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# THE STABILITY OF A SLURRY TRENCH IN COHESIONLESS SOILS

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N. Morgenstern* and I. Amir-Tahmasseb†

SYNOPSIS

Hydrostatic pressure, arching of the soil and electro-osmotic forces have each been suggested as the dominant factor to account for stability of trench excavations in cohesionless soils supported by clay slurries. The Authors suggest that the most important mechanism is the hydrostatic pressure of the slurry. However, the increase in density of the slurry due to the suspension of cuttings must be considered in computing this hydrostatic pressure.

While a concrete diaphragm cut-off was under construction at Pierre-Bénite, France, an unexpected flood occurred, causing several slips in the trench excavation. The analysis of these slips is presented and it confirms that the stability of a slurry trench in cohesionless soil can be accounted for provided that the correct density is used in computing the hydrostatic pressure of the slurry.

La pression hydrostatique, l'effet d'arc du sol et les forces électro-osmotiques ont tous été suggérés comme facteur dominant pour estimer la stabilité des excavations de tranchées dans les sols sans cohésion supportés par des boues d'argile. Les auteurs suggèrent que le mécanisme le plus important est la pression hydrostatique de la boue. Pourtant l'accroissement en densité de la boue du fait de la suspension de sédiments doit être pris en compte quand on calcule la pression hydrostatique.

Alors qu'une parafouille diaphragme en béton était en construction à Pierre-Bénite, en France, il s'est produit une inondation inattendue occasionnant plusieurs glissements dans l'excavation de la tranchée. L'analyse présentée de ces glissements confirme qu'on peut justifier la stabilité d'une tranchée de boue dans un sol sans cohésion pourvu qu'on se serve de la densité correcte en calculant la pression hydrostatique de la boue.

## INTRODUCTION

The use of clay slurries to support trenches during their excavation has been the subject of considerable discussion in recent years. Although many successful diaphragm walls have been built in trenches supported by clay suspensions, an understanding of the stability of the wall of the trench remains obscure. This is particularly the case for excavation in cohesionless soils.

It is generally agreed that the clay slurry, usually a bentonite suspension, forms an impermeable layer at the interface between the slurry and the intact soil. The mud may then be considered to be a non-penetrating fluid and support of the excavation is undoubtedly achieved by the action of the hydrostatic pressure of the slurry on the impermeable face of the excavation. The question arises whether the hydrostatic pressure of the slurry is the only important factor influencing the stability of the trench sides.

A theory of the stability of trenches full of fluid mud has been suggested by Nash and Jones (1963) based upon the equilibrium between the hydrostatic force of the mud and the force required to restrain a wedge of soil from sliding. It is of interest to extend this analysis, in the case of cohesionless soils, to consider arbitrary levels of ground water and slurry and then discuss possible factors that have been omitted from the analysis.

As shown in Fig. 1, a wedge of soil inclined at $\alpha$ to the horizontal is assumed to be on the verge of sliding. The wedge has a height $H$, and is composed of cohesionless soil with a bulk density $\gamma$ and angle of shearing resistance $\phi'$. The water level in the soil is $mH$ and the wedge is separated from the slurry by a thin impermeable membrane. The slurry in the trench is at a level $nH$ and has a bulk density of $\gamma_s$. In the figure, $S$ denotes the shear force acting along the base of the sliding wedge, $N$ denotes the reaction normal to the base, $W$ denotes the weight of the wedge and $P$ denotes the horizontal force required to stop the wedge from sliding.

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If the wedge is in horizontal equilibrium,

$$P+S\cos\alpha = N\sin\alpha \quad . \quad . \quad . \quad (1)$$

and if it is in vertical equilibrium,

$$W = S\sin\alpha+N\cos\alpha \quad . \quad . \quad . \quad (2)$$

Furthermore the shear force $S$ is given by

$$S = N'\tan\phi' = (N-U)\tan\phi' \quad . \quad . \quad . \quad (3)$$

where $N'$ is the effective force normal to the slip plane and $U$ denotes the force of the water pressure acting on the slip plane. From equations (1), (2) and (3) it is readily shown that

$$P = \frac{W(\sin\alpha-\cos\alpha\tan\phi')+U\tan\phi'}{\cos\alpha+\sin\alpha\tan\phi'} \quad . \quad . \quad . \quad (4)$$

Also,

$$W=\tfrac{1}{2}\gamma H^2\cot\alpha \quad . \quad . \quad . \quad (5)$$

$$U=\tfrac{1}{2}\gamma_w(mH)^2\operatorname{cosec}\alpha \quad . \quad . \quad . \quad (6)$$

where $\gamma_w$ is the bulk density of water.

