No. G-1

ON BANK SLIDES IN THE WHANGPOO RIVER

Wang Yen, B.S.; C.E.; Construction Engineer, Whangpoo Conservancy Board

zu vol III 4130

Bank sliding, which hitherto has attracted little attention from engineers, constitutes a serious form of failure in connection with the development of water front in Shanghai Harbour. During the progress of construction of wharves, bundings (= bank revetments) and land reclamations by liquid fill bank slides have occurred on several occasions.

This form of failure is serious in that it not only destroys all the work done above, but the slide carries with it a large mass of the original bank soil underneath, leaving a deep bay at the site and therefore rendering reconstruction much more difficult and expensive. The design of a retaining dyke, or a sheet piling bunding, or a wharf structure may be sound by itself, but the general condition of the bank, which the designer has possibly overlooked and been deceived by, is often found to be not favourable as a substantial support for the structure. With bitter experience and heavy expenses, many a lesson has been learnt in this respect. A brief review here and a suggestion to its solution may be of interest to those who are charged with the responsibility of marine construction.

The Whangpoo is a tidal river emptying into the Yangtze near its mouth. The mean water level at Woosung is + 7.32 ft. (Woosung Horizontal Zero), the ordinary spring tide high water level + 13.14 ft. (W. H. Z.) and the highest high water level + 19.30 (W. H. Z.). The general level of the land on both sides is from + 12 ft. to + 15 ft. (W. H. Z.). The soil is alluvial, strata of clay and fine sand alternating. Borings indicate a rock substratum under Shanghai at 900 ft. depth.

The lower part of the Whangpoo River for a length of about 24 miles constitutes the Shanghai Harbour, of which about 12 miles near Shanghai has been fully developed, while about 10 miles at the entrance and about 2 miles near the upper limit still remain undeveloped. Gradual expansion of commerce and industry call for new sites. Foreshore land reclamation and bunding and wharf construction go abreast with the growth of the port. With all its facilities for dredging and mud disposal the Whangpoo Conservancy Board has consistently helped the riparian owners in the way of land reclamation and bunding.

Around the foreshore property which is to be reclaimed, mud dykes are first built up forming an enclosed basin into which dredged mud in hopper barges is pumped with the aid of water dilution. Then the bunding at the water front may be constructed in the form of a solid wall resting on piling, or of a sheetpiling of wood, steel or concrete tied to a back anchorage, or of a sloping bank surfaced with stone paving and supported underneath with fascine mattresses, or wooden crib work. An open type wharf or a pontoon landing may be added to make the development complete, omitting any reference to the installations on land.

This process of construction has generally been followed in Shanghai Harbour but it is not entirely free from risk. The river bank on which the dyke is built may be a newly accreted foreshore with little bearing strength, or a steep slope affording no foothold for the dyke, or an apparent old and sound foreshore of treacherous soil. Engineers not infrequently find themselves in trouble by seeing a section of the dyke yield and disappear suddenly when it was expected to retain the filling behind.

With the development of trade in recent years, the situation becomes more acute. Large ocean liners calling at the port demand deep water at the wharf front while on the other hand the owners always desire to bring their bunding and warehouses as close to the water as possible. When the distance between the wharf front and the top of bunding becomes too small, the stability of the whole bank is threatened and the inevitable result happens.

The failure of the Tung Ka Doo Wharf in 1922, (Fig. 1 & 2) attracted much attention and led to a thorough investigation of the case. It was found that the concrete bunding would be able to retain the back fill, had the bank as a whole not been over-loaded with a high stack of coal. By this load superimposed on the land close to the bunding, the bank was thrown out of balance, and a large mass, containing the foreshore, the bunding and part of coal above, was torn asunder and launched into the river. There was also evidence that the incident happened instantly without a sign of warning and that the moving mass glided over a definite plane which seemed to offer the least resistance.

Based upon the above, an analysis of the forces which had caused the slide was undertaken. In the cross-section of the bank, a plane of rupture was assumed, governed at several points by observation. Instead of a straight line the plane thus assumed was a curve drawn to depict the actual condition as closely as possible. Above the plane, the mass was divided into a number of prisms each considered standing alone. Then by applying the fundamental principles of earth pressure, the forces acting on each prism could be easily ascertained.

The forces consist of the weight of the prism, the reaction normal to the plane of rupture and the friction and shear along the plane. Analysing the forces into vertical and horizontal components, we get a horizontal resultant force acting in a direction towards the river or in an opposite one depending upon the position of the prism relative to the plane. In the former case the horizontal components tend to move the prism forward while in the latter they act to retard the motion if the prism has been already acted upon by the one next behind. Then by summing up the horizontal components of these prisms, we get two resultant forces - the active and the passive pressure - of the whole cross-section of the bank.

Consider an individual prism abcd (Fig. 3) standing on the plane of rupture ad, which makes an angle α with the horizontal. Let ef represent the weight, P, of the prism, making an angle α with the line hf, normal to the plane of rupture. Let φ be the internal friction angle of the soil. Then the resultant force, R, of the reaction normal to the plane and the friction and shear along the plane will be fg. From the construction, we get the horizontal component eg,

$$H = P \tan (\alpha - \varphi).$$

So long as the value of α is greater than φ, H is active and the prism has the tendency to move forward or to exert a thrust to the next one before it. When α becomes smaller than φ, H reverses its direction and becomes a passive force acting to resist the movement.

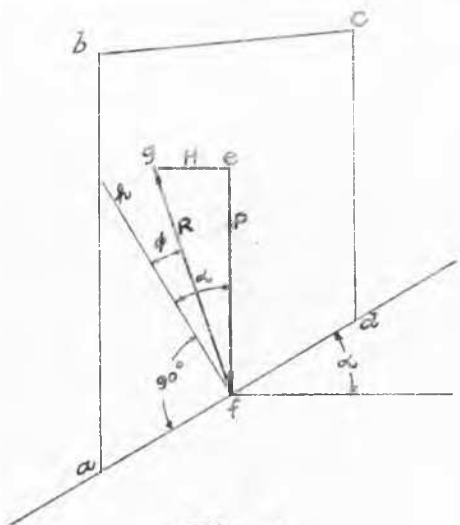


Fig. 3

Now the whole problem hinges upon the value of φ, the friction or sliding angle, which is the most doubtful factor entering into the calculation. In the case of cohesive materials, this angle is not the angle of repose and its value also varies with the pressure.

To determine this factor in local soil, a number of tests were made by the Whangpoo Conservancy Board. The results obtained were fairly consistent for soils under low pressures. By combining with test results of a very similar soil published by the American Society of Civil Engineers (Proceedings Vol. XLVI No. 6 P.923-928), a diagram was made to show the relation between the angle φ and pressure (Fig.4). It is this diagram that makes possible the analysis of the following cases.

Four cases of slides in the Whangpoo River are presented here for example. They occurred at different times and under different conditions of loading. They varied in magnitude and the extent of the damage. In one case the incident occurred at the early stage of construction, in other cases the failure happened after the back fill was completed, and, in still another one, the bunding was completed and put to use for some time when it was overloaded and collapsed. They all however alike took the same general form of failure with the slipping of a mass of soil into the river along a certain plane.

Case I - Reclamation at Jih Hui Chiang. The incident was only of minor degree (Fig. 5). A dyke was built on a concave bank on the Shanghai side in the upper Reach of the Whangpoo. When the dyke reached nearly the full height of about + 18 ft. above W. H. Z. on a fascine mattress footing and a rubble stone toe, the slope of the bank gave way and a section of the finished dyke slipped into the water (Fig. 5). It seemed that the bank was in a critical condition. Only a slight load imposed thereon could have caused a collapse.

