Tolunay-Wong Engineers, Inc.

GEOTECHNICAL ENGINEERING STUDY
PROPOSED NEW BUILDING
PORT ARTHUR, TEXAS

Prepared for:

Chica & Associates, Inc.
505 Orleans Street, Suite 106
Beaumont, Texas 77701

Prepared by:

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July 31, 2013

Project No. 13.23.174 / Report No. 60989

www.tweinc.com

Geotechnical Engineering
Environmental Consulting
Construction Materials Testing
Deep Foundations Testing

Tolunay-Wong Engineers, Inc.

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July 31, 2013

Chica & Associates, Inc.
505 Orleans Street, Suite 106
Beaumont, Texas 77701

Attn: Mr. Samuel Fountain
sfountain@chicaandassociates.com

Ref: Geotechnical Engineering Study
Proposed New Building
Port Arthur, Texas
TWE Project No. 13.23.174 / Report No. 60989

Dear Mr. Fountain,

Tolunay-Wong Engineers, Inc. (TWE) is pleased to submit this report of our geotechnical engineering study performed for the referenced project. This report contains a detailed description of the field and laboratory work performed for this study as well as the soil boring logs including tabulated laboratory test results. Also included in this report are our geotechnical design and construction recommendations for the new building to be constructed in Port Arthur, Texas.

We appreciate the opportunity to work with you on this phase of the project and we look forward to the opportunity of providing additional services as the project progresses. If you have any questions regarding this report or if we can be of further assistance, please contact us.

Sincerely,

TOLUNAY-WONG ENGINEERS, INC.
Texas Board of Professional Engineers Firm Registration No. F-000124

STATE OF TEXAS
PATRICK J. KENNEY
87994
LICENSED
PROFESSIONAL ENGINEER
7-31-13

Armando Gomez, Jr., E.I.T
Staff Professional
Beaumont, Texas

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Vice President – Southeast Region
Engineering Services

AG/PJK/ag

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1 INTRODUCTION AND PROJECT DESCRIPTION

1.1 Introduction

This report presents the results of our geotechnical engineering study performed for the proposed new building to be constructed in Port Arthur, Texas. The building location is presented on the drawing provided by the Client in Appendix A. Our geotechnical engineering study was performed in general accordance with TWE Proposal No. P13-B105 (Revision 2) dated June 9, 2013 and authorized by Andy Chica of Chica & Associates, Inc. (Client) on June 9, 2013.

1.2 Project Description

The project includes the construction a new building near the intersection of Shreveport Avenue and 4^{th} Street in Port Arthur, Texas. The building is approximately 14,000-ft^2 in size. Project information provided by the Client is presented in Appendix A.

2 PURPOSE AND SCOPE OF SERVICES

The purposes of our geotechnical engineering study were to investigate the soil and groundwater conditions within the project site and to assist our Client in the design and construction of foundations for the proposed new building.

Our scope of services for the project consisted of:

1. Drilling one (1) soil boring within the building footprint to evaluate subsurface stratigraphy and groundwater conditions;

2. Performing geotechnical laboratory tests on recovered soil samples to evaluate the physical and engineering properties of the strata encountered;

3. Providing geotechnical design recommendations for suitable shallow foundations to support the proposed new building; and,

4. Providing geotechnical construction recommendations including site and subgrade preparation, excavation considerations, fill and backfill requirements, compaction requirements, foundation installation and overall quality control monitoring, testing and inspection services.

Our scope of services did not include any environmental assessments for the presence or absence of wetlands or of hazardous or toxic materials within or on the soil, air or water at the project site. Any statements in this report or on the boring logs regarding odors, colors, unusual or suspicious items and conditions are strictly for the information of the Client. A geological fault study was also beyond the scope of our geotechnical engineering study.

3 FIELD PROGRAM

3.1 Soil Borings

As requested, TWE conducted an exploration of subsurface soil and groundwater conditions at the project site on July 19, 2013 by drilling, logging and sampling one (1) soil boring to a depth of 30-ft below ground surface. Our geotechnician coordinated the field activities, logged the boreholes and obtained groundwater level measurements during drilling and sampling. The soil boring locations are presented on Drawing No. 13.23.174-1 in Appendix B of this report.

3.2 Drilling Methods

Field operations were performed in general accordance with *Standard Practice for Soil Investigation and Sampling by Auger Borings [American Society for Testing and Materials (ASTM) D 1452]*. The soil borings were drilled using a truck-mounted drilling rig equipped with a rotary head. The boreholes were advanced using dry-auger drilling methods. Samples were obtained continuously at intervals of 2-ft from existing ground surface to a depth of 12-ft, at 13-ft to 15-ft and at 5-ft depth intervals thereafter until the boring completion depth of 30-ft was reached.

3.3 Soil Sampling

Fine-grained, cohesive soil samples were recovered from the soil borings by hydraulically pushing a 3-in diameter, thin-walled Shelby tube a distance of about 24-in. The field sampling procedures were conducted in general accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D 1587).* Our geotechnician visually classified the recovered soils and obtained a field strength measurement of the soils using a calibrated pocket penetrometer. A factor of 0.67 is typically applied to the penetrometer measurement to estimate the undrained shear strength of the Gulf Coast cohesive soils. The samples were extruded in the field, wrapped in foil, placed in moisture sealed plastic bags and protected from disturbance prior to transport to the laboratory. The recovered soil sample depths and pocket penetrometer measurements are presented on the boring logs in Appendix C.

