KIEWIT NORTH CAROLINA LNG STORAGE FACILITY GEOTECHNICAL EVALUATION
PROJECT ID: CEEN_2018CPST_008

by

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A Capstone Project Final Report

Submitted to

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Brigham Young University

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Executive Summary

PROJECT TITLE: KIEWIT NORTH CAROLINA LNG STORAGE FACILITY GEOTECHNICAL EVALUATION
PROJECT ID: CEEn_2018CPST_008
PROJECT SPONSOR: Kiewit Engineering Group, Inc.
TEAM NAME: MZM Enterprises

The following items were to be completed for a geotechnical evaluation of a proposed liquefied natural gas (LNG) storage facility near Fayetteville, North Carolina, sponsored by Kiewit Engineering Group, Inc. (referred to herein as "the client"), and undertaken by MZM Enterprises (referred to herein as "the team"):

- Seismic site classification
- Soil analysis summary
- Selection of shallow foundation type
- Determination of design values for deep foundations
- Design of truck trafficking roadway
- Discussion of constructability considerations
- Identification of potential geotechnical risks

The objective of the project was to provide a geotechnical review memorandum to the client that would enable the cost estimates crew to recommend an accurate bid on the project. Additionally, the team would produce a poster and presentation summarizing the conclusions of the project.

The following parameters have been determined:

- Seismic site classification: **D**
- Soil analysis summary: **Mostly clay and sand, design bearing capacity = 1500 psf**; see attached
- Shallow foundation type: **Strip shallow spread footings**
- Design values for deep foundations: **12-inch diameter driven pipe piles with depth range 33-75 feet and capacity range 50-150 kips**; see attached
- Design of truck trafficking roadway: **8.0-inch thick 4000 psi concrete pavement with 4-inch AASHTO A-1a base, 1.5-inch dowels centered in the concrete, and 2.5-inch deep transverse joints spaced at 15 feet on center**; see attached for alternatives.

See attached for reference. This report marks the completion of the project assigned to MZM Enterprises. Please promptly contact MZM Enterprises with concerns and questions.

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Introduction

Convenient to the city of Fayetteville, North Carolina, a facility for storing liquefied natural gas is to be constructed. The facility will consist of two LNG storage tanks, auxiliary buildings for equipment and operations, and roads for truck and shipping traffic.

The project submittal was to consist primarily of a Geotechnical Review Memorandum. The memorandum includes foundation recommendations, pavement design, soil data, and other information needed to produce an accurate cost prediction for the geotechnical design of the project.

Data regarding soil properties has been extracted from soil profiles provided by the client. Soil bearing capacity has been estimated by accepted methods from the blow count data provided for each soil profile from the client. Loads acting on shallow foundations have been approximated, and strip footings are recommended for the auxiliary structures. In accordance with the 2018 North Carolina Building Code (referred to herein as NCBC) 1613.3.5, the seismic design category has been determined. Estimated average weekly truck traffic has been provided by the client to determine average daily truck traffic. From the traffic information and the soil specifications, the roadway has been designed. Deep foundations have been designed in accordance with NCBC 1810 and accepted design practice. A graph is provided comparing individual foundation depth with bearing capacity. Constructability alternatives are briefly presented to enhance the analysis of the cost estimate, and potential hazards associated with construction on the site have been identified.

The project has been completed in the following order: seismic design category, soil property analysis, shallow spread footing foundation engineering, pavement design, constructability, hazards, deep foundation engineering, and compilation.

In addition to the memorandum and this exhaustive report, a poster has been created describing the conclusions of the memorandum. A comprehensive presentation describing the design process and the final product has also been prepared. A summary report presentation has been prepared to be given in a classroom setting to many of the civil engineering students at Brigham Young University.

This document constitutes a report declaring the project to be complete. See included for additional reference.

For convenience, all referenced tables and figures are included in Appendix B or embedded in the body report.

Schedule

The following schedule was produced at the beginning of the project and was followed with loose variation to meet the team's needs. Each week on Monday at 4:00 PM, the team held a regular team meeting to review tasks that were due that week and the following. Assignments were given to team members, and more detailed planning took place on how to complete each task. In the event of classroom instruction at this time, the team meeting was held at 3:00 PM instead. If more time was needed, additional team meetings took place at 5:00 PM and lasted up to an hour.

October 2018
- Complete and submit Statement of Work
- Seismic Site Classification
- Create team lead measures and scoreboard
- Soil settlement analysis

November 2018
- Determine soil bearing capacity
- Design shallow spread footing foundations
- Begin 30% completion report

December 2018
- Complete and submit 30% completion report
- Preliminary plan for Winter Semester

Winter Semester

Week 1 (January 7—January 11)
- Finalize plan for Winter Semester
- Set appropriate lead measures and goals

Week 2 (January 14—January 18)
- Preliminary deep foundation research
- Discuss ideas with Dr. Rollins for deep foundations for gas tanks

Week 3 (January 21—January 25)
- No meeting on Monday (MLK Day)
- Proceed with deep foundation design ideas

Week 4 (January 28—February 1)
- Deep foundation design

Week 5 (February 4—February 8)
- Begin to design deep foundations

Week 6 (February 11—February 15)

- Complete deep foundation design

Week 7 (February 18—February 22)

- Meet Tuesday February 19 (University scheduled Monday classes due to Presidents' Day)
- Preliminary pavement research

Week 8 (February 25—March 1)

- Begin pavement design
- Meet with Dr. Guthrie to discuss pavement ideas

Week 9 (March 4—March 8)

- Pavement design
- Investigate constructability and construction practices

Week 10 (March 11—March 15)

- Complete pavement design
- Examine merits of engineered fill and potential geotechnical risks

Week 11 (March 18—March 22)

- Prepare constructability report

Week 12 (March 25—March 29)

- Create a presentation to be shared in a seminar
- Brainstorm ideas for poster

Week 13 (April 1—April 5)

- Combine all report elements into a geotechnical memorandum draft
- Complete poster
- Practice presentation

Week 14 (April 8—April 12)

- Finalize geotechnical memorandum
- Prepare a final report
- Give presentation on Thursday April 11

Week 15 (April 15—April 19)

- Submit all deliverables

This schedule was modified from what is shown here to meet the needs of the team.

Assumptions & Limitations

- The SPT blow counts were assumed to be correct and to be an accurate representation of the soil under which the footings will be placed.
- The soil descriptions were assumed to be correct and to be an accurate representation of the soil under which the footings will be placed.
- A correlation was made between SPT blow counts and unconfined soil strength. The conservative value was selected.
- The bearing capacity equation used is inherently inaccurate, so a factor of safety of 3 was applied for an allowable bearing capacity. With more complete soil profile data, this factor of safety may be found to be too conservative.
- Requesting the maximum bearing capacity for the soil, equations developed by Terzaghi were used to determine the required minimum depth.
- In the seismic analysis, because exact location was unknown, the county of Cumberland containing Fayetteville, North Carolina was used.
- In regards to pavement design, it was assumed that at least one of the sides of the roadway would not feature curb and gutter or a concrete shoulder. If both sides of the roadway feature concrete curb and gutter or concrete shoulder, the design may be reducible.
- The roadway was designed assuming a 30-year life span and a traffic growth rate of no more than 14.7% increase per year. Larger life spans or growth rates may necessitate design with higher capacity. Smaller values do not enhance the design.
- 12” diameter steel pipe driven pile was the assumed member for the deep foundation design.

See included calculations for additional assumptions.

Design, Analysis, and Results

Shallow Foundation Design:

- The boring with the lowest SPT N values was used—boring B-2, with a shallow N value of 6. Soil profiles are shown below in Figure 1. Locations of the borings are shown in Figures 2 and 3.
- Per NCBC Table 1806.2, the unfactored maximum value of bearing capacity usable with the allowable stress design load combinations cannot be taken as more than 1500 psf.
- Using correlations from Karl Terzaghi and Ralph B. Peck (*Soil Mechanics in Engineering Practice*) found in Table 2, a conservative unconfined compressive strength of 1400 psf are used in subsequent calculations.
  - $q_{all} = C_u N_c S_c d_c$ / FS = 1500 psf
    - FS = 3
    - $C_u = q_u / 2$ = 1400 psf / 2 = 700 psf
    - N_c = 5.14 (Meyerhof and Hanna)
    - $S_c = 1 + 0.2B / L = 1 + 0.2(B/\infty) = 1$ (length of strip footings is assumed to be sufficiently large)
    - $d_c = (1 + 0.2d/B)$
    - Solving for depth factor, $d_c = 1.25 = 1 + 0.2d/B$
    - **1.253 = d/B**

Building loads and NCBC will govern precise footing dimensions.

