WEIDLINGER ASSOCIATES® INC

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MEMORANDUM

TO	Simon Tempest
FROM	Michael Oakland
DATE	July 11, 2011
SUBJECT	Final Report on Peer Review Preload for Foundation Construction East Bridgewater High School

COPIES TO

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□

COMMONWEALTH OF MASSACHUSETTS
MICHAEL W. OAKLAND
CIVIL
No. 35187
REGISTERED PROFESSIONAL ENGINEER

MESSAGE

This memorandum summarizes our peer review of the preloading planned to limit post construction foundation settlements of the proposed High School. The proposed structure will be two to three stories with no basement level. The new school is located behind the current High School. The site is currently used for athletic fields. A raise-in-grade of up to 5 ft. is planned over much of the site to accommodate the new High School.

A geotechnical investigation was undertaken by Pare Corporation of Foxboro, MA. The investigation found subsurface conditions at the site to generally consist of a thin layer of top soil underlain by a discontinuous layer of organic silt typically several feet in thickness were encountered. The topsoil is underlain by layers of sand, silt and clay which vary in stratigraphy and thicknesses throughout the site. However, the various materials were typically dense to very dense or stiff to very stiff in nature. Glacial till was encountered at a depth of about 74 ft. in Test Boring B-4.

Laboratory testing of the clay indicates that the clay layers are compressible under the load of the proposed fill as well as the final structure itself. Calculations made by Pare indicate that the combined consolidation settlements resulting from the weight of the fill and the structure would be more than tolerable. Therefore, Pare has recommended a program of preloading where fill is required. Preloading allows the site to settle under its weight prior to construction of the structure. In doing this, the post construction settlement will only be that from the weight of the structure.

For the purposes of calculations, Pare has used the conditions represented in Test Boring B-4 which includes two clay layers. The upper layer extends from a depth of 12 ft. to 24 ft. below grade while a lower clay layer extends from a depth of about 36 ft. to the top of the glacial till at a depth of about 74 ft. The laboratory tests related to the consolidation properties of the clay was conducted on a sample of the lower clay from this test boring.

Consolidation settlements are time dependent. That is they do not occur immediately upon loading but require time to occur. The time required is a function of the nature of the clay, thickness of the clay and the ability of the materials above and below the clay to drain. Because of the potential impact on the construction schedule, Pare has provided two options for the preloading. One is to place the fill to the final grade after removal and replacement of any unsuitable soils and allow it to settle to essentially full consolidation which is estimated to require about 100 days. The other option is to place the fill to a grade higher than that required (referred to as a surcharge) and then remove the excess material once the settlement estimated to occur under the normal fill alone is reached. This surcharging shortens the time required. The time of preloading under this option is estimated to be 45 days.

Weidlinger Associates has reviewed the calculations prepared by Pare and made independent calculations to confirm the recommendations made by Pare.

Our comments related to calculations by Pare follow:

- We agree that the Test Boring B-4 represents reasonable conditions for the site and that the properties determined by the test on the sample in the lower clay can be assumed to represent both the upper and lower clay layers.

- We agree with Pare's determination that the clays are below the previous past pressure in their current conditions as well as under the weight of the additional fill and proposed structure. However, we believe that the estimate that the rebound (swell) index is ¼ of the compression index is conservative. The rebound coefficient as determined by the laboratory results was 0.022 resulting in a swell index of 0.0407 rather than 0.063 used by Pare. This will result in reducing estimated settlements by about 35 percent. We take no exception with the conservative estimate use by Pare.

- Pare has distributed the preload fill stresses at a 2 vertical to 1 horizontal stress distribution to reduce the increase in stress in the clay layers. While this is appropriate for a rigid foundation which settles uniformly, under the flexible and very wide earthen fill it is likely that the clay below the middle of the fill will not realize this distribution and rather than an overall reduction, there will be some differential consolidation near the edges. The full preload stress should be used in calculating the potential total settlement. However for this case, the impact is relatively small and only relevant for assessing settlement platform data during construction.

