Analysis of Pile Supported Embankments Considering Soil Settlement and Load Transfer Platform Strain Compatibility

Ver. 07-30-2014
TRB Paper No. 15-2458

by

Christopher R. Byrum, PhD, PE

Project Engineer
Soil and Materials Engineers, Inc.
43980 Plymouth Oaks Blvd
Plymouth, Michigan 48170-2584
Phone: 734-454-9900
Fax: 734-454-0729
byrum@sme-usa.com

Analysis of Pile Supported Embankments Considering Soil Settlement and Load Transfer Platform Strain Compatibility

Christopher R. Byrum, PhD, PE

Abstract: This paper presents a structural analysis procedure for flexible geogrid/geotextile reinforced load transfer platforms used for pile/column supported embankments. Design of these systems must consider strain compatibility between the expected settlement of soils between piles combined with the tensile stiffness and deformation of the load transfer platform reinforcements. When relatively small settlements are expected, significantly stiffer reinforcement materials are necessary to transfer greater loads to piles at lower reinforcement strain and settlement levels. This simplified analysis procedure matches theory for uniformly loaded cables to the calculation of reinforcement elongation and force mobilization caused by the assumed deformed shape occurring between piles and using site settlement estimates and a specified geogrid/geotextile force versus strain curve. The analysis procedure described considers both a soil-arching load case and a full tributary block load case. The procedure allows for rapid estimation of load transfer platform reinforcement strength and stiffness requirements and pile loads for designs. An example problem simulating a 40-ft tall embankment overlying soft soil is provided. Sensitivity analyses demonstrate how variations in pile spacing, embankment height, subsoil compressibility, and reinforcement stiffness affect the predicted pile supported embankment behavior.

INTRODUCTION

This paper presents a structural analysis procedure for estimating the pile loading and reinforcement forces for pile supported embankment designs using flexible geogrid/geotextile reinforced load transfer platforms. Figure 1 shows a conceptual pile supported embankment cross section through a row of piling. For a given embankment fill height, pile spacing (*s*) and pile cap diameter (*d*), the forces that develop in the load transfer platform reinforcements are related to two primary factors: the stiffness of the reinforcements and the amount of downward displacement and related tensile elongation of the reinforcements that develops between the piling. The downward deflection of the reinforcements relative to the pile caps placed over the support columns or piles is necessary for mobilization of tension in the reinforcements. The displacement angle, φ, developed in the reinforcements at the edges of the pile caps allows the reinforcement tension to develop downward force on the piling that adds to embankment weight and soil arching forces, and reduces loads on the soft soil below the fill.

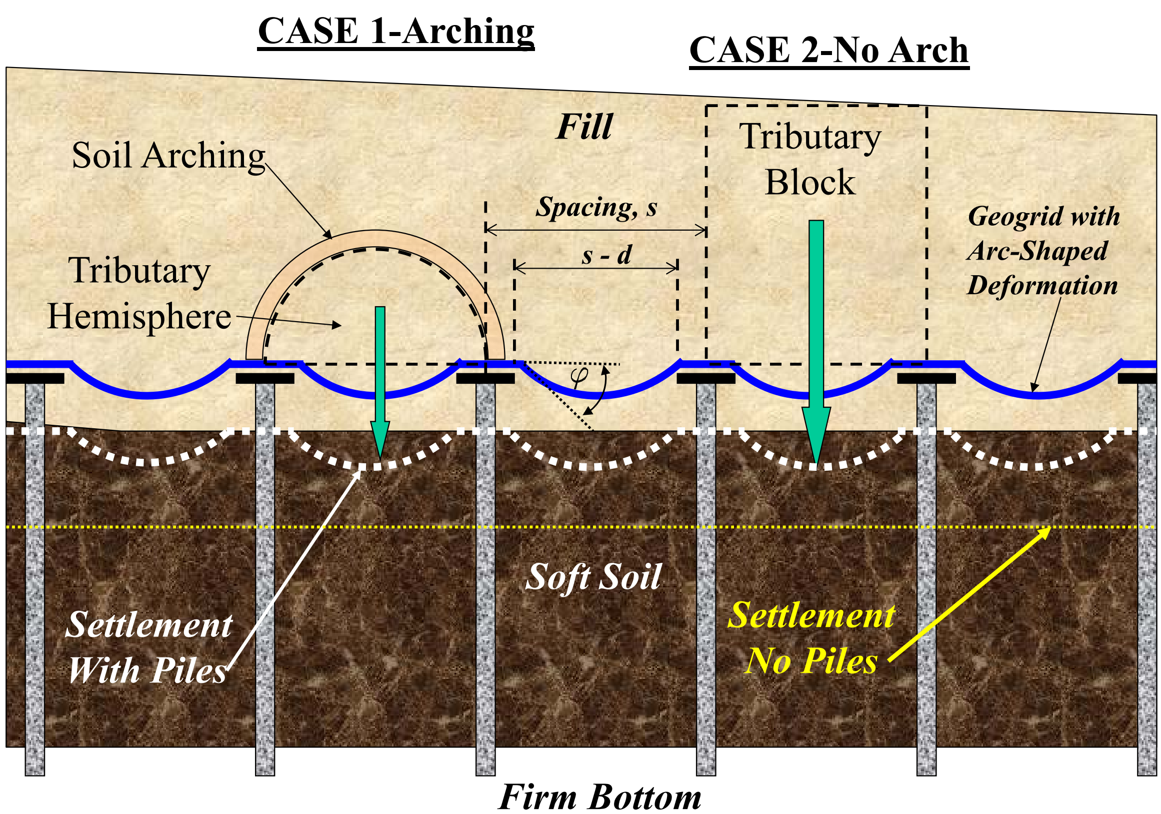

FIGURE 1 Key components of a pile supported embankment having a geogrid/geotextile reinforced load transfer platform.

In most cases, increasing upward pressures from the underlying soft soil will develop at the bottom of the load transfer platform as settlements develop. Figure 2 shows the primary vertical load and resistance forces assumed for the pile supported embankment analysis procedure described in this paper. The combination of upward resistance from the underlying soil plus the upward resistances from the deformed reinforcements must equal the applied embankment fill weight loads acting between piles. In general, if the underlying soil is relatively stiff such that predicted settlements are lower, then higher reinforcement grid stiffness is typically necessary in order to mobilize upward force in the reinforcements at a smaller strain level. Predicting the reinforcement forces is the result of analysis of strain compatibility between the estimated reinforcement deformation and estimated deformation of the soft soils beneath the load transfer platform, in response to the vertical loads.

