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DEEP FOUNDATION DESIGN NEAR FLOOD PROTECTION PROJECTS

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ABSTRACT

Flood protection structures (flood walls, pump plants, railroad or vehicular access opening and gatewell structures) have foundations subjected to hydrostatic pressures during flood conditions. The foundation sands below these structures are likely to produce hydrostatic pressure elevation heads which measure higher than the top of ground at the structures. Often the foundations for these structures are deep enough to penetrate the sands into the artesian pressures. Deep foundation design for the flood protection structures utilizes software programs based on load testing in many different types of soils. The load tests used to model soil-structure responses to loading are conducted on moist soils and sometimes saturated soils. Standard load testing does not model the hydrostatic load case. The high river stage will significantly change the foundation resistance for cohesionless materials. A procedure for adjusting the foundation sands subjected to excessive pore pressures is suggested.

INTRODUCTION

Deep foundation designers have many programs available for developing the pier capacities and the associated lateral and vertical deformations. The programs model vertical and lateral responses for many different types of loading conditions.

Deep foundation design procedures for moist and saturated ground conditions are well documented. The average practicing geotechnical engineer is not considering artesian pressures developed during a flood event. Many designers simply apply a common assumption that saturated ground conditions will model the most critical loading resistance for the deep foundation piers. This assumption is not conservative when the foundations are subjected to excessive gradients in the foundation sands due to artesian conditions. A simple procedure can be used to model the foundation reaction of foundations sands exposed to high river gradients.

Geotechnical designers should consider the pressures developed in the foundation sands for designing deep foundation which penetrate the sands in areas adjacent to flood protection projects. The confined flow in the foundation sands below an impervious blanket of cohesive material will develop artesian pressure heads at the base of the blanket. The amount of pressure head, developed in the cohesive blanket is generally modeled on the basis of the ratio of permeability between the foundation sand and the blanket, the thickness of the materials, the differential head on the foundation, and the seepage path. The foundation pressures can be determined using an underseepage analyses. The resulting pressure increase in the foundation due to the flood condition should be used to adjust the vertical and lateral soil resistance in the foundation. An appropriate factor of safety for the normal, unusual, and extreme load case should be assigned. Generally a lower factor of safety is utilized for the higher risk case.

CHANGING FOUNDATION STRESSES

The foundations in the Missouri river valley generally consist of alluvial sand deposits on bedrock varying from 25 to 40 meters thick in the Kansas City area. Alluvial deposits consisting of silt and clay overlie the foundation sands. Groundwater levels vary throughout the year and are dependent on location adjacent to the river. Low groundwater levels can represent the maximum resistance available for vertical and lateral resistance to any given loading.

During a period of heavy rainfall and recharging of the water table, the foundation can become saturated. As the river rises the pressures in the foundation sands increase due to the confinement of the cohesive blanket. An artesian pressure condition can develop at some river level. The artesian pressure can result in a pressure head higher than the ground surface but lower than the river level. The impact to the foundation sand vertical and lateral loading resistance is dependent upon the pressure in the foundation sands.

Figure 1 illustrates a possible condition that can occur adjacent to a flood protection project during high river stage. Figure 2 illustrates a typical floodwall with deep foundation support.

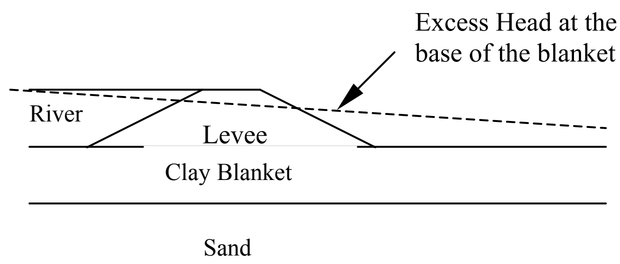

Fig. 1. Typical Levee Section in the Missouri River Valley in the Kansas City Area.