- Pare has assumed that the lower clay layer has only single drainage. However, it appears that the sand layer above the lower clay and the glacial till below the lower clay will both serve as drainage layers allowing the layer to be doubly drained. This is considered to be a conservative assumption in Pare's calculations.

- In calculating the allowable ultimate bearing capacity, the friction angles assumed, appear to be low based on the blow counts. It is believed that if other correlations between blow count and friction value such as the one in Peck, Hanson and Thornburn were used, the values would be higher. In addition, it is noted that the average value in the stratum 2A is skewed by averaging the blow counts below the clay (which tend to be lower) with those above the clay (which tend to be higher). Only the upper stratum S2 is within the zone of influence and thus Pare used a conservative approach. It is possible that a higher ultimate bearing capacity could have been realized if the analysis had kept the different Stratum 2A layers as separate layers for purposes of analysis. However, settlement is the controlling factor so the issue is moot in any case.

- It is my opinion that the Schmertmann method of calculating immediate settlements is conservative for New England soils in particular because of under estimating the elastic modulus. However, in any case, the method is generally used for sands. Pare has also applied the method for the clay layers in for this study. Additional consolidation settlements from the clay layers are then added separately. It may have been more applicable to only include the settlements estimated by the Schmertmann method for the non-cohesive portions to avoid double counting settlements in the clay. However, the method selected is considered to be conservative and no exception is taken.

- The back calculation of allowable bearing capacity based on limiting the settlement to 1 inch total and 0.5 inches differential is generally the correct approach for this project. However, Pare has assumed that the both clay layers will only be 72 percent consolidated and including the remaining 28 percent as part of the post construction settlement. However, the preload period has been based on the time required for the lower clay to consolidate. The upper clay which is thinner and doubly drained will be near 100 percent consolidated and thus its contribution to the final settlements does not need to be added. In addition, it is common to also subtract the consolidation from the upper clay (and immediate) settlements from the weight of the footing itself from the post construction settlements since this will occur prior to building construction. However, related to both issues, Pare has used a conservative approach and we take no exception.

In summary, it is our opinion that the calculations performed are in general accordance with current engineering practices and assumptions made have generally been conservative. It is believed that either option of preloading or preloading with some surcharge to accelerate the preloading process is applicable for this site and is expected to limit settlements to the tolerable limits. Weidlinger has performed independent calculations to estimate consolidation settlements under the preload as well as post construction settlements and generally agrees with the methods and time estimates provided in these recommendations.

As a final comment, we note that the 7th Edition of the Massachusetts State Building Code has been referenced as part of these calculations and in the report itself. We believe that the 8th Edition should have been used for this project. While this does not have any impact on the recommendations made with respect to the preloading or bearing capacity, it will have some impact on the format of the report and recommendations for seismic design factors. Pare should confirm which edition of the building code is applicable to this project.

Please do not hesitate to contact me if you have any questions or require further information.

Attachment:

WAI consolidation settlement calculations

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- ☐ MEETING MINUTES
- ☐ FIELD OBSERVATION
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SUBJECT CONSOLIDATION SETTLEMENT CALC.

* SETTLEMENT WILL BE DETERMINED FOR TWO MID-POINTS IN CASE OF EACH CLAY LAYER

SILT $\gamma = 100.3$ pcf

8 — $P_{1TOP} = 8 \times 100.3 + 4(115.6 - 62.4)$
$= 1015.2\ lb/ft^2$

SILT + CLAY $\gamma = 115.6$ pcf

P_1

12

CLAY ①

⊙ $P_{1M1} = 1192\ lb/ft^2$

$P_{1M1} = P_{1TOP} + 3 \times (121.5 - 62.4)$
$\Rightarrow 1192.3\ lb/ft^2$

P_{1MID} — — — CLAY $\gamma = 121.5$ — — —

⊙ $P_{2M2} = 1724\ lb/ft^2$

$P_{2M2} = P_{1TOP} + 9(121.5 - 62.4)$
$\Rightarrow 1547\ lb/ft^2$

24

SILT + SAND $\gamma = 115.6$ pcf

$P_{1BOT} = P_{1TOP} + 12(121.5 - 62.4)$
$\Rightarrow 1724\ lb/ft^2$.