Case II. The International Dock Reclamation (Figs. 6 & 7). This reclamation is situated on Pootung shore opposite to Shanghai Settlement, Yangtszepoo Reach. The dyke was built on fairly high ground and of enlarged cross-section. It stood up well. During the process of pumping the liquified mud into the basin, not the slightest sign of weakness could be found. The filling was carried to full height, about + 18.5 ft. above W. H. Z., the superfluous water drained off and the soft mud began to settle. In one morning at Low Water, a slide of about 350 ft. long and of considerable depth was reported (Fig. 7). This is one of the most remarkable slides in the Whangpoo.

Case III. Pootung Point Lower Reclamation (Figs. 8 & 9). The dyke was built in about 18 ft. of water and on fairly level bed. The heavy mud dyke was retained at the front by a wooden crib work resting on three layers of fascine mattress. During the construction, one section of the crib work together with its footing had moved forward towards the river for several feet under the pressure of the mud dyke which was being raised. The movement was successfully arrested by a line of wooden piling at the crib toe through fascine mattresses. Then filling of liquified mud was carried on as usual. After one week of the completion of the filling, another section of the dyke unexpectedly collapsed (Fig. 9).

Case IV. Tungkadoo Wharf. This was a concrete sheet piling bunding with an old wooden piled wharf standing in front for a coal yard on the Pootung side opposite the Chinese Bund. The concrete bunding was secured by tie rods to an anchorage 40 ft. behind and back-filled with dry mud. The bunding and wharf had been put to use for sometime before the slide took place. As referred to before, the accident was caused by overloading of coal immediately behind the bunding.

For each of the four cases mentioned above, calculations are made and results are tabulated (Figs. 10-17). The weight of mud is assumed to be 110 lbs. per cu. ft. above Low Water, while below this level, less the effect of buoyancy, it is reduced to 56 lbs. per cu. ft. The value of φ is taken from the diagram (Fig. 4) corresponding to the pressure. Horizontal forces are graphically represented, and, summing up, the total active and passive pressures acting on the whole cross-section are obtained. These results are quite consistent, notwithstanding the different conditions under which they fail.

The usefulness of the deduction then lies in its applicability to future problems. Consider a river bank and a project for building thereon. The first thing to look into, before actually commencing the design of the structure, is the stability of the bank in a general view. An imaginary plane of rupture is to be drawn on which the mass above, under the burden of proposed structure and liveload, is most likely to slide. If calculation based on this plane is satisfactory, try another plane and repeat the process. After several trials, some practical conclusions may be arrived at and a final decision could be made.

It must however be warned that, as so many uncertainties have to be assumed, a calculation blindly following the procedure might lead to a disastrous result. A familiar knowledge with the local soil as to friction angle φ, is indispensable. Attention must be paid to the fact that the value of φ is greatly decreased in case of extraordinary live load which has to be guarded against in the design. Another weakness of the reasoning lies in the difficulty of determining the pressure on the submerged area in view of buoyancy and the curvature of the plane of rupture. Inasmuch as no better means at the present stage of

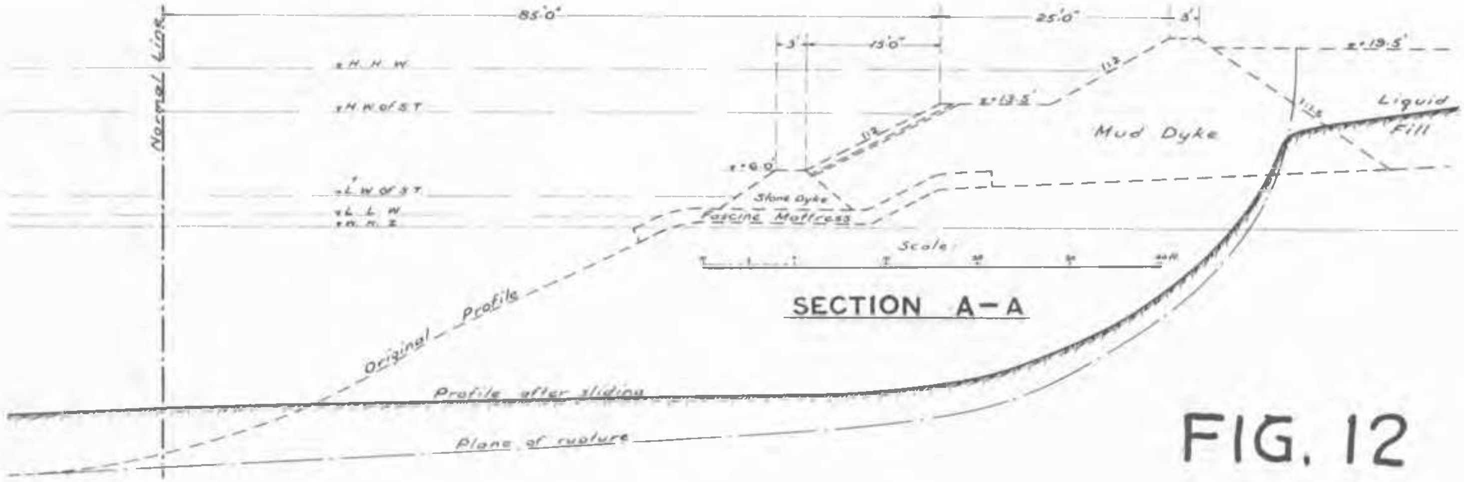
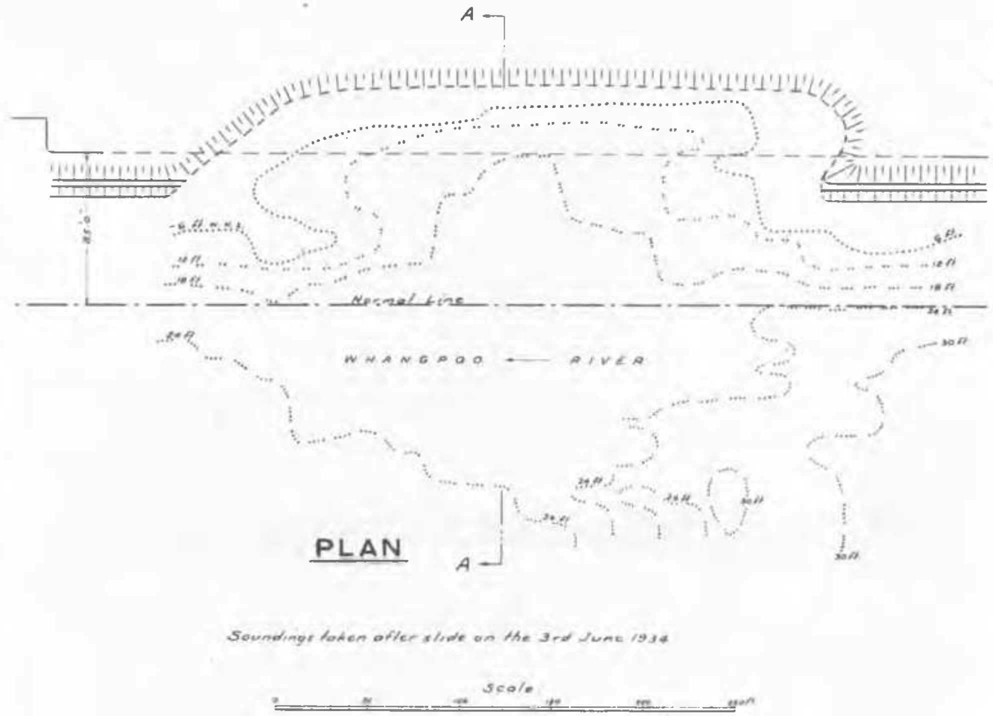


FIG. 12

BANK SLIDE AT INTERNATIONAL DOCK RECLAMATION

Fig. 7. A Close-up View of This Slide.