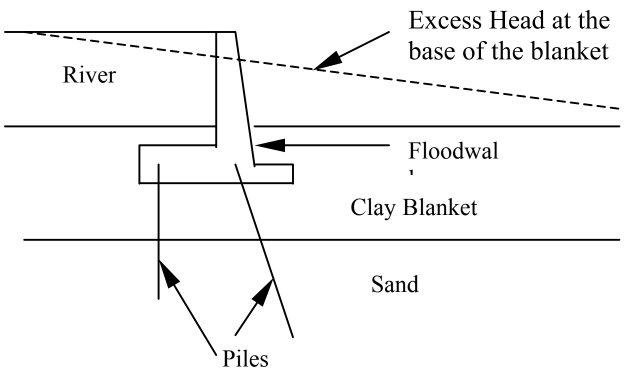

Fig. 2. Typical Floodwall Section with Deep Foundation.

The artesian head conditions established by the rise in the river stage can significantly lower the effective overburden pressures in the foundation. The effective stresses in the foundation sands and soil blanket materials decrease in magnitude below that assumed for the saturated condition. The lower effective stresses decrease the amount of vertical and lateral load resistance available. The vertical load resistance (T) with vertical deformation (z) and the lateral soil resistance (P) with lateral deflection (y) are illustrated for three conditions in figures 3 and 4.

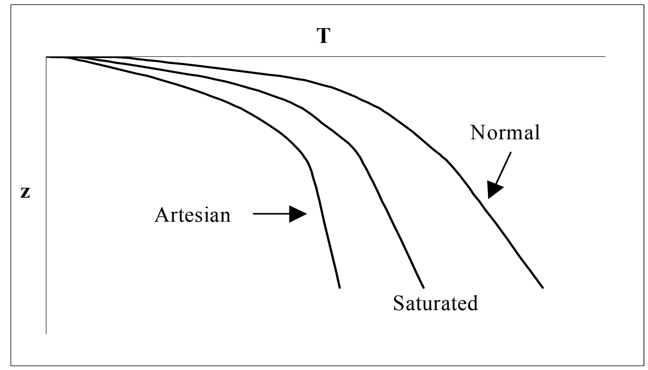

Fig. 3. T-Z Vertical Soil Resistance

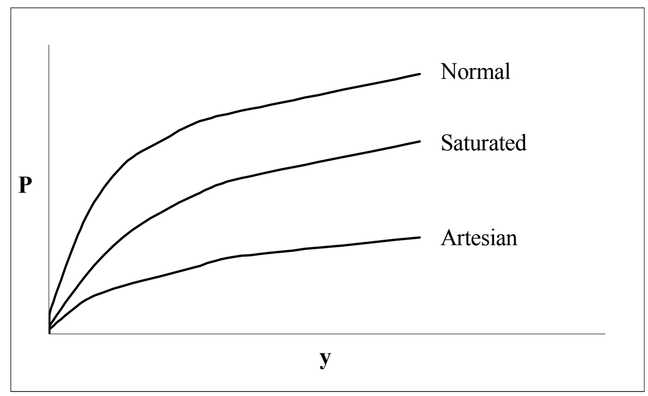

Fig. 4. P-y Lateral Soil Resistance

The decrease in the vertical and lateral load resistance is dependent upon the increase of the artesian pressure in the foundation sands.

Engineers, not generally familiar with flood protection design, design for side resistance or end bearing using the normal or saturated soil condition. Computer software has been developed to model the behavior of piles based on load tests in moist and saturated soil conditions. The programs require the maximum side friction, tip resistance, and horizontal modulus of elasticity to be provided. These input parameters should be adjusted for the flood condition.