3.4 Boring Logs

Our interpretations of general subsurface soil and groundwater conditions at the boring locations are included on the project boring logs. The interpretations of the soil types throughout the boring depths and the locations of strata changes were based on visual classifications during field sampling and laboratory testing using *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) [ASTM D 2487]* and *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) [ASTM D 2488]*. The boring logs include the type and interval depth for each sample along with the pocket penetrometer readings and blow count values for the recovered soils. The project boring logs and a key to the terms and symbols used on boring logs are presented in Appendix C.

Coarse-grained, cohesionless and semi-cohesionless soil samples were collected with the Standard Penetration Test (SPT) sampler driven 18-in by blows from a 140-lb hammer falling 30-in in accordance with the *Standard Test Method for Standard Penetration Test (SPT) and Spilt-Barrel Sampling of Soils (ASTM D 1586).* The number of blows required to advance the sampler three (3) consecutive 6-in depths are recorded for each corresponding sample on the boring logs. The N-value, in blows per foot, is obtained from SPTs by adding the last two (2) blow count numbers. The relative density of cohesionless and semi-cohesionless soils and the consistency of cohesive soils can be inferred from the N-value. The samples obtained from the split-barrel sampler were visually classified, placed in moisture sealed plastic bags and transported to our laboratory. SPT sampling intervals and blow counts are presented on the boring logs in Appendix C of this report.

3.5 Groundwater Measurements

Groundwater level measurements were attempted in the open boreholes during dry-auger drilling. Water level readings were attempted when groundwater was first encountered and at five (5) minute intervals over a fifteen (15) minute time period. The groundwater observations are summarized in Section 5.4 of this report entitled *"Groundwater Observations."*

4 LABORATORY SERVICES

A laboratory testing program was conducted on selected samples to assist in classification of the soils encountered in the project borings and to evaluate the physical and engineering properties of the strata encountered at the project site.

4.1 Laboratory Testing Program

Laboratory tests were performed in general accordance with ASTM International standards to measure physical and engineering properties of the recovered samples. The types and brief descriptions of the laboratory tests performed are presented below.

Table 4-1 **Laboratory Testing Program**	
Test Description	**Test Method**
Amount of Material in Soils Finer than No. 200 Sieve	ASTM D 1140
Water (Moisture) Content of Soil	ASTM D 2216
Unconsolidated-Undrained Triaxial Compression on Cohesive Soils	ASTM D 2850
Liquid Limit, Plastic Limit and Plasticity Index of Soils	ASTM D 4318
Dry Unit Weight	--

Amount of Materials in Soils Finer than No. 200 (75-µm) Sieve (ASTM D 1140)
This test method determines the amount of materials in soils finer than the No. 200 (75-µm) sieve by washing. The loss in weight resulting from the wash treatment is presented as a percentage of the original sample and is reported as the percentage of silt and clay particles in the sample.

Water (Moisture) Content of Soil by Mass (ASTM D 2216)
This test method determines water (moisture) content by mass of soil where the reduction in mass by drying is due to loss of water. The water (moisture) content of soil, expressed as a percentage, is defined as the ratio of the mass of water to the mass of soil solids. Moisture content may provide an indication of cohesive soil shear strength and compressibility when compared to Atterberg Limits.

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils (ASTM D 2850)
This test method determines the strength and stress-strain relationships of a cylindrical specimen of either undisturbed or remolded cohesive soil. Specimens are subjected to a confining fluid pressure in a triaxial chamber. No drainage of the specimen is permitted during the test. The specimen is sheared in compression without drainage at a constant rate of axial deformation (strain controlled). The unconsolidated-undrained (UU) triaxial shear strength of cohesive soils is applicable to situations where loads are assumed to take place so rapidly that there is insufficient time for induced pore-water pressures to dissipate and drainage to occur during the loading period.

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D 4318)
This test method determines the liquid limit, plastic limit and the plasticity index of soils. These tests, also known as Atterberg limits, are used from soil classification purposes. They also provide an indication of the volume change potential of a soil when considered in conjunction with the natural moisture content. The liquid limit and plastic limit establish boundaries of consistency for plastic soils. The plasticity index is the difference between the liquid limit and plastic limit.

Dry Unit Weight of Soils
This test method determines the weight per unit volume of soil, excluding water. Dry unit weight is used to relate the compactness of soils to volume change and stress-strain tendencies of soils when subjected to external loadings.

Soil properties including moisture content, unit weight, Atterberg limits, grain size distribution, penetration resistance and compressive strength are presented on the project boring logs in Appendix C.

5 SITE CONDITIONS

Our interpretations of soil and groundwater conditions within the project site are based on information obtained at the soil boring locations only. This information has been used as the basis for our conclusions and recommendations. Subsurface conditions may vary at areas not explored by the soil borings. Significant variations at areas not explored by the soil borings will require reevaluation of our recommendations.

5.1 Site Description and Surface Conditions

The project site is located at the intersection of Shreveport Avenue and 4th Street in Port Arthur, Texas. The surface conditions consisted of an open area with grass cover. Drainage appeared to be adequate.

5.2 Subsurface Soil Stratigraphy

The generalized subsurface profile encountered in the project borings consists of primarily firm to stiff cohesive clay from existing grade to the completion depths. Detailed descriptions of the soils encountered are provided on the boring logs in Appendix C.

5.3 Subsurface Soil Properties

Results of Atterberg limit tests on selected cohesive soil samples from the project borings indicated liquid limits (LL) ranging from 28 to 73 with corresponding plasticity indices (PI) ranging from 11 to 57. In-situ moisture contents from test on selected samples ranged from 21% to 31%. The amount of material passing the No. 200 sieve ranged from 81% to 91% within the selected cohesive soil samples tested.

