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July 22, 2013

Mr. Bert L. Bender, RA, LEED AP
Bender & Associates Architects, P.A.
410 Angela Street
Key West, Florida 33040-7402

Re: Report of Geotechnical Exploration
Glynn Archer School / City Hall Conversion
Key West, Florida

Project: #312-295
312295_00^LTR_Geotechnical Report.doc

Dear Bert:

I am writing concerning the Report of Geotechnical Exploration, Glynn Archer School / City Hall Conversion, 1300 White Street; Key West, Florida by AMEC, dated July 18, 2013 (see Appendix A). Based on additional rock cores and another geotechnical evaluation, the bearing capacity of the existing foundations and new proposed foundations at the Key West, City Hall at Glynn Archer on the Miami Limestone is 8 ksf. With this increased bearing capacity, the existing foundations are more than adequate and do not need to be underpinned. Since the Miami Limestone at this site is near the surface and is very cohesive, augercast piles will not be required at this site for the new foundations. The new foundations will be able to bear directly on the Miami Limestone.

To further clarify our Design Charrette - Structural Condition Review, Key West City Hall at Glynn Archer dated June 20, 2013, (see Appendix B), the existing perimeter concrete walls are 11 inches thick with 1 to 2 inches of hard stucco paste on the exterior of the concrete walls, leaving a concrete thickness of 9 inches as a minimum. These walls, through their history, have survived seven hurricanes with Saffir-Simpson, wind speeds of 120 mph or greater and one with a Saffir-Simpson, wind speed of 140 mph. The perimeter concrete walls are in excellent condition with little cracking and no observed spalling. At 9 inches thick, the existing walls can withstand ASCE 7-10 wind pressures generated by a 3 second gust, wind speed of 200 mph for a risk category 3, structure with a C exposure. The 9 inch thick walls were analyzed as unreinforced concrete with a minimum f'c= 1,835 psi, which is the average core compressive strength for the Building A cores performed for and presented in the CH2MHill report. The average core compressive strength for Building B is significantly higher.

It has been a pleasure serving you as a consulting structural engineer. Please contact our office if there are any questions regarding this correspondence, or if you need any additional information or assistance.

Very truly yours,
ATLANTIC ENGINEERING SERVICES OF JACKSONVILLE
FLORIDA CERTIFICATE OF AUTHORIZATION #791

Mark J. Keister, P.E.
Principal

MARK J. KEISTER
LICENSE
No. 37435
STATE OF
PROFESSIONAL ENGINEER

MJK/drg

APPENDIX A

REPORT OF GEOTECHNICAL EXPLORATION (AMEC)

GLYNN ARCHER SCHOOL
CITY HALL CONVERSION
1300 WHITE STREET
KEY WEST, FLORIDA

REPORT OF GEOTECHNICAL EXPLORATION

GLYNN ARCHER SCHOOL CITY HALL CONVERSION
1300 WHITE STREET
KEY WEST, FLORIDA

- PREPARED FOR -

BENDER AND ASSOCIATES, ARCHITECTS, P.A.

- Prepared By -

AMEC ENVIRONMENT & INFRASTRUCTURE, INC.
3901 CARMICHAEL AVENUE
JACKSONVILLE, FLORIDA 32207

AMEC PROJECT NO. 6734-13-9720

IMPORTANT NOTICE

July 18, 2013

Mr. Bert L. Bender, Architect, LEED AP
Bender and Associates, Architects, P.A.
1300 White Street
Key West, Florida 33040

Subject: **Report of Geotechnical Evaluation**
Glynn Archer School City Hall Conversion
1300 White Street
Key West, Florida
AMEC Project No. 6734-13-9720

Dear Mr. Bender:

AMEC Environment and Infrastructure, Inc. (AMEC), has performed a geotechnical exploration for the subject project in general accordance with our Revised Proposal No. 13PROPJAXV, Task 058, Rev. 1, dated March 4, 2013. You provided authorization for our services on April 24, 2013. This report supersedes our draft report issued on July 12, 2013.

In summary, the subsurface conditions in the area of the existing building to a depth of at least 20 feet consisted of about 1 to 1½ feet of fine sand over the Miami Limestone formation. The Miami Limestone consists of soft to medium, tan-white porous oolitic limestone. The groundwater table was encountered at a depth of about 4 feet below the existing ground surface, and is tidally influenced.

A total of eight unconfined compression tests were performed in the laboratory to estimate the unconfined compressive strength and elastic modulus of intact core samples so that the bearing capacity and settlement potential of shallow foundations could be estimated. The results of this analysis indicate that an allowable bearing pressure of up to 8 ksf may be used to design shallow foundations bearing within the upper few feet of the Miami Limestone.

We have enjoyed assisting you and look forward to serving as your geotechnical and construction materials testing consultant on the remainder of this project and on future projects. If you have any questions concerning this report, please contact us.

Sincerely,

AMEC ENVIRONMENT AND INFRASTRUCTURE, INC.
Florida Board of Professional Engineers Certificate of Authorization No. 5392

Kirk A. McIntosh

Kirk A. McIntosh, P.E., D.GE.
Senior Principal
Florida License No. 33703

MICHAEL B. WOODWARD LICENSE No. 42814 ★ 7/18/13 ★ STATE OF FLORIDA PROFESSIONAL ENGINEER

MBWoodward

Michael B. Woodward, P.E.
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Distribution: Addressee (email)
Atlantic Engineering Services (email)
File (1)

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Site Location Map
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Soil Test Boring Records
Double Ring Infiltrometer Test Results
Field Percolation Test Results
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Grain Size Distribution Results
Field and Laboratory Procedures
Key to Symbols and Descriptions
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1.0 PROJECT INFORMATION AND STRUCTURAL CONDITIONS

The purpose of this evaluation was to determine the strength and compressibility characteristics of the Miami Limestone in order to evaluate the bearing capacity of the existing near-surface limestone formation relative to shallow foundations. This report briefly describes the field and laboratory testing activities and presents the findings.

Project information was provided by you, by Mr. Allen Perez of Perez Engineering & Development, Inc., and by Mr. Mark Keister of Atlantic Engineering Services (AES) during the period of February 10 to July12, 2013. We were provided with the following documents:

- Report of Geotechnical Exploration, Concrete Core Testing, and Foundation Excavations
  Glynn Archer School Building
  1300 White Street
  Key West, Florida
  Prepared by: Nutting Engineers of Florida, Inc.
  Dated: August 10, 2012

- Schematic Site Plan, First Floor Plan, and Second Floor Plan
  Glynn Archer School
  Key West, Florida
  Prepared by: Bender and Associates, Architects, P.A.
  Dated: August 23 to September 29, 2010

- Design Charette-Structural Condition Review
  Key West City Hall at Glynn Archer
  Key West, Florida
  Prepared by: Atlantic Engineering Services
  Report Dated: June 20, 2013

As shown on the Site Location Map in the Appendix, the existing school building is located within the area bounded by United Street to the north, White Street to the east, Seminary Street to the south, and Grinnell Street to the west, in Key West, Florida.

The existing Glynn Archer School was constructed in two phases in 1923 and 1927, and will be converted into a City Hall building for the City of Key West, Florida. The project will involve the removal of existing wood-framed interior walls, and replacement with steel framing; therefore, some existing perimeter wall footings will be widened to support the additional loading. We understand that existing continuous wall footings are 36 to 42 inches wide, and that existing column footings are 32 inches square. Existing shallow foundations bear at least 2½ feet below ground surface. We understand that the existing shallow foundations have performed satisfactorily.

Based on a conversation with Mr. Keister, we understand that existing perimeter wall footings are currently supporting a bearing wall load of about 9 klf. The anticipated new wall loading will be on the order of 12 klf. The perimeter wall footings will be widened to support the additional load imposed by hollow-core floor slabs. In the interior of the existing building, new spread footings will be constructed to support new columns. The new columns will support loads of up to 210 kips, and will bear about 3 feet below existing grade.

The furnished Nutting Engineers report included the results of four exploratory borings drilled to a depth of 25 feet each. The borings encountered oolitic limestone (known as the Miami Limestone formation) at a depth range of about 1½ to 25 feet below grade. In their report, Nutting provided an allowable bearing pressure of at least 4,000 psf for footings bearing on the Miami Limestone.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

In order to obtain samples of the existing limestone formation for laboratory strength testing, four core/Standard Penetration Test (SPT) borings were drilled to a depth of 20 feet each around the perimeter of the existing building. The borings were initiated by auger drilling to the top of the limestone formation, which was typically encountered at a depth range of about 1 to 1½ feet. Upon reaching the Miami Limestone, a 4-inch diameter core barrel was used to core approximately 10 feet of the limestone—typically in two, 5-foot long core runs. After 10 feet of coring was completed, the remainder of each boring was performed using rotary wash drilling along with standard penetration testing. An extra SPT sample was obtained at a depth range of 11 to 12.5 feet in each boring. All standard penetration tests were performed using an automatic hammer having a calibrated efficiency of about 87 percent.

In addition to the core/SPT borings, one constant-head, open-hole hydraulic conductivity test (designated P-1) was performed to a depth of 9.5 feet below ground surface. A section of 6-inch diameter PVC pipe was installed into an 8-inch diameter borehole in order to perform this test. A double-ring infiltrometer test (designated DRI-1) was performed at a depth of 4 inches below grade.

The approximate boring and drainage test locations are shown on the Field Exploration Plan in the Appendix. The borings were drilled by Independent Drilling, Inc. (IDI) working under subcontract to AMEC. A geologist from AMEC observed and documented the drilling operations on a full-time basis, and performed the drainage tests with assistance from IDI. The boring locations were selected by a geotechnical engineer from our office and were established in the field by our personnel using a measuring tape, and should be considered approximate. The drainage test locations were determined by Mr. Perez.

The Soil Test Boring Records, in the Appendix, graphically show the penetration resistances and groundwater levels, and present the soil and rock descriptions for each SPT boring. The stratification lines and depth designations on the boring records represent the approximate boundaries between soil and rock types. In some instances, the transition between soil and rock types may be gradual. The results of the drainage tests are presented on the Double-Ring Infiltrometer Test Results sheet and the Field Percolation Test Results sheet in the Appendix. Brief descriptions of the exploratory drilling, testing, and sampling techniques used are presented in the Field and Laboratory Procedures section of the Appendix. Photographs of the drilling operations and the soil/rock samples obtained are presented in the Appendix for each boring drilled.

2.2 Laboratory Testing

In order to determine the unconfined compressive strength of the upper Miami Limestone, eight unconfined compression tests (two samples per boring) were performed on intact core samples of the limestone obtained from the borings. Deformation readings were obtained during each test using a dial gauge so that the elastic modulus of each tested sample could be estimated. In addition, one grain size distribution test was performed on a sample of surface sand obtained at the double-ring infiltrometer test location.

The results of the laboratory tests are presented on the Grain Size Distribution sheet and the Unconfined Compression Test results sheets in the Appendix. Brief descriptions of the laboratory test procedures used are presented in the Field and Laboratory Procedures section in the Appendix.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Conditions

The existing site conditions were observed by a senior geologist from AMEC during the drilling operations, which occurred on June 11 and 12, 2013. In general, the site consisted of a developed school site with maintained grass, asphaltic concrete parking areas, landscaping trees and shrubbery, concrete sidewalks, and trees. The topography encountered was relatively flat and level. Standing surface water was not observed on the property at the time of our visit. The existing building is one to two stories in height and has a stucco exterior. The existing building is understood to be supported on shallow footings. Based on observations of the exterior of the existing building, the existing foundation system appears to be performing satisfactorily.

3.2 Subsurface Conditions

3.2.1 General

The subsurface conditions outlined below highlight the major subsurface stratification. The Soil Test Boring Records, in the Appendix, should be consulted for detailed descriptions of the subsurface conditions encountered at each boring location. When reviewing the boring records, it should be understood that soil and rock conditions may vary between and away from the boring locations.

3.2.2 General Area Geology

The Florida Keys consist of a chain of small islands that extend from Miami to Key West over a total distance of about 150 miles. The average ground surface elevation of the Keys is about +3 feet (MSL), with a maximum elevation of about +17.5 feet (MSL). Key West is comprised mainly of Miami Limestone, a relatively young Pleistocene age formation of about 130,000 years in age. The Miami Limestone in Key West is generally a soft rock, with streaks or thin layers of calcite, and may contain vertical solution pits or holes that were produced by the dissolution of limestone by underground water. The Miami Limestone in Key West is comprised of oolitic and bryozoan limestones, and extends to an elevation of about -20 feet (MSL).

Beneath the Miami Limestone in Key West lies the Key Largo Limestone, which is about 100,000+ years old and consists mainly of coralline reef rock. Other lithologic types include calcarenite and calcilutite. The Key Largo Formation is believed to be about 175 feet thick in Key West.

