The use of DMT data for lateral load analyses

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Abstract: Several methods have been proposed in the literature to develop p-y curves from DMT data. Dilatometer soundings are often completed before construction begins. Construction may have an important effect upon lateral loads and lateral resistances. Construction may also have an important effect upon the parameters used to develop p-y curves. Therefore, construction effects should be addressed, to the extent practical, when estimating lateral load behavior. This paper reviews likely effects from construction and presents methods to adjust preconstruction DMT results to account for excavation.

1. INTRODUCTION

The dilatometer provides an almost continuous profile of data for lateral load analyses. The equipment and test methods for dilatometer tests (DMT) are described in ASTM D6635. There are a number of methods to estimate lateral loads including those proposed by Gabr and Borden (1988); Robertson, Davies, and Campanella (1989); Marchetti, Totanti, Calabrese and Monaco (1991) and Gabr, Lunne and Powell (1994)). Each of these papers demonstrates a reasonable match between the proposed method and limited load test data.

The practicing engineer often selects a method to compare predicted versus measured lateral load test data. If he or she does not get a good match, another method may be tried before a method of analyses is judged suitable for a given project. Most engineers are hesitant to modify published methods. However, the analysis method may not be the reason for the poor match. The poor match may be due to construction methods and equipment. This paper describes modifications to p-y curves to provide improved correlations between lateral load test data and estimated lateral loads. These modifications may be applied to p-y curves from DMT data at other locations on a project using the same construction procedures.

2. EFFECTS OF CONSTRUCTION ON LATERAL LOADS

The effects of construction upon axial capacity of piles and drilled shafts are now generally recognized. Construction may also have an important effect upon lateral capacity. However, there have been very few studies documenting the effects. Perhaps future research will help to quantify further the effects of construction on lateral capacity. In the meantime, lateral load tests may be used to calibrate a given site and provide correlations between lateral load estimates and load tests to provide confidence in lateral load design considerations.

2.1 *Possible Construction Effects on Pile Lateral Capacity*

The lateral capacity of piles is likely affected by soil type, ground water location, use of pile penetration aids (jetting, predrilling or punching), whether pipe piles or cylinder piles are driven open-ended or closed ended, whether open-ended piles are plugged or unplugged, driving equipment (impact or vibratory hammers), spacing of piles, order of pile installation, nearby fills or excavations, etc. There is some direct evidence documenting construction effects on lateral capacity. However, much of the evidence is indirect.

Schmertmann & Crapps (1993) performed a model study of the effects of jetting upon pile axial capacity. These experiments showed the axial capacity of an existing (previously driven) pile was reduced approximately 50% for piles located 5 pile widths away and the axial capacity was reduced approximately 20% for piles located 12.5 pile widths away. The study estimated the effects of jetting were close to zero at about 25 pile widths. If jetting results in pile penetration under the influence of gravity during jetting and its effect extends out almost 25 pile widths, one can readily surmise that disturbance due to jetting would influence lateral capacity. Vibrations from additional driving reconsolidate non-cohesive

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soils to some extent. However, lateral load tests may be required to provide accurate estimates of the effects of jetting or other pile penetration aids (jetting, drilled preformed holes or punched preformed holes, etc.).

Hwang et al (2001) reported the results of a study of ground response during pile driving. Measurements were made during the driving of 800 mm cylinder piles with an inside diameter of 560 mm. The piles were constructed of prestressed concrete and were driven with a closed conical shaped end. Slope inclinometer measurements showed 20 mm average horizontal ground movements 3 diameters from the pile center, movements equal to 2.5% of the pile diameter. They estimated that the horizontal ground movements were insignificant at 12 times the pile diameter. If the initial ground surface was displaced laterally, the ground was horizontally displaced, from the pile centerline, at least 0.5 times the pile diameter (440 mm) at the face of the pile. Measurements made 1.5 times the pile diameter showed vertical ground movements (heave) of 36 mm. They also estimated that horizontal ground movements extended 10 diameters or more below the center of the pile tip. These data show significant ground disturbance for considerable distance around and below a driven pile. This disturbed soil would have different properties than the undisturbed soil and the estimated lateral load behavior would certainly be different for soils data taken before and after pile driving.