Pocket penetrometer readings taken on cohesive samples indicated approximate undrained shear strengths ranging from 0.33-tsf to 0.83-tsf. Undrained shear strengths derived from laboratory UU testing ranged from 0.37-tsf to 0.64-tsf within the selected cohesive samples tested with corresponding total unit weights ranging from 118-pcf to 128-pcf. Based on the above undrained shear strength data, the cohesive soils encountered in the project borings are considered to have firm to stiff consistencies.

The above laboratory test results are tabulated at the recovered sample depths on the boring logs in Appendix C of this report.

5.4 Shrink/Swell Potential

The tendency for a soil to shrink and swell with change in moisture content is a function of clay content and type, which are generally reflected in soil consistency as defined by the Atterberg Limits. A generalized relationship between shrink/swell potential and the soil plasticity index (P.I.) is shown on the following page:

Table 5-1 General Relationship Between P.I. and Shrink/Swell Potential	
P.I. Range	***Shrink/Swell Potential***
0 – 15	Low
15 – 25	Medium
25 – 35	High
> 35	Very High

The amount of expansion that will actually occur with increase in moisture content is inversely related to the overburden pressure; that is, the larger the overburden pressure, the smaller the amount of expansion. Near-surface soils are thus susceptible to shrink/swell behavior because they experience low amounts of overburden. Shrink/swell behavior is normally considered to be limited to the upper 6 feet of the various soil formations in this Coastal Zone of Texas. The presence of a water table will tend to keep the clays near the water table saturated and thus less likely to swell. Overall, the clay soils above 6 feet at this site possess very high shrink/swell potential.

5.5 Groundwater Observations

Groundwater level measurements were attempted in the open boreholes when groundwater was first encountered during dry-auger drilling and then attempted after fifteen (15) minutes. Free water was encountered at a depth of 18-ft during dry-auger drilling and rose to a depth of 9-ft after fifteen (15) minutes.

Groundwater levels may fluctuate with climatic and seasonal variations and should be verified before construction. Accurate determination of static groundwater levels is typically made with standpipe piezometers. Installation of a standpipe piezometer to evaluate long-term groundwater conditions was not included in our scope of services.

6 GEOTECHNICAL RECOMMENDATIONS

6.1 Discussion

Our geotechnical engineering study includes the development of shallow foundation design and construction recommendations for the proposed new building. Our recommendations for shallow foundations are presented in Section 6.2 of this report entitled *"Shallow Foundation Design."* Properly designed and constructed ground-supported floor slabs used in conjunction with a shallow foundation system are also provided in Section 6.2

6.2 Shallow Foundation Design

The subsurface conditions encountered within the project borings are considered suitable for supporting shallow foundation systems for the proposed new building. We recommend that the shallow foundation system consist of drilled footings. Drilled footings could consist of drilled-and-underreamed footings or straight-sided drilled footings.

6.2.1 Drilled Footings

6.2.1.1 Foundation Depth

Drilled-and-underreamed piers should be placed at a depth of 10-ft below existing grade within the natural clay soils and have a minimum shaft diameter of 18-in. We recommend that the ratio of underream to shaft diameter be no greater than 3.0. The angle of underreamed bells to horizontal should not be less than 45° to avoid potential collapse of the bells. In case of borehole sloughing/caving at the time of bell drilling, a larger angle of 60° should be used. The clear spacing between underreams should be a minimum of one (1) underream diameter for foundation design.

If excavation of bell underreams cannot be completed because local anomalies are encountered or sloughing and/or caving occurs, TWE should be contacted to investigate the problem and modify our recommendations accordingly. The Contractor may be required to excavate to a depth specified by the Geotechnical Engineer and provide a straight-sided shaft with the shaft diameter equal to the design bell diameter.

6.2.1.2 Allowable Net Bearing Pressure

The recommended allowable net bearing pressure for drilled footings bearing on the undisturbed natural clay soils at the recommended depth of 10-ft below existing grade is 3,000-psf. This value should provide a factor of safety of 3.0 against soil shear failure.

6.2.1.3 Uplift Resistance

The allowable uplift capacity of drilled-and-underreamed piers at the project site may be calculated using the following equations:

For $D_f/B \geq 1.5$

$$Q_a = W_f/1.2 + [5.4(B^2 - b^2)/FS]$$

For $D_f/B < 1.5$

$$Q_a = W_f/1.2 + [3.0(D_f/B)^2(B^2 - b^2)/FS]$$

where:

Q_a	=	Allowable Uplift Capacity (kips)
W_f	=	Weight of Footing (kips)
D_f	=	Depth of Base of Footing below Ground Surface (ft)
B	=	Diameter of Underream (ft)
b	=	Diameter of Shaft (ft)
FS	=	Factor of Safety (2.0 for transient loads, 3.0 for sustained loads)

For straight-sided drilled piers, resistance to uplift is provided by shaft side friction plus the weight of the footing. To calculate allowable uplift capacity, an allowable side friction of 275-psf may be used for design. The side friction should be multiplied by the surface area of the shaft in contact with the natural clay soils neglecting the base. It is recommended that a buoyant unit weight of 90-pcf be used for concrete to calculate the weight of the footing. The weight of the footing should be reduced by a factor of safety of 1.2.

6.2.1.4 Lateral Resistance

Lateral resistance for drilled-and-underreamed piers is primarily due to passive resistance of soil against the side of the pier. For the conditions observed within the project borings, we recommend the soil parameters in the table below for use with lateral earth pressure designs.

