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The arching effect of soils over voids and piles incorporating geosynthetic reinforcement

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ABSTRACT: The use of geosynthetic reinforcement to act as a tensioned membrane and support embankments and infrastructure overlying voids and piles is increasing, Alexiew (1997, 2000). Currently a variety of methods exist, for predicting the amount of load carried by the geosynthetic reinforcement based on the degree of stress redistribution due to arching between the piles. This paper compares these theoretical models with the results of from a scaled physical model.

1 INTRODUCTION

This paper describes a scaled physical model which has been used to evaluate the effect of soil arching and for determination of the 'zone of influence' affecting the geosynthetic reinforcement. The physical model consists of a soil filled steel box (approximate dimensions: 1105x720x560mm) with a Perspex front and incorporating a 'trap door' with geosynthetic reinforcement above. Pressure acting on the trap door, geosynthetic tensile forces and mid-span surface displacements are monitored during the experiment. The experiment has been repeated for two fill at differing fill depths and moisture contents to determine the sensitivity of 'soil arching' to soil parameters or overlying geometry

2 THOERETICAL METHODS

Leonard (1988), Kempton & Jones (1992) & Giroud (1995), all presented a simple expression for determining the approximate value of geosynthetic strain of a deformed geosynthetic. This forms part of the basic expression used for dimensioning the support systems for most of the current theories. It assumes that the deflected shape, at a given cross-section, is a smooth curve, i.e., an arc of a circle or a parabola. The maximum strain developed, for uniformly distributed strain across the geosynthetic, can be approximated thus:

$$\varepsilon_{\max} = \frac{8}{3}\left(\frac{y}{b}\right)^2 \quad (1)$$

where y = vertical deflection at mid-span b = horizontal distance between supports. Hence an expression for relative deflection of the membrane for a given strain is provided:

$$\frac{y}{b} = \sqrt{\frac{3\varepsilon}{8}} \quad (2)$$

.

The forgoing method considers the tension membrane as two-dimensional. Later work by Villard & Giraud (1998) looked at the three-dimensional effect of geotextile support systems. The authors considered the effect of cavity shape, thread distribution (geosynthetic orientation) and boundary conditions on the maximum strains and tensions in the sheet The authors showed that for the two–dimensional analysis, the results of maximum strain and tension values obtained both numerically and analytically exhibited close similarity for relative deflections (y/b) of less than 10%.

2.1 *Zone of Influence*

Where the different dimensioning procedures exhibit considerable differences is in the extent of the soil zone (and possible surcharge), which is deemed to influence the support membrane. Essentially the design philosophy used to determine the support system above a void can be split into two categories, i.e., those that consider 'soil arching' and those that do not.

2.2 *No soil arching*

Since the simplest assumption of soil behaviour above a void is to ignore the effect of soil arching this condition is considered first. British Standard BS 8006 (BSI, 1995), provides a method for determining the maximum tensile strains that develops within the tensioned membrane supporting the zone of soil assumed as an inverted, truncated cone or trough with an angle of draw, which projects up from the edge of the void through the soil mass at an angle, θ_d to the horizontal, see Figure 1 . To simplify the analysis it assumes the load, from the zone of influence, acts over the unsupported horizontal span of the tension sheet

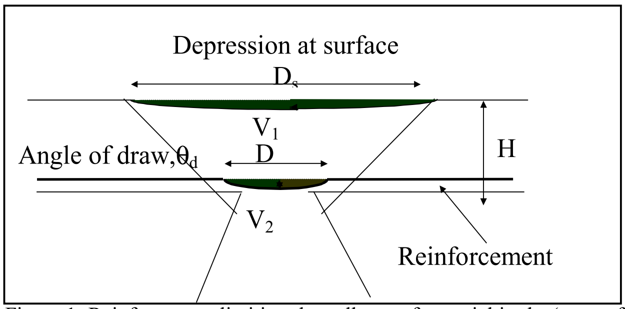


Figure 1. Reinforcement limiting the collapse of material in the 'zone of influence' above voids from BS 8006.