CAPACITY DESIGN AND DEFORMATION CONTROLS

Designers select the controlling criteria for their projects.
They can determine the ultimate capacity for a foundation based on field or laboratory results. A factor of safety is applied to the ultimate capacity to obtain the allowable capacity. The ultimate limit state (ULS) and serviceability limit state (SLS) should be

determined. An appropriate factor of safety can be applied to the ULS. The deformations are calculated for the assigned allowable capacities. The maximum loading condition may be dependent upon the probability of a high river stage. Lateral or vertical loading may dominate the project design. The worse case would be the high loading occurring simultaneously with the lowest foundation resistance. A lower factor of safety is assigned to this load case. If it is determined that the load case for the project without high river stage provides the highest risk to the client, then pier load tests are a valid representation of the foundation behavior during loading. If it is determined that the high river stage will control the design, then the designers should feel obligated to determine the effects of the hydraulic gradient on the foundation behavior.

The lateral and axial resistance of a cohesive material may not be adversely affected by pressure increases due to the low permeability characteristics of the material. Highly dessicated clay can react to the high pore pressure due to open lateral cracks and vertical fissures. Silts may behave as sands depending on the plasticity index of the silt.

Vertical Loading

The vertical load resistance of the pier foundation will consist of a combination of the side resistance and a portion of the maximum tip resistance. The side resistance in sands will be directly proportional to the effective overburden pressures in the sands as a result of the depositional history. A decrease in the side resistance of a foundation pier can result in more load being transferred to the lower portion of the pier. Table 1 and 2 present a list of axial side resistance and end bearing resistance relationships found in the geotechnical literature. Several of the relationships use the effective overburden pressure. The coefficient of lateral earth pressure is required and depends on the chemical or physical deposition of the sand materials and the maximum past overburden pressure. Some of the equations use the standard penetration test (SPT) value. The SPT is a measure of the foundation condition that depends upon the level of groundwater at the time of testing. These equations can be used to estimate the ULS. Other equations are based on the soil parameters or load testing.

Table 1. Ultimate Limit State Side Resistance Capacity Equations

Unit Side Resistance, q_s (Ton/sq ft)	Authors	Basis
$K * \sigma_v' * \tan \phi'$ (use consistent units)	Touma and Reese	Soil parameters Assessment of Depositional History
$\beta * \sigma_v'$ where $\beta > 0.25$ and $\beta = 1.5 - (0.135 * z^{0.5})$ where z = depth but qs < 2 tsf	Reese and O'Neill	Field Load Testing
N / 100 where N = standard penetration test (SPT) blow count	Meyerhof	Dependent on the SPT blow counts
0.026 * N but $q_s < 2$ tsf	Quinos and Reese	Field SPT tests
N / 34 for N<53	Reese and Wright	Field SPT Tests

Table 2. Ultimate Limit State End Bearing Side Resistance Capacity Equations

End Bearing Resistance, q_b Ton / sq ft	Authors	Basis
$\sigma_v' * N_{qp}$ where $N_{qp} = (e^{((270-\phi)/180)*\pi*\tan\phi'}) / \{2*\cos^2[45+(\phi'/2)]\}$	Bowles	Soil Parameters Assessment of Depositional History
0 for loose sand 16 / k for medium sand 40 / k for dense sand where k=0.6 for dp >1.67 only if db > 10 dp db = depth of pier and dp = diameter of pier	Touma, Reese and Quinos	Empirical based on Field Load Tests
(2*Ncorr)/(15*Dp) but < 4/3 * Ncorr in tsf where Ncorr = $(0.77*\text{Log10}(20/\sigma_v'))*N$ where N = uncorrected SPT blow count	Meyerhof	Field SPT Tests
[(2/3)*N] tsf for N < 60 40 tsf for N > 60	Reese and Wright	Field SPT Tests
0.6 N for N < 75 45 tsf for N > 75	Reese and Wright	Field SPT Tests

The SLS is selected based on the designer's tolerable amount of movement. The deformation control or limiting criteria can be dependent upon the acceptable aesthetic limit or a maximum tolerable movement. Other limits may control the allowable loading. Table 3 presents relationships found in the geotechnical literature which estimate the deformation for a given loading.