- With a factor of safety of 3, the net allowable bearing capacity of the soil is thus determined to be 1500 psf so long as it coheres with the relationship between footing width and depth established above.
- With no Atterberg limits or consolidation data, soil settlement cannot be accurately predicted or designed for. Settlement conditions could exist because the soil near the surface is predominantly fine-grained.

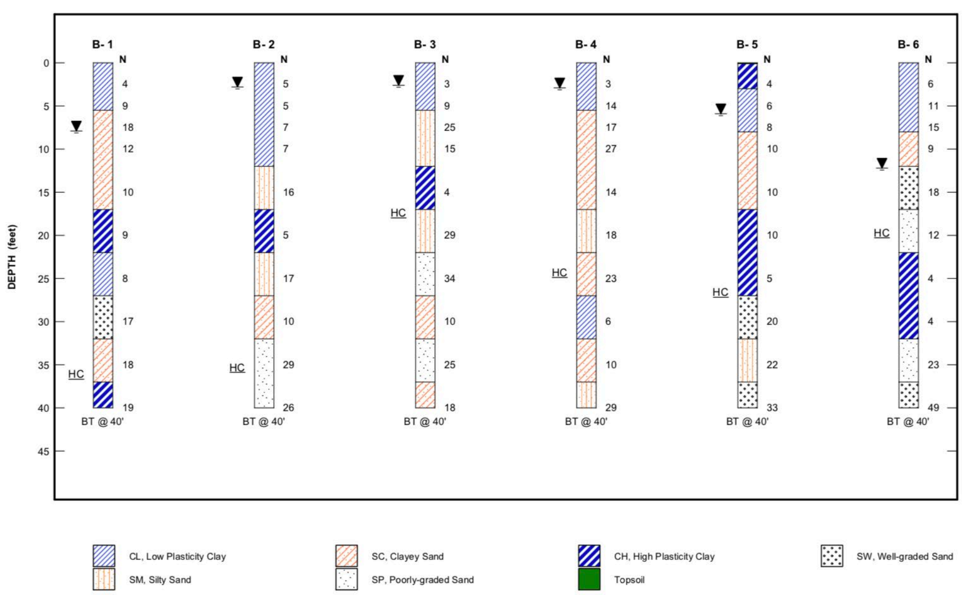


Figure 1: Test boring data.

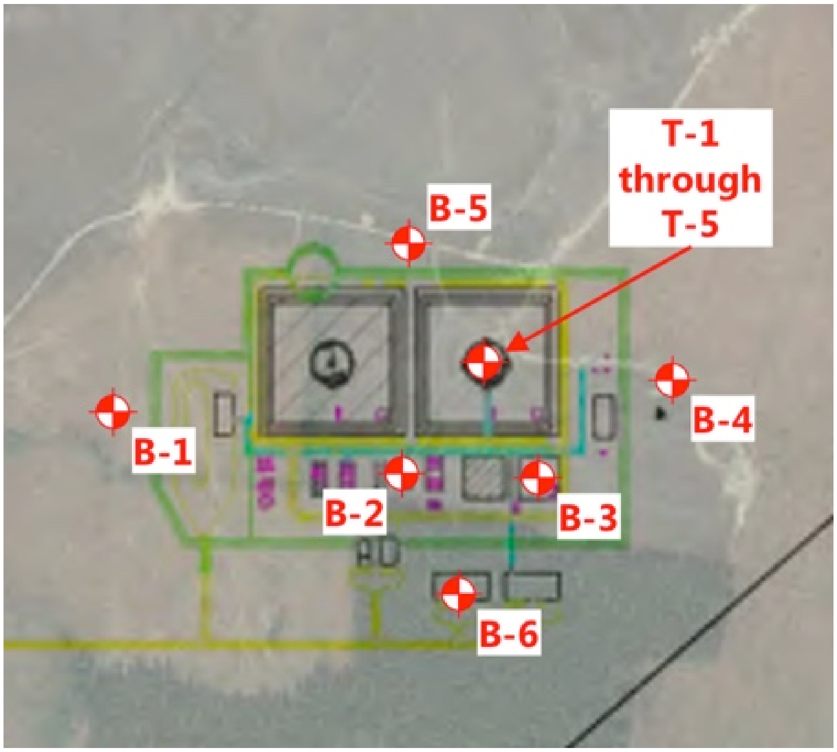


Figure 2: Location of borings in relation to structures.

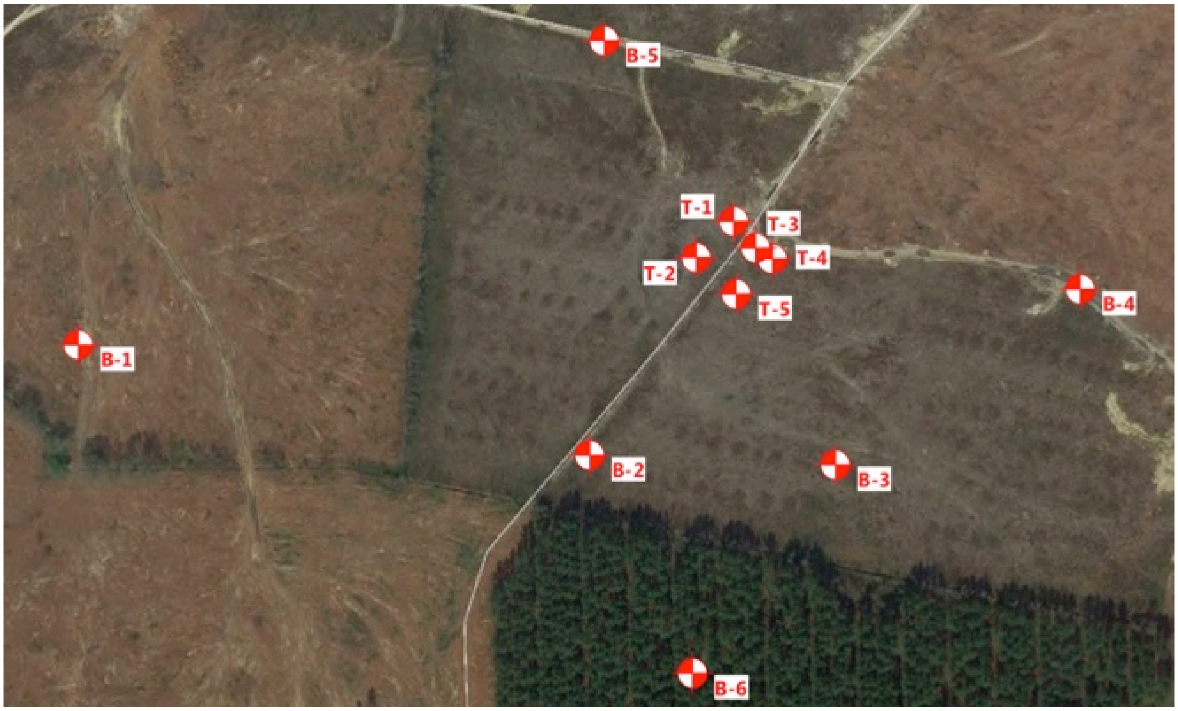


Figure 3: Location of borings in relation to existing geography.

Table 1: Cohesive Soil Consistency from SPT

N Value	Consistency	UCS (q_u)
< 2	Very Soft	< 500 psf
2–4	Soft	500 to 1000 psf
4–8	Medium	1000 to 2000 psf
8–15	Stiff	2000 to 4000 psf
15–30	Very Stiff	4000 to 8000 psf
> 30	Hard	> 8000 psf

Deep Foundation Design:

SPT blow count and soil type data were provided at five locations. Four of these tests reached 50 feet while the fifth reached 100 feet. The client requested deep foundations for these tanks and asked for an analysis on the axial capacity vs. depth of pile in order to estimate the most efficient pile count and depth. The client was specifically interested in the depth necessary for a capacity between 50 and 150 kips. In this analysis, it was assumed that a 12-inch diameter steel pipe pile was to be used.

Pile side resistance in the cohesive layers were calculated using the soil-pile adhesion method, with the American Petroleum Institute alpha coefficients from Figure 4. In the cohesionless layers, side resistance was calculated using the soil-pile friction angle based method, with Fellenius' beta coefficients from Figure 5. For point bearing, Berezantsev's curve for bearing capacity coefficient (Appendix B) was chosen for cohesionless layers. These methods were selected because of their known reliability and conservativity. It is noted that, as with any project involving deep foundations, it may be advisable to perform static or dynamic pile tests in the area. Further testing would dramatically increase the certainty of ultimate axial capacities and may increase the allowable axial capacities for each pile.