Many investigators, including Hwang et al (2001), have measured significant pore pressures during pile driving. Pore pressure induced by pile driving may create permanent changes in the soil strength even after their dissipation. For example, high pore pressures may break down the soil structure and create drainage paths that may affect lateral load behavior.

Huang et al (2001) reported on the effects of construction on laterally load pile and drilled shaft groups. They performed preconstruction and post construction CPT and DMT tests. The post-construction tests were conducted through the cap of the pile group. The authors introduced a p-multiplier to account for group effects from preconstruction DMT data and a p-multiplier to account for group effects from post-construction DMT data. The ratio of the post-construction effect to the preconstruction effect reflects the effects of construction. The authors derived a factor of 0.70 for the driven pile group which indicates that "... the installation of driven piles caused a densifying effect" (or increase in lateral stresses).

2.2 *Possible Construction Effects on Drilled Shaft Lateral Capacity*

Construction methods and equipment likely have more of an effect upon drilled shaft lateral capacity than on piles. Lateral capacity of drilled shafts is likely affected by soil type, ground water location, use of casing or no casing, sidewall relaxation, slurry buildup, nearby fills or excavations etc.

Crapps (2005) presented curves for measured slurry buildup versus time for bentonite and attapulgite. These curves showed 20 mm buildup of attapulgite and 23 mm buildup of bentonite in 2 days. Bentonite buildup was 100 mm in about 16.5 days. The filter cake or gel layer has little strength and could significantly affect lateral capacity if not removed before concrete placement. Note that before construction and after construction DMT testing would not likely detect excessive lateral movements due to slurry buildup. However, the effects of slurry buildup could be indirectly accounted for by adjusting p-y curves (say with a y-offset of the p-y curve) derived from DMT data so that lateral loads match those measured by lateral load tests.

O'Neill (2001, p.11) presented results of shear wave velocity measurements made three hours after a borehole was opened in Beaumont Clay (a stiff clay). The shear wave velocities increased with distance away from the side of the shaft excavation. These measured shear wave velocities indicate that stress relief was felt 2 to 3 borehole radii away from the wall of the shaft. The shear wave velocity was about 70% of the "free field" shear wave velocity away from the shaft. O'Neill estimated that the shear strength of the clay at the eventual concrete/shaft interface was about 50% of the undisturbed strength before excavation. Note that p-y curves estimated from DMT tests performed in undisturbed soil would be stiffer than those estimated from DMT tests performed within the zone of relaxation.

Rhyner (2005) presented a case history that demonstrated differences in lateral capacity of drilled shafts due to a difference in method of casing installation. The initial drilled shafts for the New York City World Trade Center Building 7 were installed using a vibratory hammer while new casings for replacement construction were installed using external flush. Lateral load tests showed that there were dramatic differences in lateral capacity due to different casing installation methods. The lateral load capacity of the shafts with casings installed by external flush was significantly lower than those with

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casings installed with a vibratory hammer, especially at low loads. The lateral loads for the external flush shafts were close to zero until lateral deflections of about 12.7 mm (0.5") were reached.

Huang et al (2001) reported on the effects of construction on laterally loaded pile and drilled shaft groups as previously mentioned. They performed preconstruction and post-construction CPT and DMT tests. The post-construction tests were conducted through the cap of the drilled shaft group. The authors introduced a p-multiplier to account for group effects from preconstruction DMT data and a p-multiplier to account for group effects from post-construction DMT data. The ratio of the post-construction effect to the preconstruction effect reflects the effects of construction. The authors derived a factor of 1.19 for the drilled shaft group which indicates that "... the installation of bored piles softened the surrounding soil...".

3. GROUP EFFECTS

The lateral capacity of a pile or drilled shaft group is different than the capacity of a single pile or shaft times the number of piles or shafts in the group because the effects of lateral stresses from each pile or shaft overlap. The capacity depends upon the number of rows and the spacing of the piles or shafts. The "leading" row has the highest lateral capacity and each row behind the leading row has a reduced lateral capacity. Most lateral load programs have p-multipliers to account for group effects (see Ensoft (2005) or Florida Pier (2005)).

4. ESTIMATING LATERAL LOADS USING DMT DATA

The Robertson et al (1989) method is likely the most widely used method to develop p-y curves from DMT data. This method was described in detail by Briaud and Miran (1992) in a manual prepared for the FHWA. This method will be used in this paper.