Weight of Prisms, P in lbs.	Vertical Pressure in lbs. per sq. ft.	Sliding Angle, φ		Pressure, H in lbs.	
		from Diag.	Assumed	Active	Passive
Hydraulic Pressure = ½wh² = ½ × 62.5 × 9.5² =				+ 2,800#	
P1 = 7 × 17 × 110 = 13,100; 4 × 6 × 56 = 1,350 } 14,450#	20 × 110 = 2,200	22.5°	22.5°	+ 12,700#	
P2 = 7 × 17.5 × 110 = 13,500; 7 × 10 × 56 = 3,920 } 17,420#	27.5 × 110 = 3,030	20.8°	21.0°	+ 7,400#	
P3 = 13 × 14.5 × 110 = 20,700; 13 × 16.5 × 56 = 12,050 } 31,750#	31.0 × 110 = 3,410	20.0°	20.0°	+ 6,200#	
P4 = 12 × 11 × 110 = 14,520; 12 × 22.5 × 56 = 15,100 } 29,620#	32.5 × 110 = 3,575	19.7°	19.5°		− 1,300#
P5 = 15 × 7 × 110 = 11,550; 15 × 24.5 × 56 = 20,600 } 32,150#	31.5 × 110 = 3,465	19.8°	20.0°		− 7,800#
P6 = 10 × 3.5 × 110 = 3,850; 10 × 25.5 × 56 = 14,270 } 18,120#	29 × 110 = 3,190	20.5°	20.5°		− 5,400#
P7 = 10 × 25 × 56 = 14,000#	25 × 110 = 2,750	21.2°	21.0°		− 4,200#
P8 = 10 × 18.5 × 56 = 10,350#	8.5 × 62 = 527; 18.5 × 110 = 2030 } 2,560	21.6°	21.5°		− 3,300#
P9 = 10 × 14.5 × 56 = 8,120#	12.5 × 62 = 775; 14.5 × 110 = 1595 } 2,370	22.0°	22.0°		− 2,700#
P10 = 10 × 10 × 56 = 5,600#	15.5 × 62 = 960; 10 × 110 = 1100 } 2,060	22.8°	23.0°		− 2,000#
P11 = 10 × 5.5 × 56 = 3,080#	21 × 62 = 1300; 5.5 × 110 = 600 } 1,900	23.2°	23.0°		− 1,100#
P12 = 15.5 × 3.5/2 × 56 = 1,515#	25 × 62 = 1,550; 2 × 110 = 220 } 1,770	23.6°	23.5°		− 600#
				Σ+H 29,100#	Σ−H 28,400#

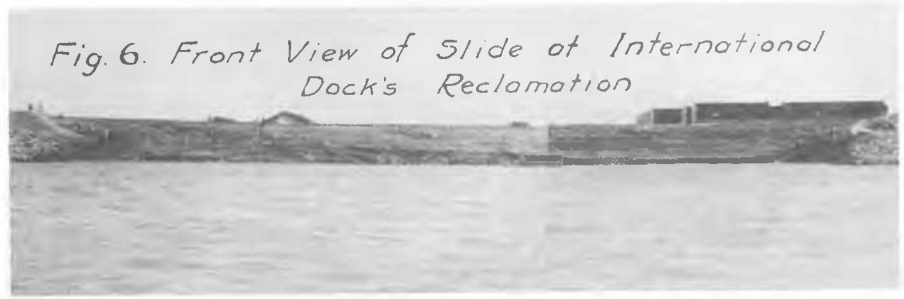


Fig. 6. Front View of Slide at International Dock's Reclamation

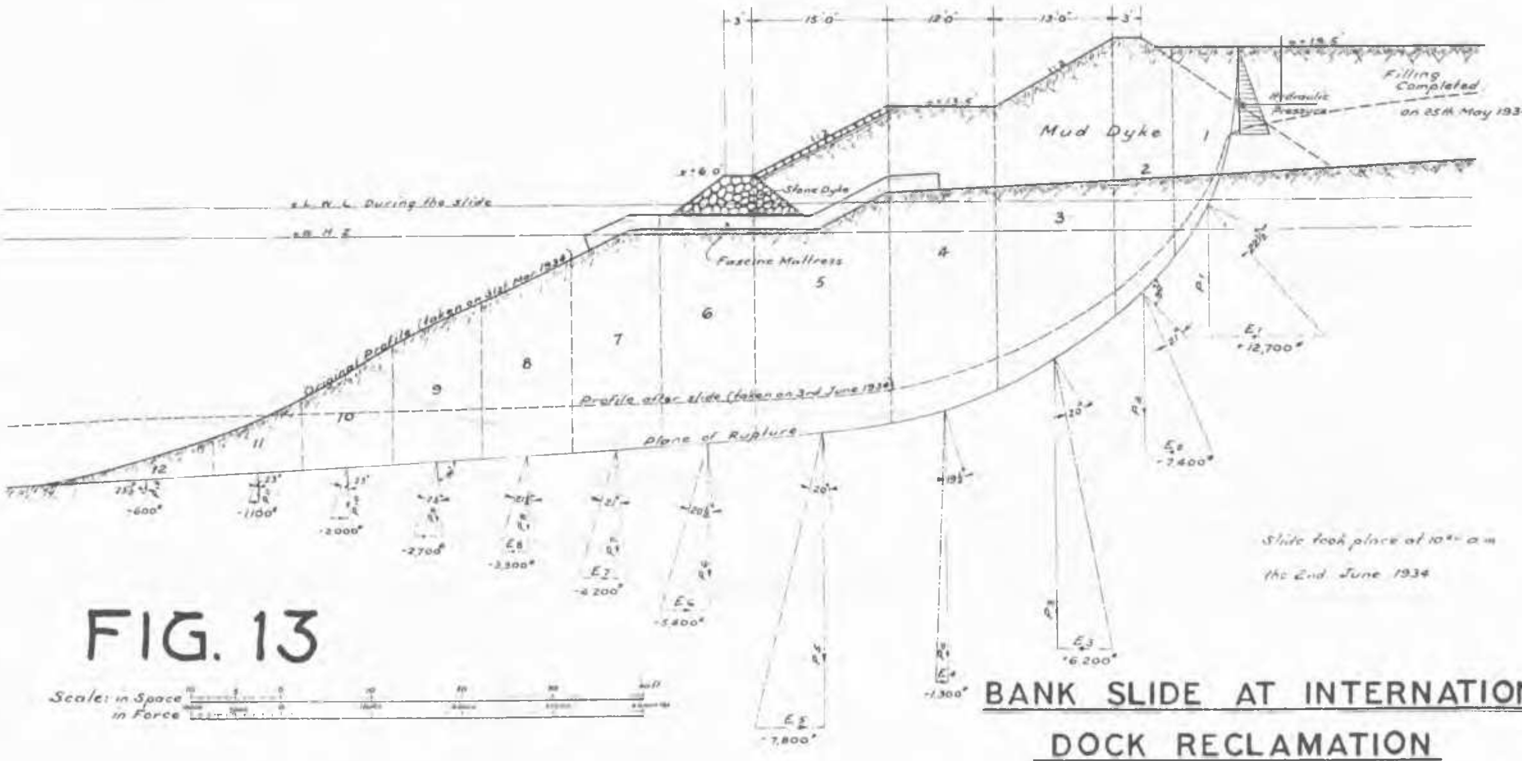
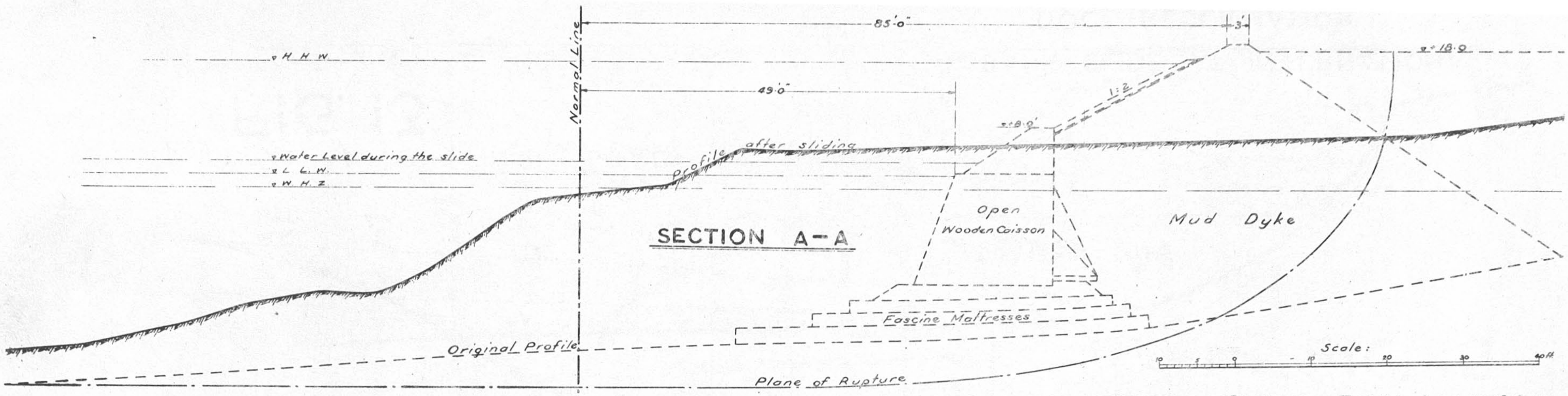