Table 6-1
Lateral Analysis Soil Design Parameters

Parameter	0-ft to 10-ft below Existing Ground Surface
Cohesion, c	1.00-ksf
Angle of Friction, ϕ	0°
Average Moist Unit Weight of Soil, γ	123-pcf

Design equations for laterally-loaded drilled footings are given in Appendix D of this report. For these equations, D_s should be taken as 2-ft if pavements are constructed up to grade beams. D_s should be taken as 6-ft if pavements are not constructed up to grade beams. A safety factor of at least 1.5 should be used for resisting moment of transient loads and 2.0 for other loads. Lateral resistance from structural select fill should be neglected for exterior piers.

6.2.1.5 Settlement

We have estimated settlements for drilled footings based on our experience. These estimates assume uniformly-loaded foundations with pressures that are no greater than the recommended allowable net bearing pressures and that the foundations are designed and constructed in accordance with the recommendations provided in this report. In addition, these estimates assume that the drilled footings will act as isolated foundations. Therefore, the clear spacing between the footings should be great enough to significantly reduce the influence from adjacent foundations.

The recommended clear spacing between drilled footings should be a minimum of one (1) underream diameter or straight-sided drilled pier diameter, respectively, for foundation design. Settlement of properly-designed and installed drilled footings is estimated to be less than about 1-in. Differential settlement is expected to be on the order of one-half (½) the total settlement.

6.2.2 Building Floor Slab

As previously mentioned, the clay soils encountered within the project borings above 6-ft at the project site possess high to very high shrink/swell potential. It is generally accepted that a primary source of foundation distress is soil movements associated with shrink/swell behavior of subgrade soils. It is therefore recommended that measures be incorporated into the design of floor slabs for the proposed new building to reduce the shrink/swell potential of the foundation soils.

6.2.2.1 Ground-Supported Floor Slabs

The floor slab of the proposed building may consist of a ground-supported floor slab provided that remedial methods are used to reduce the potential for shrink/swell movement to tolerable levels. The typical method for reducing shrink/swell potential includes the installation of a non-expansive soil layer beneath the floor slab. This method has beneficial results but does not totally eliminate the potential for shrink/swell movements.

In order to reduce the potential shrink/swell movements to tolerable limits of about 1-in or less, it is recommended that a minimum of 2-ft of non-expansive select structural fill be provided beneath the floor slab. We recommend that the 2-ft of non-expansive material consist of properly-compacted sandy lean clay (CL) material having a liquid limit less than 40 and a plasticity index between 10 and 20.

The minimum fill thickness may be provided by undercutting the existing material and replacing it with select fill, raising the building pad with select fill or a combination of these alternatives. Structural select fill should extend full depth at least 2-ft beyond the perimeter of the building area.

It should be noted that these methods for reducing shrink/swell movements are designed for normal seasonal changes in soil moisture content of the subgrade soils. Excessive shrink/swell movements can be expected if increases in soil moisture content occur as a result of broken water and sewer lines, improper drainage of surface water, landscaping planted near the foundation slab and excessive irrigation.

After stripping the building area and prior to placement of fill, the exposed subgrade should be inspected by the Geotechnical Engineer or his representative and proofrolled to identify any soft or pumping soils. Soft or pumping soils should be removed to a level of firm soils and replaced with fill material satisfying the requirements of this report.

Special care should be taken not to allow the exposed subgrade soils to become extremely wet or extremely dry of the existing moisture content. Therefore, delays between excavation and fill placement should be avoided. If construction occurs during rainy weather and the exposed subgrade soils are allowed to become wet or saturated, removal or stabilization of excessively soft and wet soils should be anticipated. The depth of undercutting or stabilization should be determined in the field by the Geotechnical Engineer or his representative. Proofrolling and undercutting should also be performed under the direction of the Geotechnical Engineer or his representative. After proofrolling, the top 6-in of subgrade soils should be scarified and compacted to 95% of the maximum dry density and within 3% of the optimum moisture content as determined by ASTM D 698 (Standard Proctor Compaction Effort).

It is recommended that a vapor barrier such as polyethylene sheeting be provided beneath soil supported floor slabs. Adequate construction joints and reinforcement should be provided to reduce the potential for cracking of the floor slab due to differential movement and volume change in concrete.

7 CONSTRUCTION CONSIDERATIONS

7.1 Subgrade Preparation

Areas designated for new construction should be stripped of all surface vegetation, loose topsoil, debris and fill material. The exposed soil subgrade should then be proofrolled with at least a 15-ton pneumatic roller, loaded dump truck or equivalent to detect weak areas. Such weak areas should be removed and replaced with properly-compacted select fill. Subsequent to proofrolling and just prior to placement of select fill, the exposed subgrade should be compacted to at least 95% of the maximum dry density near optimum (to +3%) in accordance with ASTM D 698 procedures.

Proper site drainage should be maintained during construction so that ponding of surface runoff does not occur and cause construction delays or inhibit site access. If the natural subgrade becomes wet and soft, consideration can be given to either removal and replacement of the wet material with structural fill or in-place stabilization with lime.

7.2 Structural Select Fill

Structural fill for the building area should consist of a clean, low plasticity, sandy lean clay (CL) material with a liquid limit of less than 40 and a plasticity index between 10 and 20. Select fill should be placed in thin lifts, not exceeding 8-in loose measure, moisture conditioned between -2% and +3% of optimum moisture content and compacted to a minimum 95% of the maximum dry density as determined by ASTM D 698. The maximum loose thickness for each fill lift will depend on the type of compaction equipment used. Recommended fill layers are summarized in the table on the following page.