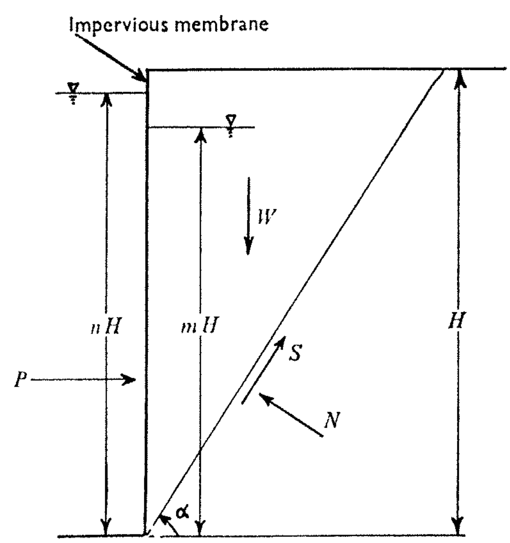


**Fig. 1. Stability of a sliding wedge**

The force of the slurry $P_s$ acting on the membrane is

$$P_s=\tfrac{1}{2}\gamma_s(nH)^2 \quad . \quad . \quad . \quad (7)$$

If, for stability

$$P=P_s \quad . \quad . \quad . \quad (8)$$

then the density of the slurry required to stop the wedge from sliding at $\alpha$ is given by

$$n^2\frac{\gamma_s}{\gamma_w} = \frac{\frac{\gamma}{\gamma_w}\cot\alpha(\sin\alpha-\cos\alpha\tan\phi')+m^2\operatorname{cosec}\alpha\tan\phi'}{\cos\alpha+\sin\alpha\tan\phi'} \quad . \quad . \quad . \quad (9)$$

By varying $\alpha$ in equation (9) the maximum slurry density which is required to ensure stability, may be found. In practice, $\alpha$ can usually be taken equal to $(45°+(\phi'/2))$ without introducing any significant error.

The description of the construction of the cut-off for the Wanapum Dam (La Russo, 1963) provided a case where the theory developed by Nash and Jones (1963) and extended in equation (9) could be checked. These calculations revealed that the slurry density required for equilibrium was considerably in excess of that of the clay suspension placed in the excavation (Morgenstern, 1963). If the influence of the yield strength of the slurry itself is considered, a shear strength of 19 lb/ft$^2$ would be required to account for the stability of the Wanapum trench, assuming the alluvium to be cohesionless. It is conceded that such high strengths are unlikely to develop except at the edges of an excavation, and as observed by Nash (1963) it would be unwise to rely on any strength of the mud in the trench in computing its stability.

It would appear that further factors remain to be considered in order to account for the stability of slurry trenches in cohesionless soils. We suggest that the main factor is the extra suspended solids contained in the slurry. During excavation a significant amount of solids is mixed with the slurry. These grains can remain suspended in the slurry if a small yield strength exists and they therefore impart to the slurry a much higher density than that of the original mix. The mechanism of support to be considered is that which includes the influence of the cuttings. It should be noted that in the case of the Wanapum trench, the clay suspension which initially had a density of 67 to 68 lb/ft$^3$ (1·08–1·09 tonnes/m$^3$) had a density at the time of backfilling of 80 lb/ft$^3$ (1·28 tonnes/m$^3$). The hydrostatic pressure of a fluid with this latter density is more than adequate to account for the stability of the excavation. An analysis of several slips which occurred during the trench excavation at Pierre-Bénite, France, support our suggestion that the influence of the suspended cuttings is an important factor in aiding the stability of mud-supported excavations. These slips will be discussed in more detail.