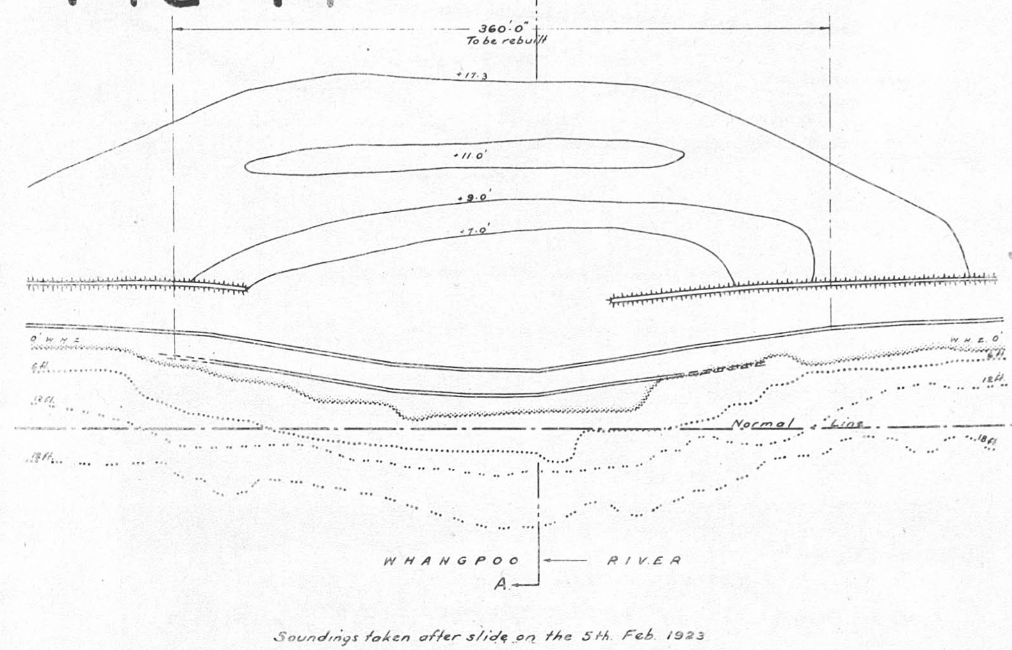
FIG. 13

BANK SLIDE AT INTERNATIONAL DOCK RECLAMATION



BANK SLIDE AT POOTUNG POINT LOWER RECLAMATION

FIG. 14



PLAN

Scale:

Fig. 8. Front View of Pootung Point Slide.

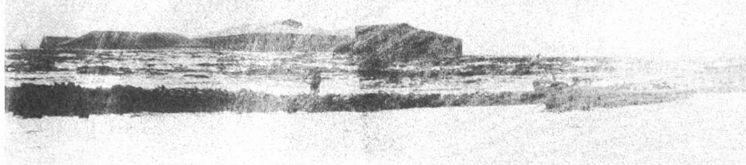


Fig. 9. Side View of Pootung Point Slide.

Weight of Prisms, P in lbs	Vertical Pressure in lbs per sq. ft.	Sliding Angle, φ from Diag.	Sliding Angle, φ Assumed	Pressure, H in lbs Active	Pressure, H in lbs Passive
Hydraulic Pressure = ½wh² = ½ × 62.5 × 11² =				+ 3,800#	
P1 = 14.5×4.5×110 = 7,200; 5×3/8×56 = 400 } 7,600#	15 × 110 = 1,650	24.0°	24.0°	+ 6,800#	
P2 = 14.5×8×110 = 12,800; 10×8×56 = 4,500 } 17,300#	24.7 × 110 = 2,720	21.7°	21.5°	+ 7,700#	
P3 = 15×9×110 = 14,900; 17×9×56 = 8,600 } 23,500#	32 × 110 = 3,520	20.0°	20.0°	+ 5,100#	
P4 = 13×11×110 = 15,700; 22×11×56 = 13,600 } 29,300#	35.5 × 110 = 3,910	19.5°	19.5°	+ 1,900#	
P5 = 9×11×110 = 10,600; 23.2×13×56 = 17,100 } 27,700#	33 × 110 = 3,630	19.8°	20.0°		− 2,900#
P6 = 4.5×(5+10½)×130 = 3,800; 18×15.5×70 = 19,500; 10×18×56 = 10,100 } 33,400#	20.5×130; 10×110 } = 3,760	19.6°	19.5°		− 8,200#
P7 = 10×23.5×56 = 13,200#	10×110; 19×62.5 } = 2,290	22.1°	22.0°		− 4,800#
P8 = 5.3×20×56 = 5,900#	5.3×110; 24.2×62.5 } = 2,100	22.8°	23.0°		− 2,500#
P9 = 4.3×20×56 = 4,800#	4.3×110; 25.2×62.5 } = 2,000	23.0°	23.0°		− 2,200#
P10 = 3.1×20×56 = 3,500#	3.1×110; 26.4×62.5 } = 2,000	23.0°	23.0°		− 1,600#
P11 = 1.2×33.5×56 = 2,300#	1.2×110; 28.3×62.5 } = 1,900	23.2°	23.0°		− 1,000#
				Σ+H 25,300#	Σ−H 23,200#