Table 8-1
Compaction Equipment and Maximum Lift Thickness

Compaction Equipment	Maximum Lift Thickness
Mechanical Hand Tamper	4-in
Pneumatic Tired Roller	6-in
Tamping Foot Roller	8-in
Sheepsfoot Roller	8-in

Prior to any filling or backfilling operations, samples of proposed fill materials should be obtained for laboratory moisture-density testing. The tests will provide a basis for evaluation of fill compaction by in-place density testing. A qualified representative of the Geotechnical Engineer should perform sufficient in-place density tests during the filling and backfilling operations to verify that proper levels of compaction are being attained.

7.3 Foundation Construction

Grade beams should be excavated with a smooth-mouthed bucket. Loose soil remaining after excavation should be removed prior to steel placement. Any excavations should be sloped sufficiently to create internal sumps for runoff collection and removal. If surface runoff water or groundwater seepage in excess of 1-in accumulates at the bottom of a foundation excavation, it should be collected and removed.

Excavations made in the building area for construction of grade beams and floor slabs should not be allowed to remain open for extended periods. If excavations are to remain open, the use of a concrete mud mat to reduce moisture changes or other damage to the natural subgrade soils should be considered. If soft or loose soils are encountered at the design excavation level, they should be undercut to firm or dense soils and the excavation backfilled with lean concrete.

The performance of the building foundation system will be highly dependent upon the quality of construction. Thus, it is recommended that foundation construction be monitored by the Geotechnical Engineer or his qualified representative to identify the proper bearing strata and depths and to help evaluate foundation construction. TWE would be pleased to develop a plan for foundation monitoring to be incorporated in the overall quality control program.

7.4 Drilled Footing Installation

The following items are important to the successful completion of drilled footing foundations.

- All footing excavations should be observed by the Geotechnical Engineer or his representative to determine when the proper bearing stratum is encountered and to record other observations regarding pier construction.

- Footing excavations should be checked for size and depth prior to the placement of concrete. Precautions should be taken during the placement of the footing reinforcement and concrete to prevent loose excavated material from falling into the excavation.

- Drilled footings should be installed in accordance with the ***"Manual on Drilled Shafts: Construction Procedures and Design Methods"***, [U.S. Department of Transportation-Federal Highway Administration (Pub. No. FHWA-IF-99-025) and ADSC: The International Association of Foundation Drilling Contractors (Pub. No. ADSC-TL-4), August 1999] by Lymon, C. Reese and Michael W. O'Neill.

- Due to the anticipated proximity of the water table to the recommended footing depth at this site, we do not expect that seepage into drilled excavations will be significant during construction. However, we recommend that groundwater levels be verified at the time of construction. If groundwater levels are shallower than 10-ft at the time of construction, TWE should be notified and recommendations provided herein reevaluated, if necessary.

- Reinforcement steel cages placed in footing shafts should be designed to be stable during the placement of concrete.

Prompt placement of concrete in excavations as they are completed, cleaned and inspected is strongly recommended to limit deterioration of the bearing stratum. Under no circumstances should a footing be drilled that cannot be filled with concrete before the end of the work day.

8 LIMITATIONS AND DESIGN REVIEW

8.1 Limitations

This report has been prepared for the exclusive use of Chica & Associates, Inc. and their project team for specific application to the design and construction of the proposed new building at the intersection of Shreveport Avenue and 4th Street in Port Arthur, Texas. Our report has been prepared in accordance with the generally accepted geotechnical engineering practice common to the local area. No other warranty, express or implied, is made.

The analyses and recommendations contained in this report are based on the data obtained from the referenced soil borings performed within the project site. The soil borings indicate subsurface conditions only at the specific locations, times and depths penetrated. The soil borings do not necessarily reflect strata variations that may exist at other locations within the project site. The validity of our recommendations is based in part on assumptions about the stratigraphy made by the Geotechnical Engineer. Such assumptions may be confirmed only during the construction phase of the project. Our recommendations presented in this report must be reassessed if subsurface conditions during construction are different from those described in this report.

If any changes in the nature, design or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions modified or verified in writing by TWE. TWE is not responsible for any claims, damages or liability associated with interpretation or reuse of the subsurface data or engineering analyses without the expressed written authorization of TWE.

8.2 Design Review

Review of the design and construction drawings as well as the specifications should be performed by TWE before release. The review is aimed at determining if the geotechnical design and construction recommendations contained in this report have been properly interpreted. Design review is not within the authorized scope of work for this study.

8.3 Construction Monitoring

Construction surveillance is recommended and has been assumed in preparing our recommendations. These field services are required to check for changes in conditions that may result in modifications to our recommendations. The quality of the construction practices will affect performance of the project structures and should be monitored. TWE would be pleased to provide construction monitoring, testing and inspection services for the project.

8.4 Closing Remarks

We appreciate the opportunity to be of service during this phase of the project and we look forward to continuing our services during the construction phase and on future projects.