Arching of the cohesionless soil along the length of an excavated panel has also been suggested by Schneebeli (1964) to be a factor contributing to the stabiliy of the trench. He adopts the Caquot theory for earth pressure in silos (Caquot and Kérisel, 1956) to deduce the active earth pressure to be resisted by the pressure of the slurry. In support of this suggestion, Schneebeli observes that slips occurred during the excavation of trenches in alluvium at Gerstheim on the Rhine when the length of the panel exceeded 5 metres and that this corresponded to the limit of the arching action when the slurry level was about 1·5 metres higher than the ground-water level. However at Pierre-Bénite, where the soil and the ground-water level are similar, panels of lengths from 9 to 25 metres were excavated without difficulty under normal conditions.

It is felt that this consideration of arching probably underestimates the actual earth pressure. For the shallow slips which occurred during the flood at Pierre-Bénite, the influence of arching would in any case be negligible.

## ANALYSIS OF SLIPS AT PIERRE-BÉNITE

A concrete diaphragm cut-off has been built to protect the construction of the power station at Pierre-Bénite in France (Berthier, 1964). The wall has a length of 1400 m (metres) and a surface area of 36 000 m$^2$. The site is on the banks of the River Rhône and during the execution of the cut-off an unexpected flood occurred. Several slips in the trench excavation ensued.

The site is covered by river alluvium containing cobbles. The alluvium can be described as sandy gravel. The general natural ground level in the vicinity of the trench excavation is approximately 156·50 m. Prior to the construction of the trench, the ground level in the area of the excavation was raised to 160 m. This was carried out by excavating alluvium using scrapers and placing the fill over the length where the trench was to be constructed. Slurry filling a trench to this new height would have under normal conditions, provided sufficient support to permit the successful completion of the diaphragm wall. From a report