Table 3. Service Limit State Deformation Control Equations

Equation	Type	Authors
$f_s = w / [(1/E_s)+(1/E_s)+(1/f_{max}*w))]$ where $E_s = A * (\sigma_t - u_w)$ $f_{max} = K_o * (\sigma_t - u_w) * \tan \phi'$ A = density constant σ_t = total overburden pressure at depth considered u_w = porewater pressure ϕ' = effective friction angle w = vertical deformation fs = calculated unit load transfer in psf fmax = maximum unit laod transfer in psf Es = soil modulus in psf/in	Vertical deformation	Mosher in a series of load tests
$\rho_t = \rho_f + \rho_{tt} + \rho_{ts}$ where $\rho_f = (Q_t+\alpha*Q_s)*(D/(A*Ec))$ and $\rho_{tt} = (C_t*Q_t)/(B*q_{ult})$ and $\rho_{ts} = (C_s * Q_s)/ (D * q_{ult})$ where q_{ult} = ultmate bearing C_t and C_s are empirical coefficients Qt = ultimate tip load Qs = ultimate side load B = pier diamter D = length of pier	Compression settlement	
$\rho_u = ((Q_u - W)/ (D/(E_s)) * I\rho$ where $I\rho$ = uplift coeffient W = weight of pier Q_u = ultimate load	Uplift Deformation	

Lateral Loading

The ultimate lateral load resistance of foundation sands has been represented using load test data or empirical relationships. The empirical relationships correlate the expected capacity to a measured deformation using field load tests and known subsurface conditions. These relationships do not model foundation conditions adjacent to flood protection levees or floodwalls during high river stages.

The relationships used to develop the horizontal resistance of the foundation sands to lateral loading are based on an initial elastic response and a secondary curvilinear response from load test behavior in foundation sands. Two of the relationships developed for defining the reaction of the sands incorporate the effective overburden pressure, the sand strength and the depositional history of the sands.

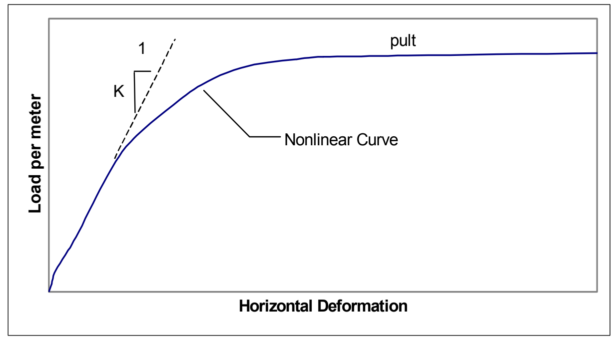

Fig. 5. Horizontal Load Deformation Resistance

The relationship proposed by Terzaghi for the initial elastic response of a sand deposit was expressed as a coefficient of horizontal subgrade reaction K, where

$$K = (A * \sigma_v') / (1.35 B) \tag{1}$$

B = pier diameter
σ_v' = effective overburden pressure
A = coefficient to represent the relative density of the sands.

Terzaghi's recommendation for the A value is provided in table 4 below.

Table 4. Terzaghi Relative Density Coefficient "A" for Sands

Sand Relative Density	Coefficient A
Loose	200
Medium	600
Dense	1500

The initial lateral soil modulus of subgrade reaction, commonly referred to as K, is shown in Figure 5. Three conditions have been developed to illustrate the loss in resistance with increase in foundation pressures as shown on Fig. 6. This chart uses a pier diameter of 0.61m and is calculated for the depth of 1.52 meters. An increase in depth will result in an increase in the reaction.