The Robertson et al method uses a cubic parabola, reproduced as Equation (1) below, to produce p-y curves:

$$\frac{P}{P_u} = 0.5\left(\frac{y}{y_c}\right)^{0.33} \tag{1}$$

Where: P/P_u = ratio of soil resistance
y/y_c = ratio of pile deflection
P_u = ultimate lateral force
y_c = critical deflection

The method to determine the values of P_u and y_c depend upon the soil type.

4.1 *P-y Curves For Clay*

Equation (2) may be used to determine the value of y_c for clays:

$$y_c = 23.67\frac{S_u D^{0.5}}{F_c E_D} \tag{2}$$

Where: y_c = critical deflection in cm
D = pile diameter in cm
S_u = undrained shear strength (from DMT)
E_D = dilatometer modulus (same units as S_U)
F_c = ratio of initial tangent modulus to the dilatometer modulus.

Robertson et al assumed a value of 10 for F_c, as a first approximation, for clay soils. The reader should note that the value of F_c is not well established and may vary. Part of the variation may be due to construction effects.

Equation (3) may be used to determine P_u for clay:

$$P_u = N_p S_u D \tag{3}$$

Where: P_u = ultimate lateral force (same units as S_u)
N_p = nondimensional ultimate resistance coefficient

$$N_p = 3 + \frac{\sigma_v'}{S_u} + \frac{Jx}{D} \tag{3a}$$

Where: J = empirical coefficient (0.25 for stiff clay and 0.50 for soft clay; stiff clay assumed in this study as $S_u > 0.5$ tsf - values of J interpolated between 0.25 and 0.50)
x = depth
σ_v' = effective vertical stress at depth x

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Note that S_u and E_D are required for y_c and S_u is required for P_u. These values are provided by DMT tests.

4.2 *P-y Curves for Sand*

Equation (4) may be used to determine the value of y_c for sand:

$$y_c = \frac{4.17\left(\sin\phi'\right)\sigma_v' D}{F_s E_D\left(1-\sin\phi'\right)} \tag{4}$$

Where: y_c = critical deflection in cm
D = pile diameter in cm
F_s = empirical stiffness factor
ϕ' = angle of internal friction

Robertson et al (1989) first assumed F_s would be equal to 1 as a first approximation. However, analyses of their data required use of a value of F_s equal to 2 for the best match of their test data. The reader should note that the value of F_s is not well established and may vary. Part of the variation may be due to construction effects.

Equations (5a) and (5b) may be used to determine possible values of P_u for sand. The value of P_u is taken as the minimum from (5a) or (5b).

$$P_u = \sigma_v'\left[D\left(K_p - K_a\right) + xK_p \tan\phi' \tan\beta\right] \tag{5a}$$

$$P_u = \sigma_v' D\left(K_p^3 + 2K_o K_p^2 \tan\phi' + \tan\phi' - K_a\right) \tag{5b}$$

Where: P_u = lesser of (5a) or (5b)
K_a = Rankine active coefficient
$= \left(1-\sin\phi'\right)/\left(1+\sin\phi'\right)$
K_p = Rankine passive coefficient = $1/K_a$
K_0 = coefficient of earth pressure at rest
$\beta = 45^o + \varphi'/2$

Note that ϕ' and E_D are required for y_c and that ϕ' and K_0 are required for P_u. These values are provided by DMT tests.

5. ACCOUNTING FOR EXCAVATIONS

The dilatometer is a valuable tool to provide design data for retaining structures. As previously mentioned, soils data, including DMT data, are often obtained before construction. This section provides a method to account for the effects of excavation. Excavations obviously have an important effect upon σ_v' and may have an important effect upon E_D, S_u, K_0, and ϕ' values used to estimate the value of y_c and P_u for p-y curves. The equations to account for excavation are included in Appendix A along with background information concerning the equations. Large projects may justify DMT testing before and after excavation to properly account for site specific changes due to excavation. However, the equations included herein may be used to estimate the effects.