δ= maximum reinforcement deflection relative to pile cap

T_{Grid} = tension in the reinforcement

F_{Soil} = upward soft soil reaction

F_V, F_H = reinforcement end reactions

F_{Block} = downward embankment load

s = pile spacing

d = pile cap diameter

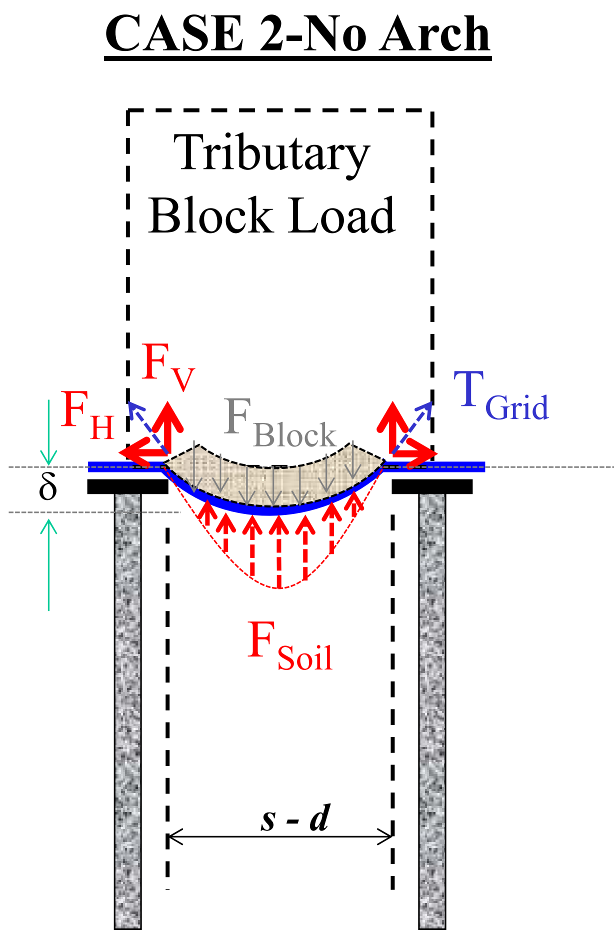

FIGURE 2 Vertical forces assumed for the deformed reinforcements in a pile supported embankment.

There is considerable variation in existing analysis procedures and opinions regarding how much of the total fill weight above the load transfer platform is assumed to transfer to the reinforcements and underlying soil (*1,2,3,4,5,6*). This variation in philosophy is related mostly to the concept of soil-arching. When full arching is present, only the weight of a hemisphere-like shaped mass of earth between four piles is expected to load the reinforcements and underlying soft soil, while the remaining fill above this hemisphere-like shape interlocks and bridges from pile to pile through the arching mechanism. However, when the embankment height is relatively small compared to pile spacing, arching will not develop because there is not enough fill height for arches to form. Arching may occur initially for relatively tall fills having short pile spacing, but heavy dynamic repeating loads may break down soil arching and the weight of nearly the full tributary block shaped earth mass could eventually act on the reinforcements.

Use of multiple layered continuous reinforcements extending well up into the hemisphere-like shape below the arches can enhance arching through a beam-like load transfer mechanism (*3*), further reducing fill surface settlements and loads on the underlying soft soils. The formation of arching and its interaction with geogrid/geotextile reinforcement is complicated by time rate of settlement effects and is also sensitive to the type of fill material used (*2*).

The type of piling or columns used to support the load transfer platform has a significant effect on ability to distribute loads onto the columns and away from the soft soils. Piling or columns that have high axial stiffness and small compression displacement will allow more force to go into the load transfer platform reinforcements and columns. Columns or piles that yield in compression during fill placement will result in more load shedding into the soft soils, because less reinforcement displacement relative to pile caps can be mobilized.

For analysis purposes, the block and arch loading assumptions can be considered boundary solutions, upper and lower extremes, respectively, for possible loading of the reinforcements. Some existing pile supported analysis procedures use arch-type or reinforced-beam type loading assumptions while others use more of a tributary block type loading assumption (*3,4*). Dynamic testing of pile supported embankments has revealed that soil arching may be present after initial construction, but repeated dynamic loading can break down the arching effect and loads in the reinforcements can approach those of the full tributary block type loading. Tall embankments using small pile spacing and not having repeated heavy dynamic loading are more likely to experience reinforcement loading closer to an arching assumption. The greater the roof thickness over the bottom of the arching zone, the more loads it will take to break down the roof and soil arch. Embankments having many repeated dynamic loads or having total heights that are less than about twice the pile spacing are more likely to transition towards a full block type loading of the reinforcements more rapidly after construction.

DESCRIPTION OF THE METHOD

This procedure simulates piles installed in a "square" regular grid spacing pattern, with geogrid/geotextile rolls to be placed in both directions and centered over the pile rows. Geogrid/geotextile rolls are to be wider than the pile spacing used to provide some overlap of adjacent rolls along with total and continuous area coverage. In the geogrid/geotextile roll direction, long overlaps at least 120 percent of the pile spacing in length, or mechanical connections are to be provided to develop tensile strength and stiffness continuity for the reinforcements over each pile row. A minimum of 6 inches of uniform fine to medium sand soil cover should be provided over and below pile top cap assemblies before placement of the first reinforcement layer. Additional reinforcement resistance and lateral pile anchoring or batter piles may be necessary to satisfy global stability design requirements for embankment side slopes, which are not discussed further in this paper. This paper focuses on the analysis of the interior portion of the load transfer platform.

This procedure analyzes both the upper and lower boundary load assumptions (the tributary block and tributary arch-hemisphere loads) as generally described in figure 1. For the lower boundary full arching loading assumption, the weight of a hemispherical volume of soil with diameter equal to the diagonal spacing between piles is simulated as the approximated embankment downward load on the reinforcement materials. For the upper boundary full tributary block loading assumption, the full weight of the tributary block shaped volume of fill between the piles is the simulated load of the reinforcement materials. In both load cases, the assumed embankment weights acting on the reinforcements are converted into approximate equivalent uniform reinforcement line-loads between the piles.

The model assumes a deflection shape for the reinforcements between the pile caps. This is necessary in order to estimate change in length (strain) in the reinforcements. The deflected shape assumption used to estimate tension and end reactions for the reinforcements near pile cap edges is that of a uniformly loaded cable with circular arc-shaped deformation (*7*). For analysis purposes, the loaded "cable" is assumed to be a unit width (1-ft) for a geogrid or geotextile reinforcement material. As this downward arc-shaped displacement and related reinforcement elongation strain occurs, upward soil pressures increase on the bottom of the load transfer platform. The approximate reinforcement displacement shape and strain magnitude can be estimated as a function of the total downward settlement, δ, occurring between the piles. The calculated strain magnitude can in turn be converted into a mobilized geogrid/geotextile tension magnitude for a specific type of reinforcement based on its actual force versus strain properties. The model incrementally increases the reinforcement deflection, δ, and estimates both the corresponding upward soil resistance magnitudes and the mobilized geogrid/geotextile vertical force components. It compares these upward resistances to the design embankment downward loadings for block and arch assumptions and estimates the deflection value, δ, needed for vertical force equilibrium for the two load cases. The model results are the estimated settlement between piles, mobilized reinforcement forces and percent strains, and general estimates of whether grid yielding around piles or in general tension will occur, for the block and arch loading assumptions. The specific analysis steps for the model are described below.