The maximum reinforcement strain is given by:

$$\varepsilon_{\max} = \frac{8}{3}\left(\frac{d}{D}\right)^2 \quad (3)$$

Where d = maximum membrane deflection & D = horizontal span.

This expression cannot be solved since it has two unknowns ε and d/D. What needs to be determined is the maximum allowable geotextile extension based on the serviceability conditions imposed for the surface. By considering the geometry of the zone of influence and assuming a constant volume and then equating the volumetric movement of the geosynthetic and the volumetric movement of the soil, a relationship is presented for the maximum allowable reinforcement strain, for long voids:

$$\varepsilon_{\max} = \frac{8\left(\dfrac{d_s}{D_s}\right)^2}{3D^4}\left(D + \frac{2H}{\tan\theta_d}\right)^4 \quad (4)$$

Where D_s= Width of void at the surface, d_s=Allowable deformation at surface, D = Design width of void; H= Height of embankment/fill, θ_d = Angle of draw of fill, approximate to the peak friction angle.

Considering the zone of influence in two dimensions the area of fill deemed to be influencing the membrane is trapezoidal in shape and as such an equation can be derived based on the volume of fill, the distributed load carried by reinforcement between the void, W_T, is given by:

$$W_T = \frac{\left(DH + \dfrac{H^2}{\tan\theta_d}\right)}{D} \quad (5).$$

The assumption of constant volume is likely to prove conservative, particularly for dense granular fills, which show considerable, increase in volume when sheared. Hence any surface settlement predictions based on the foregoing relationship will overestimate the degree of surface settlement.

2.3 Dilatancy

The tendency for a dense sand to increase in volume under increasing stress difference is known as dilatancy (Terzaghi & Peck, 1967). The beneficially effects (on surface settlement predictions) by considering dilatancy or soil expansion within the collapse zone are discussed by Villard et al (2000). A modified methodology for predicting surface settlements based on an increase in the volume of soil in the zone of influence due to dilatancy is presented by Bruhier, (1997). The methodology is very similar to the British Standard method, however, it allows for a volume increase in the zone of influence. The design approach, effectively limits surface deflections to prescribed serviceability requirements and hence determine the volume of the 'saucer' shaped depression at the surface,(V1). A dilatancy factor is then applied to the material in the zone of influence which thus determines the increased volume of the deflected membrane volume (V2), See Figure 1. This method will result in larger reinforcement strains, for prescribed surface settlement criterion, than the BS 8006 methodology.

Both methodologies assume a draw angle projecting up from the edge of the void through the soil mass at an angle, which approximates to the peak angle of friction $\varnothing_{pek'}$. Observations of sink/swallow holes post collapse (Site Investigation Steering group, SISG.1993) would endorse this. However it is also known from historical and empirical observations that the phenomenon of 'arching' occurs where the depth of fill over the void is of a sufficient height.

3 SOIL ARCHING /VERTICAL LOAD SHEDDING

Essentially for soil arching or stress redistribution to occur the depth of fill above the void has to be sufficient for the arch to be deemed to have developed. The British Standard BS 8006 (1995), does not consider the concept of soil arching in respect to geosynthetic reinforcement spanning voids, however the same document does consider the concept of soil arching in respect to geosynthetic reinforcement spanning between pile caps at the base of a piled embankment and conservatively ignores any bearing contribution offered by the ground in between the piles. The tension membrane is deemed to span over a void between the edges of the pile cap.

3.1 *Piled Embankments*

The geosynthetic reinforcement at the base of a piled embankment serves two principal functions. Firstly, to act as a tensioned membrane in supporting the weight of the fill in the embankment between the piles. Secondly, to counteract the lateral thrust on the sides of the embankment hence reduce the need for raking piles at the edge of the embankment. Whilst acknowledging the existence of the lateral thrust in determining the type and strength of the geosynthetic reinforcement, this latter requirement is ignored by the author, for comparisons of the different arching methods.