As only simple soil classification and blow count data were known, other soil characteristics were estimated for each layer using various correlations. For the cohesive soils, cohesion for each layer was estimated from the blow counts using Table 1. Unit weights of these clays and silts were estimated using Table 2. Side and end bearing capacities were found using the following equations for these layers. A factor of safety of 3 was chosen due to the uncertainty involved with estimating cohesive soil characteristics from SPT blow counts alone.

Cohesive Point Bearing

$Q_p = A_p * q_p,$

where A_p = Area of the base

q_p = bearing pressure at the base

$q_p = c_u N_c + \gamma D,$

where γD is close to the weight of the pile, so is omitted

B = Pile diameter
c_u = average undrained cohesion near pile tip (3B below tip to 8B above tip)
N_c = 9.0 for piles driven deeper than 2.5B

Cohesive Side Friction

$Q_s = \sum q_{si} * A_{si}$,
where A_{si} = surface area of pile in layer i
q_{si} = unit skin resistance in layer i

$q_{si} = c_{ai} = \alpha c_{ui}$,
where c_{ai} = adhesion between pile and clay
c_{ui} = undrained cohesion
α = correction factor dependent on clay stiffness and soil stratification

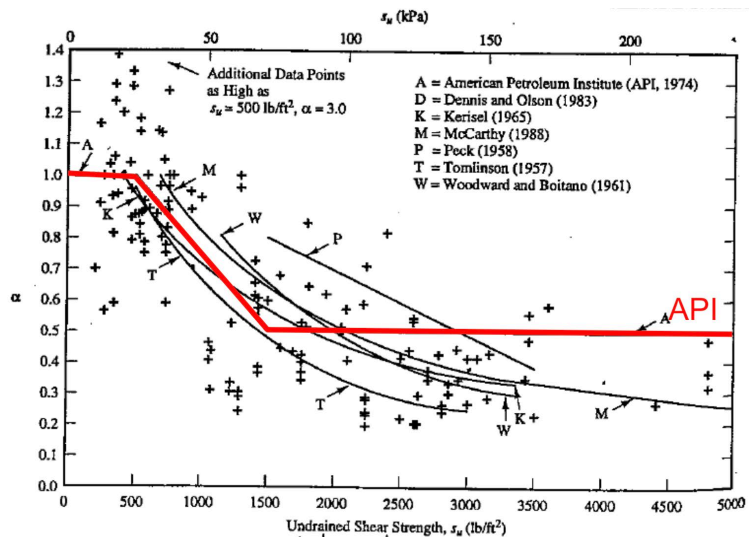


Figure 4: API alpha coefficients.

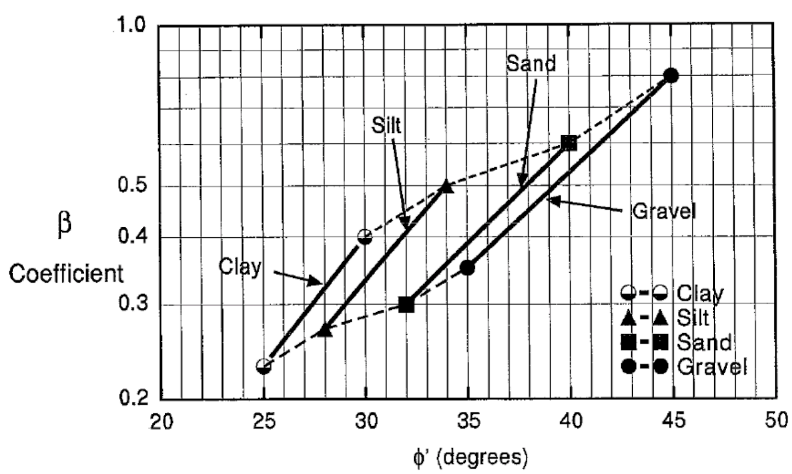


Figure 5: Chart for estimating β coefficient versus soil type and ϕ' angle.

Table 2: Typical Values of Soil Index Properties

Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids(1)					Unit Weight(2) (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	$146^{(3)}$	90	$155^{(3)}$	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	$148^{(4)}$	125	$156^{(4)}$	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5µ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62

For the cohesionless soils, relative densities were first estimated using Figure 6, after which friction angles and unit weights were chosen using Figure 7 (from Figure 7: Correlations of Strength Characteristics for Granular Soils in the Naval Facilities Engineering Command Soil Mechanics Design Manual 7.01) based on the soil type and the estimated relative densities. Using the friction angles, bearing capacity factors were found from Berazantsev's curve (Appendix B), and beta values were found from Fellenius' curves in Figure 5. Effective vertical earth pressures were calculated using the estimated unit weights, and the side and end bearing capacities were calculated according to the following equations. A factor of safety of 2 was chosen for these layers and limiting values for side and end bearing strengths were chosen in accordance with the American Petroleum Institute, as in Table 3.

<u>Cohesionless Point Bearing</u>

$Q_p = A_p * q_p,$

where A_p = Area of the pile base

q_p = bearing pressure on base

$q_p = \gamma D N_q + 0.5 \gamma B N_\gamma,$

where $0.5\gamma B N_\gamma$ is close to the weight of the pile, so is omitted

Cohesionless Side Friction

$Q_s = q_s A_s = \sum q_{si} * \Delta L * (perimeter),$

where A_s = surface area of the shaft

q_{si} = side friction on shaft in segment

ΔL = segment length of shaft

$q_s = K\sigma_v' tan\delta = \beta\sigma_v',$

where K = earth pressure coefficient

σ'_v = vertical effective stress at center of segment, DL

δ = soil-pile friction angle

Table 3: API Limiting q_p Values

Soil Type	Limiting q_p Values (ksf)	Relative Density (%)
Loose Sand	60	< 35
Medium Sand	100	35-65
Dense Sand	200	> 65

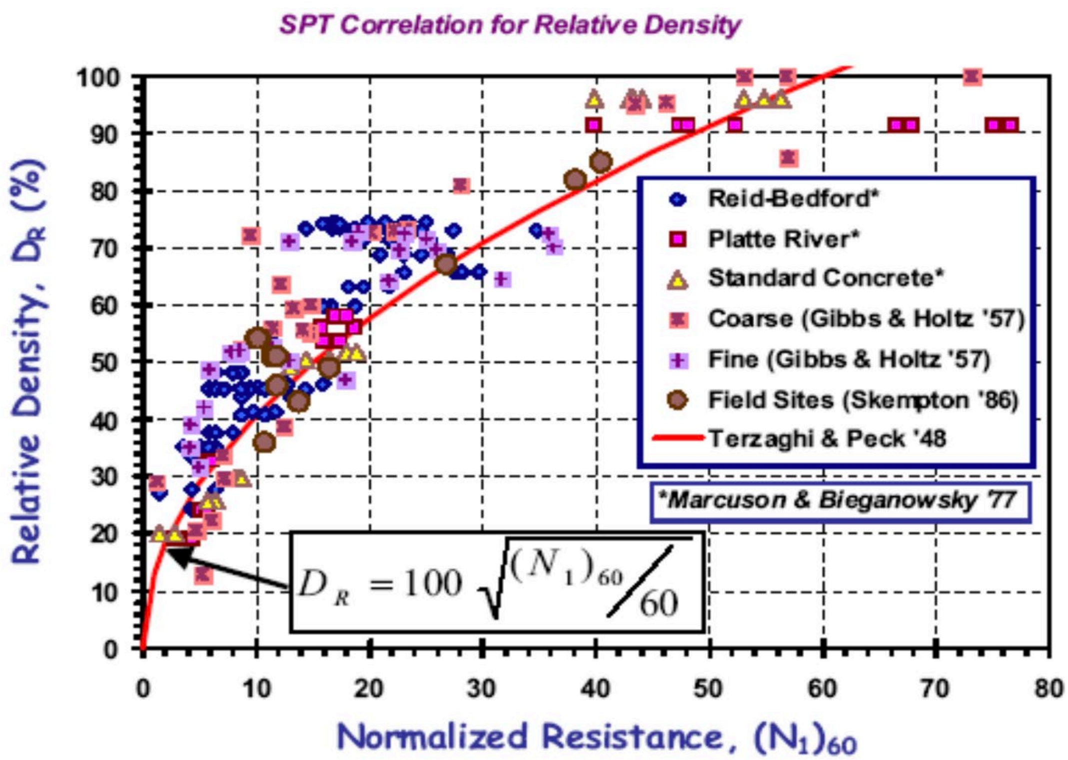


Figure 6: SPT correlation for relative density.