Step 1- Settlement Estimates: The first step in the process is to estimate the amount of settlement that would occur beneath the proposed fill height if the fill were placed without any pile support, $\delta_{NoPiles}$, and using the consolidation characteristics for the specific site soils. This site-specific settlement value is used to establish an approximate long-term static load upward resistance pressure versus total consolidation settlement relation present beneath the load transfer platform. This concept is similar to the concept of soil modulus of subgrade reaction but in this case is based on site consolidation type compressibility rather than local or "small-strain" soil elastic modulus. The soil stiffness value in this analysis has units of 'psf of vertical loading per foot of vertical long-term consolidation settlement'. A constant such as this can be used, or a more realistic or non-linear relation between fill load and settlement can be programmed if desired.

Step 2- Set-up a Pile Supported Embankment Configuration: Table 1 shows the six required inputs (shaded area) for this analysis model. The example problem presented here simulates a 40-ft tall embankment overlying a soft clay and organic soil profile from the Black River Valley near Port Huron, Michigan. Settlement analysis revealed about 48 inches of expected consolidation settlement for the weight of 40 feet of fill over this soil profile. Global stability analysis revealed that the soft soils could not support 40 feet of earth fill without experiencing lateral-squeeze or sliding-block type mass movements. This pile supported embankment analysis model only requires basic inputs including; embankment height, fill unit weight, pile spacing, pile-top load cap diameter, and the total long term estimated settlement values for the design fill height that would occur without pile support. The model also requires use of an estimated geogrid/geotextile tensile force versus strain curve for the simulation. Figure 3 shows a simplified force versus strain curve used for this example analysis. The 2% strain stiffness value is input into the model and a polynomial generates the curve shown, which simulates ultimate strength and reinforcement yielding at about a 14 percent strain level in the grid material.

TABLE 1 Required Input Data and Summary Output Data for the Model

INPUTS		
1-D Grid Force at 2% strain =	10000	lb/ft
Embankment Height, *H* =	40	feet
Fill Soil Unit Weight, γ =	125	pcf
Pile Spacing, s =	10	feet
Pile Cap width/diameter, d =	3	feet
Total Settlement w/o Piles =	4	feet
Calculated Parameters		
tributary block weight =	500	kip
tributary sphere weight =	93	kip
H/(s-d) =	5.7	L/L

SUMMARY RESULTS				
ARCH Analysis				
Settlement between Piles =	*5.8*	inches	*% settl.* =	12%
Arc-Shape Grid Tension =	*7249*	lb/ft	*%Strain* =	1.2%
Grid Force Around Piles =	*7249*	lb/ft	**%Strain* =	0.6%
Soil Load Effective Height =	*4.8*	feet	* double layers	
Pile Load =	*486*	kip		
BLOCK Analysis				
Settlement between Piles =	*15.6*	inches	*% settl.* =	32%
Arc-Shape Grid Tension =	*23781*	lb/ft	*%Strain* =	8.9%
Grid Force Around Piles =	*41578*	lb/ft	**%Strain* =	7.8%
Soil Load Effective Height =	*13.0*	feet	* double layers	
Pile Load =	*392*	kip		

Double-Layer Resistance & Forces over Piles			Tension Angle, deg	Pile Cap Angle, deg
*max. pile cap grid resistance at 5% strain** =	358	kip*		
arch - approx. pile load from grids =	25	kip	15.3	21.4
block- approx. pile load from grids =	285	kip	36.6	46.7

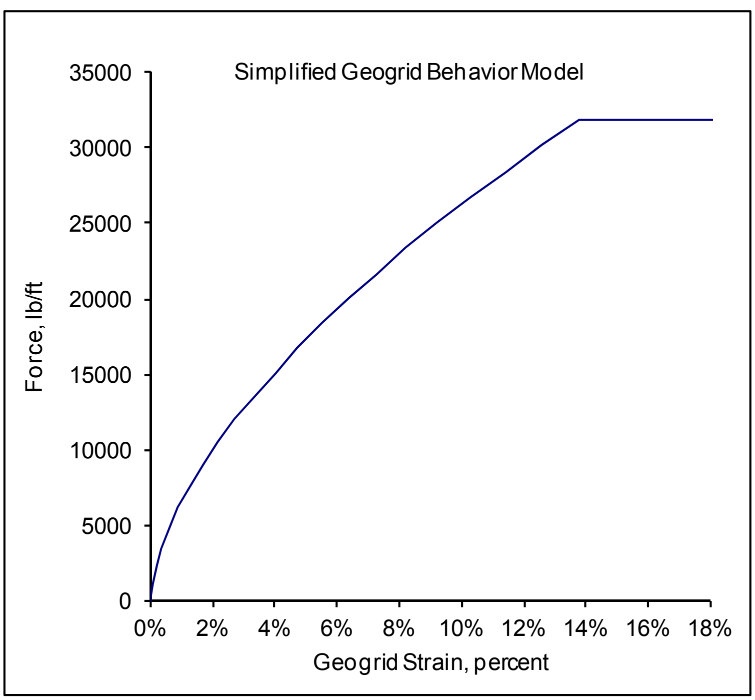

FIGURE 3 Simplified force versus strain model for the reinforcement geogrid/geotextile used in the example analysis.

Step 3- Find Equilibrium Settlement Values for Both Load Cases: Once the model parameters are established, equilibrium force calculations for the reinforcements are performed using two separate methods. The first method calculates the effective cable tension required for equilibrium from the fill weight placed above the load transfer platform as a function of settlement displacement, δ, using uniformly loaded cable theory (*7*). If a cable is uniformly loaded along its length with vertical distributed load (w) and the cable assumes an arc-shaped deformation with deflection δ at the center of the arc, it can be shown that the radius of curvature, R, for the deformed arc circle is calculated as follows:

$$R = \frac{\left(\frac{s-d}{2}\right)^2 + \delta^2}{2\delta} \tag{1}$$

The vertical and horizontal reactions at the ends of the cable are approximately:

$$F_H = \frac{w\ (s-d)^2}{8\delta} \tag{2}$$

$$F_V = \frac{w\ (s-d)}{2} \tag{3}$$

The equivalent downward uniform cable loading from the fill materials for the arch and block load analyses are as follows:

$w_{Block} = \gamma H$, in units of *lb/ft loading per 1-ft wide strip*, or psf (4)

where,

γ = fill soil unit weight, pcf
H = effective embankment plus surcharge height above original ground, feet

$$w_{Arch} = \gamma H\left(\frac{V_{Hemisphere}}{V_{Block}}\right) \tag{5}$$

where,

$V_{Block} = Hs^2$ = volume of tributary block (6)

$V_{Hemisphere} = \left(\frac{1}{2}\right)\left(\frac{4}{3}\right)\pi\left(\frac{0.5(s)}{\sin(45°)}\right)^3$ = volume of tributary hemisphere (7)

The peak upward pressure developed by and on top of the underlying soft soil, mid-way between the piles is estimated to be linearly proportional to the ratio of; the maximum reinforcement downward deflection occurring below pile caps divided by the total site-specific consolidation settlement magnitude predicted to occur if no piling were used, as follows:

$$\text{Max. Soil Upward Pressure} = \gamma H\left(\frac{\delta}{\delta_{NoPiles}}\right),\ \text{psf} \tag{8}$$

The upward soil pressure on the bottom of reinforcements is also assumed to have corresponding upward (reducing) cable-end force effects. The distribution of the upward pressure on the bottom of the load transfer platform is assumed to vary from zero near the pile caps to the maximum value between the pile caps in with a parabolic shape as shown in figure 2. Using a simplified pressure area force concept of; force equal to two-thirds of the base times height for the parabolic upward pressure distribution, the resulting upward (reduction) cable end forces are approximated to be:

$$\text{Cable End Up-Reactions} = \left(\frac{1}{2}\right)\left(\frac{2}{3}\right)(s-d)\ \text{(Max. Soil Upward Pressure), lb/ft} \tag{9}$$

These upward cable end reactions are subtracted from the downward cable end reactions calculated to be caused by the fill above the reinforcements. The overall effective downward cable loading, w_{eff} is determined based on the effective cable end reactions, where the cable end up-reactions above are subtracted from the cable down-reactions caused by w_{Block} or w_{Arch} as follows:

$$w_{eff} = \text{(effective end reaction)}/\{(0.5)(s - d)\} \tag{10}$$

The tension in the cable caused by the arc-shaped downward deformation is then estimated as the square root of the sum of F_H squared and F_V squared for varying w_{eff} and deflection, δ values. Using the geometry of a circle it can be shown that the length, L, of the stretched cable is approximately:

$$L = 2R\sin^{-1}\left\{\frac{(s-d)}{2R}\right\} \tag{11}$$

The percent strain in the geogrid material is calculated as:

$$\%Strain = \left\{\frac{L-(s-d)}{(s-d)}\right\}100 \tag{12}$$

The second method estimates the tension resistance force mobilized in the reinforcement based on the input force versus *%Strain* curve for the material and as a function of the same settlement displacement δ values and compares this to the equilibrium tension force in the theoretical cable for the same deflection. The displacement value where the mobilized reinforcement force caused by reinforcement elongation begins to exceed the calculations for required cable tension force for equilibrium from the cable theory is the solution for the downward reinforcement displacement, δ, occurring between the piles. Tables 2 and 3 show calculated values for the block and arch solutions, for the example problem. The underlined values are the approximate equilibrium reinforcement deflection solutions resulting in the summary data shown in Table 1. A 48-point discretization scheme is used in spreadsheet software to develop the solutions in tables 2 and 3. A finer discretization scheme (expanding the table vertically) can be used for greater precision if desired.

Table 2 Detailed Output for the Tributary Block Analysis

Downward Cable Line Load = 5000 (lb/ft)/ft
Downward Cable Vertical Reaction = 17500 lb/ft

BLOCK	***Upward Soil Force Analysis***			***Ideal Cable Loads Analysis***			***Grid Strain and Resistance Analysis***				***Pile Loads***
Deflection δ feet	Max. Soil Upward psf	Cable End Up Reactions	Cable Down Load, psf	End Vertical =	End Horiz =	Required Tension =	Deflection Radius	Stretched Length	Geogrid Strain, %	Mobilized Grid Tension	Column Load
0.0010	1	3	4999	17497	30619896	30619901	6125.0	7.000	0.0%	5	499917
0.03	31	73	4979	17427	1219896	1220021	245.0	7.000	0.0%	218	497917
0.12	145	338	4903	17162	259112	259680	52.9	7.005	0.1%	1373	490342
0.21	259	603	4828	16897	142973	143968	29.7	7.016	0.2%	2750	482767
0.30	372	868	4752	16632	97759	99164	20.7	7.034	0.5%	4256	475192
0.39	486	1133	4676	16367	73698	75493	16.0	7.057	0.8%	5857	467617
0.48	599	1399	4600	16101	58759	60925	13.0	7.087	1.2%	7531	460042
0.57	713	1664	4525	15836	48582	51098	11.0	7.123	1.8%	9267	452467
0.66	827	1929	4449	15571	41202	44046	9.6	7.165	2.4%	11054	444892
0.75	940	2194	4373	15306	35606	38757	8.5	7.214	3.1%	12886	437317
0.84	1054	2459	4297	15041	31217	34652	7.7	7.268	3.8%	14756	429742
0.93	1168	2724	4222	14776	27682	31379	7.0	7.328	4.7%	16660	422167
1.03	1281	2989	4146	14511	24774	28711	6.5	7.394	5.6%	18594	414592
1.12	1395	3254	4070	14246	22340	26496	6.0	7.465	6.6%	20553	407017
1.21	1509	3520	3994	13980	20273	24626	5.7	7.542	7.7%	22535	399442
1.30	***1622***	***3785***	***3919***	***13715***	***18495***	***23026***	***5.4***	***7.625***	***8.9%***	***24536***	***391867***
1.39	1736	4050	3843	13450	16950	21638	5.1	7.713	10.2%	26555	384292
1.48	1849	4315	3767	13185	15595	20422	4.9	7.806	11.5%	28588	376717
1.57	1963	4580	3691	12920	14397	19344	4.7	7.905	12.9%	30635	369142
1.66	2077	4845	3616	12655	13330	18380	4.5	8.008	14.4%	30635	361567
1.75	2190	5110	3540	12390	12374	17510	4.4	8.117	16.0%	30635	353992
1.84	2304	5375	3464	12125	11512	16719	4.2	8.230	17.6%	30635	346417
1.93	2418	5641	3388	11859	10731	15994	4.1	8.348	19.3%	30635	338842
2.03	2531	5906	3313	11594	10020	15324	4.0	8.470	21.0%	30635	331267
2.12	2645	6171	3237	11329	9370	14702	4.0	8.597	22.8%	30635	323692
2.21	2759	6436	3161	11064	8774	14121	3.9	8.728	24.7%	30635	316117
2.30	2872	6701	3085	10799	8225	13574	3.8	8.864	26.6%	30635	308542
2.39	2986	6966	3010	10534	7717	13058	3.8	9.003	28.6%	30635	300967
2.48	3099	7231	2934	10269	7247	12569	3.7	9.147	30.7%	30635	293392
2.57	3213	7496	2858	10004	6811	12102	3.7	9.294	32.8%	30635	285817
2.66	3327	7762	2782	9738	6404	11655	3.6	9.445	34.9%	30635	278242
2.75	3440	8027	2707	9473	6024	11226	3.6	9.600	37.1%	30635	270667
2.84	3554	8292	2631	9208	5668	10813	3.6	9.758	39.4%	30635	263092
2.93	3668	8557	2555	8943	5334	10413	3.6	9.920	41.7%	30635	255517
3.03	3781	8822	2479	8678	5020	10025	3.5	10.085	44.1%	30635	247942
3.12	3895	9087	2404	8413	4725	9649	3.5	10.253	46.5%	30635	240367
3.21	4009	9352	2328	8148	4446	9282	3.5	10.424	48.9%	30635	232792
3.30	4122	9617	2252	7883	4183	8924	3.5	10.598	51.4%	30635	225217
3.39	4236	9883	2176	7617	3934	8573	3.5	10.775	53.9%	30635	217642
3.48	4349	10148	2101	7352	3698	8230	3.5	10.955	56.5%	30635	210067
3.57	4463	10413	2025	7087	3474	7893	3.5	11.136	59.1%	30635	202492
3.66	4577	10678	1949	6822	3261	7561	3.5	11.318	61.7%	30635	194917
3.75	4690	10943	1873	6557	3058	7235	3.5	11.500	64.3%	30635	187342
3.84	4804	11208	1798	6292	2865	6913	3.5	11.682	66.9%	30635	179767
3.93	4918	11473	1722	6027	2681	6596	3.5	11.864	69.5%	30635	172192
4.03	5031	11738	1646	5762	2505	6283	3.5	12.046	72.1%	30635	164617
4.12	5145	12004	1570	5496	2337	5973	3.5	12.227	74.7%	30635	157042