P_{2TOP}

36 — $P_{2TOP} = P_{1BOT} + 12(115.6 - 62.4)$
$\Rightarrow 2362.4\ lb/ft^2$

CLAY ②

$P_{2M1} = 2924\ lb/ft^2$

⊙ CLAY $\gamma = 121.5$ pcf

45

ACTION

$P_{2M1} = P_{2TOP} + 9.5(121.5 - 62.4)$
$\Rightarrow 2923.85\ lb/ft^2$

— — — — — — — — — —

64.5

⊙ $P_{2M2} = 4046\ lb/ft^2$

$P_{2M2} = P_{2TOP} + 28.5(121.5 - 62.4)$

74

$\Rightarrow 4046.35\ lb/ft^2$

79

SAND

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SUBJECT

CONSOLIDATION TEST RESULT PARAMETERS:—

P_c = PRECONSOLIDATION PRESSURE ≈ 6 tsf Approx
(FOR B-4 CLAY LAYER 2) = 12000 lb/ft^2

e_0 = INITIAL VOID RATIO = 0.85

C_{RE} = 0.022
(FROM STRAIN VS VERTICAL STRESS PLOT)

SETTLEMENT EQUATION:—

$$S_C = C_{RE} H_0 \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p}$$

★ CASE WITH 5' FILL WITH γ = 120 pcf
TOTAL SURCHARGE PRESSURE = 5 × 120 = 600 #/ft^2

ACTION

CLAY LAYERS ARE DIVIDED INTO HALF.

SETTLEMENT IN THE 1ST HALF OF CLAY LAYER ①

$$S_{C_{11}} = 0.022 \times 6 \times 12'' \times \log\left[\frac{1192 + 600}{1192}\right]$$

$$S_{C_{11}} \Rightarrow 0.28''$$

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SUBJECT SETTLEMENT IN THE 2nd HALF OF CLAY LAYER ①

$$S_{c_{12}} = 0.022 \times 6 \times 12'' \times \log\left[\frac{1724+600}{1724}\right]$$

$$= 0.20''$$

TOTAL SETTLEMENT IN CLAY LAYER ① =

$$\Rightarrow 0.28 + 0.20'' = 0.48''$$

AGAIN,

SETTLEMENT IN 1st HALF OF SECOND CLAY LAYER-②

$$S_{c_{21}} = 0.022 \times 19.5 \times 12 \times \log\left[\frac{2924+600}{2924}\right]$$

ACTION

$$\Rightarrow 0.42''$$

SETTLEMENT IN 2nd HALF OF SECOND CLAY LAYER-②

$$S_{c_{22}} = 0.022 * 19.5 \times 12 \times \log\left[\frac{4046+600}{4046}\right]$$

$$\Rightarrow 0.31''$$

TOTAL SETTLEMENT IN CLAY LAYER ② =

$$0.42 + 0.31 = 0.73$$

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SUBJECT

TOTAL SETTLEMENT IN CLAY LAYER ① + ②

$\Rightarrow 0{\cdot}48 + 0{\cdot}73 = 1{\cdot}21''$

* CASE WITH POST CONSTRUCTION LOADING :–

ASSUMING 3-FLOOR BUILING @ 100 Psf ⇒ 3×100 = 300 psf
1 TOP ROOF @ 40 Psf ⇒ 1×40 = 40 Psf

340 Psf

SETTLEMENT INCLUDING POST CONSTRUCTION LOADING

CLAY LAYER ①

NOW $\sigma_{v0}' = 1792 \ \#/ft^2$, $\Delta\sigma_v = 340 \ \#/ft^2$

$$S_{c11} = 0{\cdot}022 \times 6 \times 12'' \times \log\left[\frac{1792+340}{1792}\right]$$