3.2.3 Soils /Rocks

From the existing ground surface to depths of about 1 to 1-1/2 feet, brown fine sands (SP) were encountered. Beneath the fine sands, and extending to the maximum depth drilled at 20 feet, the Miami Limestone formation was encountered. The Miami Limestone can be classified as a soft to medium, tan-white porous oolitic limestone. In the portion of the borings where the Miami Limestone was sampled at a depth range of about 10.3 to 13.3 feet below grade using standard penetration testing, SPT N-values ranged from 39 to 47 blows/foot using an automatic SPT hammer, with an average N-value of 42 blows/foot. Below depths of about 12 to 13 feet, SPT N-values ranged from about 14 to 38 blows/foot, with an average of about 21 blows/foot. It can be seen that there is a reduction in limestone cementation or an increase in porosity below a depth of about 12 to 13 feet.

Four-inch diameter core recoveries in the upper 10 to 12 feet of the subsurface profile in the Miami Limestone ranged from 68 to 100 percent, with an average recovery of 87 percent. The rock quality designation (RQD) ranged from 43 to 96 percent, with an average RQD value of about 66 percent.

3.2.4 Groundwater

The depth to the groundwater table was measured at some of the boring locations at the time of drilling. The groundwater level was encountered at a depth of about 4 feet below existing grade. Fluctuation in groundwater levels should be expected due to seasonal climatic changes, construction activity, rainfall variations, tidal fluctuations in the nearby ocean and gulf, surface water runoff, and other site-specific factors. Since groundwater level variations are anticipated, design drawings and specifications should accommodate such possibilities and construction planning should be based on the assumption that variations will occur.

4.0 EVALUATION

4.1 Miami Limestone Strength and Compressibility Parameters

In order to estimate the ultimate bearing capacity of the Miami Limestone, it was first necessary to determine the compressive strength and elastic modulus of the limestone formation. The unconfined compressive strength of the limestone was determined by performing laboratory unconfined compression tests. The unconfined compressive strength (UCS) of intact core samples ranged from 14.9 to 59.3 ksf. The average UCS of the rock in Borings AB-1 and AB-2 was 20.5 ksf. The average UCS of intact core samples from Borings AB-3 and AB-4 was 39.9 ksf. The strength data obtained from Borings AB-1 and AB-2 were used for all bearing capacity and settlement calculations for conservatism.

The elastic modulus of the limestone formation was estimated from the laboratory rock core unconfined compression test results. A stress-strain curve was prepared for each unconfined compression test performed. The slope of a tangent line drawn to the stress-strain curve at 50 percent of the ultimate strength was selected as the elastic modulus (E_{50}) of the intact core sample. The E_{50} values ranged from 10,400 to 57,000 ksf, with an adjusted average value of 12,700 ksf from Borings AB-1 and AB-2.

The rock core unconfined compression tests provided an estimate of the intact rock elastic modulus. The in-place rock mass elastic modulus will be lower than the modulus determined from intact core samples because the in-place rock mass includes more cavities and voids (i.e., porosity) and other planes of weakness than the small diameter cores. In order to estimate the in-place elastic modulus of the rock mass, an empirical equation developed by Deere, Merritt, and Coon (1969) was utilized:

$$E_d = [(0.0231)\ (RQD) - 1.32\ ] \times E_{50}$$

Where: E_d = in-situ modulus of deformation
RQD = rock quality designation (in %)
E_{50} = laboratory tangent modulus at 50% of the UCS

Using the average rock core RQD value of 65 percent, and an average rock core E_{50} value of 12,700 ksf, we calculated an in-situ elastic modulus of the rock mass (E_d) equal to 2,280 ksf. The E_d value of 2,280 ksf was used for all footing settlement computations.

4.2 Miami Limestone Bearing Capacity Evaluation

In order to estimate the ultimate bearing capacity of existing and proposed footings bearing within Miami Limestone, the following document was utilized:

- Engineering Policy Guidelines for Design of Spread Footings (EPG 751.38)
  Prepared by: Univ. of Missouri and Missouri Univ. of Science and Technology
  Dated: October, 2011

Section 751.38.3.2- Bearing Resistance for Spread Footings on Weak Rock (5 ksf < q_u < 100 ksf), was utilized to calculate the ultimate bearing capacity of footings bearing on weak limestone. The following equation was used:

$$q_{ult} = \frac{\overline{q_u}}{2} \text{ x } N_c \text{ x } S_c \text{ x } d_c \text{ x } i_c \leq 200 \text{ ksf}$$

Where:
q_{ult} = ultimate bearing capacity of the weak limestone
N_c = bearing capacity factor = 5.0
S_c = correction factor to account for footing shape
d_c = correction factor to account for footing depth
i_c = correction factor to account for inclination of the factored load = 1.0 for column loads with no deviation from the vertical.
$\overline{q_u}$ = mean value of the uniaxial compressive strength of the rock from lab tests.

In our calculation, we used a $\overline{q_u}$ value of 20 ksf, an N_c value of 5, S_c values of 1.01 (continuous footings) to 1.20 (column spread footings), d_c values of 1.09 to 1.2 for column spread footings and 1.20 to 1.24 for continuous footings, and an i_c value of 1.0. A footing depth-of-embedment (D_f) of 3 feet was assumed, along with arbitrary footing widths of 2.5 to 3 feet for continuous footings, and 3 to 7 feet for individual column spread footings. Our calculated values of ultimate bearing capacity varied from about 60 ksf for continuous footings to about 67 ksf for individual column spread footings. An alternate method was also used for individual column footings (side width of 4 feet assumed), which yielded a calculated ultimate bearing capacity of about 75 ksf, similar in magnitude to the EPG Method value.

In order to analyze the limestone layer or "mat" for its ability to support shallow foundations, four additional failure modes were considered:

1. Punching failure of the limestone mat.
2. Local crushing at the contact of the shallow foundation and the limestone.
3. Beam tension failure of the limestone mat.
4. Settlement of the limestone mat and any underlying soils.

Punching shear failure of the limestone was determined to be unlikely to occur due to the relatively large H/B ratio, where H is the rock thickness below the footing and B is the footing width, and due to the presence of rock in Key West from about existing grade to a depth of at least 200 feet (Miami and Key Largo Limestone formations).

Beam tension failure occurs when the H/B ratio is large, and the flexural strength of the limestone mat is small. It is more likely to occur when loose sand or soft soil is present immediately beneath the limestone mat, allowing bending of the limestone mat to occur when loaded. Again, because significant additional rock thickness is anticipated below the boring depth of 20 feet, and because no known documented cases of this type of failure have occurred in South Florida, this failure mode is unlikely.

Local crushing can occur when the <u>confined</u> compressive strength of the limestone is exceeded by the footing contact stress on the rock. Our experience indicates that this failure mechanism can occur with small footings when the contact stress is approximately two times the <u>unconfined</u> compressive strength of the rock. Therefore, a minimum factor-of-safety (FS) against a local crushing type failure of about two will be available as long as the applied rock contact stress does not exceed the unconfined compressive strength of the rock. As stated previously, the minimum rock core unconfined compressive strength obtained for this project was about 15 ksf, with an overall average of 20.5 ksf for samples from Borings AB-1 and AB-2. The average UCS from Borings AB-3 and AB-4 was about 40 ksf.

The above evaluation indicates that allowable bearing pressures for footings bearing within the Miami Limestone on the order of 6 to 8 ksf should provide a FS against failure on the order of 3.5 to 5, when considering local crushing. Much higher safety factors would apply to general shear failure, as discussed earlier in this section. It is important to note that, because laboratory unconfined compression tests require intact samples having a minimum L/D ratio of two, only the best quality samples can be tested in the laboratory. This should be taken into consideration when safety factors are determined.

4.3 Footing Settlement Evaluation

Settlements of proposed square footings bearing on limestone and proportioned for allowable bearing pressures of 6 to 8 ksf were estimated using the following published equation:

$$S = \frac{1.12\, q\, B\, (1 - \mu^2)(\frac{L}{B})^{0.5}}{E_d}$$

Where:
- $S =$ settlement at center of column footing
- $q =$ applied rock contact stress
- $B =$ footing width
- $\mu =$ Poisson's ratio = 0.25
- $L =$ footing length
- $E_d =$ in-situ modulus of deformation = 2280 ksf

The above equation applies to "flexible" footings. Using applied rock contact stresses of 6 to 8 ksf and arbitrary spread footing widths of 4 to 7 feet, calculated footing settlements were on the order of 0.2 to 0.3 inch. These footing settlements will occur simultaneously with application of the structural dead load during construction.

5.0 Recommendations

The following recommendations are based upon the previously presented project information and structural conditions along with the data obtained in this evaluation. The field and laboratory data have been compared with previous performances of structures bearing in subsurface conditions similar to those encountered at this site. If the structural information is incorrect, please contact us so that our recommendations can be reviewed. The discovery of any site and/or subsurface condition during construction which deviates from the data obtained in this exploration should also be reported to us for our evaluation.

5.1 Shallow Foundation Design and Construction

5.1.1 Shallow Foundation Design

We anticipate that a series of spread and/or continuous footings will be utilized to support the new structural loads. The footings may be designed using an allowable bearing pressure of up to 8,000 psf for footings bearing at least one foot into the Miami Limestone. The allowable bearing pressure may be increased by about one-third for transient edge stress loading considerations. Long-term edge stresses should not exceed the allowable bearing pressure.

Footings should bear at least 3 feet below the finished exterior grade to generate the 8,000 psf allowable bearing pressure. We recommend that footings be constructed without forming, where possible. An allowable bearing pressure of 4,000 psf may be utilized for shallow foundations bearing at a minimum embedment depth of 18 inches in compacted fill or backfill soils. Minimum footing widths of 24 inches for individual footings and 18 inches for continuous footings are recommended, even though the allowable bearing pressure may not be fully developed in all cases.

5.1.2 Shallow Foundation Construction

The Miami Limestone surface may be pinnacled in some areas of the site. In order to generate the recommended maximum allowable bearing pressure, all footing excavations should extend into undisturbed natural limestone. If the natural limestone surface is below the planned footing bearing level, the excavation should be backfilled to the planned footing elevation using compacted structural fill material or lean concrete (2,000 psi). Excavations made into intact limestone may be made vertically. In sandy zones, excavations should be sloped no steeper than 1½:1 (H:V) or should be properly braced or shored. The footings should be unformed and cast directly against the limestone walls. Unformed footings cast directly against the limestone will provide a better bond and increased lateral load and uplift resistance.

Construction of footings in the Miami Limestone will generally require the following preparatory work:

1. The surficial overburden sand and/or fill should be removed in order to bear footings into the Miami Limestone.
2. The nature of the limestone may prevent the removal of all sand from the excavations. Loose sands and rock fragments should be removed by hand, such that the sides of the excavation are rough and less than one inch of sand remains in the bottom of the footing excavation prior to concrete placement.
3. Any natural pits exposed at the footing bearing elevations should be thoroughly cleaned, removing loose sand, clayey sand, and limestone fragments. The cleaning of the pits should in general extend from the bearing surface to a depth of at least three times the pit diameter. After the pits have been cleaned, they should be filled with lean concrete (1,000 psi) to restore the limestone below the footing bearing level to a relatively uniform competency and grade.

Our experience with the excavation of Miami Limestone indicates that footing excavations can probably be made using a medium to heavy track-mounted backhoe. Based on the Nutting report, some material having a standard penetration value (N, blows per foot) in excess of 100 exists within the planned excavation depths; therefore, some variability in the excavateability of the rock should be anticipated.

We anticipate that, because of the nature of the limestone, some sand pockets between limestone pinnacles may be encountered in the footing excavations. Because of the anticipated pinnacled nature of the limestone and the need to bear the footings in limestone, each footing excavation should be observed by a geotechnical engineer from AMEC in order to ascertain whether the intended bearing strata has been reached and whether its condition is acceptable. Based on the observation of each individual footing, the geotechnical engineer would then recommend whether any additional excavation, cleaning, or compaction was needed.

5.2 Groundwater Control

The need for significant groundwater control is not anticipated for footing construction. If required due to heavy rainfall conditions or other climatic conditions, groundwater in permeable materials can probably be controlled by pumping from sumps located in perimeter ditches or pits. All sump pump inlets should be located outside the bearing areas to avoid loosening of the bearing materials. The sump should be dug a few feet deeper than the intended depressed water level. The groundwater level should be maintained at least one foot below the bottom of any excavations made during construction and two feet below the surface of any vibratory compaction operations.

5.3 General Site Preparation and Earthwork

Initially all vegetation, topsoils and any other deleterious materials should be stripped and removed from the new foundation areas. The depth to which stripping will be required will vary to some degree; however, we anticipate that this depth would be on the order of 6 inches, or less.