TABLE 3 Detailed Output for the Tributary Sphere Arching Analysis

Downward Cable Line Load = 926 lb/ft
Downward Cable Vertical Reaction = 3240 lb

ARCH	*Upward Soil Force Analysis*			*Cable Loads Analysis*			*Grid Strain and Resistance Analysis*				*Pile Loads*
Deflection δ feet	Max. Soil Upward psf	Cable End Up Reactions	Cable Down Load, psf	End Vertical =	End Horiz =	Required Tension =	Deflection Radius	Stretched Length	Geogrid Strain, %	Mobilized Grid Tension	Column Load
0.0010	1	3	925	3237	5664037	5664038	6125.0	7.000	0.0%	5	499971
0.03	31	73	905	3167	221662	221685	245.0	7.000	0.0%	218	499271
0.12	145	338	829	2901	43807	43903	52.9	7.005	0.1%	1373	496620
0.21	259	603	753	2636	22308	22463	29.7	7.016	0.2%	2750	493968
0.30	372	868	677	2371	13938	14138	20.7	7.034	0.5%	4256	491317
0.39	486	1133	602	2106	9484	9715	16.0	7.057	0.8%	5857	488666
0.48	***599***	***1399***	***526***	***1841***	***6718***	***6966***	***13.0***	***7.087***	***1.2%***	***7531***	***486015***
0.57	713	1664	450	1576	4834	5085	11.0	7.123	1.8%	9267	483363
0.66	827	1929	374	1311	3468	3708	9.6	7.165	2.4%	11054	480712
0.75	940	2194	299	1046	2432	2648	8.5	7.214	3.1%	12886	478061
0.84	1054	2459	223	780	1620	1798	7.7	7.268	3.8%	14756	475410
0.93	1168	2724	147	515	965	1094	7.0	7.328	4.7%	16660	472758
1.03	1281	2989	71	250	427	495	6.5	7.394	5.6%	18594	470107
1.12	1395	3254	0	0	0	0	6.0	7.465	6.6%	20553	467456
1.21	1509	3520	0	0	0	0	5.7	7.542	7.7%	22535	464805
1.30	1622	3785	0	0	0	0	5.4	7.625	8.9%	24536	462153
1.39	1736	4050	0	0	0	0	5.1	7.713	10.2%	26555	459502
1.48	1849	4315	0	0	0	0	4.9	7.806	11.5%	28588	456851
1.57	1963	4580	0	0	0	0	4.7	7.905	12.9%	30635	454200
1.66	2077	4845	0	0	0	0	4.5	8.008	14.4%	30635	451548
1.75	2190	5110	0	0	0	0	4.4	8.117	16.0%	30635	448897
1.84	2304	5375	0	0	0	0	4.2	8.230	17.6%	30635	446246
1.93	2418	5641	0	0	0	0	4.1	8.348	19.3%	30635	443595
2.03	2531	5906	0	0	0	0	4.0	8.470	21.0%	30635	440943
2.12	2645	6171	0	0	0	0	4.0	8.597	22.8%	30635	438292
2.21	2759	6436	0	0	0	0	3.9	8.728	24.7%	30635	435641
2.30	2872	6701	0	0	0	0	3.8	8.864	26.6%	30635	432990
2.39	2986	6966	0	0	0	0	3.8	9.003	28.6%	30635	430338
2.48	3099	7231	0	0	0	0	3.7	9.147	30.7%	30635	427687
2.57	3213	7496	0	0	0	0	3.7	9.294	32.8%	30635	425036
2.66	3327	7762	0	0	0	0	3.6	9.445	34.9%	30635	422385
2.75	3440	8027	0	0	0	0	3.6	9.600	37.1%	30635	419733
2.84	3554	8292	0	0	0	0	3.6	9.758	39.4%	30635	417082
2.93	3668	8557	0	0	0	0	3.6	9.920	41.7%	30635	414431
3.03	3781	8822	0	0	0	0	3.5	10.085	44.1%	30635	411780
3.12	3895	9087	0	0	0	0	3.5	10.253	46.5%	30635	409128
3.21	4009	9352	0	0	0	0	3.5	10.424	48.9%	30635	406477
3.30	4122	9617	0	0	0	0	3.5	10.598	51.4%	30635	403826
3.39	4236	9883	0	0	0	0	3.5	10.775	53.9%	30635	401175
3.48	4349	10148	0	0	0	0	3.5	10.955	56.5%	30635	398523
3.57	4463	10413	0	0	0	0	3.5	11.136	59.1%	30635	395872
3.66	4577	10678	0	0	0	0	3.5	11.318	61.7%	30635	393221
3.75	4690	10943	0	0	0	0	3.5	11.500	64.3%	30635	390570
3.84	4804	11208	0	0	0	0	3.5	11.682	66.9%	30635	387918
3.93	4918	11473	0	0	0	0	3.5	11.864	69.5%	30635	385267
4.03	5031	11738	0	0	0	0	3.5	12.046	72.1%	30635	382616
4.12	5145	12004	0	0	0	0	3.5	12.227	74.7%	30635	379965