$\Rightarrow 0{\cdot}12''$

ACTION

SIMILARLY, 2nd HALF OF CLAY LAYER ①

$\sigma_{v0}' = 2324 \ lb/ft^2$, $\Delta\sigma_v = 340 \ \#/ft^2$

$$S_{c12} = 0{\cdot}022 \times 6 \times 12'' \times \log\left[\frac{2324+340}{2324}\right]$$

$\Rightarrow 0{\cdot}09''$

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SUBJECT

TOTAL SETTLEMENT IN CLAY LAYER ①
DUE TO POST CONSTRUCTION

$\Rightarrow \quad 0.12'' + 0.09'' = 0.21''$

SETTLEMENT INCLUDING POST CONSTRUCTION
LOADING CLAY LAYER ②
1st HALF CLAY LAYER ②

NOW $\sigma_{v_0}' = 3524\ \#/ft^2$, $\Delta\sigma_v = 340\ \#/ft^2$

$$S_{c_{21}} = 0.022 \times 19.5 \times 12 \times \log\left[\frac{3524 + 340}{3524}\right]$$

$\Rightarrow \quad 0.21''$

SIMILARLY,
2nd HALF OF CLAY LAYER ②

ACTION

$\sigma_{v_0}' = 4646\ \#/ft^2$, $\Delta\sigma_v = 340\ \#/ft^2$

$$S_{c_{22}} = 0.022 \times 19.5 \times 12 \times \log\left[\frac{4646 + 340}{4646}\right]$$

$= \quad 0.16''$

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SUBJECT

TOTAL SETTLEMENT IN CLAY LAYER (2) DUE TO POST CONSTRUCTION

$\Rightarrow S_{C_{21}} + C_{C_{22}}$

$\Rightarrow 0.21'' + 0.16'' = 0.37''$

TOTAL SETTLEMENT DUE TO POST CONTRUCTION

$\Rightarrow$ $\boxed{0.21 + 0.37 = 0.58'' \approx 0.6''}$

SAY $0.6'' < 1.0''$ OK

ACTION

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SETTLEMENT DUE TO INDIVIDUAL FOOTING LOAD

ASSUME TOTAL LOAD OF 340 #/ft² ON 30 x 30 Sq ft FOOTING

∴ 340 × 30 × 30 ⇒ 306,000 #

q_{ALLOW} = 2000 lb/ft²

SIZE OF FOOTING = $\frac{306000}{2000}$ ⇒ 153 ft²

or Say 12' × 12'