3.2 *Counter pressure*

The reliance on partial support from the ground in between the piles has created much debate in geotechnical circles (Tonks et al, 1998, Kempfert et al, 1999). Certainly the inclusion of counter pressure beneath the tensioned membrane will greatly reduce the loadings on the reinforcement. Jones et al, (1990), suggested that current simplified analytical procedures are conservative inasmuch as they overestimate the tensile requirements of the reinforcement and cannot accurately take into account the partial foundation support beneath the geotextile. This agrees with evidence from field observations from instrumented embankments, Rogbeck et al, 1998, indicating that post construction geosynthetic loadings and strains were lower than predicted by analytical solutions.

However the reliance on long term support from the compressible soil between pile caps, is still a matter of engineering judgement. Tonks et al (1998) acknowledge this support in the short term but suggest that in the long-term it will be necessary to model the soil consolidation and address the long- term settlements.

3.3 *Stress redistribution variations*

The vertical stress acting at the base of a piled embankment due to its self-weight and surcharge is non-uniformly distributed. This is due to the stress acting on the relatively rigid piles being significantly greater than the stress acting on the compressible soil in between. The stress variations can be attributed to the influence of soil arching between adjacent pile caps and due to the influence of the internal shear strength of the soils. Assumptions in the degree of soil arching, considered by the differing dimensioning procedures are considerable and accounts for the main variation in the strength of the geosynthetic reinforcement predicted. There are a number of factors, which will influence the degree of stress redistribution including pile/ embankment geometry, fill properties and the relative stiffness of the piles and surrounding soil.

3.4 *Boundary Conditions*

In attempting to make comparisons between the different arching theories it is important to gain an appreciation of the boundary conditions or support system that anchors the tension membrane. For geosynthetics spanning voids, restraint is provided all around the perimeter, for a circular void or continuously along opposite edges, for an infinitely long trench, Terzaghi's assumption. However when considering piled embankment design restraint is provided only at discrete points along the boundary edge, i.e., at the pile caps. To enable comparison of different theories plain conditions will be assumed.

3.5 *Comparison of Arching Theories*

The methodologies compared were for Terzaghi, (1943), Hewlett & Randolph, (1988), BS 8006, (1995), the Carlsson method described by Rogbeck et al (1998), enhanced arching approach, Jenner et al, (1998), also referred to as the Guido approach (Guido et al, 1987), each of which assume some form of arching or stress redistribution. In order to compare spanning applications and allow comparison with results from the physical model, each of the methods above will be discussed and considered in two-dimensions. All the above procedures (except Guido) treat soil arching, enhanced or otherwise, as a redistribution of the vertical stress but still utilize a tension membrane to carry the weight of soil in the zone of influence.

3.6 *Terzaghi Approach*

Soil arching was also discussed by Terzaghi (1943) by assuming a lateral load transfer through shear stresses along vertical planes located at the edges of the clear span. An expression is presented for stress redistribution and is discussed in more detail by Giroud et al (1990). The distributed load influencing the geosynthetic reinforcement:

$$p = \frac{\gamma_{ave} D}{2K \tan\phi}\left[1 - e^{-2kTan\phi H / D}\right] \tag{6}$$

where p= Pressure acting on the geosynthetic over the void; γ= Unit weight of fill; H = Height of fill above void; D= Width of the void, K = the lateral earth pressure coefficient.

3.7 *Hewlett & Randolph Approach*

Hewlett & Randolph (1988), derived a theoretical solution for the arching action of free–draining granular material above piles, based on observation from laboratory model test on moist sand were performed for different boundary conditions. Based on these observations the authors consider the limiting equilibrium of stresses in a curved region of sand between adjacent pile caps. for plane strain conditions. The overall principal behind the Hewlett and Randolph analysis is that: arches of sand' shed the uniform overburden of the embankment onto the pile caps and that the infilling sand beneath the arch offers no support or counter pressure, that pore pressures are zero, hence total and effective stresses are therefore equal in the analysis.

The stress acting across the geosynthetic can be determined by considering the limiting equilibrium of stresses in a curved region of sand between adjacent pile caps. The critical location was shown to be at the crest of the arch.