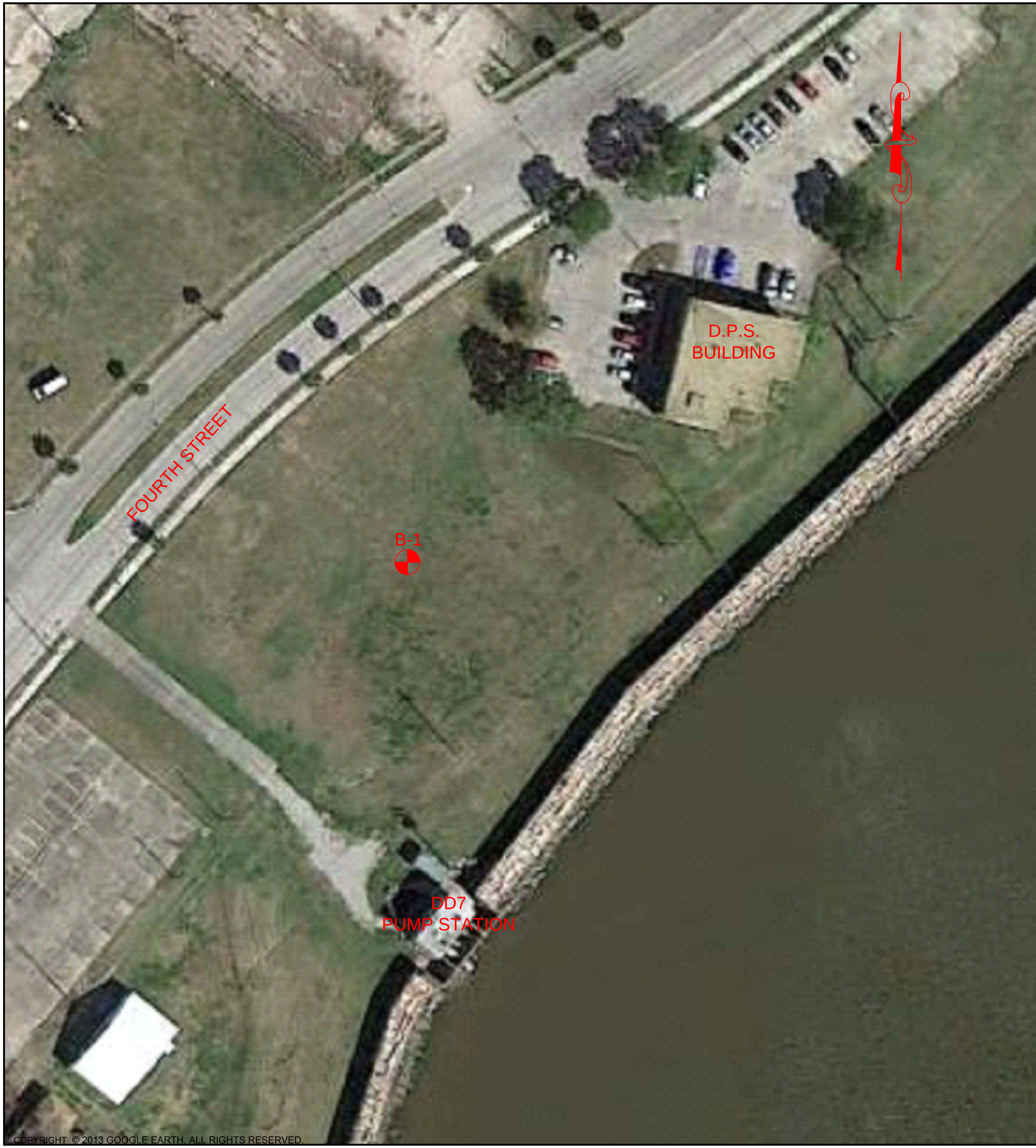
APPENDIX A

PROJECT INFORMATION
CHICA & ASSOCIATES, INC.

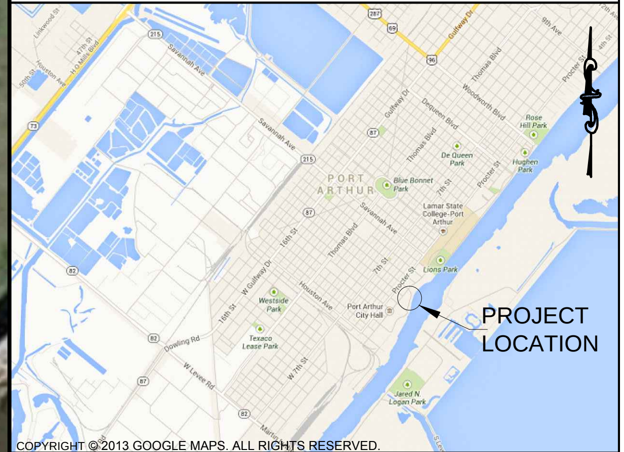
Atanta Ave.
Shreveport Ave
4th St.
EXIST. PARKING
(Approx. 72 Cars)
Prop. Sidewalks
PROPOSE 14,000 SQ FT
Prop. Bld. Location
(Not to Scale)
Jeff Co Maint. Bl
DPS Bld.
(DD7 PUMP STATION)
© 2012 Google
Google earth
feet
100
200
300
400
meters
100

APPENDIX B

SOIL BORING LOCATION PLAN
DRAWING NO. 13.23.174-1



VICINITY MAP



SOIL BORING COORDINATES

BORING	DEPTH	LATITUDE	LONGITUDE
B-1	30'	29° 52' 23.70" N	93° 55' 52.00" W

LEGEND

SOIL BORING LOCATION

Tolunay-Wong Engineers, Inc.