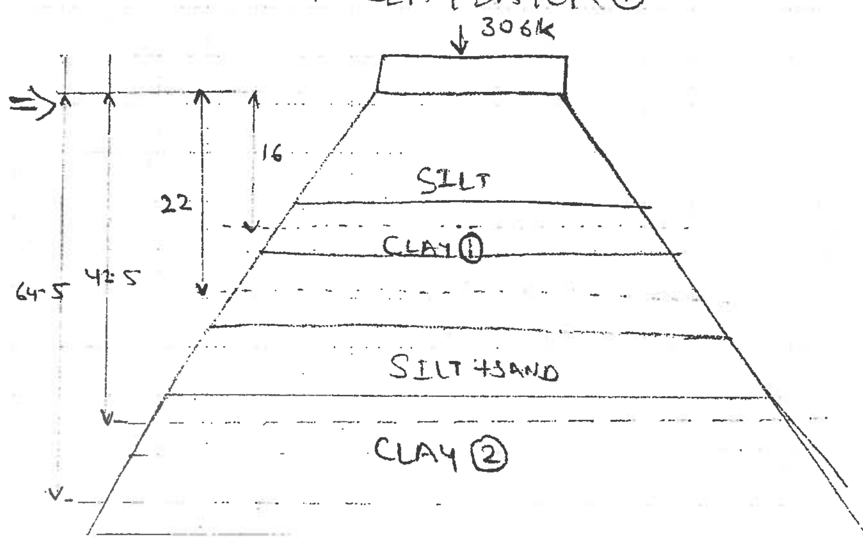
THE FOOTING WILL BE 4' BELOW THE GROUND

∴ STRESS ON THE TOP HALF OF CLAY LAYER ①

ACTION

$\frac{306000}{(12+16)(12+16)}$

⇒ $\frac{306000}{(28)^2}$



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= 390 Psf

SETTLEMENT IN THE TOP HALF OF THE 1ST CLAY LAYER

$$S_{c_{11}} = 0.022 \times 6 \times 12 \times \log\left[\frac{1792+390}{1792}\right]$$

$S_{c_{11}} \Rightarrow 0.14''$

STRESS ON THE BOTTOM HALF OF CLAY LAYER ①

$$\Rightarrow \frac{306000}{(12+22)(12+22)} \Rightarrow \frac{306000}{(34)^2}$$

$\Rightarrow$ 298.8 psf

SETTLEMENT IN THE BOTTOM HALF OF CLAY LAYER ①

ACTION

$$S_{c_{12}} = 0.022 \times 6 \times 12 \times \log\left[\frac{2324+299}{2324}\right]$$

$\Rightarrow 0.08''$

TOTAL SETTLEMENT CLAY LAYER ①

$\Rightarrow 0.14 + 0.08 \Rightarrow 0.22'$

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SIMILARLY,
STRESS ON THE TOP HALF OF II CLAY LAYER

$$\frac{306000}{(12+42.5)(12+42.5)} \Rightarrow 103.02 \text{ psf}$$

SETTLEMENT IN TOP HALF OF THE CLAY LAYER II

$$S_{C_{21}} = 0.022 \times 19.5 \times 12 \times \log\left[\frac{3524+103.02}{3524}\right]$$

$$S_{C_{21}} \Rightarrow 0.064''$$

STRESS ON THE BOTTOM HALF OF II CLAY LAYER

$$\frac{306000}{(12+64.5)(12+64.5)} \Rightarrow 52.28 \text{ psf}$$

ACTION

SETTLEMENT IN BOTTOM HALF OF II CLAY LAYER

$$S_{C_{22}} = 0.022 \times 19.5 \times 12 \times \log\left[\frac{4646+52.28}{4646}\right]$$

$$= 0.025''$$

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TOTAL SETTLEMENT IN CLAY LAYER 2

⇒ 0.064 + 0.025

⇒ 0.089 ≈ 0.09 SAY

TOTAL SETTLEMENT DUE TO INDIVIDUAL FOOTING LOAD

⇒ $S_{c11} + S_{c12} + S_{c21} + S_{c22}$

⇒ 0.22 + 0.09

⇒ 0.31" < 1.0" O.K

ACTION

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SUBJECT

TIME RATE CALCULATIONS :-

FROM LAB TEST :-

$C_v = 0.008 \text{ in}^2/\text{sec}$

ASSUMING DOUBLY DRAINED LAYERS ① AND ②

FOR 60% CONSOLIDATION, $T = 0.48$ (INTERPOLATION FIG 9.3)

$$T = \frac{C_v t}{H^2}$$

FOR CLAY LAYER ①

$$t = \frac{H^2 T}{C_v} \Rightarrow \frac{(6 \times 12\text{in})^2 \times 0.48}{0.008} \Rightarrow 311040 \text{ sec}$$

$\simeq$ 3.6 days.