6. RECOMMENDED MODIFIERS FOR P-Y CURVES

The author proposes three modifiers (C_y, C_P and Δ_y) for p-y curves to account for the effects of construction. The first modifier, C_y, adjusts the estimated value of y_c as shown in Equation 6a and the second modifier, C_P adjusts the value of P_u as shown in Equation (6b). The value of Δ_y denotes the y-movement required before the value of P begins to increase from zero.

$$y_c' = C_y y_c \tag{6a}$$

$$P_u' = C_p P_u \tag{6b}$$

Equations (1a) and (1b) reflect the changes in Equation (1) after introducing the modifiers.

$$P' = 0 \qquad \text{when } y' \le \Delta_y \tag{1a}$$

$$\frac{P'}{P_u'} = 0.5\left(\frac{y}{y_c'}\right)^{0.33} \qquad \text{when } y' > \Delta_y \tag{1b}$$

$$y' = y + \Delta_y \tag{1c}$$

The intent is to offset the p-y curve by an amount equal to Δ_y to account for conditions that allow lateral movement before lateral resistance is encountered. The modified curves, P' versus y', are used in the lateral load analyses. Note that one may make a p-y curve stiffer by increasing the value of P_u or by decreasing the value of y_c. A value of C_P greater than 1.0 or a value of C_y less than 1.0 makes the p-y curve

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stiffer; and, conversely a value of C_P less than 1.0 or a value of C_y greater than 1.0 makes the p-y curve softer (less stiff). One may note that the use of a value of C_y other than 1.0, effectively modifies F_c or F_s which may vary depending upon the effects of construction, as previously noted. One may also note that the use of a value of C_p other than 1.0, effectively modifies S_u, which may also be affected by construction as previously noted. A value of Δ_y greater than zero offsets the entire p-y curve but does not change the stiffness. Also note that the introduction of multipliers for P_u and y_c and the use of an offset, Δ_y , for y may be used for p-y curves generated by any method.

7. CASE HISTORY

This case history is from the Puerto Nuevo Project, a U.S. Army Corps of Engineers Project located in San Juan, Puerto Rico. Rains swell mountainous streams which flow through San Juan to the ocean. The streams are narrow and development in San Juan has reached both sides of the streams at some locations. These existing natural waterways are being widened and/or deepened to improve the drainage in San Juan. At some locations, retaining walls are required to protect existing construction. This case history is from the load test program for this project.

One of the wall designs included 1220 mm (48 inch) diameter pipe "king" piles providing lateral support for steel sheet piles placed between the pipe piles. The plan excavation in front of the wall was to elevation -4880 mm (-16 feet). The elevation of the ground surface at the time the DMT soundings were made was about elevation +1220 mm (+4 feet). Therefore, there would be about 6100 mm (20') of excavation in front of the wall after it was constructed.

The load test program included the lateral testing of two steel 1220 mm piles of different lengths. The pipe piles, with 19 mm (0.75 inch) wall thickness, were driven and a cap constructed on each of the piles at the Contract 2A test site. Two separate static lateral load tests were performed at the site by jacking one cap against the other. Test 1 was conducted before excavation and Test 2 was constructed after a cofferdam was constructed and excavated to approximately the design excavation elevation. Additional details are available in the project report (see Crapps (2000)).

The lateral load test site was moved from its intended location due to a conflict with a fly-over bridge subsequently constructed after the original testing was completed. New DMT tests were completed at the test site by GEOCIM (see GEOCIM (2000) or Crapps (2000)). The DMT data at the test site were adjusted for the effects of excavation (a small excavation primarily to remove construction debris before the first test and a deep excavation before the second lateral load test), p-y curves were developed and appropriate values of C_p were developed by trial and error using LPILE3 (see Reese and Wang (1997)). A value of C_p equal to 1.1 before excavation and 1.2 after excavation provided a good match with the load test results. Note that C_y was set equal to 1.0 and Δ_y was set equal to 0.0. Note that relatively small adjustments (C_p values of 1.1 and 1.2 versus 1.0) were required for a good match between predicted and measured results, after making the adjustments for the effects of excavation.

Anderson et al (2003) used *FloridaPier* (*FLPier*) with p-y curves derived from SPT, CPT, DMT and PMT data to compare predicted versus measured lateral deflections. The Puerto Nuevo Project test program data were included in their analyses. One of their conclusions was that "On the average, DMT derived p-y curves predict well at low lateral loads." However, they did not have a good correlation between predicted deflections using DMT data and measured deflections at high lateral loads. The differences in the match for lateral load behavior determined by Anderson et al. (2003) and Crapps (2000) are likely due to construction effects. This paper and the Anderson et al. paper demonstrate the need for future research to provide a better understanding of the effects of construction.