If required, we recommend that structural fill or backfill soils consist of either crushed limestone or an inorganic, granular material with less than 5 percent passing the No. 200 mesh sieve. Any crushed limestone structural fill material should meet the following material and placement criteria:

1. The fill should be an inorganic, non-plastic, granular mixture of locally available limerock and sand from GAEC-approved pits for limerock base and fill material.

2. At least 97 percent (by weight) of the material should pass a 3-inch sieve and the material should be well graded down to dust. The fine material (portion passing the No. 200 mesh sieve) should consist entirely of dust from fracturing only and not exceed 5 percent by weight.

3. Crushing which might be necessary in order for the fill material to meet the above gradation requirements may be done before or after the material is placed.

4. Each lift should be placed with a loose lift thickness not exceeding 12 inches and compacted with appropriate equipment to at least 98 percent of the Modified Proctor maximum dry density (ASTM D-1557). We recommend that at least a 5 ton (static at-drum weight) vibratory roller be used for the compaction work, except where working within 10 feet of walls or in confined areas where a vibratory sled or small vibratory drum roller is recommended. Also, if the contractor forms the footing and then backfills to grade, a small vibratory sled or rammer should be used to densify the fill around the footing to at least 98 percent of the Modified Proctor maximum dry density.

5. The mixture of limerock should have a minimum Limerock Bearing Ratio (LBR) of 100.

We recommend that, prior to initiating compaction operation, representative samples of the fill material to be used be collected and tested to determine its compaction and classification characteristics. The maximum dry density, optimum moisture content, gradation and plasticity characteristics should be determined. These tests are needed for compaction quality control of the compacted fill and existing soils and to determine if the fill material is acceptable.

The moisture content of the fill soils should be within two percent above and four percent below the optimum moisture content based on the Modified Proctor maximum dry density test (ASTM D-1557). In confined areas, the loose lift thickness of the backfill should be reduced to 4 inches to facilitate compaction with smaller, walk-behind equipment.

A representative number of field density tests should be made in the compacted fill soils in order to confirm that the specified degree of compaction has been achieved. As a general recommendation, one field density test should be made in each lift of compacted fill soil or exposed acceptable existing soil per 3,000 square feet of area. In addition, at least one field density test per 20-foot length (or 100 square feet of area) of compacted structural fill placed in footing excavations should be performed. Testing and compaction monitoring of the fill soils by the geotechnical engineer are essential in confirming satisfactory placement procedures.

5.4 Construction Plans and Specifications Review

We recommend that this office be provided the opportunity to make a general review of the foundation and earthwork plans and specifications prepared from the recommendations presented in this report. We would then suggest any modifications such that our recommendations are properly interpreted and implemented. Our report has been written in a guideline recommendation format and is not appropriate for use as a specification without in-part being reworded into a specification-type format. We recommend that this report not be made a part of the contract documents; however, it should be made available to prospective contractors for information purposes.

The evaluation of conditions which may be encountered in construction requires engineering judgment and interpretation. For this reason, we recommend that AMEC remain involved with this project during the construction process, particularly during foundation construction. If we are not retained during construction, we cannot assume responsibility for misinterpretation of our recommendations, or for unfavorable foundation or floor slab performance as a result of judgments rendered by others.

APPENDIX

REFERENCE:
KEY WEST QUADRANGLE; FLORIDA
TOPOGRAPHIC MAP
DATED: 2012
U.S. GEOLOGICAL SURVEY

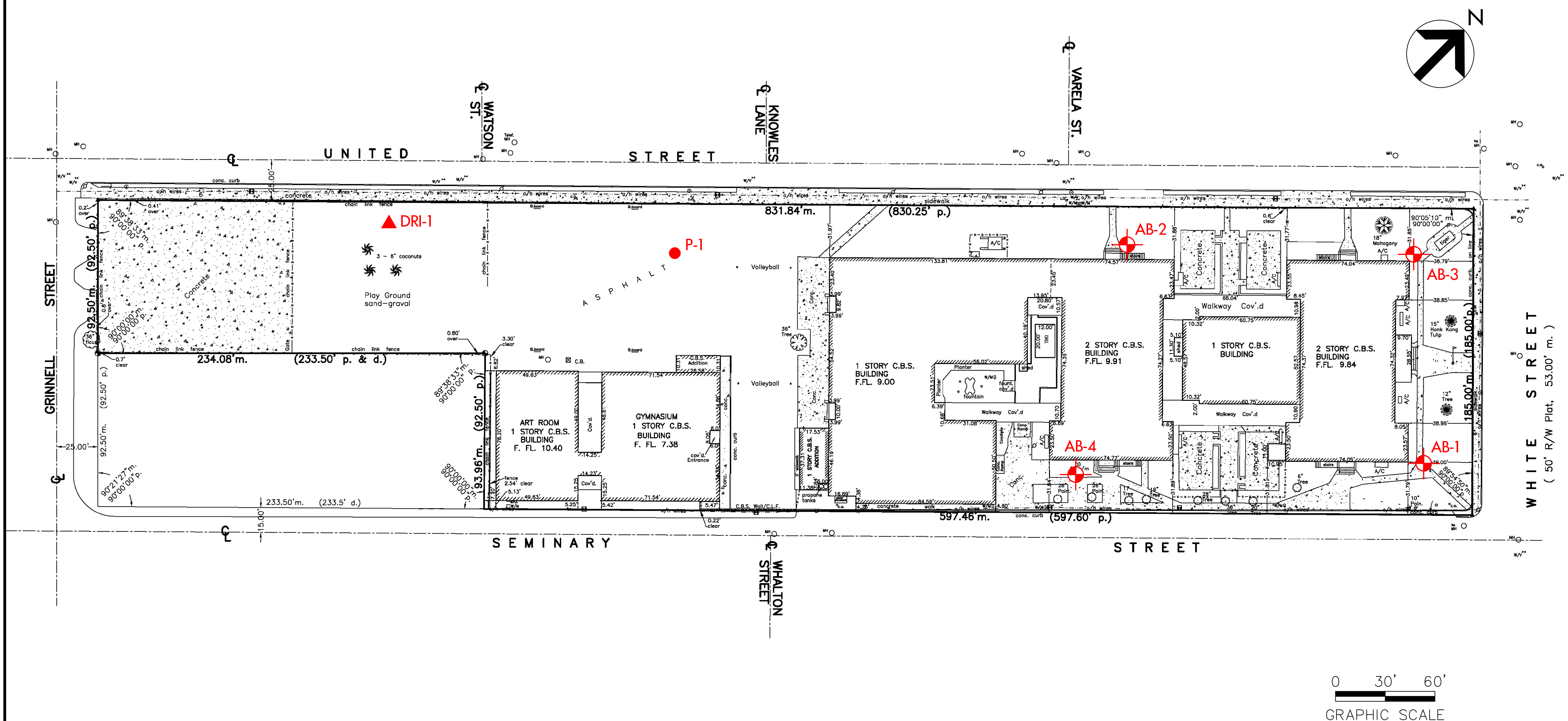
0 1000' 2000'
GRAPHIC SCALE

amec

3901 CARMICHAEL AVENUE
JACKSONVILLE, FL 32207
(904) 396-5173

SITE LOCATION MAP
Glynn Archer School City Hall Conversion
1300 White Street
Key West, Florida

DRAWN: JP	DATE: 7/11/13	SCALE: 1"=2000'
CHECKED: KAM	PROJ. NO. 6734-13-9720	APPROX.



LEGEND

- CORE/SPT BORING LOCATION (AMEC)
- OPEN HOLE PERCOLATION TEST LOCATION
- DOUBLE-RING INFILTROMETER TEST LOCATION

REFERENCE:
1) Boundary Survey (Dwn No. 98-201)
Prepared by: Frederick H. Hildebrandt
Dated: June 8, 1998

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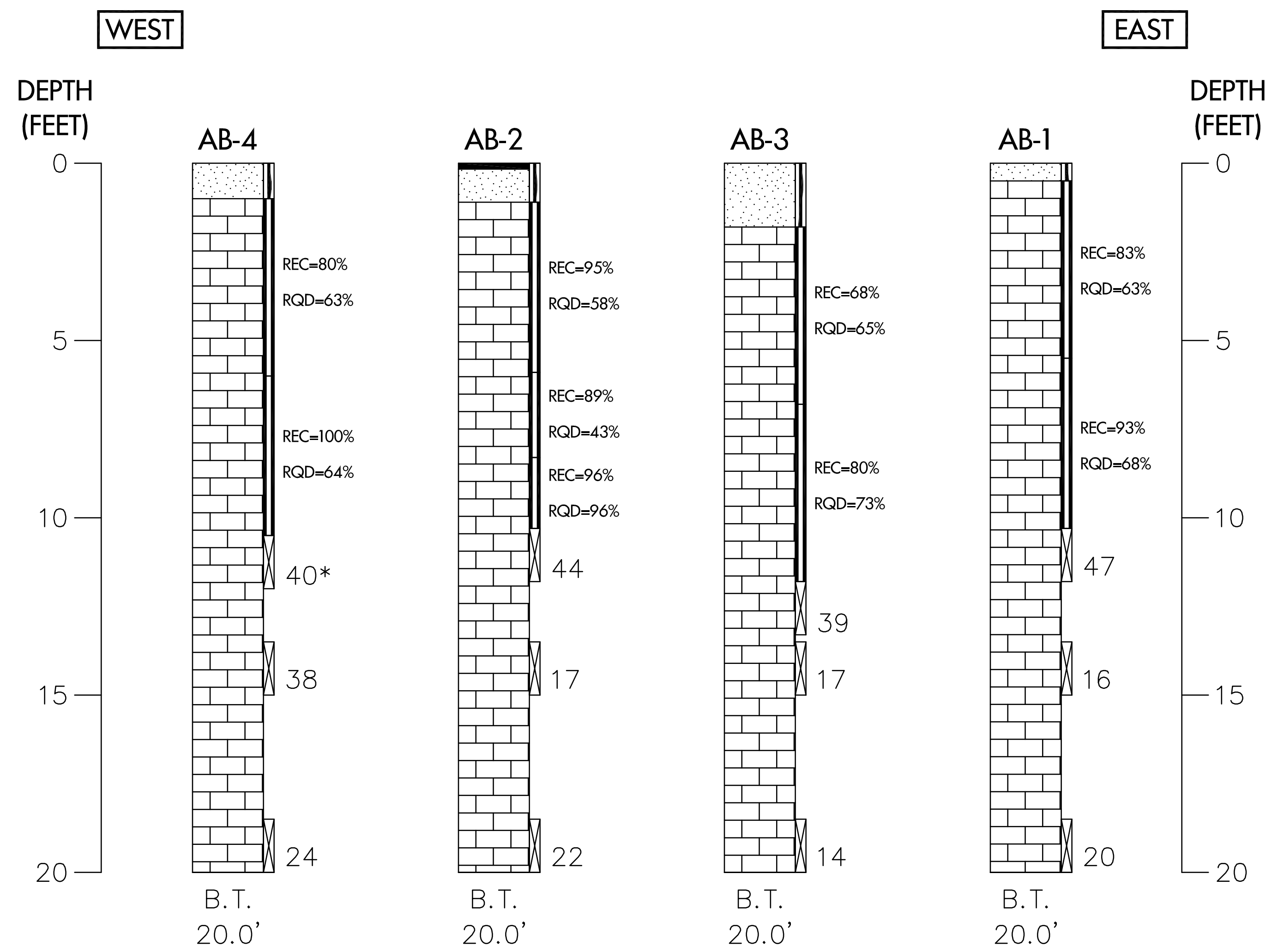
3901 CARMICHAEL AVENUE
JACKSONVILLE, FL 32207
(904) 396-5173

FIELD EXPLORATION PLAN

Glynn Archer School City Hall Conversion
1300 White Street
Key West, Florida

DRAWN: JP	DATE: 7/11/13	SCALE: 1"=60'
CHECKED: KAM	PROJ. NO. 6734-13-9720	APPROX.