FOR CLAY LAYER ②

ACTION

$$t = \frac{H^2 T}{C_v} = \frac{(19.5 \times 12)^2 \times 0.48}{0.008} \Rightarrow 3285360 \text{ sec}$$

$\simeq$ 38 days

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FOR 70% CONSOLIDATION, T = 0.59 (INTERPOLATION) (FIG. 93)

$$t = \frac{TH^2}{C_v}$$

TOP LAYER

$$\Rightarrow \frac{0.59 \times (6 \times 12)^2}{0.008} = 382320 \text{ SEC}$$

$\Rightarrow$ 4.42 days

BOTTOM CLAY LAYER

$$t \Rightarrow \frac{0.59 \times (19.5 \times 12)^2}{0.008} = 4038255 \text{ SEC}$$

$\approx$ 46.7 SAY 47 DAYS

ACTION

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FOR 80% CONSOLIDATION, T = 0.75 (FIG 9.3).

$t = \frac{TH^2}{C_v}$, $C_v = 0.008\ in^2/sec$

TOP LAYER

$t \Rightarrow \frac{0.75 \times (6 \times 12)^2}{0.008} \simeq 486000\ SEC$

$\simeq 5.6$ DAYS

BOTTOM LAYER

$t = \frac{0.75 \times (19.5 \times 12)^2}{0.008} = 5133375\ SEC$

$\Rightarrow \simeq 59.41$ DAYS.

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FOR 90% CONSOLIDATION, T = 1.0 (Fig 9.3)

$t = \frac{TH^2}{c_v}$, $c_v = 0.008\ in^2/sec$

TOP LAYER

$t = \frac{1.0 \times (6 \times 12)^2}{0.008} \Rightarrow 648000\ SEC$

≈ 7.5 DAYS

BOTTOM LAYER

$t = \frac{1.0 \times (19.5 \times 12)^2}{0.008} \Rightarrow 6844500\ SEC$

≈ 79.21 DAYS

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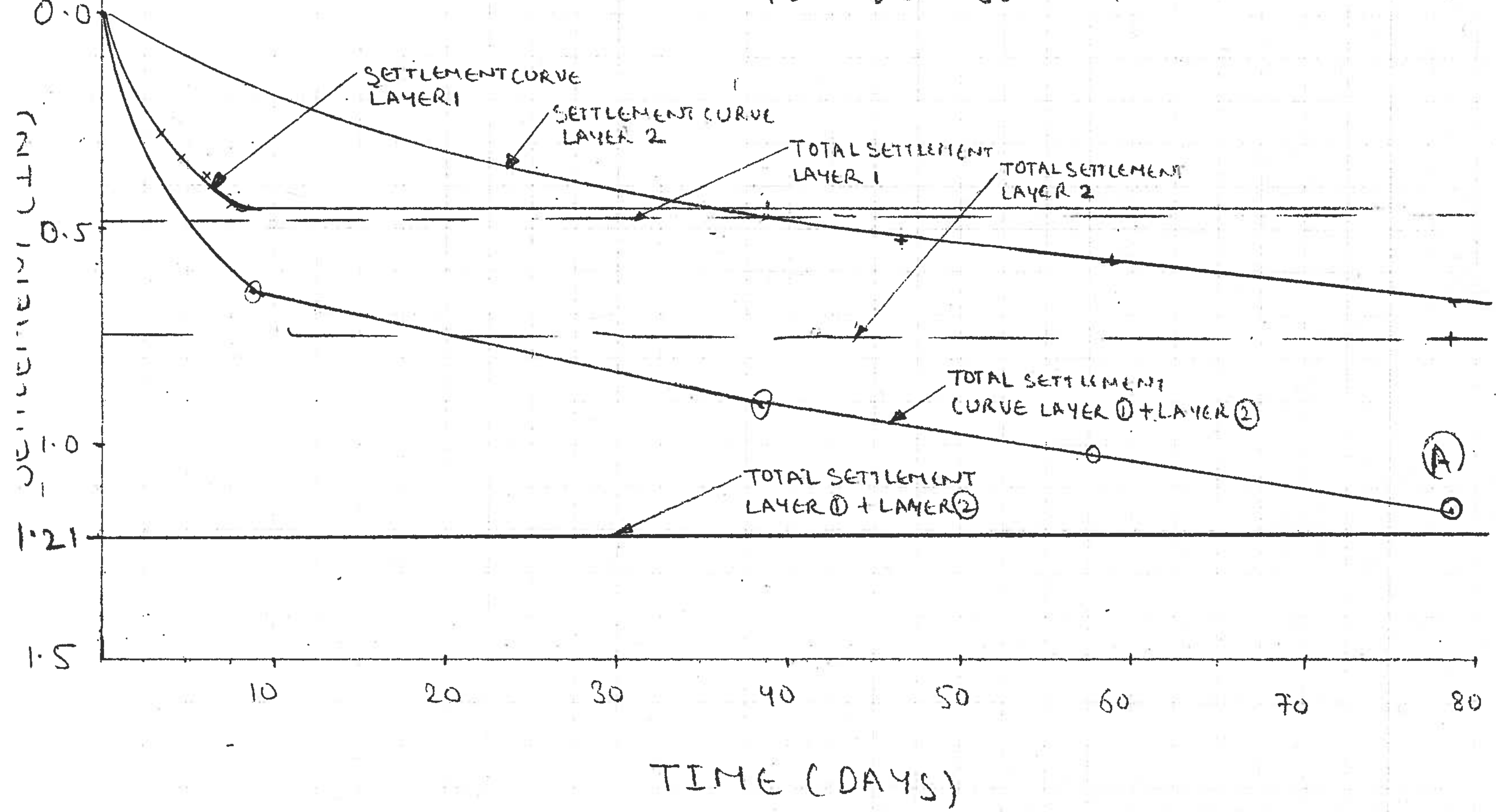
SETTLEMENT VS TIME GRAPH