Stress conditions at the crest are given by:

$$\sigma_i = \gamma\left(H - \frac{s}{2}\right)\left(\frac{s-a}{s}\right)^{(Kp-1)} \tag{7}$$

The pressure acting on the under side of the arch is given by σi this is at a height, s-a/√2 above the reinforcement. Hence the total stress acting along the reinforcement is given by:

$$\sigma_s = \sigma_i + \gamma(s-a)\sqrt{2} \tag{8}$$

where a = Pile cap size;`s = Spacing between adjacent piles; H = Height of embankment or fill depth; σ_i = Stress on the interior of the arch; σ_s = Stress on the geosynthetic; γ= Unit weight of fill; K_p= the passive earth pressure coefficient, 1+sinϕ'/1-sinϕ'.

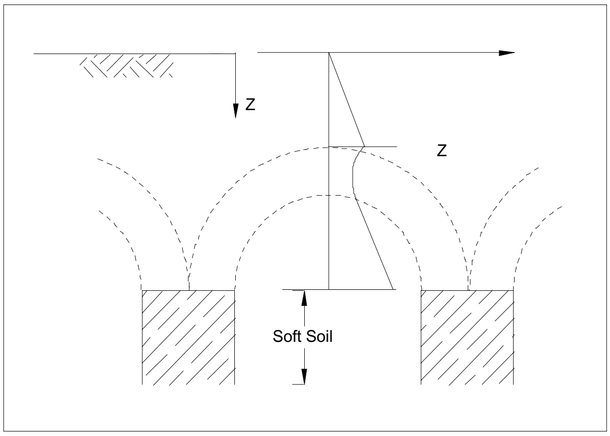


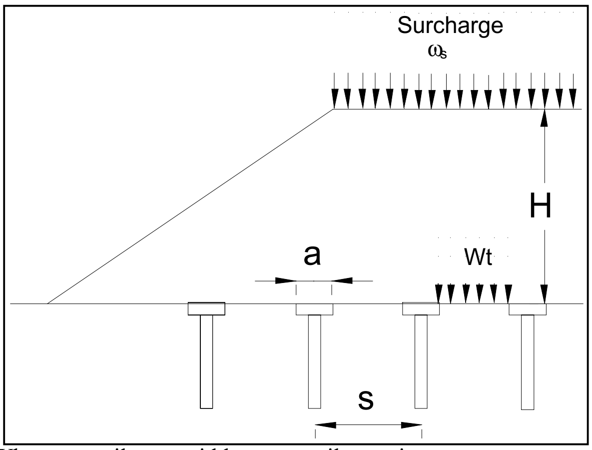
Figure 2. Zone of influence, stable hemispherical arches after Hewlett and Randolph

3.8 *BS8006*

The BS identifies a ***'critical height'*** concept whereby the depth of fill is sufficient for the full arch to be deemed to have developed and any additional overburden or surcharge loads do not influence the tensioned membrane, but distribute to the boundary supports i.e., the pile caps. The fill depth is commonly expressed in terms of the clear span dimension, as a ratio of depth of fill/span width.

According to the BS 8006 (See Figure 3) the 'critical height' *Hc* is given by:

$$H_c => 1.4(s-a) \tag{9}$$



Where a = pile cap width; s = pile spacing

Figure 3. Piled embankment incorporating horizontal reinforcement above pile caps,

A relationship is given in BS 8006 to determine the vertical pressure acting on the top of the pile caps due to the presence of soil arching and is based on work by Jones et al. (1990). Modelling of different deformation due to the relative stiffness of the piles Marston's formula. Since end bearing piles are generally stiffer and attract more vertical load. Marstons work related to the stress variance on buried rigid pipes. This provides the ratio of vertical stress on pile caps to average stress at embankment base.

$$\frac{p'_c}{\sigma'_v} = \left[\frac{c_c a}{H}\right] \tag{10}$$

End bearing piles $C_c = 1.95\,{}^{H}\!/_{a} - 0.18$ (11)

Friction & other piles $C_c = 1.5\,{}^{H}\!/_{a} - 0.07$ (12)

Where p'_c = Vertical stress on pile cap, σ'_v= Factored effective vertical stress at embankment; γ= Unit weight of fill; H= Height of embankment; a = Size of pile caps; C_c= Arching coefficient.