LEGEND

Asphalt

Fine SAND (SP) to Slightly Silty Fine SAND (SP–SM)

Miami LIMESTONE

Augered

Standard Penetration Test Sample

* Standard Penetration Resistance (Blows/ft.) Measured Using an Automatic Hammer System (Efficiency = 87%)

B.T. Boring Terminated

20.0' Depth Terminated

Rock Core (4" Diameter)

REC Percent Recovery

RQD Rock Quality Designation

NOTE: Please refer to text of report for additional information relative to groundwater conditions and potential fluctuations which could occur.

amec

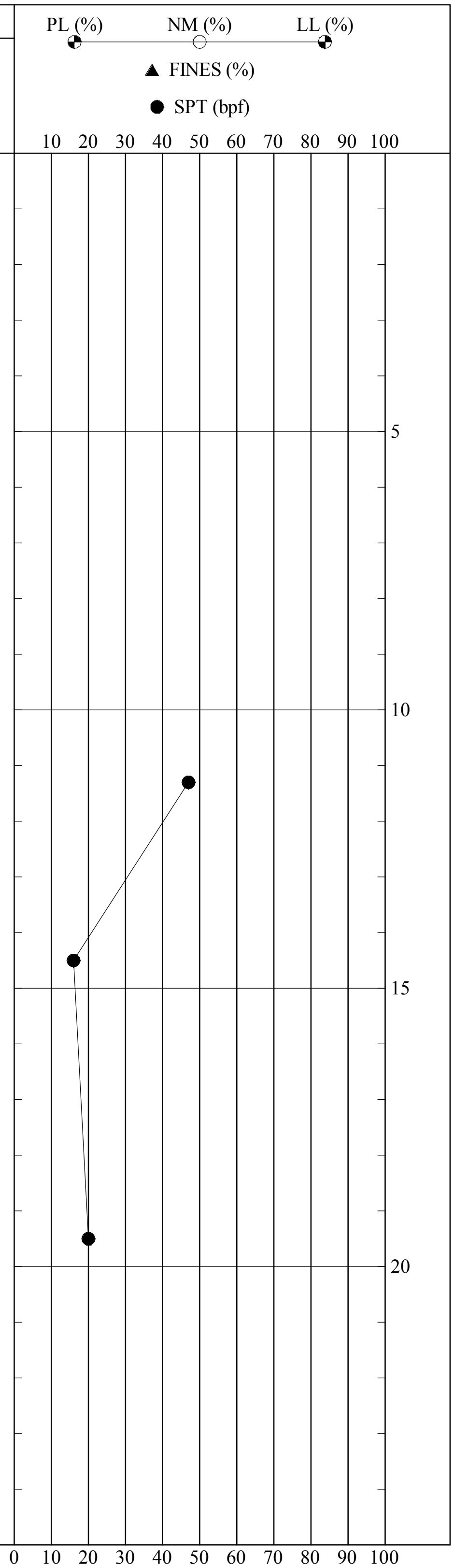
3901 CARMICHAEL AVENUE
JACKSONVILLE, FL 32207
(904) 396-5173

GENERALIZED SUBSURFACE PROFILE

Glynn Archer School City Hall Conversion
1300 White Street
Key West, Florida

DRAWN: JP	DATE: 7/11/13	SCALE: AS SHOWN
CHECKED: KAM	PROJ. NO. 6734-13-9720	

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS (SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.)	ELEV (ft)	SAMPLES IDENT	N-COUNT (1st 6" / 2nd 6" / 3rd 6")
0	SAND, mostly fine-grained ooids with trace silt, little coarse to fine gravel, moist, strong HCl reaction, dark grayish brown (SP)	0.0	AU-1	
	LIMESTONE, oolitic limestone, soft to medium, with few soft layers of poorly cemented ooids, slightly weathered, tan (LS)		RC-1	REC=83% RQD=63%
5		-5.0		
	LIMESTONE, oolitic limestone, soft to medium, with few soft layers of poorly cemented ooids, slightly weathered, tan (LS)		RC-2	REC=93% RQD=68%
10		-10.0		
	LIMESTONE, oolitic limestone, soft to medium, wet, strong HCl reaction, tan, (2" thick layer of lime mud at bottom of sample) (LS)		SPT-1	14-18-29
	LIMESTONE, oolitic limestone, very soft, wet, strong HCl reaction, tan (LS)		SPT-2	9-8-8
15		-15.0		
	LIMESTONE, oolitic limestone, very soft to soft, wet, strong HCl reaction, tan (LS)		SPT-3	8-9-11
20	BORING TERMINATED	-20.0		
25		-25.0		



CONTRACTOR: Independent Drilling, Inc.
DRILLER: J. Wilkerson
EQUIPMENT: CME 45B (DR-8, Eff. 87%) - Auto. Hammer
METHOD: Auger/Mud Rotary/4" Dia. Rock Core
HOLE DIA.: 4"
REMARKS: Groundwater table estimated at 4 feet below grade

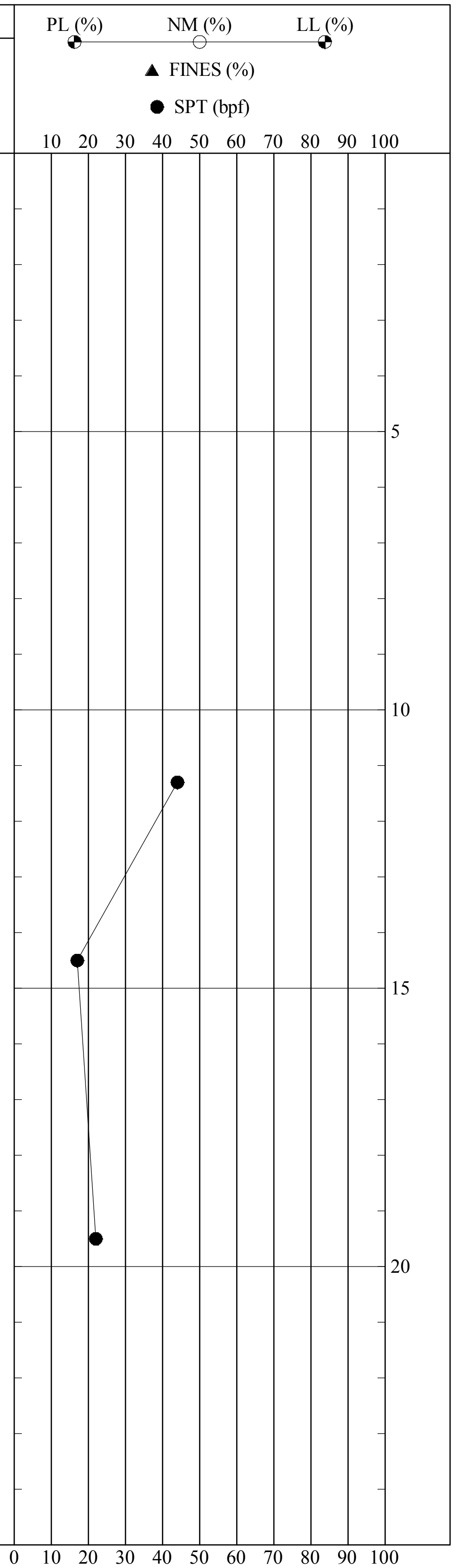
THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

Project: Glynn Archer School City Hall Conversion
Coord N:
Coord E:
Drilled: June 11, 2013
Proj. No.: 6734-13-9720
Boring No.: AB-1
Checked By: KAM

amec

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS (SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.)	ELEV (ft)	SAMPLES IDENT	N-COUNT (1st 6" / 2nd 6" / 3rd 6")
0	Asphalt (2" thick)	0.0		
	SAND, mostly fine-grained ooids with trace silt, wet (due to drilling fluid), strong HCI reaction, dark grayish brown (SP)		AU-1	
	LIMESTONE, oolitic limestone, soft to medium, slightly weathered, layers of poorly cemented ooids or very soft limestone between moderately hard layers, tan (LS)		RC-1	REC=95% RQD=58%
5		-5.0		
	LIMESTONE, oolitic limestone, soft to medium, with few soft layers of poorly cemented ooids, trace shell fragments, slightly weathered, tan (LS)		RC-2	REC=89% RQD=43%
	LIMESTONE, oolitic limestone, soft to medium, slightly weathered, layers of poorly cemented ooids, tan (LS)		RC-3	REC=96% RQD=96%
10		-10.0		
	LIMESTONE, oolitic limestone, medium, fine-grained ooids, wet, strong HCI reaction, tan (LS)		SPT-1	11-22-22
	LIMESTONE, oolitic limestone, very soft, fine-grained ooids, wet, strong HCI reaction, tan (LS)		SPT-2	10-9-8
15		-15.0		
	LIMESTONE, oolitic limestone, soft, fine-grained ooids, wet, strong HCI reaction, tan (LS)		SPT-3	7-11-11
20	BORING TERMINATED	-20.0		
25		-25.0		



CONTRACTOR:	Independent Drilling, Inc.
DRILLER:	J. Wilkerson
EQUIPMENT:	CME 45B (DR-8, Eff. 87%) - Auto. Hammer
METHOD:	Auger/Mud Rotary/4" Dia. Rock Core
HOLE DIA.:	4"
REMARKS:	Groundwater table estimated at 4 feet below grade

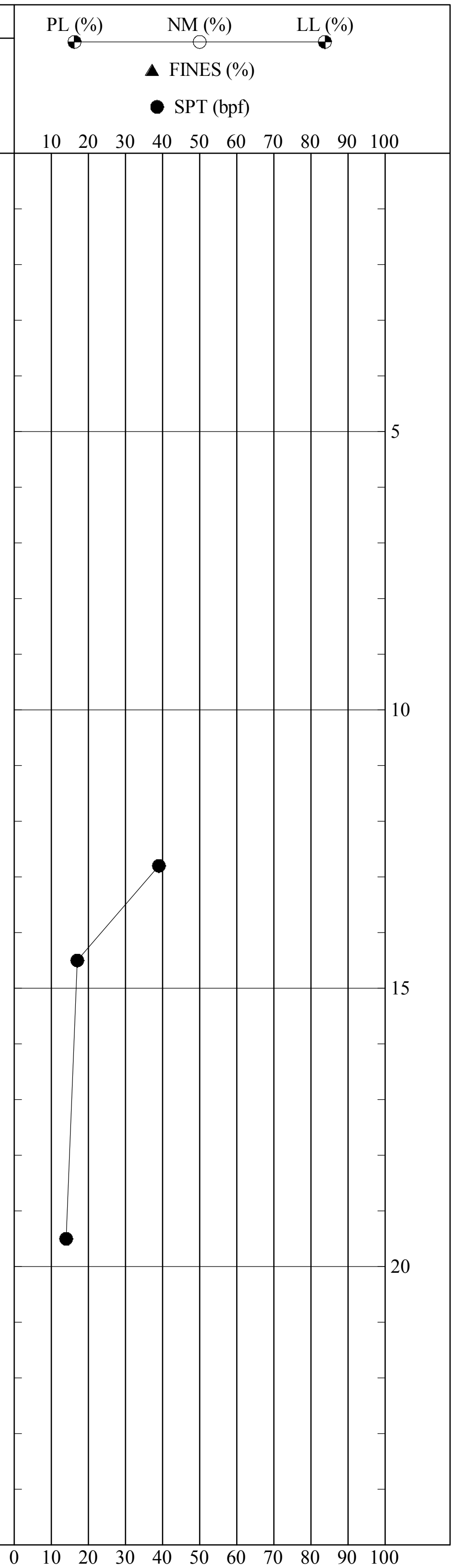
THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

Project: Glynn Archer School City Hall Conversion
Coord N:
Coord E:
Drilled: June 11, 2013
Proj. No.: 6734-13-9720
Boring No.: AB-2
Checked By: KAM

amec

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS (SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.)	ELEV (ft)	SAMPLES IDENT	N-COUNT (1st 6" / 2nd 6" / 3rd 6")
0	SAND, mostly fine sand-sized oolitic granuals with trace silt, little coarse to fine gravel and few organics, moist, strong HCI reaction, dark grayish brown (SP)	0.0	AU-1	
	LIMESTONE, oolitic limestone, soft to medium, fine grained, slightly weathered, rock fragments in upper 2' of core run, tan (LS)		RC-1	REC=68% RQD=65%
5		-5.0		
	LIMESTONE, oolitic limestone, soft to medium, fine grained, slightly weathered, tan (LS)		RC-2	REC=80% RQD=73%
10		-10.0		
	LIMESTONE, oolitic limestone, medium, wet, strong HCI reaction, tan (LS)		SPT-1	15-18-21
	LIMESTONE, oolitic limestone, very soft, wet, strong HCI reaction, tan (LS)		SPT-2	7-8-9
15		-15.0		
			SPT-3	8-7-7
20	BORING TERMINATED	-20.0		
25		-25.0		



SOIL 9720-01LOGS.GPJ LAW_GIBB.GDT 7/12/13

CONTRACTOR:	Independent Drilling, Inc.
DRILLER:	J. Wilkerson
EQUIPMENT:	CME 45B (DR-8, Eff. 87%) - Auto. Hammer
METHOD:	Auger/Mud Rotary/4" Dia. Rock Core
HOLE DIA.:	4"
REMARKS:	Groundwater table estimated at 4 feet below grade

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

Project: Glynn Archer School City Hall Conversion
Coord N:
Boring No.: AB-3
Coord E:
Checked By: KAM
Drilled: June 11, 2013
Proj. No.: 6734-13-9720