POINT (A) REPRESENTS SETTLEMENT OF 1.14 in AFTER APPROXIMATELY 80 DAYS.

$\therefore \frac{1.14}{1.21} \times 100 = 94\%$

i.e., 94% OF THE TOTAL SETTLEMENT IS REACHED IN 80 DAYS.



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Michael W. Oakland, PhD, PE

Senior Geotechnical Engineer/Senior Associate

Dr. Oakland has over 25 years of geotechnical engineering experience related to the design and construction of building foundations requiring assessment of bearing capacity and settlement evaluation. His extensive experience encompasses a wide range of building projects, including office and residential buildings, treatment plants, tanks, walls and general site grading. Dr. Oakland is proficient at many types of ground modification to control settlement and improve bearing capacity including surcharging, compaction grouting, deep dynamic compaction and other techniques. He has used a wide range of analysis methods to assess settlements including finite elements to calculate stresses and displacement below structures.

He has served as the project manager for all geotechnical aspects of various projects, including field investigations, laboratory testing, analysis, design, report and specification preparation, and expert witness services. As manager of geotechnical instrumentation services, Dr. Oakland gained experience with installing, reading and interpreting inclinometers, piezometers, strain gages, load cells, tilt meters, and other monitoring devices.

Dr. Oakland is also a member of the geotechnical advisory committee for the Massachusetts State Building Code and has assisted with drafting both the 7th and 8th editions of the Code.

Education

PhD, Purdue University, West Lafayette, ID, 1986, Civil Engineering
MSCE, Purdue University, West Lafayette, ID, 1981
BSCE, Pennsylvania State University, University Park, PA, 1980

PE Registration

MA (35487) 1990, Civil Engineering; Also OH, NJ, KY, MI

Affiliations

American Society of Civil Engineers (ASCE)
Association of State Dam Safety Officials
Boston Society of Civil Engineers
  Geotechnical Group Chairman, 1995-1996
  Computer Group Chairman, 1990-1991
Massachusetts State Building Code Committee

Select Project Experience Related to Building Foundation Design and Preloading

Expert Witness, Condominium Settlement, Confidential. Assisting with the assessment of excessive settlement at a condominium complex in NJ where deep dynamic compaction and surcharging was used to limit settlements.