8. SUMMARY & CONCLUSIONS

1. Factors, related to construction, which may have an effect upon the lateral load capacity of piles and drilled shafts are summarized.

2. A method to account for excavation (decrease in effective stresses) is presented for DMT data.

3. Modifiers for p and y are proposed to account for the effects of construction upon lateral load behavior.

4. A case history was presented using the methods to account for excavation.

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APPENDIX A – ACOUNTING FOR THE EFFECTS OF EXCAVATION

A1. INTRODUCTION

Appendix A provides the background for derivation of equations to estimate the effects of excavation.

A2: CHANGE IN EFFECTIVE STRESS DUE TO EXCAVATION

Elastic methods may be used to estimate the effects of excavation upon effective stress (for example, see Poulos and Davis (1974)).

A3. DEFINITIONS

Marchetti (1980) provided Equations (A1), (A2) and (A3 which define three key DMT variables:

$$E_D = 34.7\left(p_1 - p_0\right) \tag{A1}$$

$$K_D = \frac{p_0 - u_0}{\sigma_0'} \tag{A2}$$

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \tag{A3}$$

A4. UNDRAINED SHEAR STRENGTH

The undrained shear strength is required for a number of methods to estimate P-y curves for clay. Many sites have clays that are overconsolidated or will be overconsolidated upon excavating in front of the walls. Equation (A4), from Schmertmann (1978) and/or Tang & Tsuchida (1999) provides a method to estimate the effects of overconsolidation ratio on the undrained shear strength of clays:

$$\frac{S_{u1}/\sigma_{01}'}{S_{u2}/\sigma_{02}'} = \left(\frac{OCR_1}{OCR_2}\right)^{\Lambda} = \left(\frac{\sigma_{p1}'/\sigma_{01}'}{\sigma_{p2}'/\sigma_{02}'}\right)^{\Lambda} \tag{A4}$$

Where: S_{u1} = undrained shear strength for cond. 1
S_{u2} = undrained shear strength for cond. 2
OCR_1 = over consolidation ratio for cond. 1
OCR_2 = over consolidation ratio for cond. 2
σ_{p1}' = preconsolidation stress for condition 1
σ_{p2}' = preconsolidation stress for condition 2
σ_{01}' = vertical effective stress for condition 1
σ_{02}' = vertical effective stress for condition 2
Λ = coefficient ranging from 0.7 to 0.9

A4.1 *Effect of Excavation on S_u*

Noting that the preconsolidation stress remains the same when there is an excavation ($\sigma_{p1}' = \sigma_{p2}'$) and using the average value of 1-Λ = 0.2 provides equation (A5):

$$S_{u2} = \left(\frac{\sigma_{02}'}{\sigma_{01}'}\right)^{0.2} S_{u1} \tag{A5}$$

A5. EFFECT OF EXCAVATION ON E_D

A5.1 Undrained E_D

Marchetti (1980) presented the Equation (A6) for undrained shear strength (also see Schmertmann (1988) or Briaud and Miran (1992).

$$S_u = 0.22\sigma_o'\left(0.5K_D\right)^{1.25} \tag{A6}$$

Equation (A7a) may be derived from equations (A1), (A2) and (A3).

$$E_D = 34.7\left(K_D I_D\right)\sigma_0' \tag{A7a}$$

Solving Equation (A7a) for K_D and substituting in Equation (A6) provides Equation (A7b).

$$E_D = 233 S_u^{0.8} I_D \left(\sigma_0'\right)^{0.2} \tag{A7b}$$

The value of I_D remains constant with a change in effective stress ($I_{D2} = I_{D1}$). Equation (A8) may be used to estimate the effects of excavation upon undrained values of E_D.

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$$\frac{E_{D2}}{E_{D1}} = \left(\frac{S_{u2}}{S_{u1}}\right)^{0.8} \left(\frac{\sigma'_{01}}{\sigma'_{02}}\right)^{0.2} \qquad \text{(A8)}$$

A5.2 *Drained E_D*

The drained value of E_D is expected to remain constant with excavation. Therefore, assume $E_{D2} = E_{D1}$.