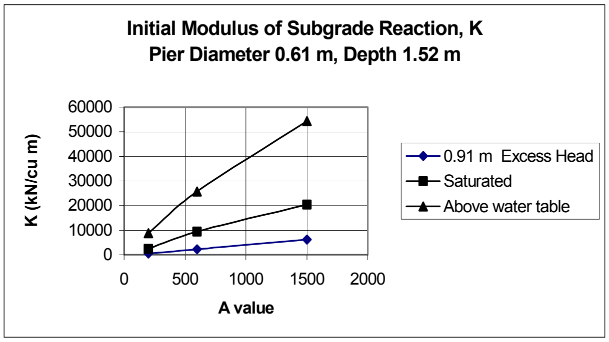

Figure 6 : Comparison of the Terzaghi "K" for Given Pier and water Conditions

Beyond the initial elastic portion of the relationship defined by the coefficient K, the lateral deformation increases without any increase in the ultimate load. The limiting load has been reached. Reese, Cox and Koop utilized a series of tests to develop the expected curvilinear response approaching the limiting ultimate resistance. Their equation uses the effective overburden pressure.

The curvilinear portion of the lateral resistance can be modeled using the smaller of the relationships (2) and (3) to determine the ultimate resistance for given depth, effective overburden pressure, sand strength, and depositional history.

$p_u = \sigma_v' * [D* (K_p-K_a)+2*Kp*\tan \phi'*\tan (45 + \phi'/2)]$
(2)

$p_u = \sigma_v' * D* [K_p^3 + 2 *K_o*K_p^2* \tan \phi' + \tan \phi'-K_a]$
(3)

D : pier diameter
σ_v' : effective overburden pressure at depth considered
K_a : active earth pressure coefficient = $(1-\sin\phi')/(1+\sin\phi')$
(4)
K_p: passive earth pressure coefficient = $(1/K_a)$
(5)
K_o : at rest earth pressure
ϕ' = the effective friction angle of the soil

The effective overburden pressure developed during a flooding stage will control the amount of axial and lateral resistance in the foundation sands. The effective overburden pressures should be carefully considered. The foundation design for a building structure constructed on the landside of a flood protection project is usually controlled by the axial loading. A communication tower or a transmission tower foundation may be dependent on a combination of uplift, lateral and axial loading. A floodwall design will experience large lateral loads during flooding. The lateral resistance usually controls the foundation design.

SUGGESTED PROCEDURE FOR MODELING THE FLOOD CONDITION

A procedure is suggested to model the changes in the foundation soils resistance parameters due to high river stages. The designer should make an assessment of the existing foundation conditions below the proposed location for the structure(s). The designers should identify the cohesive layer and the limits of the pervious sands. The pressure increase due to high river stages can be determined by the underseepage analysis, flow net, method of fragments, finite element, or other methods.

Considering the influence of the effective overburden described above, the following procedure is suggested for modeling the soil response for deep foundation in the critical zone of a flood protection project:

a) Perform a literature search and geotechnical investigation to characterize the foundation materials.

b) Coordinate with the structural engineer and determine the location of the structure(s) with respect to the centerline of an existing levee or floodwall.

c) Calculate the hydraulic gradient using an underseepage analysis.

d) Calculate the normal (non-flood), saturated, and artesian geotechnical soil parameters, side friction, tip resistance and horizontal modulus.

e) Assess each loading condition using dead loads, live load and transient loads to determine the critical case.

f) Apply the adjusted side friction, tip resistance and lateral load deformation relationship to the computer model to determine the foundation response to the given loading.

g) Adjust the number of piers or spacing or depth to satisfy limiting deformation criteria developed for the specific structure(s).

EXAMPLE CALCULATION

<u>Axial Capacity</u>

The vertical resistance of a foundation pier was calculated to illustrate the impact subsurface pressures can have on the ultimate capacity of a pier. Two moisture conditions are demonstrated as shown on Fig. 7. A normal condition uses water below the top of ground surface. The artesian condition uses an excess gradient head of 0.914 meters. The length of the pier is 4.572 meters and the foundation consists of medium dense sand. The figure emphasizes the importance of considering the gradient in the foundation sands.