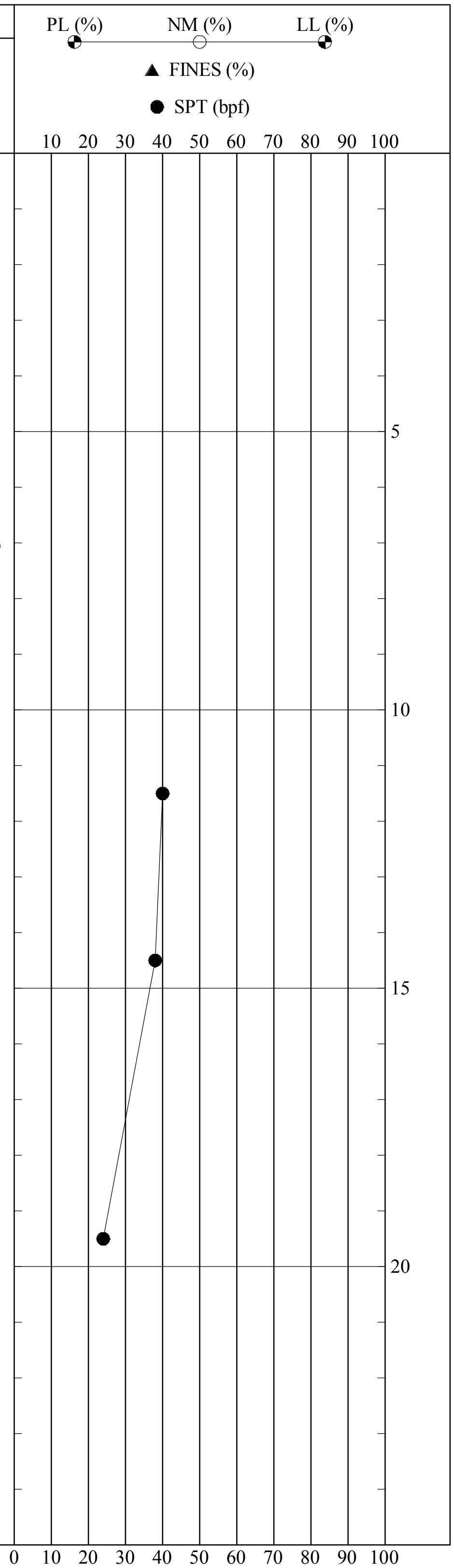
amec

SOIL TEST BORING RECORD

Project: Glynn Archer School City Hall Conversion
Coord N:
Coord E:
Drilled: June 11, 2013
Proj. No.: 6734-13-9720
Boring No.: AB-4
Checked By: KAM

SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS	ELEV (ft)	SAMPLES IDENT	N-COUNT (1st 6" / 2nd 6" / 3rd 6")
0	SAND, mostly fine-grained ooids with trace silt, few fine to coarse gravel, moist, strong HCI reaction, dark grayish brown (SP)	0.0	AU-1	
	LIMESTONE, oolitic limestone, soft to medium, slightly weathered, tan (LS)		RC-1	REC=80% RQD=63%
5		-5.0		
	LIMESTONE, oolitic limestone, soft to medium with very soft layering between harder layers, slightly weathered, tan (LS)		RC-2	REC=100% RQD=64%
10		-10.0		
	LIMESTONE, oolitic limestone, medium, wet, strong HCI reaction, tan (LS)		SPT-1	15-18-22
	LIMESTONE, oolitic limestone, medium, some lime mud, wet, strong HCI reaction, tan (LS)		SPT-2	16-19-19
15		-15.0		
	LIMESTONE, oolitic limestone, soft, some lime mud, wet, strong HCI reaction, tan (LS)		SPT-3	8-10-14
20	BORING TERMINATED	-20.0		
25		-25.0		



CONTRACTOR: Independent Drilling, Inc.
DRILLER: J. Wilkerson
EQUIPMENT: CME 45B (DR-8, Eff. 87%) - Auto. Hammer
METHOD: Auger/Mud Rotary/4" Dia. Rock Core
HOLE DIA.: 4"
REMARKS: Groundwater table estimated at 4 feet below grade

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

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Double-Ring Infiltrometer Test Results

Glynn Archer School City Hall Conversion
1300 White Street
Key West, Florida
AMEC Project No. 6734-13-9720

Date Performed: June 12, 2013

Test Location	Test Depth (ft)	Infiltration Rates (in/hr)		
		Inner Ring	Annular Space Between Rings	Recommended Value
DRI-1	0.3	16.6	20.2	16.6

KAM

Stratification	
Depth Range (ft)	Material Description
0.0'-1.5'	Sand
1.5'-20'+	Miami Limestone

Field Percolation Test Results

Glynn Archer School City Hall Conversion
1300 White Street
Key West, Florida
AMEC Project No. 6734-13-9720

Date Performed: June 12, 2013

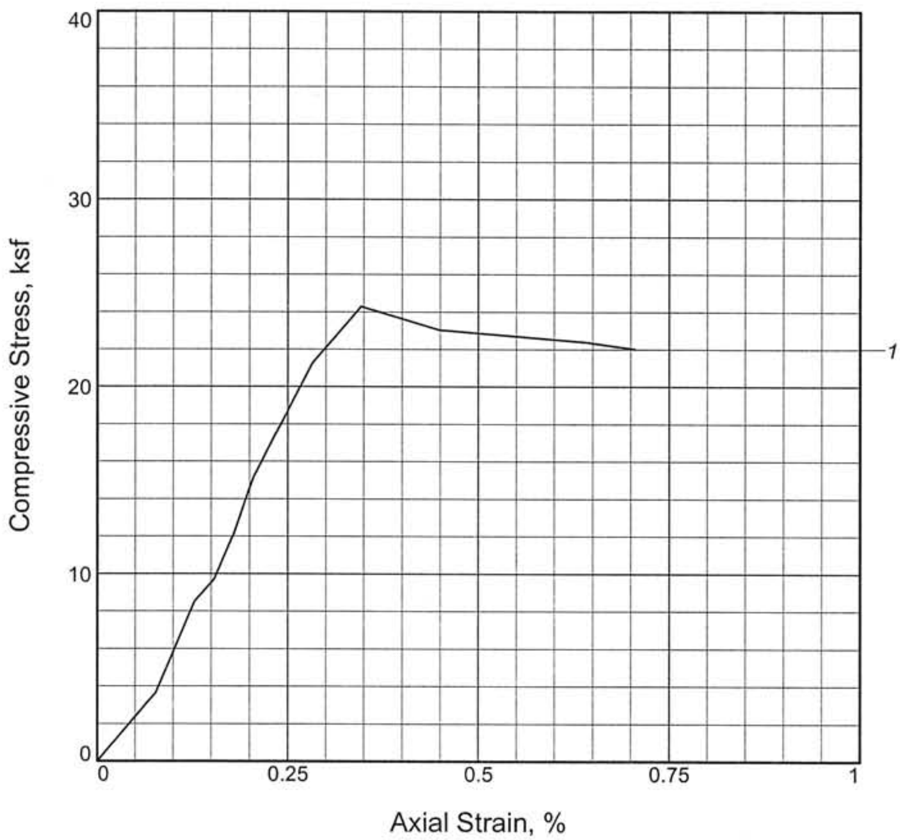
Location	Test Depth (ft)	Depth to Groundwater Level (ft)	Diameter of Drilled Hole (in)	Diameter of Casing (in)	Flow Rate, Q (cfs)	Head (ft)	Hydraulic Conductivity, k (cfs/ft^2 - ft. of head)
P-1	9.5	4.0	8	6	6.9×10^{-4}	4	1.45×10^{-5}

KAM

Stratification	
0.0' - 0.7'	3" asphalt over 5" limerock base
0.7' - 20'+	Miami Limestone

Test Method: South Florida Water Management District (March 22, 2009)
Usual Open-Hole Test (Fig. F-1)
Constant Head Method

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	24.27			
Undrained shear strength, ksf	12.14			
Failure strain, %	0.3			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	126.4			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.880			
Specimen height, in.	7.800			
Height/diameter ratio	2.01			

Description: Tan-white oolitic limestone

LL =	PL =	PI =	Assumed GS= 2.45	Type: Core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date tested: 6-21-13
E50: 10,400 ksf

Figure ________

Client: Bender & Associates Architects, P.A.

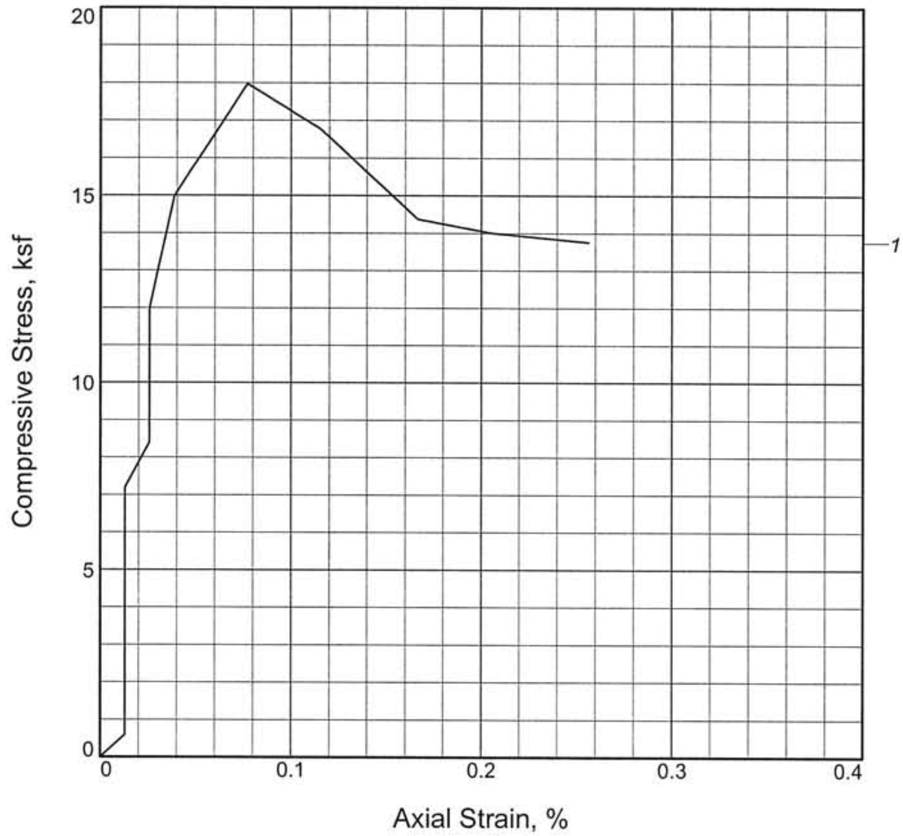
Project: Glynn Archer School City Hall Conversion

Sample Number: AB-1 **Depth:** 3.3'-4.0'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

Tested By: CM **Checked By:** K. McIntosh, P.E.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	17.98			
Undrained shear strength, ksf	8.99			
Failure strain, %	0.1			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	126.7			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.910			
Specimen height, in.	7.800			
Height/diameter ratio	1.99			

Description: Tan-white oolitic limestone

LL =	**PL =**	**PI =**	**Assumed GS=** 2.45	**Type:** Core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date tested: 6-21-13
E50: 50,000 ksf

Figure ________

Client: Bender & Associates Architects, P.A.

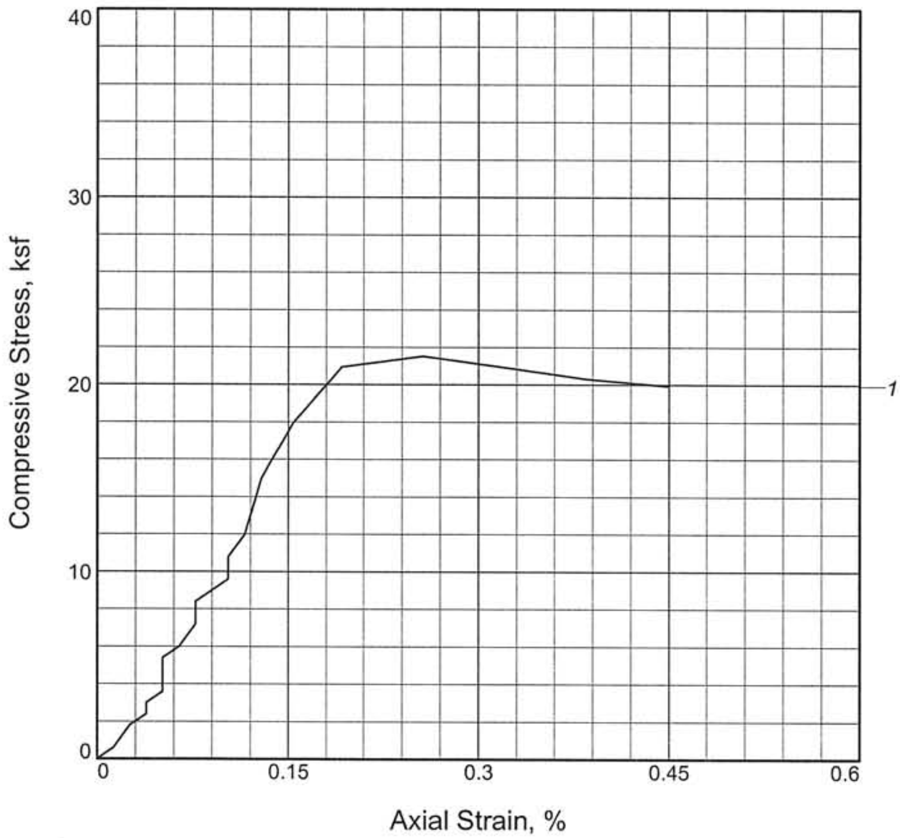
Project: Glynn Archer School City Hall Conversion

Sample Number: AB-1 **Depth:** 6.2'-7.0'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

Tested By: CM **Checked By:** K. McIntosh, P.E.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	21.53			
Undrained shear strength, ksf	10.77			
Failure strain, %	0.3			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	126.7			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.910			
Specimen height, in.	7.800			
Height/diameter ratio	1.99			

Description: Tan-white oolitic limestone

LL =	PL =	PI =	Assumed GS= 2.45	Type: Core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date Tested: 6-21-13
E50: 14,300 ksf

Figure ________

Client: Bender & Associates Architects, P.A.