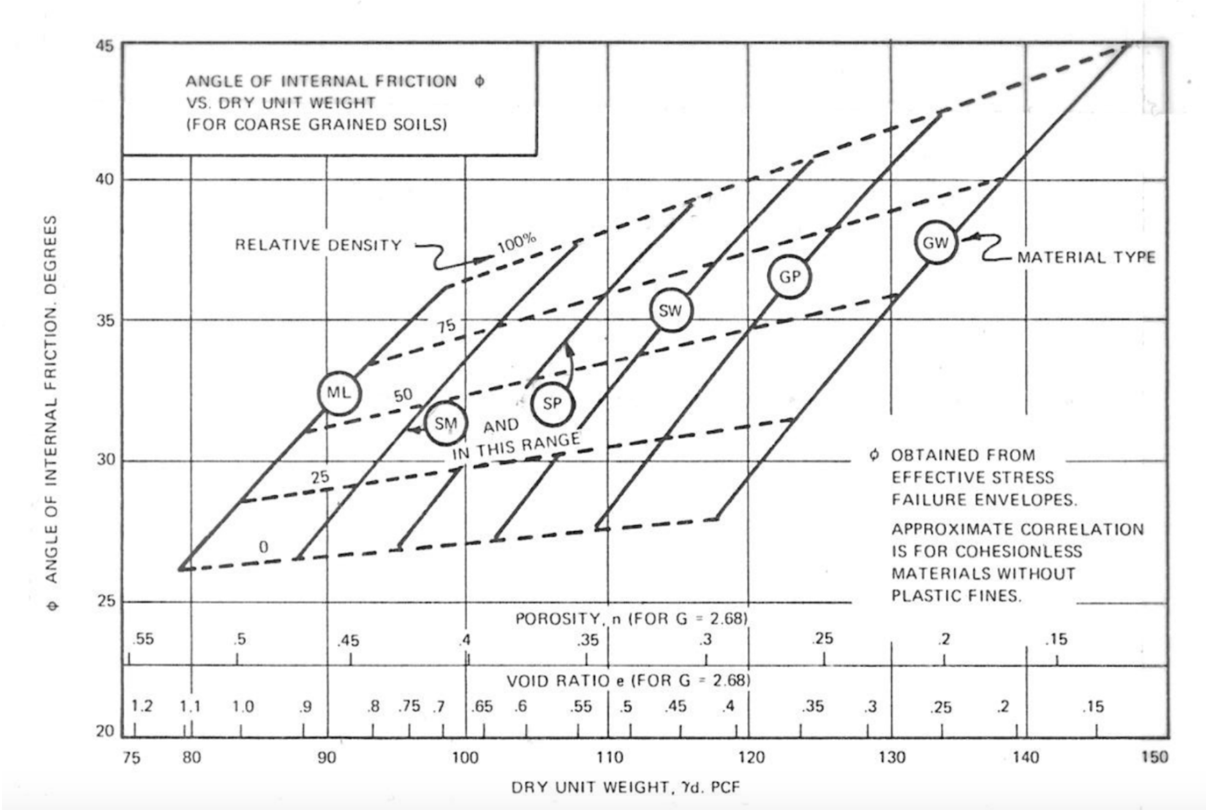


Figure 7: Friction angles and dry unit weights from relative density.

A minimum envelope for allowable side bearing and for allowable total bearing was found and plotted with depth in Figure 8. Values from the graph are also presented in Table 4. The total allowable capacities with depth of each test area were also plotted for the client to see the variability in the results. Depths with relatively large end bearing capacities correspond to layers of cohesionless soil, which we recommend for pile placement if possible. According to our calculations, for an allowable axial capacity between 50 and 150 kip, a pile depth between about 33 and 75 feet must be achieved.

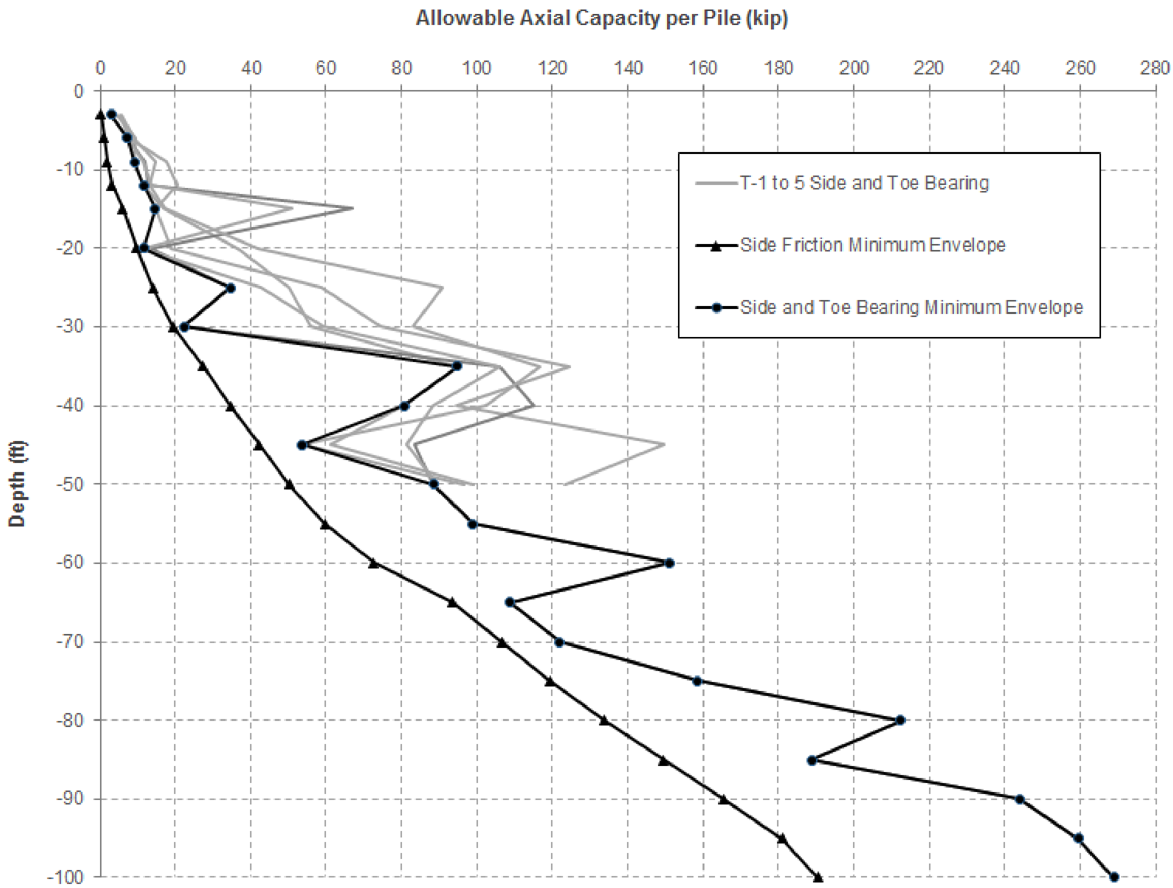


Figure 8: Individual pile capacity against pile depth.

Table 4: Side and Total Allowable Capacities at Various Depths

Minimum Envelope		
Depth (ft)	$(Q_{Side})_{Allow}$ (kips)	$(Q_{Total})_{Allow}$ (kips)
3	0.4	3.0
6	1.1	7.3
9	2.0	9.2
12	3.1	11.7
15	6.1	14.8
20	9.7	11.6
25	14.0	34.6
30	19.4	22.4
35	27.5	94.9
40	34.7	80.6
45	42.2	53.7
50	50.3	88.5
55	59.8	99.0
60	72.6	151.2
65	93.6	108.9
70	106.7	122.0
75	119.3	158.6
80	133.9	212.4
85	149.6	188.9
90	165.3	243.9
95	181.0	259.6
100	190.4	269.0

Seismic Design Category:

- Risk Category (I, II, III, or IV) can be determined from NCBC Table 1604.5. A risk category of **IV** was selected on the grounds of hazardous material storage.
- Seismic Spectral Response Acceleration for site class B:
  - 1-second acceleration, S_1, determined from NCBC Figure 1613.3.1(4) = **0.11**
  - 0.2-second acceleration, S_S, determined from NCBC Figure 1613.3.1(3) = **0.30**
- Site Class, according to NCBC 1613.3.2: **D** *(Insufficient data to determine site class from ASCE 7 chapter 20)*
- Site Coefficients:
  - F_a determined from NCBC Table 1613.3.3(1) = **1.56**
  - F_v determined from NCBC Table 1613.3.3(2) = **2.36**
- Adjusted spectral responses to site class D for maximum considered earthquake:
  - S_{MS} according to NCBC 1613.3.3 = 1.6*0.30 = **0.468**
  - S_{M1} according to NCBC 1613.3.3 = 2.4*0.11 = **0.260**

- Design spectral responses from NCBC 1613.3.4
  - $S_{DS} = (2/3)S_{MS} =$ **0.312**
  - $S_{D1} = (2/3)S_{M1} =$ **0.173**
- Seismic Design Category from NCBC Table 1613.3.5(1) and 1613.3.5(2) ((2) governs): **D**

Seismic design category is based off design spectral response acceleration parameters, S_{DS} and S_{D1} (site is assigned the more severe category from these two parameters). S_{DS} and S_{D1} are determined by multiplying (2/3) by S_{MS} and S_{M1} respectively. S_{MS} is the product of the site coefficient F_a and 0.2-second spectral response acceleration for site class B S_s, while S_{M1} is the product of the site coefficient F_v and 1-second spectral response acceleration for site class B S_1 (F_a and F_v modify the accelerations of site class B into the accelerations for a specific site class). Site coefficients are derived from Site Class, which is determined from soil properties by methods contained in ASCE 7. If data is insufficient to determine site class according to ASCE 7 chapter 20, site class can be taken as D. Site coefficients are contained in NCBC Tables 1613.3.3(1) and NCBC 1613.3.3(2).