SOIL BORING LOCATION PLAN
PROPOSED NEW BUILDING
PORT ARTHUR, TEXAS

DRAWN BY:	M.M.	DWG. NO.	13.23.174-1
CHECKED BY:	T.G.H.	SCALE:	N.T.S.
APPROVED BY:	P.J.K.	DATE:	JULY 31, 2013

APPENDIX C

PROJECT BORING LOG B-1 AND A
KEY TO SYMBOLS AND TERMS USED ON BORING LOGS

LOG OF BORING B-1

PROJECT: Proposed New Building
Port Arthur, Texas

CLIENT: Chica & Associates, Inc.
Beaumont, Texas

COORDINATES: N 29° 52' 23.70"
W 93° 55' 52.00"

SURFACE ELEVATION: --

DRILLING METHOD:
Dry Augered: 0' to 20'
Wash Bored: 20' to 30'

ELEVATION (FT) / DEPTH (FT): 0, 5, 10, 15, 20, 25, 30, 35

MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
Stiff gray FAT CLAY (CH) -with shell and organics from 0' to 2'	(P)2.00		21	106	51	35				91	
-becomes gray and reddish brown at 2' -with ferrous nodules from 2' to 4'	(P)1.25		26	100			1.28	11	3		
-becomes firm at 4'	(P)1.25		29		71	57					
-with ferrous nodules from 6' to 10'	(P)1.00		30	93			0.90	15	6		
-with calcareous nodules from 8' to 15'	(P)1.25		31	90	73	56	0.74	13	8		
-becomes stiff at 10'	(P)1.75										
-becomes reddish brown at 13' -with sand pockets from 13' to 15'	(P)2.50										
Stiff brown and reddish brown LEAN CLAY with SAND (CL)		7/6" 8/6" 9/6"	28		28	11				81	
Stiff brown and gray FAT CLAY (CH), with sand seams	(P)2.00										
-becomes firm at 28' -with silt layers from 28' to 30'	(P)1.25										
Bottom @ 30'											

COMPLETION DEPTH: 30 ft
DATE BORING STARTED: 07/19/13
DATE BORING COMPLETED: 07/19/13
LOGGER: T. McClain
PROJECT NO.: 13.23.174

NOTES: Free water was encountered at a depth of 18-ft during dry-auger drilling and rose to a depth of 9-ft after fifteen (15) minutes. The open borehole was backfilled with cement-bentonite grout.

SYMBOLS AND TERMS USED ON BORING LOGS

Most Common Unified Soil Classifications System Symbols

Fill	Silt w/ Sand (ML)
Pavement	Well Graded Sand (SW)
Lean Clay (CL)	Well Graded Sand w/ Gravel (SW-GM)
Lean Clay w/ Sand (CL)	Poorly Graded Sand (SP)
Sandy Lean Clay (CL)	Poorly Graded Sand w/ Silt (SP-SM)
Fat Clay (CH)	Silt (ML)
Fat Clay w/ Sand (CH)	Elastic Silt (MH)
Sandy Fat Clay (CH)	Elastic Silt w/ Sand (MH-SP)
Silty Clay (CL)	Silty Gravel (GM)
Sandy Silty Clay (CL-ML)	Clayey Gravel (GC)
Silty Clayey Sand (SC-SM)	Well Graded Gravel (GW)
Clayey Sand (SC)	Well Graded Gravel w/ Sand (SP-GM)
Sandy Silt (ML)	Poorly Graded Gravel (GP)
Silty Sand (SM)	Peat

Sampler Symbols	Meaning
	Pavement core
	Thin - walled tube sample
	Standard Penetration Test (SPT)
	Auger sample
	Sampling attempt with no recovery
	TxDOT Cone Penetrometer Test

Field Test Data

2.50	Pocket penetrometer reading in tons per square foot
8/6"	Blow count per 6 - in. interval of the Standard Penetration Test
	Observed free water during drilling
	Observed static water level

Laboratory Test Data

Wc (%)	Moisture content in percent
Dens. (pcf)	Dry unit weight in pounds per cubic foot
Qu (tsf)	Unconfined compressive strength in tons per square foot
UU (tsf)	Compressive strength under confining pressure in tons per square foot
Str. (%)	Strain at failure in percent
LL	Liquid Limit in percent
PI	Plasticity Index
#200 (%)	Percent passing the No. 200 mesh sieve
( )	Confining pressure in pounds per square inch
*	Slickensided failure
**	Did not fail @ 15% strain

RELATIVE DENSITY OF COHESIONLESS & SEMI-COHESIONLESS SOILS

The following descriptive terms for relative density apply to cohesionless soils such as gravels, silty sands, and sands as well as semi-cohesive and semi-cohesionless soils such as sandy silts, and clayey sands.

Relative Density	Typical N_{60} Value Range*
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	Over 50

* N_{60} is the number of blows from a 140-lb weight having a free fall of 30-in. required to penetrate the final 12-in. of an 18-in. sample interval, corrected for field procedure to an average energy ratio of 60% (Terzaghi, Peck, and Mesri, 1996).

CONSISTENCY OF COHESIVE SOILS

The following descriptive terms for consistency apply to cohesive soils such as clays, sandy clays, and silty clays.

Pocket Penetrometer (tsf)	Typical Compressive Strength (tsf)	Consistency	Typical SPT "N_{60}" Value Range**
pp < 0.50	qu < 0.25	Very soft	≤ 2
0.50 ≤ pp < 0.75	0.25 ≤ qu < 0.50	Soft	3-4
0.75 ≤ pp < 1.50	0.50 ≤ qu < 1.00	Firm	5-8
1.50 ≤ pp < 3.00	1.00 ≤ qu < 2.00	Stiff	9-15
3.00 ≤ pp < 4.50	2.00 ≤ qu < 4.00	Very Stiff	16-30
pp ≥ 4.50	qu ≥ 4.00	Hard	≥ 31

** An "N_{60}" value of 31 or greater corresponds to a hard consistency. The correlation of consistency with a typical SPT "N_{60}" value range is approximate.