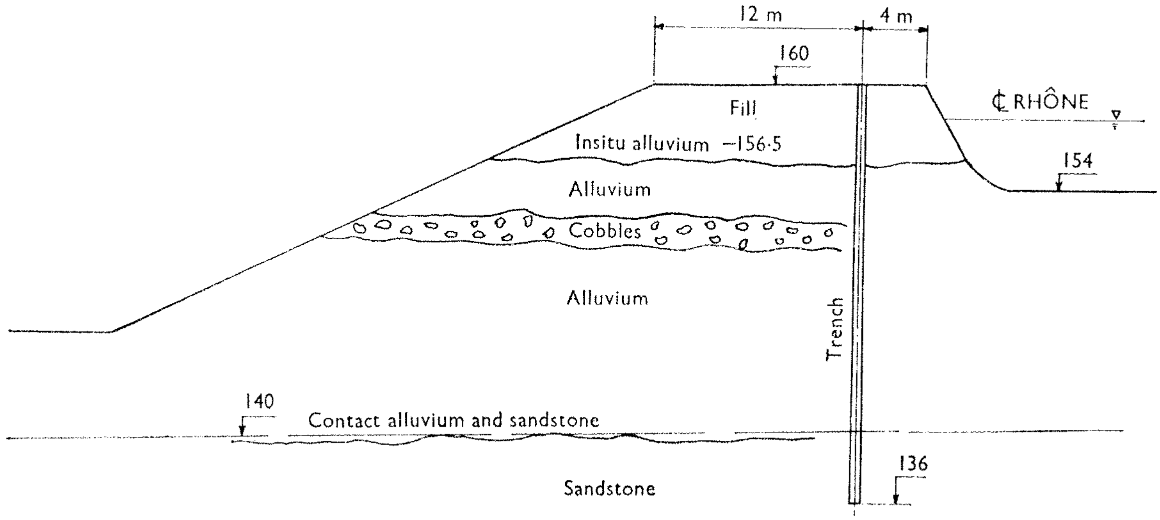


**Fig. 2. Cross-section at panel 77**

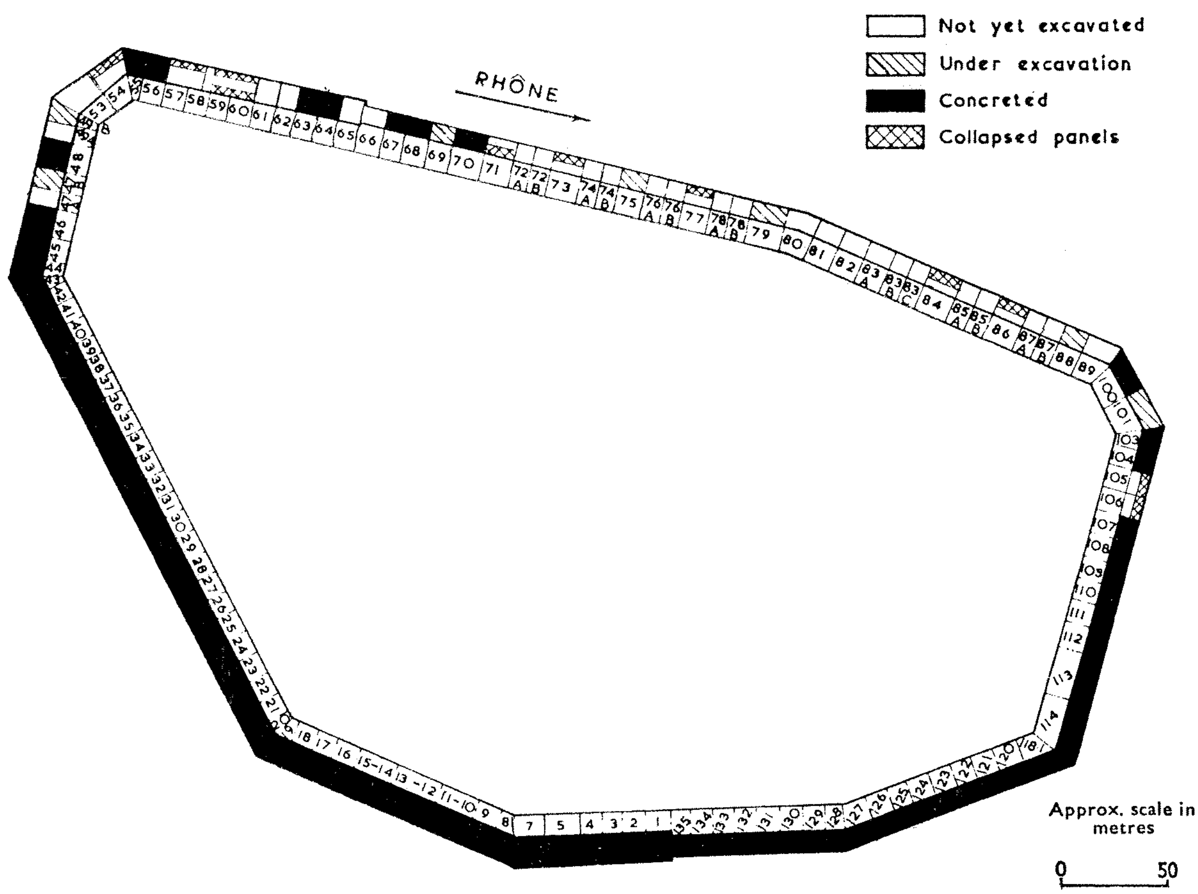


**Fig. 3. Site plan (30 March, 1963)**

of the Compagnie Nationale du Rhône it appears that the saturated density of the fill varied between 1·8 and 1·9 tonnes/m$^3$, and had a permeability of 1 to 0·1 cm/sec. The in-situ alluvium was undoubtedly denser but since all the slips took place in this loose fill, the properties of the strata beneath the fill are of little relevance. A typical cross-section through the excavation is shown in Fig. 2.

The diaphragm wall was constructed by excavating alternate panels and backfilling with concrete. The technique of excavation involved the use of rotary boring in the first few metres of alluvium and percussion boring in the underlying denser alluvium and sandstone. The cuttings were sucked from the bit level itself through reverse circulation and the bentonite from the trench, after some cleaning was returned to the excavation. The width of the panels was 0·6 m and the lengths varied between 9 and 25 m. The plan of the diaphragm and state of the work on 30 March, 1963 is shown in Fig. 3. Panels completed, under excavation, collapsed or not yet begun are distinguished by different symbols.