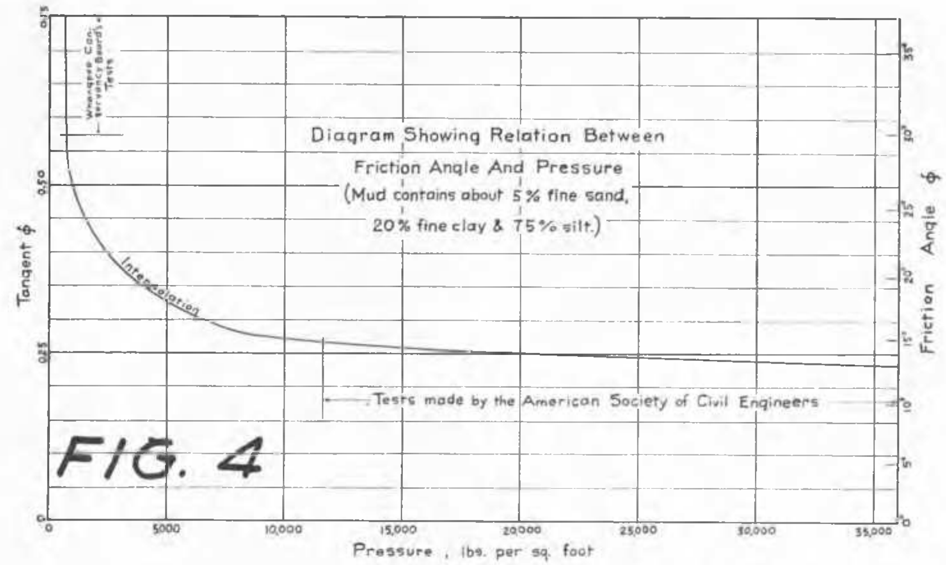
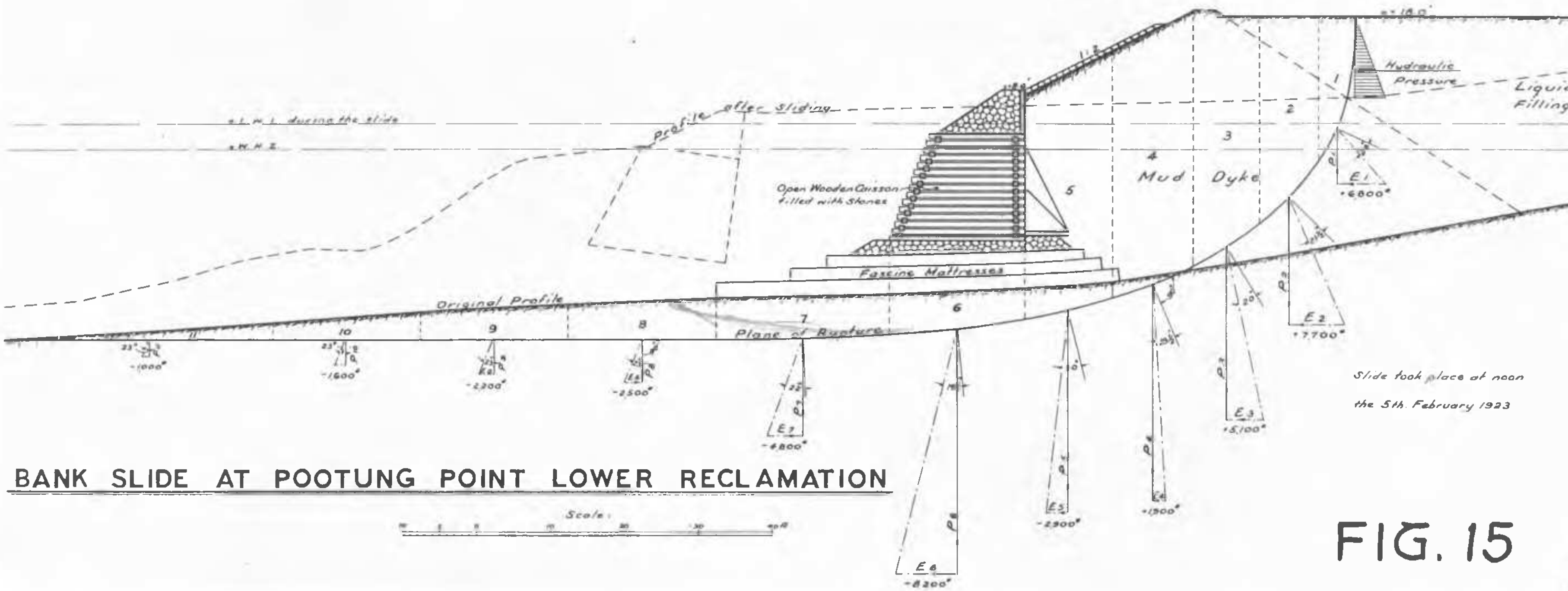


FIG. 4



BANK SLIDE AT POOTUNG POINT LOWER RECLAMATION

FIG. 15

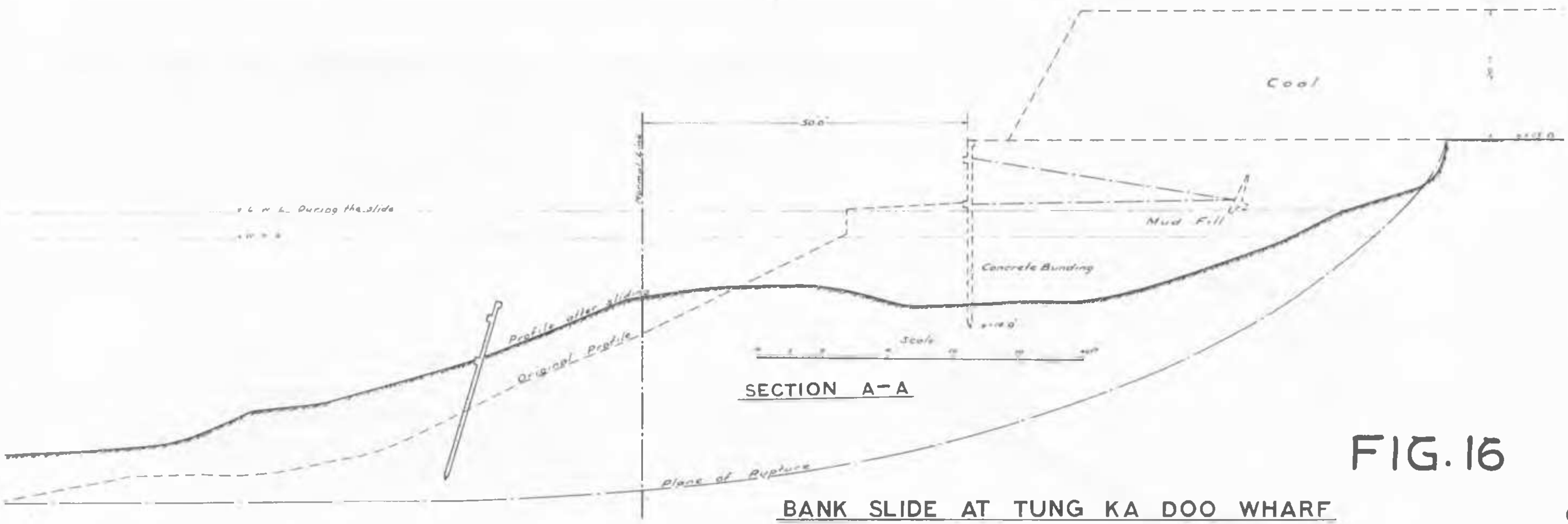


FIG. 16

BANK SLIDE AT TUNG KA DOO WHARF

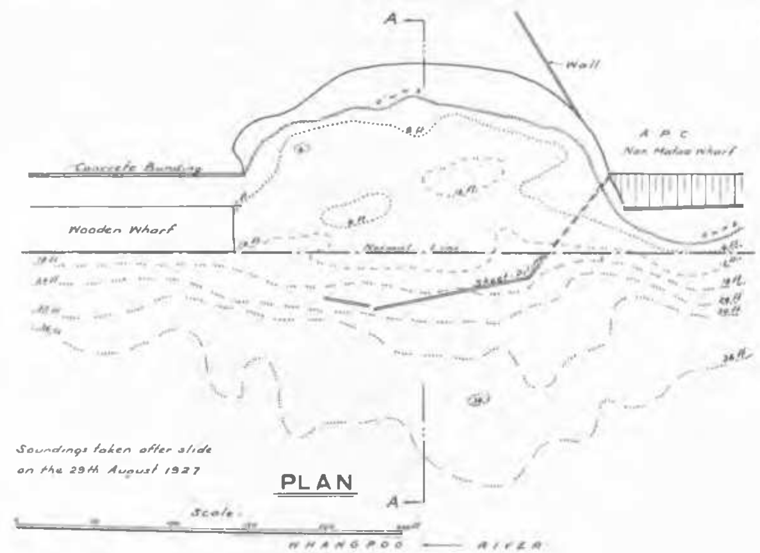


Fig. 1. Bank Slide at Tung Ka Doo Wharf.