Having established the proportion of the vertical stress acting across the pile caps the remainder of the vertical embankment weight exerts an averaged uniform load distributed on the geotextile reinforcement, WT, with plain strain conditions assumed whereby the boundary supports are a continuous strip and partial load factors ignored , W_T, is given by:

$$W_T = \frac{1.4(s-a)\gamma}{s-a}\left[s-a\left(\frac{p'_c}{\sigma'_v}\right)\right] \qquad (13)$$

3.9 *Carlsson method*

The Carlsson method described by Rogbeck et al (1998) also adopts a critical height approach and considers the cross sectional area under the soil arch to be approximated by a soil wedge (See Figure 4) with an internal angle at the apex of the wedge equivalent to 30°.

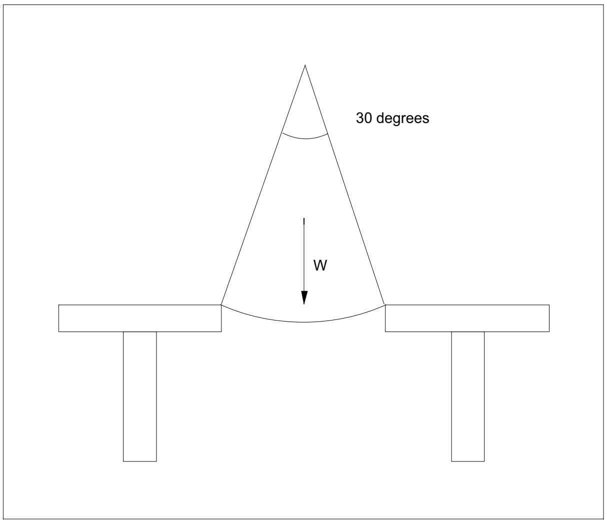


Figure 4. The soil wedge influencing the reinforcement after Carlsson

Hence the height of the wedge is given by:

$$H = (s-a)/2.\tan 15^\circ \qquad (14)$$

Which corresponds to a critical height ratio equivalent to:

$$H => \left(\frac{1}{2.\tan 15^\circ}\right)(s-a) = 1.86(s-a) \qquad (15)$$

where a = Pile cap size; s = Spacing between adjacent piles; H = Height of wedge

In common with the British Standard approach at the ***'critical height'*** the depth of fill above the arch is sufficient such that the arch is deemed to have developed and any additional overburden or surcharge loads on the surface of the embankment fill do not influence the tensioned membrane, but distribute to the boundary supports i.e., the pile caps. The weight of the soil wedge influencing the tensioned membrane is given by:

$$W_T = \left(\frac{s-a}{2}\right)\left(\frac{s-a}{2.\tan 15^\circ}\right)\gamma = \frac{(s-a)^2}{4.\tan 15^\circ}.\gamma \qquad (16)$$

Carlsson considers the weight of the soil wedge to be acting across the deflected length of the geosynthetic membrane rather than across the horizontal clear span. The displacement of the membrane is calculated for an allowable strain using Equation 2. Hence the force in the reinforcement in two dimensions can be determined from:

$$F_{2D} = W_T.\left(\frac{s-a}{8d}\right)\sqrt{1+\frac{16d^2}{(s-a)^2}} \qquad (17)$$

where a = Pile cap size; s = Spacing between adjacent piles; H = Height of wedge, W_T= weight of the soil wedge; d= membrane deflection; γ = Unit weight of fill.

In considering the concept of a critical height, assumptions are made regarding loading conditions based purely on embankment/pile geometry. As can be observed from the last two methods, disagreement exists on when this critical height develops

3.10 *Enhanced arching approach*

Jenner et al (1998) have applied the findings of early work by Guido et al (1987). The work carried out by Guido et al (1987) work was on plate loading test on geogrid reinforced sand. However Bell et al (1994), Maddison et al (1996) & Jenner et al (1998) have all applied this work to the design of piled embankments to promote enhanced arching.