Truck Trafficking Roadway Design:

The roadway was designed using accepted practices established by the American Concrete Pavement Association (ACPA), as contained in the publication *Concrete Information: Design of Concrete Pavement for Streets and Roads*, IS184-P, 2006. The team selected concrete as the primary construction material rather than asphalt to minimize creep deformations that standing truck traffic may inflict on asphalt pavements.

Table 5 was provided by the client for traffic information:

Table 5: Projected Average Weekly Traffic

AASHTO Vehicle Class	Estimated Total Weight (kip)	Passes per Week
3	7	200
5	25	50
8	48	50
10	80	50

American Association of State Highway and Transportation Officials (AASHTO) vehicle class 3 is not considered a truck in the calculation of average daily truck traffic (ADTT). In IS184-P, ACPA excludes all two-axle, four-tire trucks. Thus, vehicle class 3 traffic counts were excluded in the following calculations. Propagation of the ADTT over the 30-year design life is described in Table 6. Initial average daily truck traffic was calculated as follows:

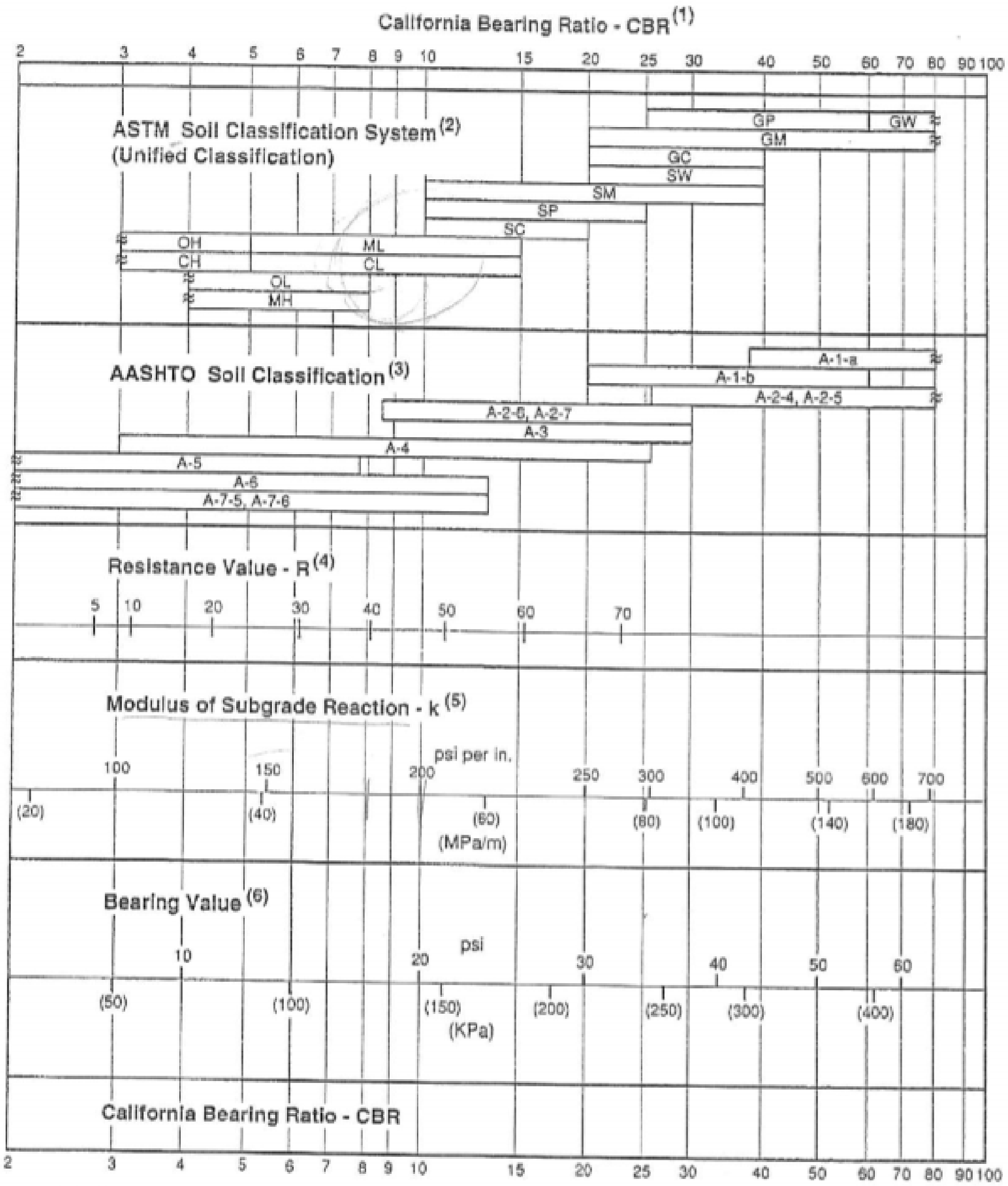
ADTT = Total number of passes per week from classes 5-10 / 7 days per week = 22 passes per day.

Table 6: Propagation Goal-Seek of ADTT

Average Daily Truck Traffic (ADTT)			
Growth rate per year			14.7%
Year	ADTT	Year	ADTT
1	22	16	172.1369
2	25.234	17	197.441
3	28.9434	18	226.4648
4	33.19808	19	259.7552
5	38.07819	20	297.9392
6	43.67569	21	341.7362
7	50.09602	22	391.9715
8	57.46013	23	449.5913
9	65.90677	24	515.6812
10	75.59506	25	591.4863
11	86.70754	26	678.4348
12	99.45355	27	778.1647
13	114.0732	28	892.555
14	130.842	29	1023.761
15	150.0758	30	1174.253
		Design ADTT	**300**

IS184-P assigns this project a traffic classification of "Industrial." For industrial traffic classifications, the minimum ADTT value used for concrete pavement design is 300. ADTT is equal to the average ADTT of each year over the design life span of the roadway, with each ADTT value increased by the growth rate from the previous value, and with the first value being the expected ADTT during the first year of pavement life. Using the design ADTT of 300 passes per day (prescribed by IS184-P) as an input, the table above represents a goal-seek function intended to identify a maximum allowable growth rate per year. The goal-seek yielded the growth rate of 14.7% increase in traffic per year.

The design of concrete pavement according to IS184-P requires a modulus of subgrade reaction, k, which is obtained from Figure 9 (referenced in IS184-P as figure 1). The modulus of subgrade reaction is dependent on the grade upon which the concrete pavement is laid. If concrete pavement is placed directly on the existing sandy clay subgrade, the value of the modulus of subgrade reaction is estimated to be k = 150 pci. If, however, AASHTO A-1 or A-2 engineered fill is used as 4-6-inch subbase, a larger value (k = 300 pci) may be used (use greater fill depth for A-2).



(1) For the basic idea, see O.J. Porter, "Foundations for Flexible Pavements," Highway Research Board *Proceedings of the Twenty-Second Annual Meeting,* 1942, Vol. 22, pages 100-136.
(2) ASTM Designation D2487.
(3) "Classification of Highway Subgrade Materials," Highway Research Board *Proceedings of the Twenty-Fifth Annual Meeting,* 1945, Vol. 25, pages 376-392.
(4) C.E. Warnes, "Correlation Between *R* Value and *k* Value," unpublished report, Portland Cement Association, Rocky Mountain-Northwest Region, October 1971 (best-fit correlation with correction for saturation).
(5) See T.A. Middlebrooks and G.E. Bertram, "Soil Tests for Design of Runway Pavements," Highway Research Board *Proceedings of the Twenty-Second Annual Meeting,* 1942, Vol. 22, page 152.
(6) See item (5), page 184.