G-1

Weight of Prisms, P in lbs		Vertical Pressure in lbs per sq. ft		Sliding Angle φ		Pressure, H in lbs.	
				from Diag.	Assumed	Active	Passive
Coal	Pressure	=				+ 1,880#	
$P_1 = \left.\begin{matrix}20.5 \times 7.3 \times 60 \\ 7 \times 7.3 \times 110\end{matrix}\right\}$ =	14,600#	$\left.\begin{matrix}20.5 \times 60 \\ 7 \times 110\end{matrix}\right\}$ =	2,000	23.0°	23.0°	+ 9,000#	
$P_2 = \left.\begin{matrix}20.5 \times 8.7 \times 60 \\ 11 \times 8.7 \times 110 \\ 4 \times 8.7 \times 56\end{matrix}\right\}$ =	23,200#	$\left.\begin{matrix}20.5 \times 60 \\ 15 \times 110\end{matrix}\right\}$ =	2,880	21.2°	21.0°	+ 8,800#	
$P_3 = \left.\begin{matrix}20.5 \times 10 \times 60 \\ 11 \times 10 \times 110 \\ 11.5 \times 10 \times 56\end{matrix}\right\}$ =	30,900#	$\left.\begin{matrix}20.5 \times 60 \\ 22.5 \times 110\end{matrix}\right\}$ =	3,700	19.7°	20.0°	+ 9,100#	
$P_4 = \left.\begin{matrix}20.5 \times 10 \times 60 \\ 11 \times 10 \times 110 \\ 17.5 \times 10 \times 56\end{matrix}\right\}$ =	31,000#	$\left.\begin{matrix}20.5 \times 60 \\ 28.5 \times 110\end{matrix}\right\}$ =	4,360	18.7°	19.0°	+ 5,800#	
$P_5 = \left.\begin{matrix}20.5 \times 10 \times 60 \\ 11 \times 10 \times 110 \\ 23 \times 10 \times 56\end{matrix}\right\}$ =	37,300#	$\left.\begin{matrix}20.5 \times 60 \\ 34 \times 110\end{matrix}\right\}$ =	4,970	18.1°	18.0°	+ 4,700#	
$P_6 = \left.\begin{matrix}20.5 \times 10 \times 60 \\ 11 \times 10 \times 110 \\ 27.5 \times 10 \times 56\end{matrix}\right\}$ =	39,800#	$\left.\begin{matrix}20.5 \times 60 \\ 38.5 \times 110\end{matrix}\right\}$ =	5,460	17.6°	17.5°	+ 2,500#	
$P_7 = \left.\begin{matrix}20.5 \times 5.5 \times 60 \\ 11 \times 17 \times 110 \\ 27.5 \times 10 \times 56\end{matrix}\right\}$ =	58,300#	$\left.\begin{matrix}10 \times 60 \\ 43.5 \times 110\end{matrix}\right\}$ =	5,380	17.7°	17.5°	+ 300#	
$P_8 = 38 \times 18.5 \times 56$ =	39,400#	38×110 =	4,180	19.1°	19.0°		− 4,100#
$P_9 = 33.5 \times 15 \times 56$ =	28,200#	$\left.\begin{matrix}33.5 \times 110 \\ 7 \times 62.5\end{matrix}\right\}$ =	4,100	19.2°	19.0°		− 4,800#
$P_{10} = 27.5 \times 15 \times 56$ =	23,100#	$\left.\begin{matrix}27.5 \times 110 \\ 14.5 \times 62.5\end{matrix}\right\}$ =	3,900	19.5°	19.5°		− 5,800#
$P_{11} = 21 \times 20 \times 56$ =	23,500#	$\left.\begin{matrix}21 \times 110 \\ 22.5 \times 62.5\end{matrix}\right\}$ =	3,720	19.7°	19.5°		− 6,700#
$P_{12} = 13 \times 20 \times 56$ =	14,600#	$\left.\begin{matrix}13 \times 110 \\ 31.5 \times 62.5\end{matrix}\right\}$ =	3,400	20.0°	20.0°		− 4,900#
$P_{13} = 5 \times 30 \times 56$ =	8,400#	$\left.\begin{matrix}5 \times 110 \\ 40 \times 62.5\end{matrix}\right\}$ =	3,050	20.7°	20.5°		− 3,000#
$P_{14} = 3.5 \times 32.5 \times 56$ =	6,400#	$\left.\begin{matrix}3.5 \times 110 \\ 42 \times 62.5\end{matrix}\right\}$ =	3,050	20.7°	21.0°		− 2,300#
						Σ+H 42,080#	Σ−H 31,600#