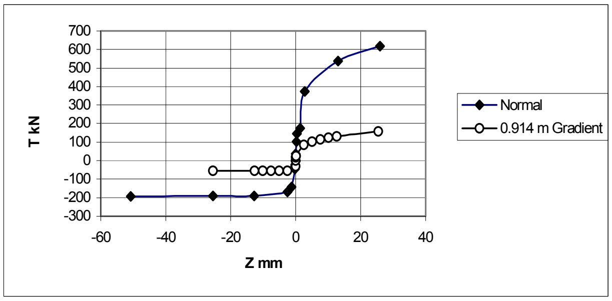

Fig. 7. Axial Capacity of a Single Pier with Gradient

Lateral Capacity

The impact to the allowable lateral resistance of a foundation pier is illustrated in Figure 8. The pier diameter is 0.61 meters and the initial length was selected at 15.24 meters. The hydraulic gradient was calculated to be 1 meter above the ground surface. The initial water level was measured at 6.1 meters below the ground surface. The axial loading for the pier is determined to be 89 kN and the allowable lateral loading is needed for varying foundation conditions. Five foundation conditions were considered; 1) water at 6.1 meters depth, 2) water at 3.05 meters depth, 3) saturated condition, 4) excessive gradient head of 0.914 meters and 5) gradient head of 3.05 meters.

The equation proposed by Terzaghi was used to develop the initial lateral response of the sands for a medium dense sand, A = 600, and a pier width of 0.61 meters. The calculated K values are plotted in Figure 6. The range of the ultimate lateral resistance for each load case was calculated using minimum value from equations (2) or (3) and the anticipated effective overburden pressure. The range of ultimate lateral resistance for each case is shown in Table 5.

The resulting allowable load is based on the pier diameter and soil conditions with a factor of safety of 2.5. These results are shown in Figure 8. The figure shows the loss of working lateral load resistance due to the increase in the pressures in the foundation sands.

Table 5. Allowable Loads Considering End Fixity and Gradient for a Single Pier.

Free			Water Level	Fixed		
Allow Pt	y_t	Moment		Allow Pt	y_t	Moment
kN	mm	m-kN	m	kN	mm	m-kN
102	6.6	152	-6.1	120	2.4	182
94	5.8	136	-3.1	111	2.3	176
85	8.4	146	0	102	3.2	180
78	10.2	143	0.9	98	3.8	181
53	15.2	132	3.1	75	7.1	181

Diameter of Pier : 0.61 meters
Ec = 24,994,375 kPa
I = 0.0068 m^4
Length of Pier = 15.24 meters

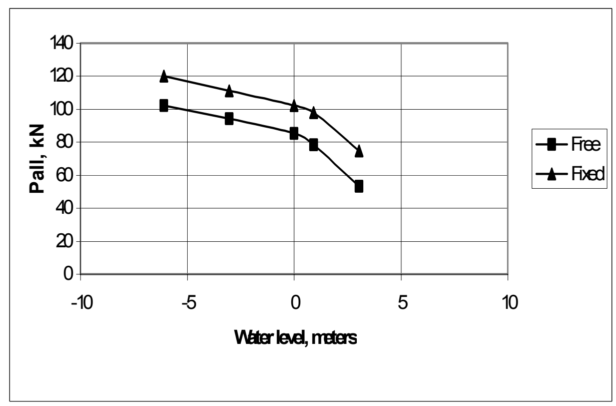

Fig. 8. Range of Lateral Resistance for Varying Water levels

CONCLUSIONS

A simplified procedure is suggested to adjust for artesian pressure developed near a flood protection project to account for the loss in resistance in sands due to high river stages. The impact on both vertical and horizontal load resistance can be adjusted using the anticipated increase in the foundation pressure and decrease in effective overburden pressures. After the adjustments are made, the designer can determine which case controls and make a recommendation for their project. The artesian condition near flood protection levees or flood walls can decrease the available vertical and lateral resistance to that below any measured resistance of a load test conducted under normal non-flooding conditions. Future research should be considered to model the gradient conditions to verify the theoretical assumptions and predictions.

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