Figure 9: Approximate interrelationship of soil classifications and bearing values.

Table 7 (referenced in IS184-P as Table 6(b)) is used to determine the minimum acceptable road thickness for a given traffic classification, modulus of subgrade reaction, and ADTT count (Table 6(a) may be used if both sides of the road feature concrete curb and gutter or concrete shoulder). The thickness of the pavement is given as a range of values dependent on the modulus of rupture of the concrete.

Table 7: Concrete Thickness (inches), 30-Year Design

WITHOUT concrete curb and gutter or concrete shoulders

		k = 100 pci			k = 150 pci			k = 200 pci			k = 300 pci		
		Modulus of Rupture (psi			Modulus of Rupture (psi			Modulus of Rupture (psi			Modulus of Rupture (psi)		
Traffic Classification		550	600	650	550	600	650	550	600	650	550	600	650
Light Residential (Cat LR, SF = 1.0)	ADTT = 3	6.0	5.5	5.5	6.0	5.5	5.5	5.5	5.5	5.0	5.5	5.0	5.0
Residential (Cat 1, SF = 1.0)	ADTT = 10	7.0	6.5	6.0	6.5	6.0	5.5	6.0	6.0	5.5	6.0	5.5	5.5
	ADTT = 20	7.0	6.5	6.0	6.5	6.0	6.0	6.5	6.0	5.5	6.0	5.5	5.5
	ADTT = 50	7.0	6.5	6.5	7.0	6.5	6.0	6.5	6.0	6.0	6.0	6.0	5.5
Collector (Cat 2, SF = 1.1)	ADTT = 50	8.0	7.5	7.0	7.5	7.5	7.0	7.5	7.0	6.5	7.0	6.5	6.5
	ADTT = 100	8.5	8.0	7.5	8.0	7.5	7.0	7.5	7.0	7.0	7.0	7.0	6.5
	ADTT = 500	9.0	8.5	8.0	8.5	8.0	7.5	8.0	7.5	7.0	7.5	7.0	7.0
Business (Cat 2, SF = 1.1)	ADTT = 400	9.0	8.5	8.0	8.5	8.0	7.5	8.0	7.5	7.0	7.5	7.0	7.0
	ADTT = 700	9.0	8.5	8.0	8.5	8.0	7.5	8.0	7.5	7.5	8.0	7.5	7.0
Minor Arterial (Cat 2, SF = 1.2)	ADTT = 300	9.0	8.5	8.0	8.5	8.0	8.0	8.5	8.0	7.5	8.0	7.5	7.0
	ADTT = 600	9.5	9.0	8.5	9.0	8.5	8.0	8.5	8.0	8.0	8.0	7.5	7.5
Industrial (Cat 3, SF = 1.2)	ADTT = 300	10.0	9.5	9.0	9.5	9.0	8.5	9.5	9.0	8.5	9.0	8.5	8.0
	ADTT = 800	10.5	10.0	10.0	10.0	9.5	9.5	9.5	9.0	9.5	9.0	8.5	8.5

The use of dowels is not necessary and does not affect the rest of the design according to IS184-P. However, we do recommend the use of 1.5-inch dowels to provide the concrete with additional resistance to joint faulting under truck loading. Typical practice places dowels in the center of the concrete cross-section.

Although the use of engineered fill with a modulus of subgrade reaction value of k = 300 pci only offers a roadway thickness reduction of 0.5 inches, the use of a 4-6-inch subbase is recommended for constructability purposes, especially if engineered fill is desired for other applications in the project. Placing the concrete pavement directly on the existing grade will require careful compaction of the pre-existing clay material. However, if engineered fill is used, the subbase can be compacted in as little as a single lift, and the reduction in labor costs will likely exceed the cost of the fill.

The final recommendation for the concrete pavement is summarized in Table 8. The precise thickness of the pavement may vary slightly with the modulus of rupture of the concrete. The selection of the concrete modulus of rupture is therefore left to the client, but, to be complete, we have listed a modulus of rupture of 600 psi.

Table 8: Recommended Pavement Thicknesses Across Modulus of Rupture

WITHOUT concrete curb and gutter or concrete shoulders, Pavement Thickness with Dowels			
k = 300 pci			
	Modulus of Rupture (psi)		
	550	600	650
Industrial ADTT ≤ 300	**9.0"**	**8.5"**	**8.0"**

Modulus of rupture is related to compressive strength in the following equation 13.6b provided by Sidney Mindess, J. Francis Young, and David Darwin in their text, *Concrete, Second Edition*:

$$f'_r = 2.30f'_c{}^{2/3}$$

Solving for compressive strength f'_c, the recommended concrete compressive strength rounds to 4000 psi.

Because of the low flexural strength of concrete, control joints are required in concrete pavement. Joints were designed in accordance with Table 9 (referenced as Table 7 in IS184-P).

Table 9: Recommended Joint Spacing for Plain Concrete Pavements

Pavement Thickness	Joint Spacing*
5"	10-12.5 ft
6"	12-15 ft
7"	14-15 ft
8" or more	**15 ft**

*Can vary if local experience indicates; depends on climate and concrete properties

The following roadway design is prescribed: 8.0-inch thick concrete pavement on 4 inches of AASHTO A-1a subbase. Concrete should feature 1.5-inch reinforcing dowels and 2.5-inch deep transverse joints spaced at 15 feet. Concrete should have compressive strength of 4000 psi.

<u>Construction Considerations:</u>

The findings presented in this section are based on common judgment and are not necessarily authoritative. An engineer should review these claims with authoritative sources before implementing them.

Table 10 summarizes the current soil conditions found on the site in OSHA classifications.

Table 10: Summary of Subsurface Conditions

Physical Soil Type	Average Blow Count	Number of Values Averaged	Unconfined Compressive Strength	OSHA Soil Classification
Clayey Sand	7	5	>1.5 tsf	A
Low Plasticity Clay	6	16	0.5-1.5 tsf	B
High Plasticity Clay	4	1	0.5-1.5 tsf	B

Depending on the depth and location of excavation, excavated soil from the projected site can be reused as fill. The available soils are fine grained, and fine-grained soils of low to medium plasticity can be effectively used as backfill (Suryakanta Padhi, "6 Types of Backfill Materials Used in Construction). The sites with clayey sand have the most desirable material for fill. The clays can also be used if contact with free water is avoided. However, the soil may not be usable if the excavation goes below the water table. It is not recommended to reuse saturated clay as fill (US Department of Transportation FHWA).

The best material for structural backfill is well-graded, capable of being well-compacted, and has an optimal amount of moisture for compaction. While the soil on site is usable, it would be more ideal to have a backfill of well-graded cohesionless material to provide a higher bearing capacity and to minimize consolidation costs. Additionally, NCBC 1804 gives the following backfill requirements:

- Contains no organic material
- Contains no construction debris
- Contains no cobbles & boulders
- Placed in lifts
- Compacted in a manner so as not to damage the foundation, waterproofing, or dampproofing material

If engineered backfill is desired, the team recommends the use of AASHTO A-1 or A-2 soil for maximized bearing capacity and minimized labor costs.

Lessons Learned

- Accepted pavement design practice is not standardized but is rather left to the engineer's best judgment. Nevertheless, documents are provided by associations such as ACPA to facilitate the design process.
- Economical applications of the building code.
- Optimization of constructability and economical design.
- Application of resources such as the NCBC, ACPA IS184-P, and a mentor to assert effective decisions.
- Effective compilation of work so conclusive deliverables reflect perspectives and conclusions of all team members.
- Collaborative research to enhance the collective understanding of the team.
- Communication optimization.
- How following up can facilitate a project.
- Value of note-taking.
- Value of frequent regular meetings.
- Utility of extensive planning.

Conclusions

Data regarding soil properties has been extracted from soil profiles provided by the client. Using accepted methods, Soil bearing capacity has been estimated from the blow count data provided for each soil profile. Loads acting on shallow foundations have been approximated, and strip footings have been recommended for the auxiliary structures. In accordance with NCBC 1613.3.5, the seismic design category has been determined.

The LNG storage facility was determined conservatively to have a risk category of IV for seismic design category purposes. Following procedures in NCBC 1613.3.5, the seismic design category was determined to be D.

The soil was found to be mostly clay or silty clay with little variation. Using accepted approximation methods from Terzaghi (*Soil Mechanics in Engineering Practice*), the soil's bearing capacity was determined conservatively to be 1500 psf.

On recommendation from the client and based on common construction practice, shallow spread strip footing foundations are recommended for the construction of auxiliary structures to maximize economy and performance.