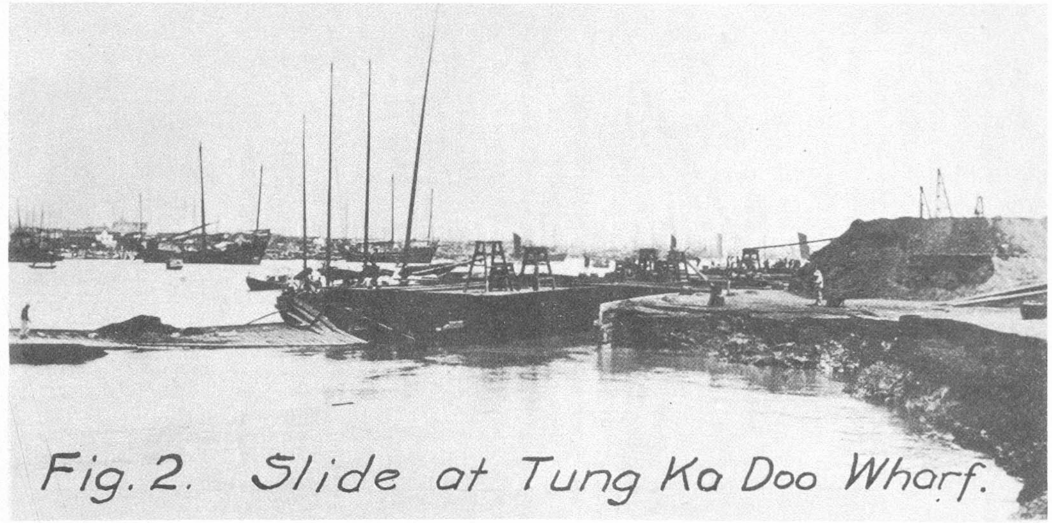
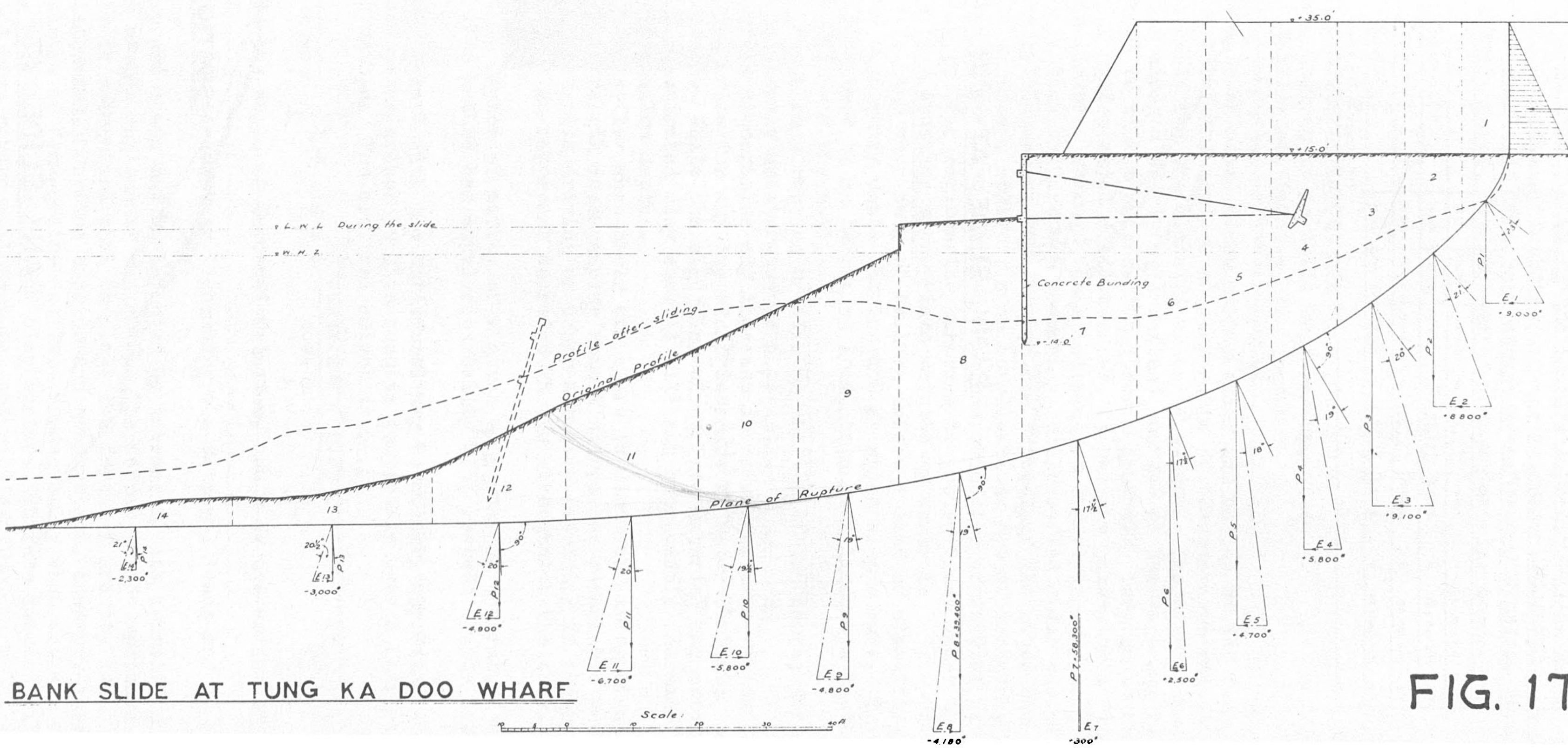


Fig. 2. Slide at Tung Ka Doo Wharf.



BANK SLIDE AT TUNG KA DOO WHARF

FIG. 17

Weight of Prisms, P in lbs.	Vertical Pressure in lbs per sq. ft.	Sliding Angle φ From Diag.	Sliding Angle φ Assumed	Pressure H in lbs Active	Pressure H in lbs Passive
$P_1 = 9 \times 5 \times 110 = 4{,}950^{\#}$	$10 \times 110 = 1{,}100$	26.0°	26.0°	+ 2,900#	
$P_2 = \left.\begin{matrix} 12 \times 9 \times 110 = 11{,}880 \\ 3 \times 8 \times 56 = 1{,}340 \end{matrix}\right\} = 13{,}220^{\#}$	$15 \times 110 = 1{,}650$	23.9°	24.0°	+ 3,000#	
$P_3 = \left.\begin{matrix} 7.5 \times 10 \times 110 = 8{,}250 \\ 8.5 \times 10 \times 56 = 4{,}760 \end{matrix}\right\} = 13{,}010^{\#}$	$16 \times 110 = 1{,}760$	23.5°	23.5°	Neutral	
$P_4 = \left.\begin{matrix} 4 \times 10 \times 110 = 4{,}400 \\ 12.5 \times 10 \times 56 = 7{,}000 \end{matrix}\right\} = 11{,}400^{\#}$	$16 \times 110 = 1{,}760$	23.5°	23.5°		−1,000#
$P_5 = 13.5 \times 10 \times 56 = 7{,}560^{\#}$	$\left.\begin{matrix} 13.5 \times 110 \\ 2 \times 62.5 \end{matrix}\right\} = 1{,}610$	24.0°	24.0°		− 900#
$P_6 = 12.5 \times 10 \times 56 = 7{,}000^{\#}$	$\left.\begin{matrix} 12.5 \times 110 \\ 6 \times 62.5 \end{matrix}\right\} = 1{,}750$	23.5°	23.5°		−1,100#
$P_7 = 9.5 \times 10 \times 56 = 5{,}320^{\#}$	$\left.\begin{matrix} 9.5 \times 110 \\ 11.5 \times 62.5 \end{matrix}\right\} = 1{,}760$	23.5°	23.5°		−1,000#
$P_8 = 5 \times 10 \times 56 = 2{,}800^{\#}$	$\left.\begin{matrix} 5 \times 110 \\ 17.5 \times 62.5 \end{matrix}\right\} = 1{,}640$	23.9°	24.0°		− 700#
$P_9 = 1.5 \times 26 \times 56 = 2{,}520^{\#}$	$\left.\begin{matrix} 1 \times 110 \\ 25 \times 62.5 \end{matrix}\right\} = 1{,}670$	23.9°	24.0°		− 700#
				Σ·H 5,900#	Σ·H 5,400#

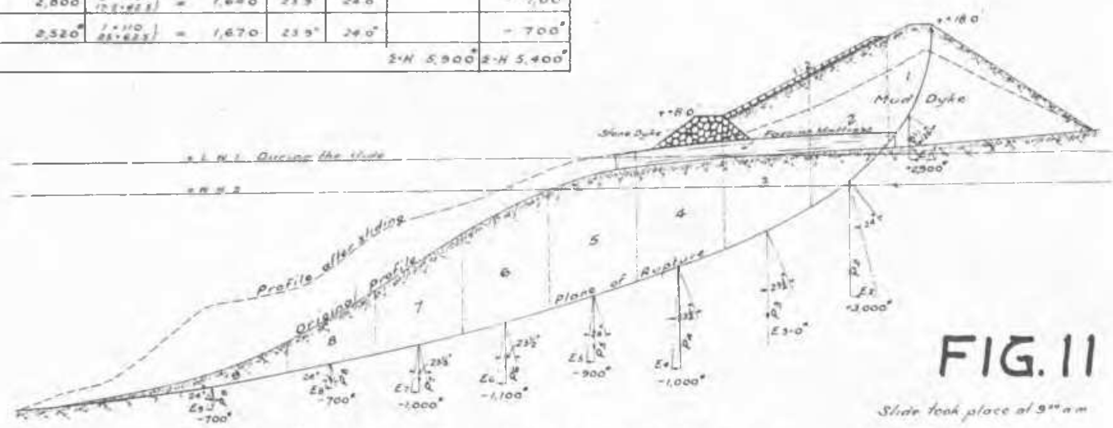
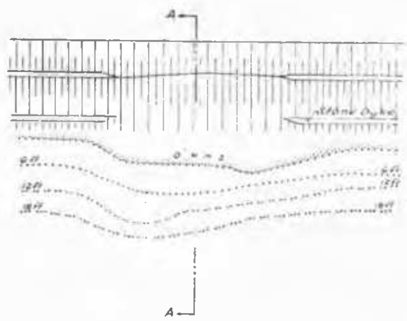