The bentonite used for the slurry was the Carbonisation et Charbons Actifs 'FB 2–5' and the concentration of the slurry was 4%, giving a bulk density of approximately 1·025 tonnes/m$^3$. Generally the level of the slurry was maintained at 10 to 20 cm from the top of the trench. A cake of clay was seen to form on the sides of the excavation, occasionally reaching a thickness of 5 cm. Observations of the density of the slurry within the excavation were also carried out at regular intervals using a Ruttner bottle.* The density was found to vary between 1·15 and 1·25 tonnes/m$^3$ although it lay more often between 1·20 and 1·25. This high density resulted from the fact that a considerable amount of sand from the cuttings was left in the slurry. Measurements of the density at different depths when the slurry was at rest revealed that no settlement of suspended matter occurred and that the density was constant with depth.

During the months of March and April, 1963, several unexpected floods occurred on the Rhône. The records of the river level and the ground-water levels at several observation wells are given in Fig. 4. It will be seen that the maximum river and ground-water levels occurred on 20 March, bringing the ground-water level in the vicinity of several panels which were in the process of excavation virtually to the top of the fill. During 19, 20 and 21 March slips occurred in the exterior walls of seven panels. The water levels on both interior and exterior sides of the slips were inferred from the observation well data. Two other panels slipped in April. The slip at panel 71 occurred on the interior side of the excavation. However, it should be noticed that for this case the internal water level was at its maximum, although the level of the Rhône was decreasing at the time.

The length of the slips ran for the whole length of the panels under excavation and their width at the top varied from 1·5 to 2·5 m. The slips terminated at the base of the alluvial fill, taking the form of a Coulomb wedge. Details of the nine cases are given in Table 1.

It is necessary to make several assumptions to analyse the slips. For example, the angle of shearing resistance of the fill is not known exactly but, considering its loose state, it is reasonable to assume that it lay between 30 and 35°.

The thickness of the top fill is also not known with sufficient accuracy. However, the ground-water levels at each slipped panel were estimated as reported in Table 1 and they are known to be close to the ground surface. In order to encompass the variation from panel to panel, calculations have been carried out using equation (9) to find the slurry density required at the point of sliding for different heights of ground-water level and the two values of angle of shearing resistance mentioned above. In the calculations, the level of the slurry is determined by a value of $n$ of 0·96. This corresponds to a level 15 cm from the top of the sliding wedge if the top level is at 160·0 m and its base at 156·5 m. Three values of ground-water level have been considered. They are specified by $m$ values of 0·87, 0·93 and 1·00. The first

* This is a sampling bottle whose base and top valves can be closed at depth.

