ the bearing capacity factors which were expressed as functions of ϕ.

Assuming $c=2$ t/m^2, $\phi=25^\circ$, the value of T_u computed from formula (3) are plotted in Fig.4. The computed values are also much too high as compared with the results of field tests.

(3) Formula based on model tests

During the seventies, Neely(1973), Das(1977) and Ranjan(1974) had performed model tests separately on anchor slabs in sand. They arrived at the same conclusion that the ultimate carrying capacity may be expressed as

$$T_u = SAM_qB^3 \qquad (4)$$

in which S represents the shape factor, A the coefficient which varies with ϕ, M_q the coefficient which increases with the ratio of embeddment (H/B). All the coefficients S,A, M_q are dimensionless constants which were given in the reports of their model tests. The values of T_u calculated from formula (4) is compared with the results of field tests in Fig. 4, and again it can be seen that the calculated values of formula (4) is too high for deep embedded slabs.

(4) Results of finite element analysis using Duncan-Chang model

The third author, Z.S.Zhang, had performed finite element analysis on some of the tests. In order to avoid the complexity of three dimensional analysis, the stress-deformation of the soil in front of the anchor slab is assumed to be axial symmetrical with respect to the tie-bar on which the pulling force is applied. The stress-strain relationship of soil is assumed to follow the Duncan-Chang model. The results of finite element analysis is shown in Table 2.

Table 2 Result of FEM Analysis

Location	Backfill properties	Depth H (m)	Dimension of slabs B(m)	FEM Result T_u (t)	FEM Result Displ. (cm)
Taie-Lan	$\gamma=1.74$ t/m^3	3.0	0.5	15.0	16.1
			0.75	18.0	8.7
			1.0	30.0	12.8
	c= 2 t/m^2		0.5	18.0	9.3
	$\phi=26.5^\circ$	6.0	0.75	24.0	6.7
			1.0	40.0	10.4
Wu-Chang	$\gamma=1.94$ t/m^3	5.0	1.0	48.0	12.5
			1.4	70.0	12.4
	c=2.7 t/m^2	6.7	1.0	56.0	10.7
	$\phi=22.5^\circ$		1.4	92.0	15.7

It is better than the foregoing formulae, but still differs appreciably from the field test results.

(5) Semi-empirical formula suggested by X.M.Wu

The second author, X.M.Wu, suggested a semi-empirical formula based on the results of field tests:

$$T_u = \frac{\gamma(B^2+1)}{2B}\beta\left[H^2-(H-B)^2\right] + ULf + \mu\gamma A(H-B/2) \qquad (5)$$

in which β represents a coefficient from the empirical data, U the perimeter of tie-bar, L the length of tie-bar, A the side area of the anchor slab, f the unit frictional force along tie-bar, and μ the friction coefficient between soil and concrete. Since this formula is highly empirical, more evidence is needed to clarify whether it is adoptable to most cases.

(6) Discussion

The comparison between the computed values and the results of field tests had arrived at a conculusion that both the classical theory of carrying capacity and the empirical formulae from model tests are unsuitable for deep seated anchor slabs.

The classical theories were derived with the traditional assumption that the vertical compressive stress in soil is equal to its overburden pressure γz, in which z denotes the vertical depth of the point considered. But this assumption holds true only when the vertical compressive stress is uniform over a large area. Since the stress of soil around an anchor slab is non-uniform, local shearing strain in the soil will induce complex stress conditions, and the vertical stress will by no means be equal to γz. In the authors' opinion, this is the main explanation for the discrepancy between classical theory and the field tests.

In case the soil around the anchor slab contract locally during shear, arch action will take place and the vertical stress of the contracted portion will be smaller than γz. This may explain the phenomena in Fig. 4 that the carrying capacity does not increase with depth.

On the other hand, if the soil dilate locally during shear, the vertical stress of the dilated portion will be greater than γz. Murry(1979) reported in the 7th European Conference of SMFE that the apparent angles of friction between soil and reinforcement obtained from the pull-out tests were always greater than was measured in direct shear tests. The authors would rather explain this phenomena by the increase of stress due to dilatancy.

PRACTICAL CONSIDERATIONS AND MONITORED RESULTS

Since anchor slabs are usually used in groups in an engineering project, the effect of mutual influence should be studied.