FIG. 11

Slide took place at 9 a.m. the 29th August 1927

BANK SLIDE AT JIH HUI CHIANG

Scale:



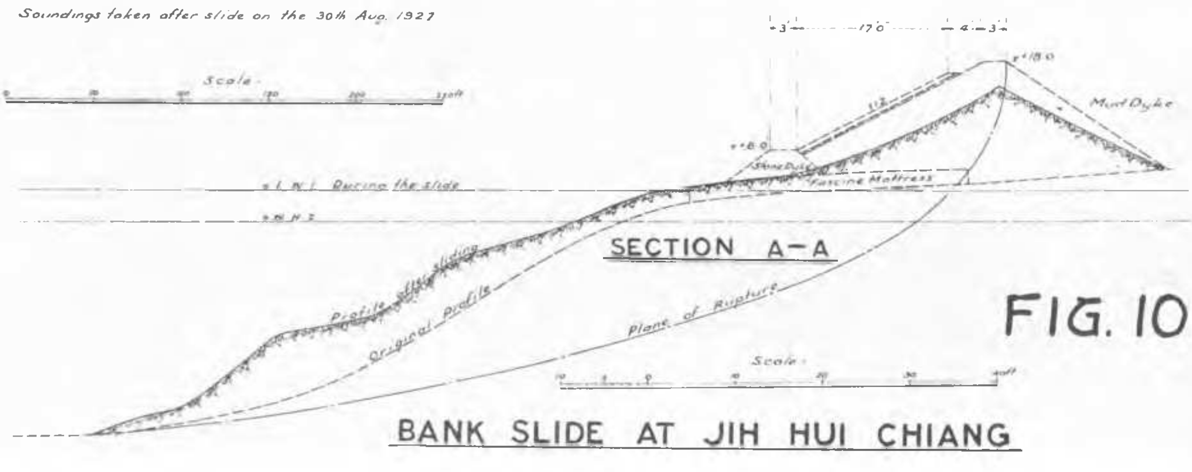
WHANGPOO ⟶ RIVER

PLAN

Soundings taken after slide on the 30th Aug. 1927

Scale

Fig. 5. Reclamation at Jih Hui Chiang



SECTION A–A

FIG. 10

Scale:

BANK SLIDE AT JIH HUI CHIANG

development of soil mechanics is available for the treatment of a problem of this nature, the simple cut and try method as outlined above proves to be quite helpful in many practical cases.

Indebtedness is acknowledged to Dr. H. Chatley, Engineer-in-Chief, Whangpoo Conservancy Board, for his direction in preparing the paper and his permission for using the Board's records, and to Mr. H. F. Meyer, formerly Construction Engineer, Whangpoo Conservancy Board (Now Chief Engineer & General Manager, Free Port of Copenhagen, Denmark), who made numerous soil tests and advanced the theory of slide (See "Mechanics of sheet piling", by H. Chatley & H. F. Meyer, Proceedings of the Engineering Society of China, 1924-1925).

No. G-2

THE CONTROL OF LANDSLIDES

Robert G. Hennes, Instructor in Civil Engineering,
University of Washington, Seattle, Washington

see vol III pp 130

A landslide may be defined as the failure of an earth slope mainly under the action of its own weight, in which the displacement has both vertical and horizontal components of considerable magnitude. The manner of failure serves as a convenient basis for the classification of slides. The displacement of a body of soil moving essentially as a unit along a fairly well defined surface of cleavage may be termed rupture. If the shear is not restricted to such a surface but spreads generally throughout some zone of disturbance in the soil in the form of differential displacements, it has the nature of a flow. When the failure occurs less in shear than in flexure, through the withdrawal of underlying support, the result is collapse. While the nature of the initial failure can generally be classified under one of these three headings, the conditions immediately succeeding may involve one or both of the other two types of failure. From the standpoint of the engineer, however, such complications are of secondary importance, inasmuch as the ends of slide control work are achieved when the condition of initial failure is prevented from materializing. It is sufficient for engineering purposes to group slides under the three headings of rupture, flow and collapse in discussing appropriate methods of control.

Regardless of the type of failure, however, the very fact of failure infers a loss of stability; an existing slope has a factor of safety greater than unity until the instant at which slip occurs. This reduction in the factor of safety is traceable either to an increased load, to a less favorable distribution of stress, or to a decrease in the strength of the soil.

An increase in the load may occur through new construction or fill on the top of the slope; through snow or ice loads; or through saturation of previously unsaturated sand pockets.

A decrease in the strength of cohesive soils accompanies any increase in water content. Thick clay beds under heavy pressures require years or centuries for any important decrease in water content; the reversal of the process involves the same factors. While the sloughing of surface material may often be attributed to softening, the water content of a saturated clay subsoil will not appreciably increase during a single rainy season beyond relatively shallow depths.

All clay in their natural state will become softer when their structure is disturbed at constant water content. In some clays this decrease in strength accompanying remolding reaches remarkable proportions; and in such clays it is conceivable that pile driving or other activity resulting in severe vibration or distortion of the clay may result in an important decrease in the strength of the soil.

Rupture in Homogeneous Soil. 1. Analysis. The factor of safety of a given slope may be determined by the modified Swedish method. No other published method has equal flexibility with respect to the inclusion of complicating factors.

The critical height of a slope making an angle θ with the horizontal may be quickly approximated by the following semi-empirical formula, devised by the author to give results comparable over a fair range of values with those obtained by the Swedish method. The critical height is

$$h = \frac{4\,c}{w \cos\varphi\ (1 - \cot\theta \tan\varphi)}\left[1 + \sin\varphi + \frac{\cos\theta\,(2\theta + 0.2)}{\cos\varphi}\right]$$

where c is the coefficient of cohesion, φ is the angle of internal friction, and w is the weight of a unit volume of soil.

2. Control. Stresses in the subsoil may be decreased by lightening the external loads and by flattening the slope.

The ability of the soil to carry the imposed loads may be improved by increasing its cohesive properties, and by providing additional support at the regions of weakness. The former is accomplished by decreasing the water content of the soil. It is recognized that in clays the force of gravity is insufficient to provide drainage of interstitial water. Surface tension and evaporation, however, have been found to be effective agents in the drainage of fine-grained soils. Consequently when dry air passes through drill holes and tunnels, evaporation will drain the exposed clay surface; surface tension will seek to replenish this loss by drainage of the more remote soil regions, and will eventually result in appreciably increased cohesion. Ventilation of the necessary ducts may be either forced or natural; the choice will depend, among other things, upon the permeability of the clay, as there is no purpose in evaporating the water faster than it can be replenished through the capillaries.

The second method of increasing the stability of a given slope is by providing auxiliary support. This may be done by the use of piles. There are two criteria for the design of pile reinforcement.

A. The spacing of piles must be less than that which would permit soil rupture between piles.