This example analysis estimates that for the reinforcement material simulated (stiffness of 10,000 lb/ft force mobilized at 2% strain, placed in both directions continuously over pile caps), the lower boundary loading full arching simulation generates just over a one percent strain level in the reinforcement materials, with an estimated settlement of about 6 inches between piles. If the arching were to completely break down from vibrations or creep effects, resulting in full tributary block loading, settlement is predicted to rise to an estimated 16 inches with reinforcement strain levels approaching 7 to 8 percent. This is compared to a consolidation settlement estimate of about 48 inches for the 40-ft tall fill without pile support. With full soil-arching present, the pile/column loads are estimated to be about 486 kips. If the arches break down, allowing the soil between the piles to take more of the load, pile top forces are estimated to drop to about 392 kips. This analysis does not consider the increase in total pile loads due to increasing negative skin friction forces from settlement. The results noted are the estimated forces from the fill and reinforcements applied to the pile tops. These values are compared to the tributary block soil fill weight of 500 kips per pile. The estimated effective height of the embankment being applied to the underlying soft soil between the piles is reduced to about 5 and 13 feet for the arch and block loading conditions, respectively, for this 40-ft tall embankment simulation.

Step 4- Estimate Grid Force Concentrations Around Pile Caps: For the analysis assuming full arching is present, the reinforcement near the pile caps is assumed to remain oriented like the ideal loaded cable with arc-shaped deformation. However, for the analysis assuming a full break-down of soil arching, the reinforcement immediately near the pile cap is assumed to re-orient somewhat to have a steeper load transfer angle right near the cap edge. At worst case, the localized punching-type deformation would cause the reinforcement to reach a nearly vertical orientation surrounding the pile cap assemblies (*10*). One check performed is to assume this vertical orientation exists and to quantify the maximum reinforcement punching resistance present at 5 percent strain that would be generated if this highly localized deformation condition were to develop. The 5 percent strain level is considered to be just below a creep deformation threshold for some common types of geogrid reinforcement material. The model reports the vertical resistance force that would be mobilized around the pile cap at a 5 percent strain level in the reinforcements being simulated. This resistance is quantified by multiplying the pile cap perimeter length times the number of reinforcement layers provided over the cap (minimum of two layers over each pile, one in each direction) times the reinforcement material 5% strain mobilized force value in lb/ft. This is one limiting value to consider in the reinforcement design. This index of maximum available reinforcement upward resistance around the pile cap is compared to two parameters. The first is the estimated reinforcement tension magnitude caused by the arc curvature shape and the second is the estimated total pile cap load divided by the pile cap perimeter length. It is assumed that by keeping the calculated reinforcement forces around the pile cap less than the 5% strain limit for the grid material, long term yielding of reinforcements around pile tops will be kept low. The mobilized grid deflection angle near the pile cap edge due to the pure arc shaped deflection is estimated as follows:

$$\text{Reinforcement Arc-shape Tension Deflection Angle near Cap} = \tan^{-1}\left(\frac{F_V}{F_H}\right) \tag{13}$$

The mobilized deflection angle right near the pile cap edge is estimated as follows:

$$\text{Localized Pile Cap Edge Deflection Angle} = \tan^{-1}\left(\frac{0.5(w_{eff})(s-d)}{F_H}\right) \tag{14}$$

where, the w_{eff} value is the effective uniform Cable Down Load values in column 4 from Tables 2 and 3.

Table 1 also shows the summary results for the example problem analysis. The full block loading would result in an estimated 8 percent strain level in the reinforcements, which would likely be beyond a tensile creep limit threshold for some types of reinforcements. However, it is generally understood that the full block loading is not likely to fully mobilize at the site because it is a conservative upper boundary type loading assumption, and the 'fill height to reinforcement span' ratio, $H/(s\text{-}d)$, value for this simulation is 5.7, which is greater than the minimum ratio needed for arching to develop. The 5 percent strain level resistance provided around the pile caps in this design example is about 358 kips and this is roughly 90 percent of the full block analysis predicted loading value around the pile caps. The designer would generally assume in this case that the reinforcement loading near the pile caps would start at some level slightly above the lower boundary arch-analysis results shortly after construction, say approximately 100 kips per pile cap perimeter. Continuing soil consolidation and repeated heavy truck traffic could eventually force the reinforcement loading near pile cap edges to approach the full block and in this case we have placed the 5 percent strain level resistance in the reinforcements at about 80 to 90 percent of the full block loading estimated tension for the simulated grid material. This would be considered a heavy duty design for numerous repeating loads and almost complete arching breakdown.

SENSITIVITY ANALYSES FROM THE NEW PROCEDURE

Figure 4 shows the sensitivity of the calculated grid stiffness requirements to variation in pile spacing and embankment height for two soil profile simulations, one having a thick layer of compressible organic soils (48 inches of settlement for 40 feet of fill, without piles) and one having a thinner layer (16 inches of settlement without piles). The grid stiffness trends shown are the stiffness values needed to keep the estimated strain level in the reinforcements to a level of about 5% strain for the full block loading conditions, except for where the lines are paralleling the gray banded zone on the plot. The grey banded zone represents the critical minimum stiffness for the reinforcements for the lower-settlement soil profile, where reinforcement stiffness values lower than the highlighted boundary are predicted to be too soft to substantially reduce settlement or transfer loads to piles. For the lower-settlement profile simulations, less than 5% settlement reduction is predicted for reinforcement stiffness values below the gray banded zone. The grid stiffness required is less for the lower-settlement profile for short pile spacing, but because the settlements are lower, the amount of strain that can ultimately be mobilized by grid deformation is limited. Therefore, stiffer reinforcements are needed to mobilize any upward resistance beyond a point. Regarding the chosen 10-ft, 28-ft and 40-ft tall simulations, 10-ft tall soil embankments are typically too tall to be constructed over soft to very soft soils without special reinforcements and/or ground improvements to enhance global stability and lateral squeeze resistances. A 28-ft tall embankment is a common height for highway bridge approach embankments for bridges over roadways. A 40-ft tall embankment is a common bridge approach embankment height for highways passing over railroad tracks.