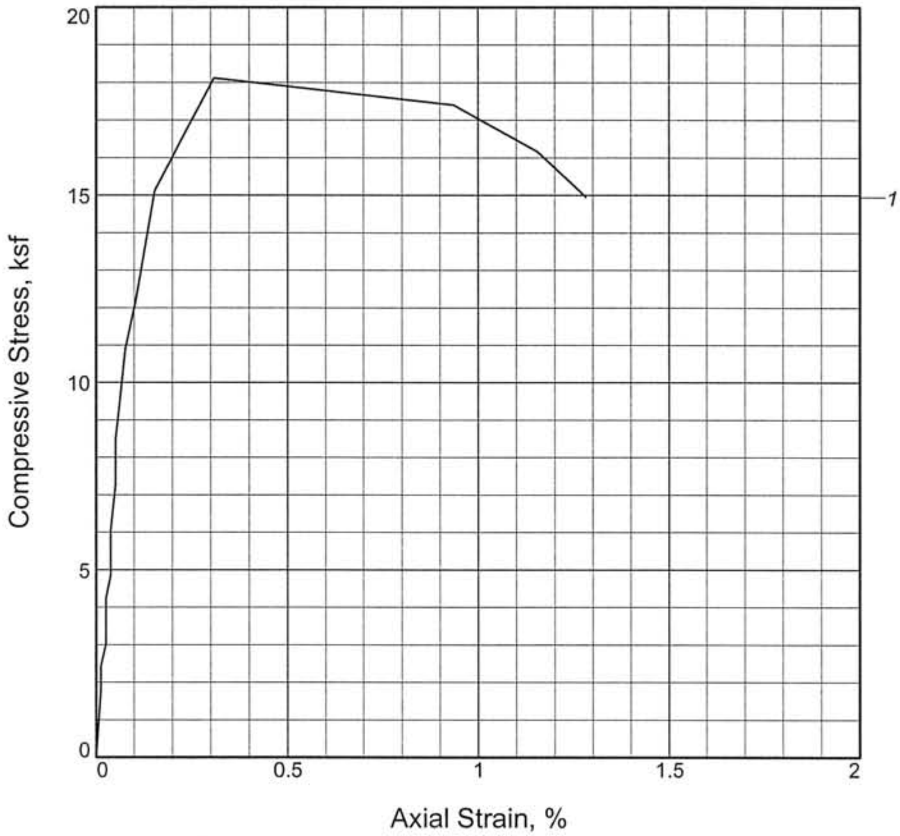
Project: Glynn Archer School City Hall Conversion

Sample Number: AB-2 **Depth:** 2.4'-3.2'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

Tested By: CM **Checked By:** K. McIntosh, P.E.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	18.12			
Undrained shear strength, ksf	9.06			
Failure strain, %	0.3			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	129.0			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.890			
Specimen height, in.	7.800			
Height/diameter ratio	2.01			

Description: Tan-white oolitic limestone

LL =	PL =	PI =	Assumed GS= 2.45	Type: core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date tested: 6-21-13
E50: 13,000 ksf

Figure _______

Client: Bender & Associates Architects, P.A.

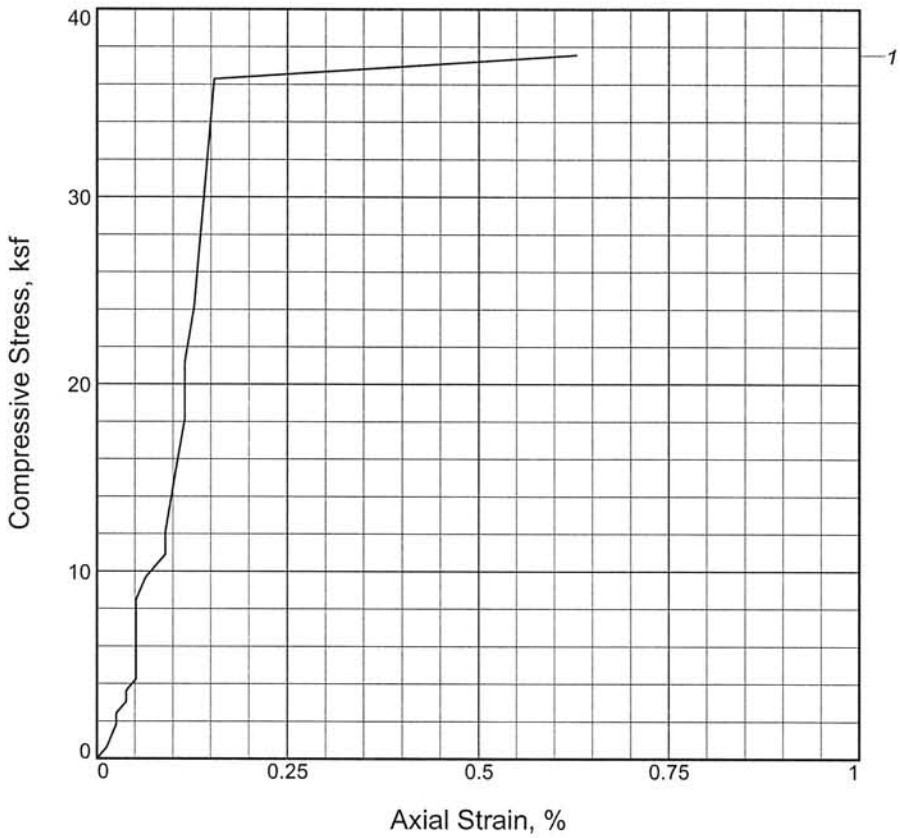
Project: Glynn Archer School City Hall Conversion

Sample Number: AB-2 **Depth:** 8.3'-9.0'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

Tested By: CM **Checked By:** K. McIntosh, P.E.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	37.57			
Undrained shear strength, ksf	18.78			
Failure strain, %	0.6			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	125.1			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.890			
Specimen height, in.	7.800			
Height/diameter ratio	2.01			

Description: Tan-white oolitic limestone

LL =	PL =	PI =	Assumed GS= 2.45	Type: core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date tested: 6-21-13
E50: 52,000 ksf

Figure ______

Client: Bender & Associates Architects, P.A.

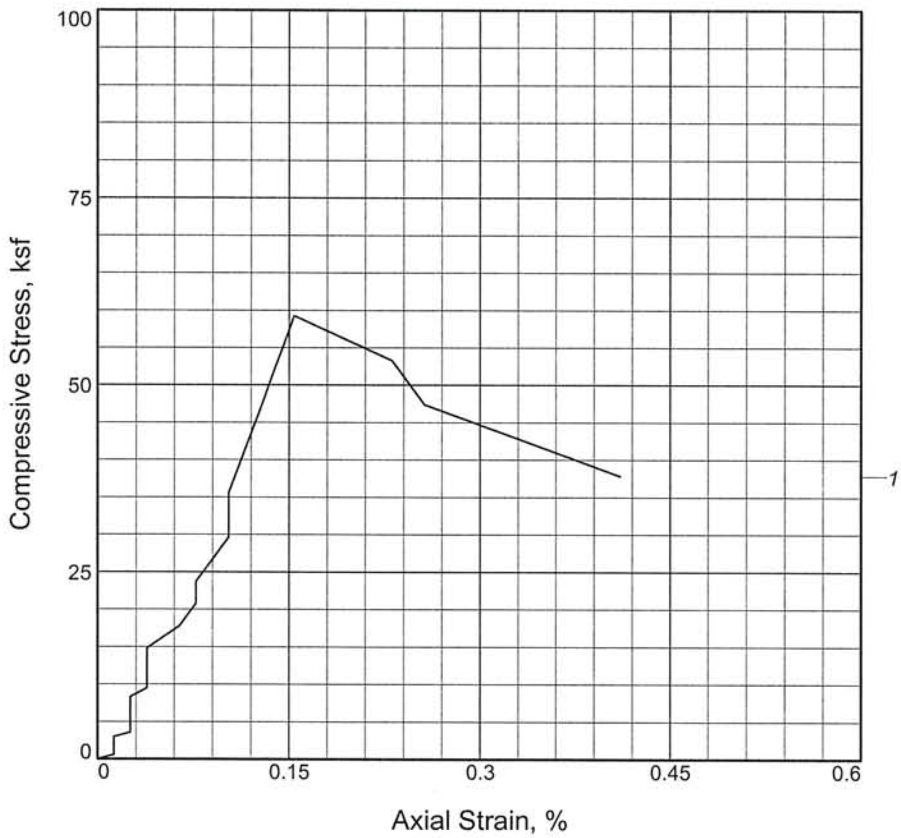
Project: Glynn Archer School City Hall Conversion

Sample Number: AB-3 **Depth:** 3.2'-3.9'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

Tested By: CM **Checked By:** K. McIntosh, P.E.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	59.26			
Undrained shear strength, ksf	29.63			
Failure strain, %	0.2			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	129.2			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.930			
Specimen height, in.	7.800			
Height/diameter ratio	1.98			

Description: Tan-white oolitic limestone

LL =	**PL =**	**PI =**	**Assumed GS=** 2.45	**Type:** core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date Tested: 6-21-13
E50: 26,650 ksf

Figure _______

Client: Bender & Associates Architects, P.A.

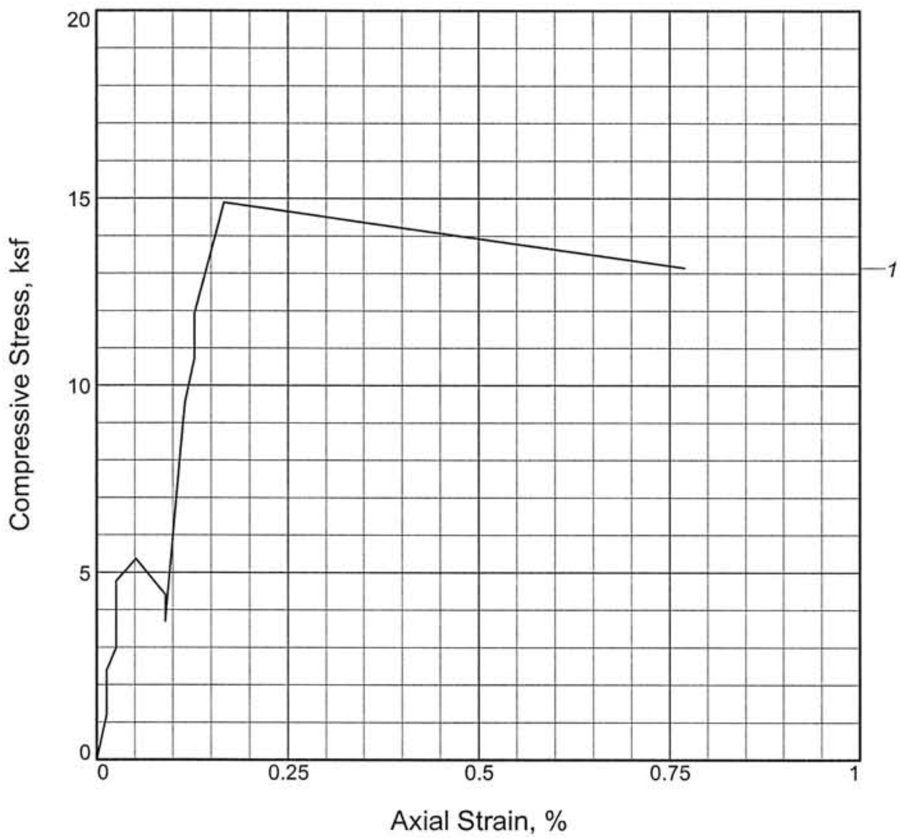
Project: Glynn Archer School City Hall Conversion

Sample Number: AB-3 **Depth:** 6.8'-7.5'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

Tested By: CM **Checked By:** K. McIntosh, P.E.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	14.89			
Undrained shear strength, ksf	7.44			
Failure strain, %	0.2			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	126.3			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.920			
Specimen height, in.	7.800			
Height/diameter ratio	1.99			

Description: Tan-white oolitic limestone

LL =	PL =	PI =	Assumed GS= 2.45	Type: core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date tested: 6-21-13
E50: 20,000 ksf

Figure _______

Client: Bender & Associates Architects, P.A.