Central Artery North Area, Boston, MA. Designed wick drains and surcharging program for the transition embankment sections from the viaducts to the boat sections in Charlestown. Reviewed all monitoring data and rendered opinion as to the required duration of the surcharge.

Electrical Building, Brayton Point Power Plant, Somerset, NJ. Provided foundation design recommendations which included preloading the site to limit post construction settlements of the deep compacted fill placed below the embankment.

Turbine Building Settlement Analysis, AP1000, Voglte Nuclear Power Plant, GA. Used detailed finite elements to assess potential settlements and impact on below grade piping for the proposed Units 3 and 4 at the existing power plant.

University Campus Expansion, Confidential. Performed a settlement analysis below a mat foundation supporting a major university expansion supported over deep varved silts using three dimensional finite element methods to assess the feasibility of avoiding end-bearing deep piles. The study included both a conventional mat foundation and a shallow pile supported raft-slab.

Building Assessment, D-11A, Central Artery, Boston, MA. Used detailed finite elements to assess the impact of the Central Artery construction which was to extended below and actually cut off one corner of the mat foundations supported each of two high rise towers in downtown Boston. The finite element procedures were used to predict the total and distribution of settlement below the critical structures at each stage of excavation.

Randolph Houses, New York, NY. Conducted a geotechnical investigation and provided foundation design recommendations for a new 12 story building to replace a number of 7 story row houses in the Harlem area near Columbia University.

121 Portland Street, Boston, MA. Conducted a geotechnical investigation and provided foundation design recommendations for underpinning of the caisson foundation to support 3 additional stories to be added to the early 19th century structure. Drilled in minipiles were used to provide the additional foundation capacity and were designed to be compatible with the existing foundations.

745 Atlantic Ave., Boston, MA. Field engineer during site development and construction of a new highrise structure in downtown Boston. Work included demolition of the existing post office, site clearing and pre-excavation, lateral earth support construction and foundation construction.

Connecticut Legislative Office Addition, Harttford, CT. Provided geotechnical design recommendations for a 5 story addition with one level below grade on to the existing caisson supported office building in downtown Hartford. New caissons had to be designed to be compatible with the existing building support.

Water Treatment Plant, Cambridge, MA. Provided foundation design recommendations a new water treatment plant which included large below grade clearwells. Settlement was a significant issue for the treatment plant which was founded over deep compressible clay deposits requiring rigorous assessment using several different consolidation settlement and stress distribution programs to model the impact of settlements from the non-uniformly loaded mat which was founded at a variety of different elevations.

Atkins Water Treatment Plant Clearwell, Greenville, SC. Provided foundation design recommendations for a new 10 million gallon prestressed concrete water storage tank to be constructed at the existing treatment plant. The plant required ground modification to limit settlements which had affected previous areas of the plant. The water storage tank was to be added to the two existing tanks and settlement assessments had to include the impact of additional settlements on the adjacent tanks.

Interim Sludge Storage Tanks, Deer Island, MA. As part of his work on Deer Island, Dr. Oakland provided foundation design recommendations for support of two 2 million gallon conical bottom sludge storage tanks. Both steel and concrete options were evaluated. The tanks were founded over soft organic silts and the steel tanks were selected as the temporary vessels due to there ability to tolerate the anticipated settlement.

Sumat Prakarn Treatment Plant, Bangkok, Thailand. Conducted a technical review of the geotechnical design for a new treatment plant. Very soft soil conditions require structures to be pile supported and special consideration for fills required for access roads, berms and other sitework features. The project included numerous containment structures including circular clarifiers and settling basins. Extensive settlement predictions were made related to the earthen berms so that adequate freeboard could be provided to compensate for long term settlement.

Wastewater Storage Tanks, IBM, Essex Junction, VT. Provided foundation design recommendations and construction phase consultation for two 8 million gallon double containment vessel with a steel tank inside a prestressed concrete tank. The load of the filled steel vessel within the empty concrete tank create some unique settlement concerns on the concrete slab. Worked with the independent tank venders to develop and acceptable system.