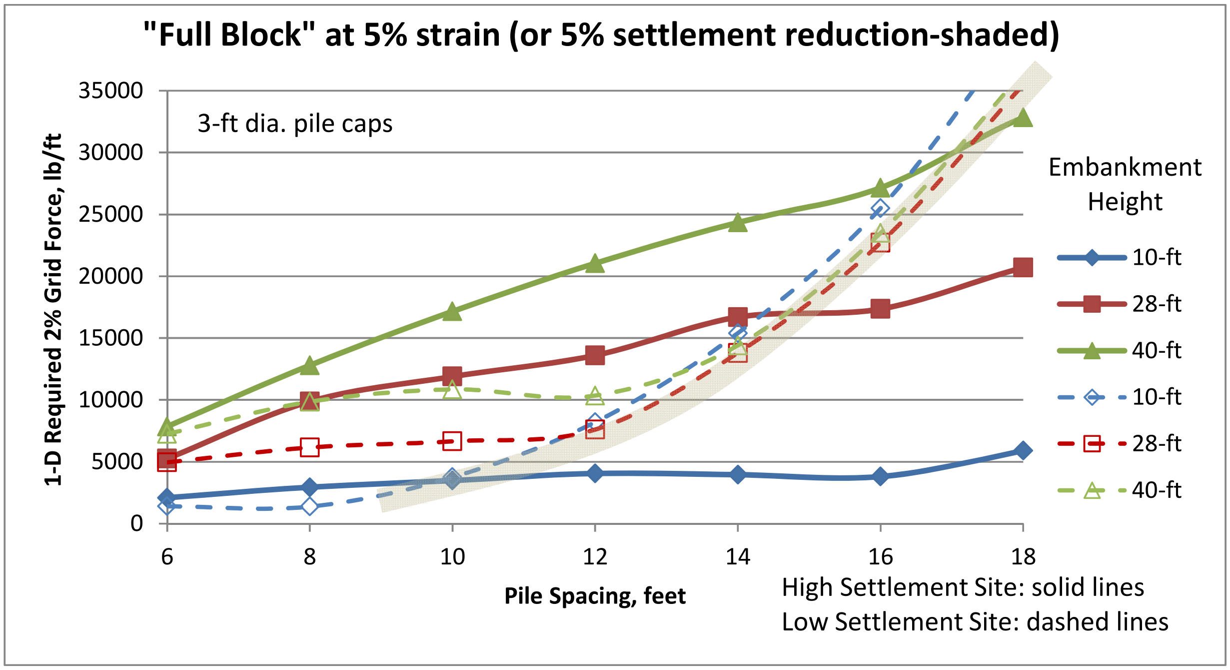

FIGURE 4 Sensitivity of the analysis procedure to fill height and pile spacing and showing grid stiffness values needed to have an estimated 5% strain level in the reinforcements, and the apparent minimum-stiffness boundary concept.

Figure 5 shows solutions for the same high and low settlement profiles but now showing the estimated reinforcements 2 percent strain level stiffness needed to develop a 50 percent settlement reduction between the piles as compared to if piles were not used. Significantly greater reinforcement stiffness is needed to reduce settlements from 16 to 8 inches over the low settlement profile, as compared to reducing settlements from 48 to 24 inches over the high settlement profile. If an engineer wanted to use a pile spacing of 10 feet for a design and also provide a settlement reduction of at least 50 percent for both soil profiles, a reinforcement tensile stiffness of about 30,000 lb/ft would be needed for the low settlement site, where as only about 5,000 lb/ft would be needed for the high settlement site. These equal settlement percent reductions provide approximately equal vertical stress reductions on the underlying soft soils. If the engineer wanted to use a constant 10,000 lb/ft at 2 percent strain reinforcement platform design, the pile spacing selected would need to be about 8 feet for the low settlement profile and about 12 feet in the higher settlement profile area. Greater pile spacing would be allowed for the higher settlement area, which is purely related to strain compatibility effects between the reinforcements and the underlying subsoil settlements.

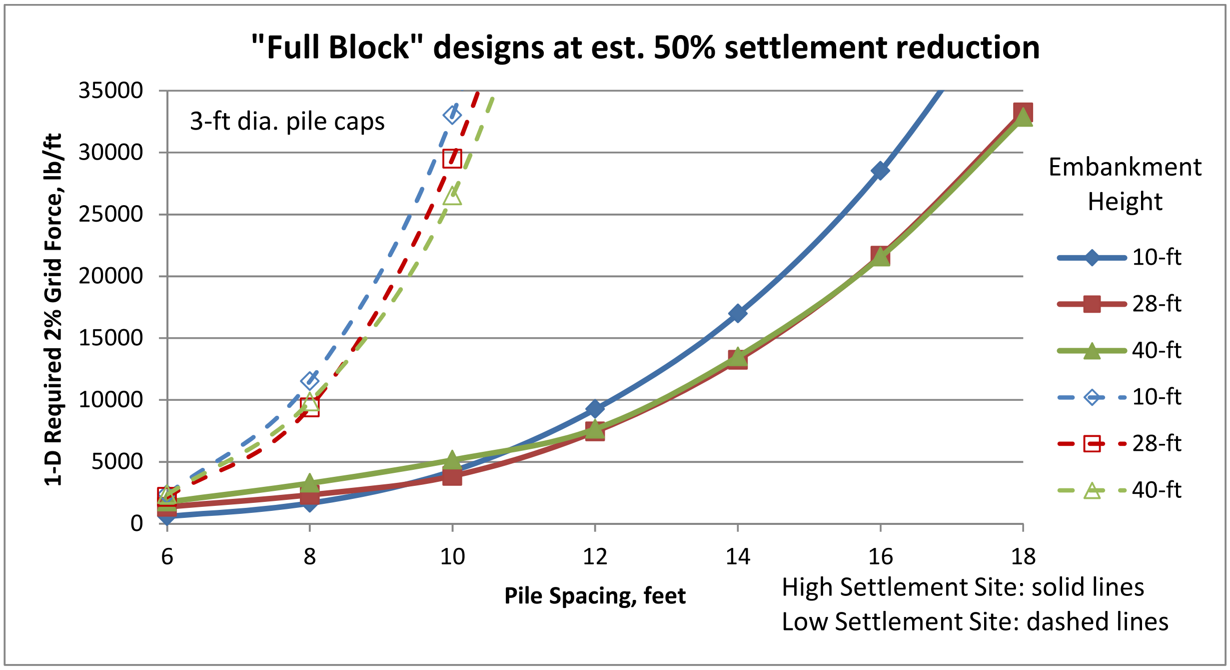

FIGURE 5 Sensitivity of the analysis procedure to site soil compressibility and fill height for constant percent settlement reduction solutions of 50 percent.

It should be noted that high strength geogrids and geotextiles are often specified using a required minimum mobilized reinforcement force at a given strain level. The 2 percent or 5 percent strain mobilized force values are often used for reinforcement specifications and provided on manufacturer's specification data sheets for the products. It is for this reason that this analysis procedure converts the results back to a "required reinforcement stiffness/force mobilized at 2 percent strain, in units of lb/ft" as the solution results.