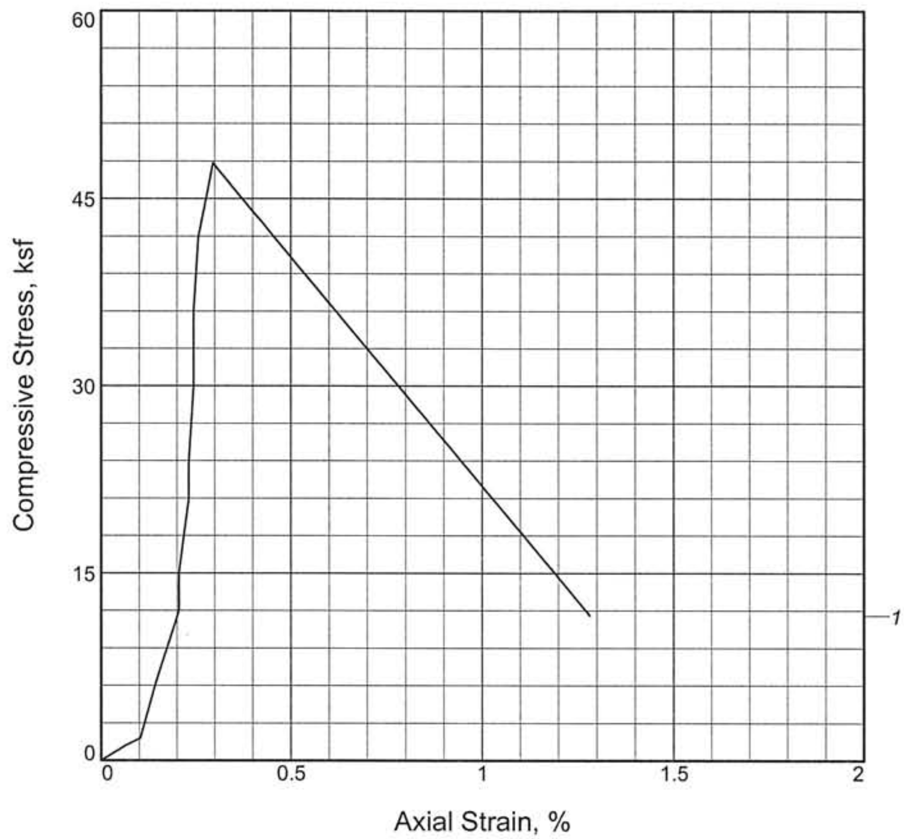
Project: Glynn Archer School City Hall Conversion

Sample Number: AB-4 **Depth:** 3.1'-3.9'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

Tested By: CM **Checked By:** K. McIntosh, P.E.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	47.83			
Undrained shear strength, ksf	23.91			
Failure strain, %	0.3			
Strain rate, in./min.	0.005			
Water content, %	N/A			
Wet density, pcf	127.4			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	3.910			
Specimen height, in.	7.800			
Height/diameter ratio	1.99			

Description: Tan-white oolitic limestone

LL =	PL =	PI =	Assumed GS= 2.45	Type: core

Project No.: 6734-13-9720

Date Sampled: 6/11/13

Remarks:
Date Tested: 6-21-13
E50: 57,000 ksf

Figure ________

Client: Bender & Associates Architects, P.A.

Project: Glynn Archer School City Hall Conversion

Sample Number: AB-4 **Depth:** 4.8'-5.5'

UNCONFINED COMPRESSION TEST
AMEC E&I
Jacksonville, Florida

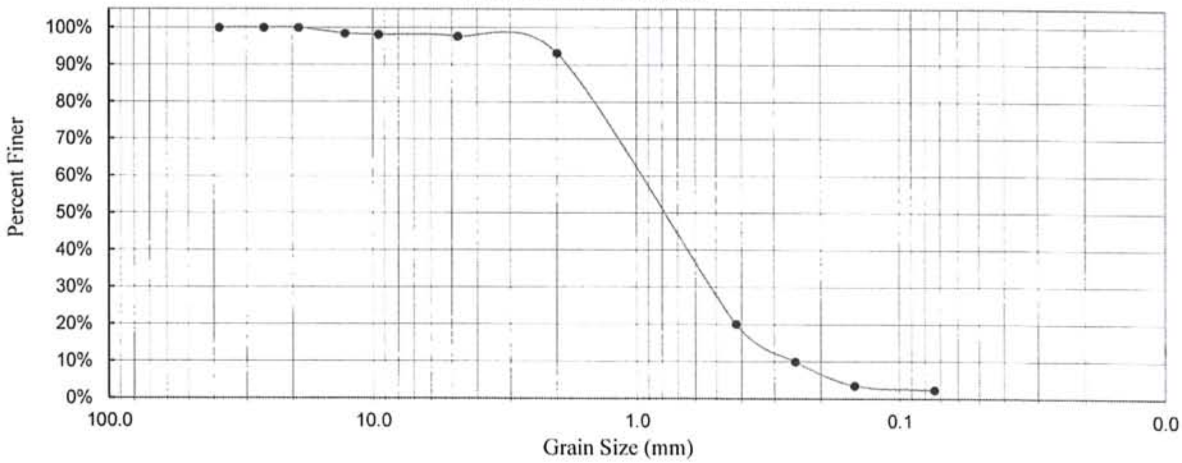
Tested By: CM **Checked By:** K. McIntosh, P.E.

AMEC Environment & Infrastructure, Inc.
2580 Metrocentre Blvd., Suite # 6
West Palm Beach, Florida 33407

Grainsize Analysis

Project:	Glynn Archer School	Project #:	6734-13-9720
Tested by:	MCh	Test Date:	6/17/2013
Sample Description:	Gray, poorly graded SAND (SP), collected from sandy playground area.	Sample:	No. 1

Grain Size Distribution



Sieve	Size (mm)	Weight Retained On Sieves (cum) (g)	Percent Retained on Sieve (cum)	Percent Passing Sieve
1 1/2	38.1	0.0	0.0%	100.0%
1	25.7	0.0	0.0%	100.0%
3/4	19	0.0	0.0%	100.0%
1/2	12.7	36.4	1.6%	98.4%
3/8	9.51	42.9	1.9%	98.1%
4	4.76	53.7	2.3%	97.7%
10	2	158.0	6.9%	93.1%
40	0.42	1838.9	79.9%	20.1%
60	0.25	2070.9	90.0%	10.0%
100	0.149	2219.2	96.4%	3.6%
200	0.074	2246.9	97.6%	2.4%
	Pan	2288.1	99.4%	

Total Weight Before Wash : 2301.2
Percent finer than # 200 sieve : 2.4%
Unified Soil Classification System : A-1-b

D_{10} = 0.250 mm
D_{30} = 0.634 mm
D_{60} = 1.283 mm

Coefficient of Curvature, C_c = 1.25
Coefficient of Uniformity, C_u = 5.13

Checked by:

6-19-2013

Type of Test: ASTM D-422 and D-2487

Field and Laboratory Procedures

Field Procedures

Soil Test Borings - The soil test borings were performed in general accordance with ASTM D 1586, "Penetration Test and Split-Barrel Sampling of Soils." The borings were initially advanced by augering. A rotary drilling process was subsequently used and bentonite drilling fluid was circulated in the boreholes to stabilize the sides and flush the cuttings. At regular intervals, the drilling tools were removed and soil and rock samples were obtained with a standard 1.4-inch I.D., 2.0-inch O.D., split-tube sampler. An internal liner was not utilized in the sampler. The sampler was first seated 6 inches and then driven an additional foot with blows of a 140-pound automatically tripped hammer falling 30 inches. This hammer had been previously calibrated for efficiency by AMEC—which indicated an efficiency of about 87 percent. The number of hammer blows required to drive the sampler the final foot is designated the "Penetration Resistance." The penetration resistance, when properly interpreted, is an index to the soil or rock strength and density.

Representative portions of the rock samples, obtained from the sampler, were placed in glass jars and transported to our laboratory. The samples were classified by a geologist in the field.

Rock Coring -Samples of the Miami Limestone were obtained using a diamond-studded bit fastened to the end of a hollow, double tube core barrel, which was, in turn, fastened to the end of the drill rods. The coring procedure employed was similar to that described by ASTM D 2113. Core samples of the material penetrated were protected and retained in a swivel-mounted inner tube. Upon completion of each core run, the core barrel was brought to the surface and the samples removed and placed in wooden boxes.

The field geologist classified the rock obtained, and determined the percent core recovery and the rock quality designation (RQD) for each core run. The recovery is defined as the ratio of the sample length obtained to the depth drilled, expressed as a percent. The percent recovery is related to the rock soundness and continuity. In addition, the Rock Quality Designation (RQD) was determined. The RQD is defined as the sum of the lengths of recovered pieces equal to or larger than 4 inches divided by the length of rock cored, expressed as a percentage. The rock description, percent recovery, and RQD values are shown on the appropriate Soil Test Boring Record. The coring performed utilized a core barrel which obtained core samples having an approximate diameter of 4 inches.

Laboratory Procedures

Unconfined Compression - Test samples were obtained from unfractured core samples of rock-like materials. The sample diameters varied from about 2 to 4 inches with the height and twice the sample diameter. For sample heights less than twice the diameter, the test results were corrected using established correction factors from ASTM Designation C-42, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete". The ends of the samples were either precisely trimmed or were "capped" by a cementing agent in order to form a smooth surface for testing. The test samples were then individually placed in the testing device, and vertical loads applied continuously until the sample failed in shear. Vertical deformation during some of the test was measured with a micrometer dial indicator at the top of the specimen. This test was performed in general accordance with ASTM Designation D 2938.

Direct Shear (Core Specimen) – The direct shear test allows the determination of the shear strength parameters along a pre-determined failure plane. The core specimen is placed in a split container and grouted in-place with leadite or gypsum cement. Prior to testing, a normal stress approximately equal to the sample overburden pressure is applied perpendicular to the shear plane and located in by spring-loaded tie rods. The device is then rotated 90°, and the shearing load applied to one-half of the container, with the other half held stationary. During the test, the shear displacement is measured by micrometer dial gauges or LVDTS and the shearing force is read directly from the compression machine. This method is essentially that outlined in ASTM Publication STP 479 (1970).

KEY TO CLASSIFICATION AND SYMBOLS

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

GRANULAR MATERIAL	SPT N VALUE (BLOWS/FOOT)	
RELATIVE DENSITY	SAFETY HAMMER	AUTOMATIC HAMMER
VERY LOOSE	0 - 4	0 - 3
LOOSE	5 - 10	4 - 8
MEDIUM DENSE	11 - 30	9 - 24
DENSE	31 - 50	25 - 40
VERY DENSE	> 50	> 40

SILTS AND CLAYS	SPT N VALUE (BLOWS/FOOT)	
CONSISTENCY	SAFETY HAMMER	AUTOMATIC HAMMER
VERY SOFT	0 - 2	0 - 1
SOFT	3 - 4	2 - 3
FIRM	5 - 8	4 - 6
STIFF	9 - 15	7 - 12
VERY STIFF	16 - 30	13 - 24
HARD	> 30	> 24

ROCK HARDNESS DESCRIPTION

VERY SOFT	Rock core crumbles when handled **N < 20**
SOFT	Can break core easily with hands **N = 21-30**
MEDIUM HARD	Can break core with hands **N = 31-45**
MODERATELY HARD	Thin edges of rock can be broken with fingers **N = 46-60**
HARD	Thin edges of rock cannot be broken with fingers **N = 61-100**
VERY HARD	Rock core rings when struck with a hammer (cherts) **N > 50/2"**

MODIFIERS

APPROXIMATE PERCENTAGE	MODIFIERS
0 to 5%	Trace
5% to 10%	Few
15% to 25%	Little
30% to 45%	Some

The modifiers provide our estimate of the percentages of gravel, sand, and fines (silt or clay size particles).