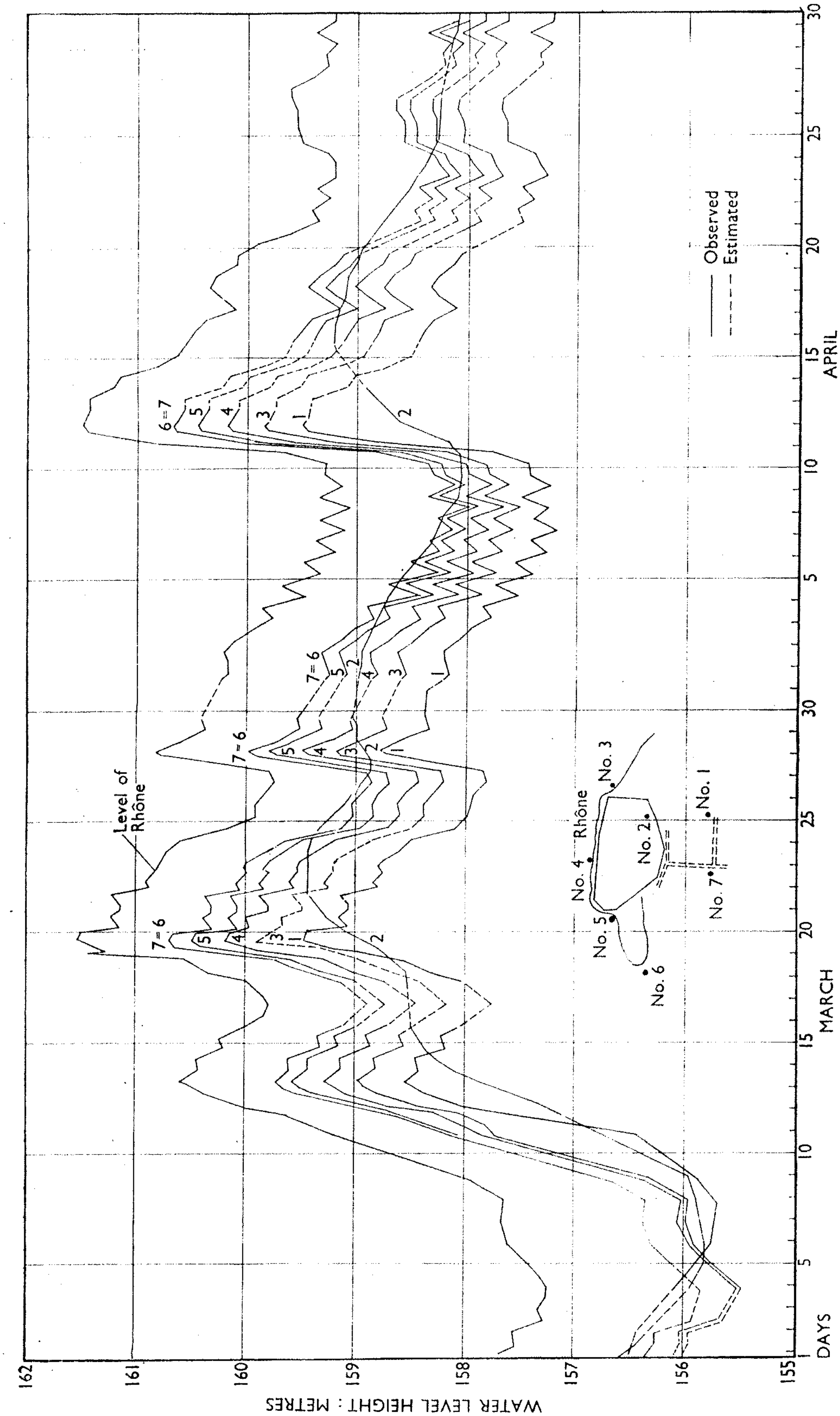


**Fig. 4. Ground and river water levels in March and April, 1963**

**Table 1**

**Slips at Pierre-Bénite**

| Panel No. | Length: m | Depth of excavation: m | Width: m | Internal water level | External water level | Side collapsed | Date of collapse | Remarks |
|---|---|---|---|---|---|---|---|---|
| 105–106 | 19 | 20 | 0·6 | 158·6 | 159·5 | Exterior | March 19 | 7 p.m. |
| 54 | 15 | 28 | 0·6 | 158·8 | 160·3 | ,, | ,, 19 | 12 p.m. |
| 57–58 | 13 | 8 | 0·6 | 158·9 | 160·3 | ,, | ,, 20 | Observed at 7 a.m. |
| 84 | 16 | 17 | 0·6 | 158·9 | 160·05 | ,, | ,, 20 | ,, |
| 59–60 | 20 | 26·5 | 0·6 | 159·2 | 160·3 | ,, | ,, 21 | ,, |
| 77 | 15 | 20 | 0·6 | 159·2 | 159·9 | ,, | ,, 21 | ,, |
| 73 | 15 | 23 | 0·6 | 159·2 | 160·1 | ,, | ,, 21 | ,, |
| 71 | 9 | 20 | 0·6 | 159·4 | 159·8 | Interior | April 22 | Observed at 6.45 a.m., max. interior water level |
| 69 | 13 | 26 | 0·6 | 159·2 | 159·4 | Exterior | April 16 | |

value corresponds to panels 105 and 106 assuming the ground surface to be at 160·0 m and the last value corresponds to a ground-water level at the top of the fill.