A6. EFFECT OF EXCAVATION ON ϕ'

A detailed discussion of estimates of ϕ' from DMT test data may be found in Schmertmann (1988). The value of ϕ' for sands is dependent upon effective stress due to the non-linearity of the failure envelope. The values of ϕ' presently reported in the DMT data reduction program provided by GPE, Inc. are based upon a standard reference failure pressure of 2.72 bars as explained in Schmertmann (1983). Schmertmann (1983) and Schmertmann (1984) presented an equation (presented below as Equation (A9)) as well as a figure to estimate ϕ' for other failure pressures. Both the figure and Equation (A9) require an iteration procedure for a solution based upon a change in effective stress. However, Equation (A9) converges rapidly even if the value of ϕ'_2 is set equal to ϕ'_1 for the first trial. Note that the value of ϕ' provided by the DMT is a plane-strain parameter.

$$\phi'_2 = \tan^{-1}\left\{\begin{matrix} \tan\phi'_1 + 0.0446 - \\ 0.105\log\left((1+\sin\phi'_2)\,\sigma'_{02}\right) \end{matrix}\right\} \qquad \text{(A9)}$$

Where: $\phi'_1 = \phi'$ before excavation

$\phi'_2 = \phi'$ after excavation

$\sigma'_{02} = \sigma'_0$ after excavation

One may note that the effect of excavation typically increases the value of ϕ'_2. In the event that the calculated value of ϕ'_2 is greater than 45 degrees, a value of 45 degrees should be used.

A7. EFFECT OF EXCAVATION UPON K_o FOR SANDS

The value of K_o is required to determine the value of P_u for sands. Schmertmann (1992) derived the following expression relating the OCR to K_o.

$$OCR = \left[K_0/(1-\sin\phi'_{ax})\right]^{(1/0.8\sin\phi'_{ax})} \qquad \text{(A10a)}$$

Where: OCR = overconsolidation ratio

ϕ'_{ax} = axisymetric ϕ'

Solving Equation (A10a) for K_o provides Equation (A10b).

$$K_0 = \left(1-\sin\phi^1_{ax}\right)OCR^{\left(0.8\sin\phi'_{ax}\right)} \qquad \text{(A10b)}$$

Equation (10b) provides Equation (A11).

$$\frac{K_{01}}{K_{02}} = \frac{\left(1-\sin\phi'_{ax1}\right)OCR_1^{\left(0.8\sin\phi'_{ax1}\right)}}{\left(1-\sin\phi'_{ax2}\right)OCR_2^{\left(0.8\sin\phi'_{ax2}\right)}} \qquad \text{(A11)}$$

The excavation does not change the value of the preconsolidation stress. Therefore, $\sigma'_{p2} = \sigma'_{p1}$ and Equation (11a) may be derived.

$$\frac{K_{02}}{K_{01}} = \left\{\frac{(1-B)}{(1-A)}\frac{\left(\sigma'_{01}\right)^{0.8A}}{\left(\sigma'_{02}\right)^{0.8B}}\right\}\left(\sigma'_{p1}\right)^{0.8(B-A)} \qquad \text{(A11a)}$$

Where: $A = \sin\phi'_{ax1}$ and $B = \sin\phi'_{ax2}$

K_{o1} = before excavation value of K_o

K_{o2} = after excavation value of K_o

ϕ'_{ax1} = before exc. value of axisymetric ϕ'

ϕ'_{ax2} = after exc. value of axisymetric ϕ'

A8. ESTIMATING AXISYMETRIC ϕ' FROM PLANE STRAIN ϕ'

Note that all the values of ϕ' prior to Equation (A10) have been plane-strain parameters provided by the DMT test. One may use Equation (A12) from

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Schmertmann (1992) to estimate axisymetric parameters.

$$\phi'_{ax} = \phi'_{ps} \quad \text{for } \phi'_{ps} \le 32^0 \qquad \text{(A12a)}$$

$$\phi'_{ax} = \phi'_{ps} - \left[\left(\phi'_{ps} - 32\right)/3\right] \quad \text{for } \phi'_{ps} \rangle 32^0 \qquad \text{(A12b)}$$

Where: ϕ'_{ax} = axisymetric ϕ'
ϕ'_{ps} = plane strain ϕ'

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