APPENDIX D

DESIGN EQUATIONS FOR ECCENTRICALLY-LOADED DRILLED-AND-UNDERREAMED FOOTINGS

P_v = Total Vertical Load

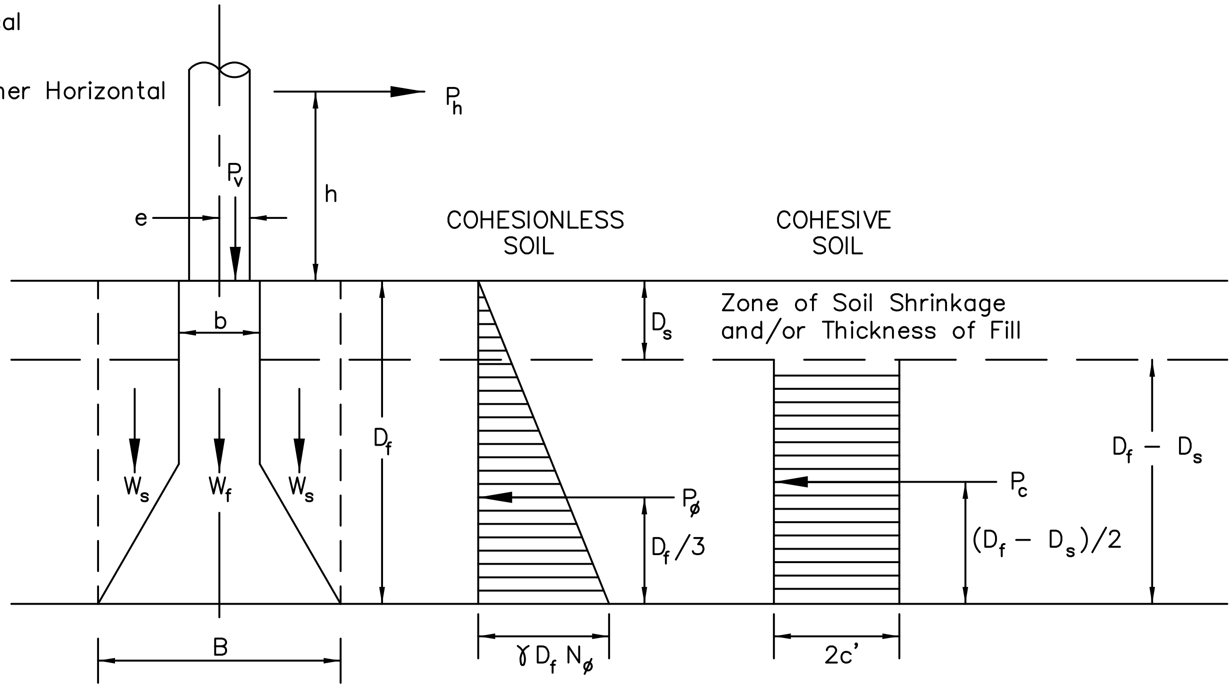
P_h = Wind or Other Horizontal Load

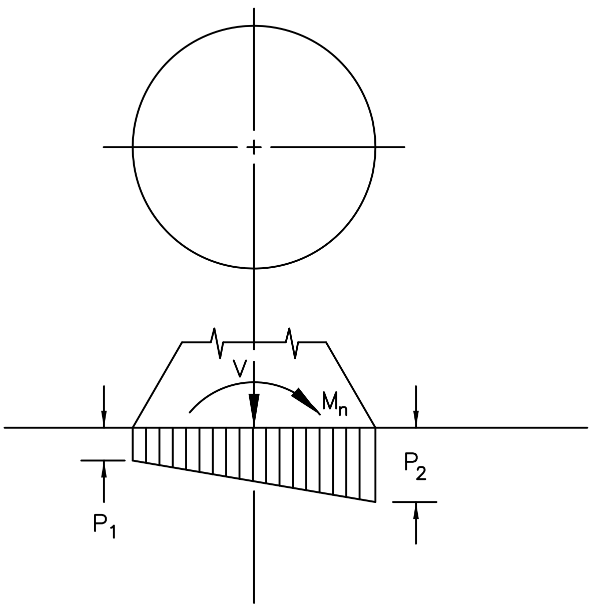
W_s = Weight of Soil

W_f = Weight of Foundation

b = Shaft Diameter

B = Base Diameter





COHESIONLESS SOIL

$$P_\phi = (\gamma/2)(D_f)^2 bN_\phi$$

γ = Effective Unit Weight of Soil

$N_\phi = \tan^2(45 + \phi/2)$

ϕ = Angle of Internal Friction of Soil

COHESIVE SOIL

$$P_c = 2c'(D_f - D_s)b$$

$c' = c/F.S.$

c = Cohesion of Soil

F.S. = Factor of Safety

AT BASE OF FOOTING:

(1) Applied Vertical Load:

$$V = P_v + W_f + W_s$$

(2) Applied Overturning Moment:

$$M_o = P_v e + P_h (h + D_f)$$

(3) Resisting Moment from Lateral Earth Pressure:

$$M_r = P_\phi (D_f/3) + P_c[(D_f - D_s)/2]$$

(4) Net Moment Resisted By Base:

$$M_n = M_o - M_r$$

(5) Soil Bearing Pressures:

$$P_1 = (4V/\pi B^2) - (32M_n/\pi B^3)$$

$$P_2 = (4V/\pi B^2) + (32M_n/\pi B^3)$$

(6) Maximum Pressure, P_2, should not exceed Allowable Gross Bearing Pressure, q_{ga}, where:

$$q_{ga} = q_{na} + \gamma Df$$

q_{na} = Allowable Net Bearing Pressure

DESIGN EQUATIONS FOR ECCENTRICALLY LOADED DRILLED AND UNDERREAMED FOOTINGS

TOLUNAY-WONG ENGINEERS, INC.