Only a few model tests have been made on this complex problem and it was found that the effect of mutual influence will reduce the carrying capacity of an individual anchor slab if the spacing is small. But, no definite conclusion has been reached on the spacings and amount of reduction.

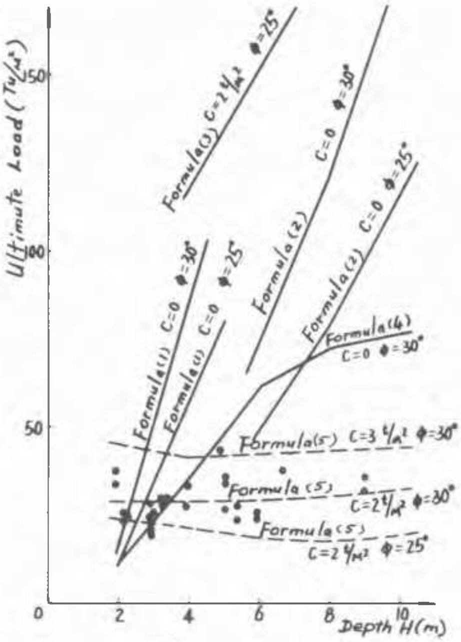


Fig.4 Comparison between computed and field test • results

The influence of earth pressure on the carrying capacity of anchor slabs in the vicinity near the active zone behind a retaining wall is another very complex problem. This problem will arise when a structure with relatively short tie-bars is to be designed, and the present opinions on this problem are still in a state of controversy.

In view of the complexities involved in these practical considerations, it is decided to make judgements not only based on the results of field tests on individual anchor slab, but also in combination with the experiences obtained from the monitored result of practical anchor slab structures.

Table 3 gives the monitored results of four anchor slab structures which were constructed in 1978 and have been monitored for six years during service. In the first year, the stresses and deformations increased slightly due to traffic. And since that, they remained practically unchanged.

It can be seen from Table 3 that the loads on the anchor slabs of the Wu-Bo abutment are relatively small (5.8 -5.9t/m^2), and consequently the actual deformation of this abutment is also small (5mm). While the load on the anchor slabs of the other structures are comparatively larger (10 -16.4t/m^2), the deformation of these structures are also larger (16 - 49mm). It would be reasonable then to suggest an allowable carrying capacity of 10 -15t/m^2, as the deformations associated with this suggestion has been proved to be acceptable. Since this suggestion is in agreement with the result of field test given in Table 1, the conclusion can be drawn as following.

Table 3 Result of Pulling Force Measured

Location	Displ. Top of Column (mm)	Slab Width B (m)	Depth H (m)	Pulling Force Measured T_u (t)	Force on Slab (t/m^2)
Wu-Bo Abutm.	5	1.2	3.02	11.7	5.95
		1.2	7.52	11.4	5.80
Yuan-Z. Abutm.	49	0.8	5.55	8.92	14.0
Bin-He Abutm.	16-	1.2	4.36	9.2-13.3	6.4-9.2
	30	0.8	5.53	6.4-6.44	10.0
Tai-L. Abutm.	35	0.65	1.61	6.91	16.4
		0.65	3.32	4.12	9.75
		0.70	5.12	11.88	24.24
		0.70	6.88	6.80	13.87

CONCLUSION

Owing to the complexities of stresses and deformations in soil around deep embedded anchor slabs, neither the classical theory nor the formulae so far developed could predict the carrying capacity of a deep seated anchor slab with sufficient accuracy. Therefore, at the present stage of development the answer to this problem can only be obtained by field tests. The result of field tests had shown that, in sandy clay fills (PI $<$ 17) compacted to 90% of the Proctor density, the ultimate carrying capacity of an anchor slab embedded at depth of 3 - 10meters is approximately within the range of 30 - 45t/m^2. Adopting a safety factor of 3, the allowable carrying capacity may be suggested tentatively as following:

(1) For anchor slab embedded at depth 3-5m, allow 10 - 12t/m^2;

(2) For anchor slab embedded at depth 5-10m, allow 13 - 15t/m^2;

(3) For anchor slab whose depth of embeddment is less than 3m, the allowable carrying capacity should be determined by the classical formula

$$T=\frac{1}{6}\gamma BH^2(K_p-K_a)(1+0.5\frac{H}{B}\tan\ )$$

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