Jenner et al (1998) suggest the purpose of the multi layers of geogrids are to enhance the transfer of vertical loading and to mobilise the maximum shear strength of the granular layer to distribute the imposed loads efficiently and evenly into the piles and that some support to the fill beneath the developed arch will always be provided by the underlying soil.

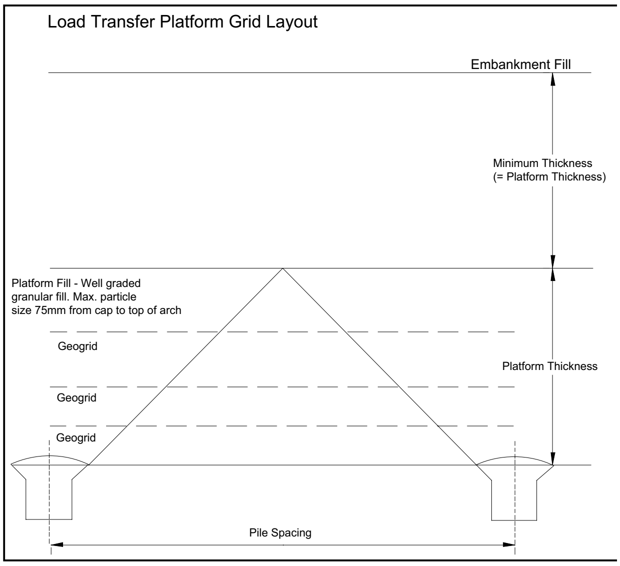


Figure 5. Enhanced arching approach after Jenner et al (1998)

The authors suggest two different ways of designing the reinforcement, (i) Tensioned membrane approach, (ii) Enhanced Arching approach. All the previous procedures treat soil arching, enhanced or otherwise, as a redistribution of the vertical stress but still utilise a tension membrane to carry the weight of soil in the zone of influence. Jenner et al (1998) identify a critical height or thickness above the piles to ensure that the arch is always loaded. This equates to a minimum height of fill above the load transfer platform of at least the platform thickness or 1 times the clear span between the supports. See Figure 5.

Assuming plain strain conditions and considering a triangular zone of influence projecting up at an angle of 45° from the boundary edge. The distributed load influencing a tensioned membrane is given by:

$$W_T = \frac{(s-a)\gamma}{4} \tag{18}$$

where a = Pile cap size; s = Spacing between adjacent piles; W_T= weight of the soil wedge; γ = Unit weight of fill.

4 COMPARISON OF ARCHING THEORIES

4.1 *Stress Reduction Ratio*

Various authors (Giroud et al, (1990), Russell & Pierpoint, (1997), & Kempert et al, (1999)), have used the concept of a stress reduction ratio to quantify the reduction in vertical stress due to the influence of soil arching. The stress reduction ratio is given by:

$$SRR = \frac{Pr}{Po} = \frac{Pr}{\gamma H} \tag{19}$$

Where: *Pr* is the average vertical stress carried by the geosynthetic: *Po* is the original average vertical stress due to the overlying fill.

To enable plain strain comparison between pile spanning applications and the void spanning applications Stress Reduction Ratio(SRR) will again be used however SRR > 1 will signify an increase in the average vertical stress influencing the membrane. Comparisons between different theoretical methods are given in Table 1., where stress reduction ratios for different fill depth to span ratios are compared

Table 1. Stress reduction ratios for different arching methods

Stress Reduction Ratio, values of p/p_o for different fill depth to span ratios, H/D or H/(s-a)							
H/D	0.3	0.6	1	1.4	2	3	5
1	3.42	1.439	0.742	0.55	0.394	0.244	0.143
2	0.929	0.864	0.787	0.71	0.632	0.518	0.367
3	1.3	1.6	2	2.4	3	4	6
4	3.3	2.59	0.933	0.66	0.466	0.31	0.155
5	4.672	2.33	1.414	1.02	0.724	0.494	0.311
6	1.2	0.41	0.25	0.178	0.125	0.083	0.05