COMPARISON TO EXISTING FIELD CASE STUDIES

Camp and Siegel (*8*) describe a settlement related functional failure of a pile supported embankment constructed in South Carolina. It was a shallow embankment reportedly less than 4 feet tall constructed over concrete piles spaced at about 8.2 feet and using 3-ft diameter pile caps. Light-duty reinforcements were used providing only approximately 1150 lb/ft of tensile resistance at a 2% strain level. Analysis of this case study using this procedure indicates the reinforcements used would have been too flexible, resulting in relatively large settlements between piles, and that the strain levels in the reinforcements would have likely been below creep levels in the reinforcements. This was a case study that highlighted the need for evaluation of strain compatibility between reinforcement stiffness and expected underlying soil settlements.

Hoppe and Hite (*9*) describe an instrumented pile supported embankment constructed in Virginia. The embankment used piles spaced at about 7 feet with 3-ft diameter pile caps and total embankment height of about 6 feet. Heavy reinforcements were used resulting in about 5,000 lb/ft of reinforcement resistance available at a 2% strain level. The researchers reported a maximum settlement between piles of about 2.45 inches. The analysis model presented here predicts that about 3.4 inches of settlement would have occurred between piles and that the reinforcements would be functioning at a low 1 to 2 percent strain level.

Van Eekelen et al (*4*) describe an instrumented pile supported embankment and compare results to the EBGEO German design procedure and the BS8006 design procedure from Britain describing the key features of each procedure. It was a shallow roughly 3.8-ft tall fill placed on timber piles spaced at about 4.2 feet apart and using approximately 1-ft diameter pile caps. Their analysis showed significant differences between the two design procedures. The instrumentation data used indicated that the upward resistance offered by the underlying soft soil must be considered. Their analysis showed a high sensitivity for required reinforcement stiffness versus percentage load transferred to the piles versus the underlying soft soil. Their data indicates that as soft soil consolidation occurred after construction, pile loads increased and arching continued to develop over a period of about 1 year and then stabilized.

SUMMARY AND CONCLUSIONS

This paper presents a simplified structural analysis procedure for estimating the geogrid/geotextile reinforcements and pile forces for pile supported embankments having flexible reinforced load transfer platforms. None of the previously existing procedures for design of pile supported embankments reviewed were purely mechanistic and directly considering strain compatibility between the reinforcement deformation and the estimated settlements of the soft soils underlying the load transfer platform reinforcements. Therefore, this procedure was visualized and developed in spreadsheet software. Force mobilization in the reinforcement material requires that some settlement occur, stretching the reinforcement materials and mobilizing tensile forces. This analysis procedure simulates this strain compatibility effect.

There has been considerable debate whether or not embankment loads acting on the load transfer platform reinforcements behave more like an arching assumption, or more like a full tributary block assumption. This analysis procedure provides solutions for both cases and allows the design engineer to decide whether the system will have arching or will perhaps lose the arching and evolve towards full tributary block type loading. Factors inluding repeated heavy dynamic loads, such as interstate truck traffic, have been shown to be able to break down soil arching, slowly increasing reinforcement loads over time. In general, if embankment height is significantly greater than pile spacing, arching is more likely to remain intact for a longer time period and over more dynamic loads. When embankment height is about the same as the pile spacing, arching will not fully develop and loading closer to full tributary block conditions is likely to develop. Piles or columns that are axially weak and yield as a result of embankment related axial loads will simply allow more fill weight to be applied to the soft soil and less into the piles or reinforcements. It is the differential deflection occurring in the soft soil between the pile caps and the pile tops that mobilizes the reinforcement tension.

This new procedure estimates the forces in the reinforcements near the pile cap assemblies. For the arch loading case the reinforcements are assumed to remain oriented like the ideal uniformly loaded cable assumption while most fill soil weight is distributed to the piles and shear forces around the pile cap perimeter are assumed to remain small. For the transition to a full tributary block loading, the reinforcement is assumed to reorient into a steeper load transfer angle locally around the pile cap assemblies as shear forces become significant at the pile cap edges and piles begin to try to punch upward through the load transfer platform and into the fill materials. Estimates of percent strain and load transfer angles in the reinforcements near the pile cap are provided.

In general, this model was developed as part of the author's consulting engineering practice for design of embankments over soft soils. A review of existing models revealed a need for more rational design procedures for pile supported embankments based on strain compatibility between the reinforcement elongation and the expected settlements beneath the reinforcements. A simplified mechanistic structural analysis procedure that can be used by practicing engineers was needed. This simplified framework is easy to program and use, and could be calibrated using empirical coefficients to match output from more costly and advanced analytical procedures and data from instrumented test sites. The spread sheet software is available from the author.

REFERENCES

1. Filz, G.M., Smith, M.E. *Design of Bridging Layers in Geosynthetic-Reinforced Column-Supported Embankments*. Virginia Transportation Research Council (VTRC) Report No. 06-CR12. 2006.

2. Naughton, P.J. The Significance of Critical Height in the Design of Piled Embankments. In the *proceedings of the GEODENVER 2007 conference*. American Society of Civil Engineers, 2007.

3. Collin, J.G., Han, J., and Huang, J. (2005), Geosynthetic-Reinforced Column-Supported Embankment Design Guidelines, *proceedings of the North American Geosynthetics Society/Geosynthetics Institute Conference*, Las Vegas, Nevada.

4. Van Eekelen, S., Bezuijen, A., Alexiew, D. Piled Embankments in the Netherlands, a Full-Scale Test Comparing 2 Years of Measurements with Design Calculations. In the *proceedings of EuroGeo4*, paper number 264. 2008.

5. Chen, R.P., Chen, Y.M., Han, J., Xu, Z.Z. Theoretical Solution for pile-supported embankments on soft soils under one dimensional compression. *Canadian Geotechnical Journal*, Volume 45, 2008.

6. Filz, G.M., Smith, M.E. Net Vertical Loads on Geosynthetic Reinforcement in Column-Supported Embankments. *Soil Improvement, GSP 172*, Geo-Denver conference (on CD-ROM), ASCE, Reston, VA, 10 p. 2007

7. Sack, R.L. *Structural Analysis*. The McGraw-Hill Book Company. 1984.

8. Camp, W.M. III and Siegel, T.C. Failure of a Column-Supported Embankment over Soft Ground. In the *proceedings of the Fourth International Conference on Soft Soils*, 2006.

9. Hoppe, E.J., Hite, S.L. *Performance of a Pile-Supported Embankment*. Virginia Transportation Research Council (VTRC) Report No. 06-R36. 2006.

10. Liu, H.L., Charles, W.W., Fei, K. Performance of a Geogrid-Reinforced and Pile-Supported Highway Embankment over Soft Clay: Case Study. *Journal of Geotechnical and Geoenvironmental Engineering*. Vol. 133, No. 12, December 2007.