The composite results are presented in Fig. 5. It is seen that for a given strength the required slurry density is very sensitive to the ground-water level. Indeed, a small change in ground-water level is much more influential than a small change in angle of shearing resistance. Taking the slurry density in the excavation to be between 1·20 and 1·25 tonnes/m$^3$, the corresponding relative ground-water levels at the time of slip for both strengths are seen to be close to the top of the fill. These values, taken from Fig. 5, are presented in Table 2 and corroborate the mechanism discussed in a previous paragraph, provided that the influence of the suspended cuttings is taken into account.

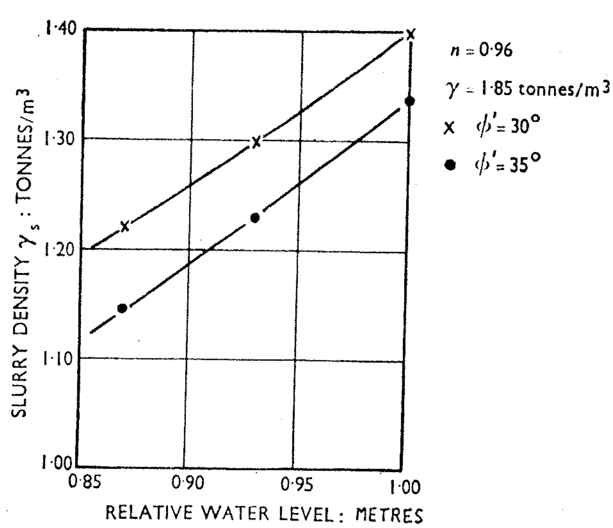


**Fig. 5. Relation between relative water level and slurry density for slips at Pierre-Benite**

**Table 2**

**Required relative ground-water level at time of slips**

| Slurry density: tonnes/m$^3$ | $m$ at time of slips | |
|---|---|---|
| | $\phi' = 35°$ | $\phi' = 30°$ |
| 1·20 | 0·903 | 0·856 |
| 1·25 | 0·945 | 0·894 |

It should be noticed that the required slurry densities for the possible variation of relative ground-water level and shear strength are considerably greater than the initial density of the slurry prepared at a 4% concentration.

## CONCLUSIONS

During a mud-supported excavation in granular deposits, the density of the slurry is increased by the suspension of cuttings within it. These cuttings are held in suspension by the small yield strength of the mud. Cardwell (1941) has suggested that the diameter of a particle supported by a slurry is

$$D = \frac{6C_s}{(\gamma - \gamma_s)g} \qquad (10)$$

where $D$ is the diameter of the particle,
$\gamma$ is the density of the particle,
$\gamma_s$ is the density of the slurry,
$g$ is the acceleration due to gravity
and $C_s$ is the shear strength of the slurry.

Using this equation, it is readily shown that a shear strength of only 0·03 lb/ft$^2$ is required to support a 1 mm sand particle. Although the strength of the slurry itself is not an important factor contributing to the stability of an excavation, it does provide the basis whereby the density of the slurry can be increased without the expense of adding extra clay. Jones (1963) quotes a Bingham yield value of 0·04 lb/ft$^2$ for a 4% suspension. This would be adequate to support particles finer than coarse sand.

The analysis of the slips at Pierre-Bénite confirms the suggestion of Nash and Jones (1963) that an impervious membrane develops at the junction of the slurry and the surrounding material and that hydrostatic force is exerted on the walls of the trench. The use of bentonite in manufacturing the slurry is necessary because of its capacity to form an impervious boundary at the sand interface and because its yield strength allows higher densities to be achieved without the addition of extra clay. If calcium montmorillonite were used in making the slurry a higher clay concentration would be necessary in order to obtain the same strength as that obtained with a lower concentration of sodium montmorillonite.

It is likely that other factors enter into the analysis such as the resistance to sliding due to shear at the ends of the wedges and the shearing resistance of the slurry that has penetrated the sand and has gelled under quiescent conditions. However it is felt that these usually have only a minor influence and that a rational design can be obtained if the correct density is used.

## ACKNOWLEDGEMENT

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