The truck trafficking roadway was designed from ACPA IS184-P *Concrete Information: Design of Concrete Pavement for Streets and Roads,* 2006. Combined with constructability considerations, this document conservatively yielded the following roadway design: 8.0-inch deep concrete on 4 inches of AASHTO A-1a subbase, compressive strength 4,000 psi, with 2.5-inch deep joints at 15 feet and 1.5-inch reinforcing dowels centered in the concrete cross-section.

Individual deep foundations can have a capacity of 50-150 kips if depths of 33-75 feet are achieved.

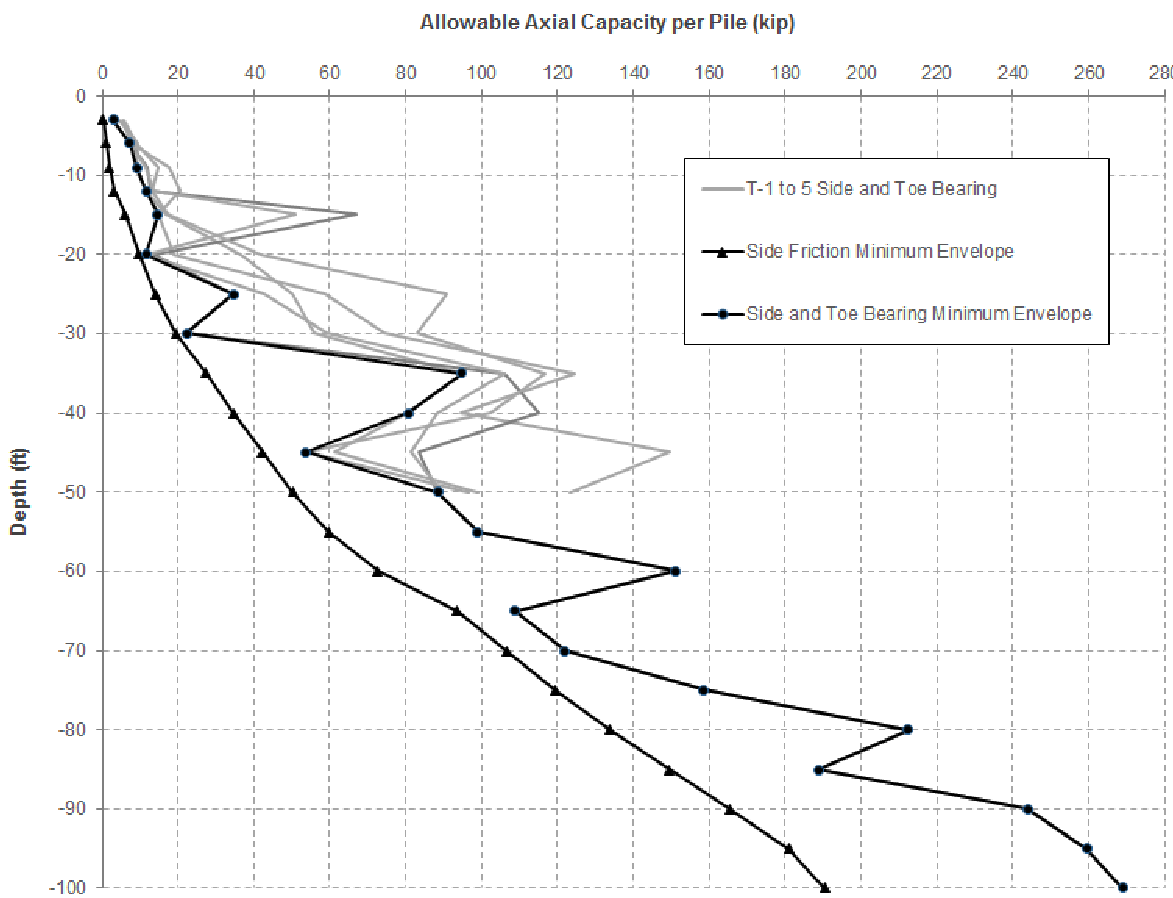
The cost estimates crew should strongly consider the merits of engineered fill to reduce labor costs associated with compacting fine-grained soil and to provide slightly greater foundation strengths. Some recommended soil types include AASTHO A-1 and A-2. Others may be used as the engineer of record deems adequate.

Engineering and construction should be performed to mitigate the possible effects of soil consolidation, soil expansion, shear plane developments during excavation, flooding, and a high water table.

Please contact the team with any concerns or questions regarding these conclusions. Consult the "Data, Analysis, and Results" section for additional details. Examine Appendix B for referenced figures and tables. This information is summarized in the following section, "Recommendations".

Recommendations

- Subsurface conditions – **Medium-soft clay** with some **sandy clay**
- Design bearing capacity – **1500 psf**
- Seismic site classification – **D**
- Shallow spread foundation footing type – **Strip**
- Deep foundation design chart –



- Roadway design – **8.0-inch thick concrete pavement on 4 inches of AASHTO A-1a subbase. Concrete should feature 1.5-inch reinforcing dowels and 2.5-inch deep transverse joints spaced at 15 feet. Concrete should have compressive strength of 4000 psi.**

The team notes that additional soil and site analysis may permit more economical design parameters. Contact the team for details.

Appendix A

Résumés

Matthew D. Martino *EIT*

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EDUCATION

PASSED CIVIL FUNDAMENTALS OF ENGINEERING EXAMINATION *Apr 2018*

BACHELOR'S OF SCIENCE: CIVIL ENGINEERING *Dec 2019*
BRIGHAM YOUNG UNIVERSITY *Provo, UT*

- GPA: 3.70
- Relevant Coursework: Linear Finite Element Methods, Reinforced Concrete Design, Structural Analysis, Computational Methods, Drafting with CAD Applications, Applications of ArcGIS
- Civil Engineering Capstone: Collaborated with a team to engineer deep and shallow foundations for a liquefied natural gas storage complex in North Carolina for Kiewit Engineering, Inc.

EXPERIENCE

PRODUCTION ENGINEER - STUDENT *Jul 2018 –*
ACUTE ENGINEERING, INC. *Orem, UT*

- Engineered 200+ light frame residential homes
- Communicated with 15+ clients and researched code to provide 200+ building official letters

RESEARCH ASSISTANT – CIVIL ENGINEERING *Apr 2018 – Jun 2018*
BRIGHAM YOUNG UNIVERSITY *Provo, UT*

- Analyzed and extracted 50+ highway coupons for structural maintenance tests

TEACHER'S ASSISTANT *Aug 2016 – Jul 2017, Jan 2018 – Jul 2018*
BRIGHAM YOUNG UNIVERSITY *Provo, UT*

- Taught Structural Analysis and Engineering Mechanics: Statics, Strength of Materials, and Dynamics
- Created 50+ online class components, including quizzes and homework assignments
- Led 4+ review sessions of 20-60 students each in preparation for exams

ENGINEERING INTERN *Jul – Aug 2016*
HOMEYER ENGINEERING, INC. *Flowermound, TX*

- Engineered 3+ specialized water resource improvements currently in development
- Qualified 3+ civil construction plans to comply with local code
- Met deadlines for 5+ individually prepared submittals

SKILLS & ABILITIES

- AutoCAD, Revit, Civil 3D, ArcGIS Pro, Microsoft Excel (including Visual Basic), and Microsoft Word

VOLUNTEER EXPERIENCE

- Served in leadership positions for groups of 14+ missionaries while serving a 2-year proselytizing mission for the Church of Jesus Christ of Latter-Day Saints in Las Vegas, NV

INTERESTS

- Music, skiing, and food

MELANIE LATHAM

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melanielatham5@gmail.com
https://www.linkedin.com/in/melanie-latham

I am currently working toward licensure as an EIT and want to pursue a license as a professional engineer. I have practical experience working both in teams and individually to present creative solutions to problems. My specializations and interests include **water resources planning and management, transportation, geotechnical engineering, pavement engineering, interpersonal communication, mathematic computation,** and **music.**

EDUCATION

B.S., Civil Engineering, BRIGHAM YOUNG UNIVERSITY — PROVO, UT — **EXPECTED APRIL 2019**

GPA 3.47

SKILLS

- ArcGIS
- Microsoft Office Suite
- GMS-MODFLOW
- Google Suite
- LANGUAGE: English
- LANGUAGE: Spanish

WORK EXPERIENCE

Intern — **January 2019-Present**
PLEASANT GROVE PUBLIC WORKS — PLEASANT GROVE, UT

- Observing civil engineers in a real working environment.
- Aiding with administrative tasks.