1:BS 80006 (Arching), 2: Terzaghi, 3: BS 8006 No arching, 4: Carlsson
5: Hewlett & Randolph, 6: Enhanced Arching

5 LABORATORY WORK

The physical model consists of an earth filled steel box (approximate dimensions: 1105x720x560mm) with a Perspex front incorporating a 'trap door' consisting of a steel plate (12mm thick) cut accurately to fit between the two steel plates (approx. 152mm x 12mm) acting as sides supports welded on opposite sides of the steel box. The steel plate was supported by a hydraulic jack connected in series to another jack, beneath a proving ring supported on two 25mm thick steel plates welded to the outside of the steel box. The proving ring would allow variation in the pressure acting on the trap door to be observed, See Figure 7.

A hinged roller was fabricated with a clamp strip attached the reverse side to allow the geotextile to be fixed to the lever arm with the other end attached to a clamp strip fixed to side support. The lever was connected by a threaded bar passing through an oversized locating hole in the side of the apparatus through the centre of a hollow stemmed load cell with end plate. Measuring the compressive force in the load cell on the side of the apparatus indirectly determines the average tensile force across the geotextile.

Figure 7. Apparatus set up

Mid span deformation at the surface of the fill were recorded using a dial gauge placed on a small coin on the fill, on a steel plate 12mm thick placed across the top of the apparatus. The trap door was lowered incrementally resulting in deflection of the membrane. Pressure acting on the trap door, tensile forces within the geosynthetic and mid-span surface displacements are monitored during the experiment

Figure 8. Test No. 3, showing soil arch, moisture content 4.7%.

The experiment was repeated for two fill types, a coarse sand used for Test No.3,4, & 6 and a 10mm stone used for Test No. 1,2 &5. A thin piece of black paper approximately 25mm wide was placed at the front edge of the fill, providing horizontal indicators within the fill, and a black marker used to trace the original position of the indicator to enable visual comparison between the original and deflected shape. Visual comparisons of the observed zones of influence for Test No. 3, see Figure. 8 and Test No. 4, see Figure 9 show significant differences yet the only variance between the tests was the moisture content of the sand. Partially saturated soils will exhibit suction due to differences in pressure between the fluids occupying the voids in the soil structure and highlights the significant effect that soil suction can have on the development of the zone of influence particularly with scaled models where the overburden stress is not significant.

Figure 9. Test No. 4, showing wedge, moisture content 1.3%

6 RESULTS

The results from the physical model are compared (plane strain) to the current theoretical methods.

Table 2: Observed Stress Reduction Ratio & surface displacements

Test No.	2***	3	4	5***	6
H/D ratio	1.430	1.545	1.545	1.97	1.92
No Arching	1	1	1	1	1
Observed	1.101	0.717	0.92	0.27	0.29
Carlsson	0.25	0.23	0.23	0.18	0.18
BS 8006	0.79	0.69	0.66	0.63	0.64
Terzaghi	0.71	0.69	0.66	0.63	0.64
H & R*	0.99	0.98	0.98	0.72	0.81
Enhanced Arching	0.174	0.161	0.161	0.126	0.13
Surface** Deflection	-	7%	7%	1.2%	2%

* Hewlett & Randolph method, ** As a percentage of total membrane deflection *** 10mm Stone Fill..

7 CONCLUSION

The surface deflections observed were considerable lower than those predicted by current methods BS 8006, 1995 and Bruhier, 1997. There appears good agreement between authors on simple expression to determine deflection of tensioned membranes.

Where considerable variation still exists is in the degree of stress redistribution assumed, and this area warrants further work. Some methods consistently underestimate the actual stress levels and should be avoided.

The observed results show a disproportionate additional stress redistribution occurred when fill depth/span ratios increase from 1.545 to 1.92 supporting the concept of a critical height. The critical height identified by the Carlsson method 1.86 lies between these two values.

The author suggests that the findings from scaled physical models using partially saturated soils should not be directly relied upon without correlation of those results to full-scale tests.

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