Research Intern — **June 2018-August 2018**
TEXAS A&M UNIVERSITY — COLLEGE STATION, TX

- Examined recharge rates in the Gulf Coast aquifer of Texas using MODFLOW and Excel.
- Developed 20+ contour maps to compare the aquifer at different recharge rates.
- Contributed research to a funded research project and its associated journal article.

Research Intern — **June 2017-August 2017**
NORTH CAROLINA STATE UNIVERSITY — RALEIGH, NC

- Modeled 30+ dams in Excel; created and modified regional maps in ArcGIS
- Presented report at university-wide symposium

AFFILIATIONS AND HONORS

American Society of Civil Engineers, MEMBER — **2015-PRESENT**
Tau Beta Pi Induction, ENGINEERING HONORS SOCIETY — **MARCH 2016**
Spanish Language Certificate: Advanced, BRIGHAM YOUNG UNIVERSITY — **MARCH 2017**

- BASED ON AMERICAN COUNCIL OF THE TEACHING OF FOREIGN LANGUAGES GUIDELINES AND SUPPORTING COURSEWORK.

VOLUNTEER

Full-Time Volunteer Representative — **August 2013-February 2015**
RELIGIOUS ORGANIZATION — OSORNO, CHILE

- Taught 30+ English-language workshops to native Chileans and other Spanish speakers
- Taught 1000+ character-improving lessons to community members in Spanish

Zachary Farnsworth EIT

496 North 750 East, Provo, UT 84606 | (210) 332-7640 | zachfarns@gmail.com

Education

Passed Civil Fundamentals of Engineering Examination *Mar 2018*

Bachelor of Science, Civil Engineering; Minor, Mathematics *(anticipated Apr 2019)*
Brigham Young University *Provo, UT*

- 3.76 GPA
- Civil Engineering Capstone: Designed deep and shallow foundations for a liquified natural gas storage facility in North Carolina for Kiewit Engineering, Inc.
- Relevant Coursework: Foundation Engineering, Reinforced Concrete Design, Structural Steel Design, Structural Analysis, Computational Methods, Drafting with CAD Applications

Engineering Experience

Research Assistant - Civil Engineering *Jun 2018–*
Brigham Young University *Provo and Lehi, UT*

- Oversaw the design and analysis of all 25+ structural steel components of the project
- Collaborated with a team on the geotechnical analysis of data from over 900 strain gauges
- Performed 30+ nuclear density gage tests and 200+ total station, digital electronic level, and surveyors level measurements
- Operated light and heavy excavation and compaction machinery on the dismantling and rebuilding of an MSE wall

Field Assistant - Civil Engineering *Mar–Jun 2018*
Brigham Young University *Provo, UT*

- Conducted a GIS survey and detailed inventory of 400+ catch basins and manholes
- Performed data entry for the hydraulic computer modeling of BYU's storm water system

Other Work and Volunteer Experience

Delivery Driver *May–Aug 2017*
Domino's Pizza *San Antonio, TX*

- Demonstrated a willingness to act as a team player in taking undesirable shifts, assignments, and responsibilities

Missionary Representative *Jun 2014–Jun 2016*
The Church of Jesus Christ of Latter-day Saints *Anchorage and Fairbanks, AK*

- Trained and oversaw groups of 8–16 other volunteers; resolving conflicts and fostering unity
- Developed interpersonal and intercultural skills, confidence in public speaking, and professionalism

Skills and Honors

- Proficient in Microsoft Excel with Visual Basic; limited ability in SAP 2000, Mathcad, and Revit
- Tau Beta Pi member: Engineering Honor Society
- Heritage Scholarship recipient: 4-Year, Full Tuition (merit based)
- Eagle Scout

Appendix B

Other Referenced Tables and Figures

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Risk Categories I, III and IV.
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing Group E occupancies with an occupant load greater than 250. • Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000.[a] • Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV. • Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released.[b]
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures. • Buildings and other structures containing quantities of highly toxic materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released.[b] • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.2 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.
b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

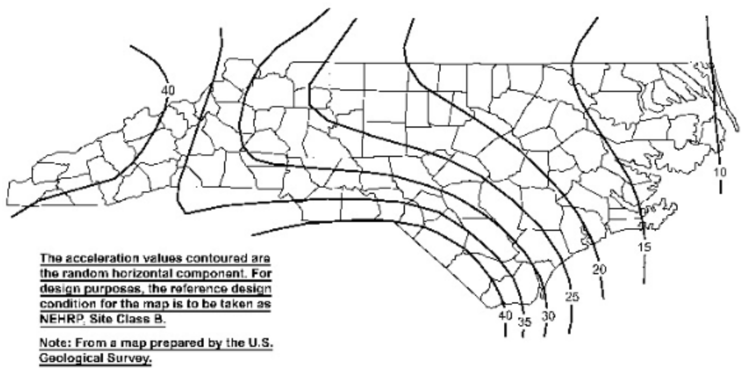


FIGURE 1613.3.1(3)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR NORTH CAROLINA OF 0.2 SECOND SPECTRAL RESPONSE ACCELERATION (5 PERCENT OF CRITICAL DAMPING), SITE CLASS B

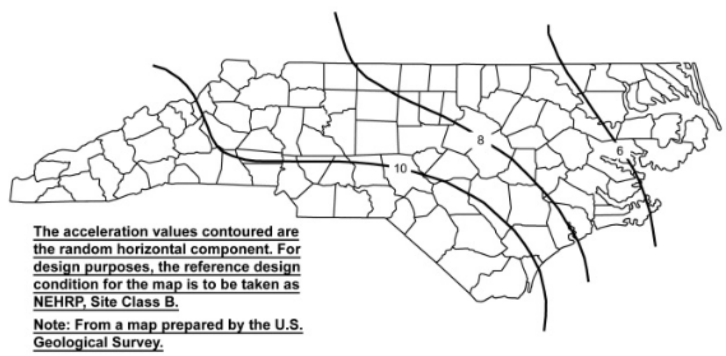


FIGURE 1613.3.1(4)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR NORTH CAROLINA OF 1.0 SECOND SPECTRAL RESPONSE ACCELERATION (5 PERCENT OF CRITICAL DAMPING), SITE CLASS B

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_V[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

1613.3.5.1 Alternative seismic design category determination.
Where S_1 is less than 0.75, the *seismic design category* is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:

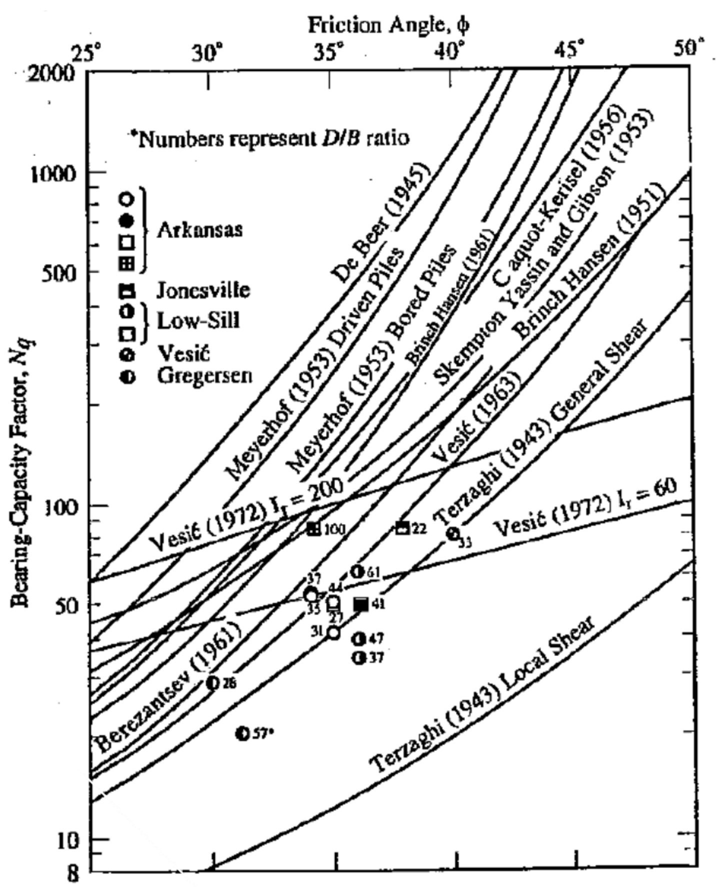
1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.5 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid or are permitted to be idealized as rigid in accordance with Section 12.3.1 of ASCE 7 or, for diaphragms permitted to be idealized as flexible in accordance with Section 12.3.1 of ASCE 7, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

TABLE 1806.2
PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

For SI: 1 pound per square foot = 0.0479kPa, 1 pound per square foot per foot = 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.
b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.



Bearing-capacity factor N_q curves (Berezantsev's curve was used for deep foundations).