SYMBOLS	DESCRIPTION
UD	Undisturbed sample (UD) recovered.
100/2"	N, Number of blows (100) to drive the support spoon or cone a number of inches (2").
NX, 4", 6"	Corel Barrel sizes which obtain cores 2-1/8", 3-7/8", and 5-7/8" diameter respectively.
65%	Percentage (65) of rock core and soil sample recovered
RQD	Rock Quality Design - Percent of rock core 4 or more inches long
▼	Water table at least 24 hours after drilling
△	Water table one hour or less after drilling
◄	Loss of drilling fluid

Photo Documentation of Drilling Operations

Glynn Archer School City Hall Conversion **AB-1**
Key West, Florida Photograph Date: <u>June 11, 2013</u>

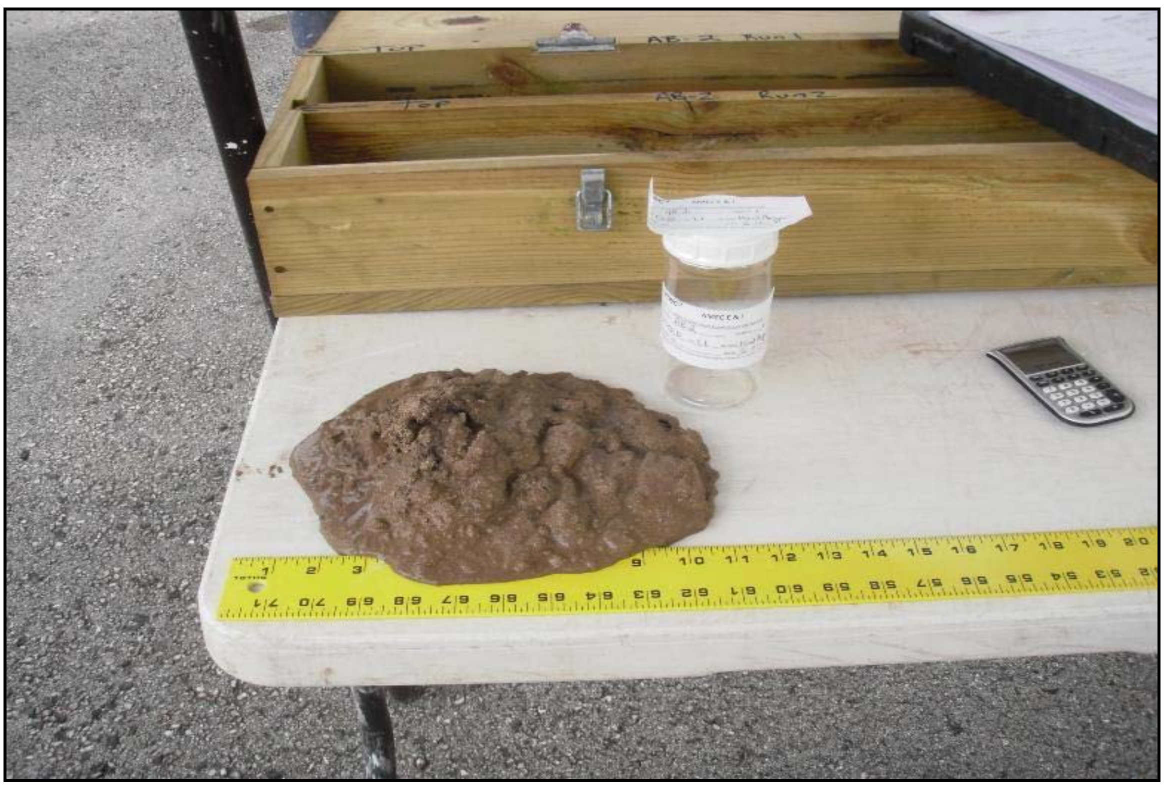

AB-1
1
0.5'-5.5'
60"
50/60
38/60 63%

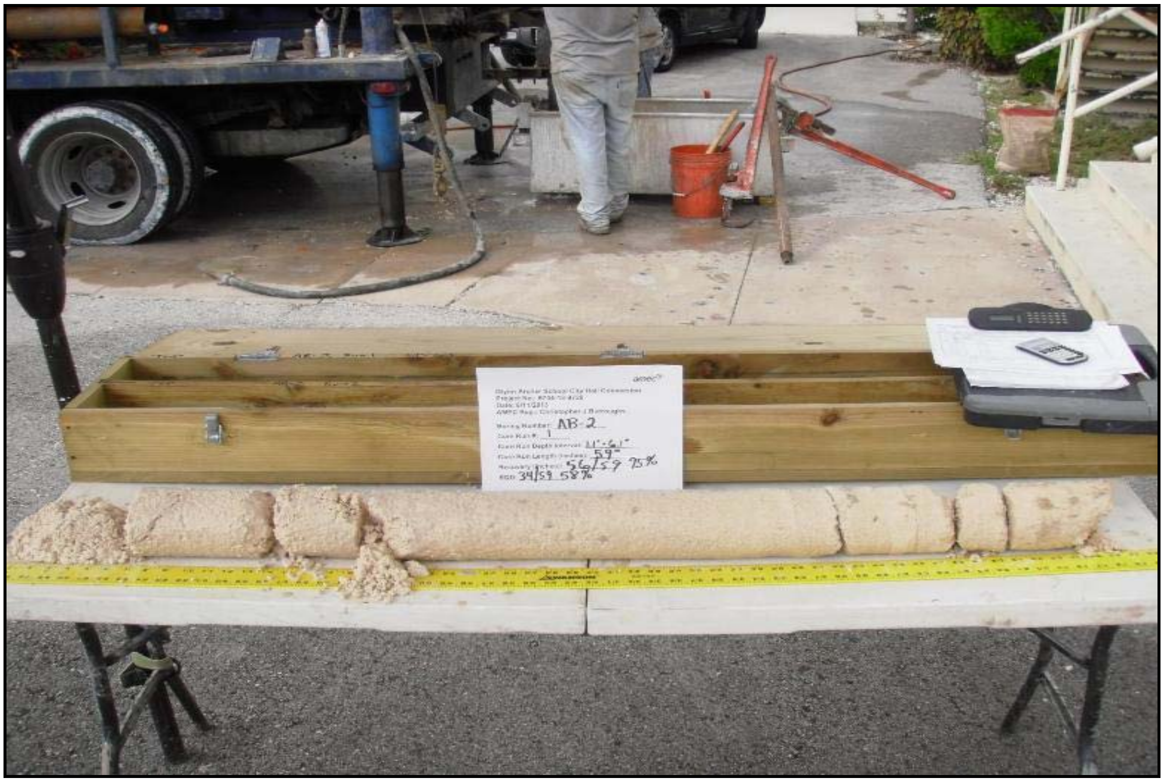

AB-1
2
5.5'-10.3'
57"
53/57
68%
← Bottom
Top →

Glynn Archer School City Hall Conversion **AB-1**
Key West, Florida Photograph Date: June 11, 2013


amec

AB-2
Photograph Date: June 11, 2013




AB-2
1
59"
56/59 75%
34/59 58%


AB-2 Run 1
AB-2 Run 2
amec
Glynn Archer School City Hall Conversion
Project No.: 6734-13-9720
Date: 6/11/2013
AMEC Rep.: Christopher J Burroughs
Boring Number: AB-2
Core Run #: 2
Core Run Depth Interval: 6.9'-83'
← Bottom
Top →


amec
Glynn Archer School City Hall Conversion
Project No.: 6734-13-9720
Date: 6/11/2013
AMEC Rep.: Christopher J Burroughs
Boring Number: AB-2
Core Run #: 3
Core Run Depth Interval: 8.3-10.3'
Core Run Length (inches): 24"
Recovery (inches): 23/24 96%
RQD: 23/24 96%
← Bottom
Top →


DIESEL


amec
Glynn Archer School City Hall Conversion
Project No.: 6734-13-9720
Date: 6/11/2013
AMEC Rep.: Christopher J Burroughs
Boring Number: AB-3
Core Run #: 1
Core Run Depth Interval: 18'-68
Core Run Length (inches): 60"
Recovery (inches): [redacted] 41/60
RQD: 39/60 65%


amec
Glynn Archer School City Hall Conversion
Project No.: 6734-13-9720
Date: 6/11/2013
AMEC Rep.: Christopher J Burroughs
Boring Number: AB-3
Core Run #: 1
Core Run Depth Interval: 18'-68
Core Run Length (inches): 60"
Recovery (inches): [redacted] 41/60
RQD: 39/60 65%
← Bottom
Top →
QUIK-GEL


AB-3
2
60
48/60
73%
Bottom
Top


429


Boring Number AB-4
Core Run # 1
1.0-6.0
60
48/60 80%
38/60 63%
Bottom
Top


AB-4
2
6.0-10.5
55
55/55 100%
35/55 64%
Bottom

Glynn Archer School City Hall Conversion **AB-4**

Key West, Florida Photograph Date: <u>June 11, 2013</u>

Glynn Archer School City Hall Conversion **AB-4**

Key West, Florida Photograph Date: <u>June 11, 2013</u>

APPENDIX B

STRUCTURAL REVIEW
KEY WEST CITY HALL AT GLENN ARCHER
1302 WHITE STREET
KEYWEST, FLORIDA

Atlantic Engineering Services

AES

Structural Review
Key West City Hall at Glynn Archer
1302 White Street
Key West, Florida

Prepared for
Bender & Associates Architects, P.A.
410 Angela Street
Key West, FL 33040-7402

Prepared by
Atlantic Engineering Services of Jacksonville
6501 Arlington Expressway, Building B, Suite 201
Jacksonville, FL 32211
(904) 743-4633

AES Project No. **312-295**
June 20, 2013

Pittsburgh ▪ Jacksonville

Atlantic Engineering Services
6501 Arlington Expressway, Building B, Suite 201
Jacksonville, FL 32211
Phone: 904.743.4633 Fax: 904.725.9295
E-mail: jax@aespj.com

June 20, 2013

Mr. Bert L. Bender, RA, LEED AP
Bender & Associates Architects, P.A.
410 Angela Street
Key West, Florida 33040-7402

Re: Design Charette – Structural Condition Review — Project: #312-295
Key West City Hall at Glynn Archer — 312295_00^RPT_Structural Condition Review.doc
Key West, Florida

Dear Bert:

I am writing, at the request of Mr. Don Craig, to follow-up on my limited structural condition review during the design charette, to confirm the condition of the structure presented in the Property Condition Assessment, Glynn Archer School dated September 7, 2012, prepared by CH2MHill. My limited structural condition review consisted of a visual review of the structure referenced above on June 11, 2013, and continuing through June 13, 2013. The review was performed by Mark J. Keister, P.E.; Atlantic Engineering Services of Jacksonville (AES).

The Glynn Archer Elementary School located on White Street between Seminary Street and United Street in Key West, Florida is the former Key West High School and consists of two buildings. Building A, with the auditorium was constructed in 1923 and Building B, which was constructed in 1927. Both buildings are two-story structures and the auditorium in Building A is one-story. Construction consists of wood framed roof and floors supported by perimeter concrete walls and interior wood framed walls. The foundations consist of shallow foundations, which bear on the shallow rock. Supporting the wood framed roof over the auditorium are three steel trusses and a wood truss.

On June 11 and 12, 2013, AMEC Environmental & Infrastructure, Inc. (AMEC) performed four, 20 foot rock cores, adjacent to the borings performed by Nutting Engineers of Florida (Nutting), as reported in their Report of Geotechnical Exploration Concrete Core Testing and Foundation Excavations dated August 2012, to confirm the consistency and bearing capacity of the shallow rock. Rock was encountered between 1’-0” to approximately 1’-6” below the surface and was very cohesive with a few voids (see Photographs 1, 2, and 3). In the Nutting report, their rock core compression tests varied from a low of 1,717 psi to a high of 4,229 psi and Nutting recommended a foundation bearing capacity of 4,000 psf. Our experience in Miami Limestone is that it has a minimum contact bearing pressure of between 6,000 psf and 8,000 psf and depending on consistency and voids, can be significantly higher. AMEC will be performing compression tests on eight samples as part of their geotechnical investigation to determine the bearing capacity of shallow foundations bearing on and in this Miami Limestone. There final results and recommendations will be forthcoming.

During our investigation of the ground floor, crawl space and foundations, cisterns were discovered in the southeast corner of Building A (see photographs 4 and 5) and the southeast corner of Building B (see photograph 6). The cistern in Building A was not noted in the CH2MHill report. Also, in Building A, an old abandoned cistern was discovered from a previous structure on the site (see photograph 7). The ground floor timber is in excellent condition and many of the 5-1/2” x 5-1/2” timber beams noted in the CH2MHill report are actually 8”x 8” timber beams (see photograph 8). There were also many framing discrepancies noted from the CH2MHill report. As can be seen in the crawl space photographs, the ground surface is weathered rock and the concrete foundations bear on the rock or are socketed into the rock.

The perimeter concrete walls consist of 11 inch concrete, which widens to 1'-6" or wider at the ground floor and widens again to 2'-0" or wider just above the ground surface and in many cases, the concrete walls widen again in the bearing rock (see photograph 9). The only place in the facility with a thinner concrete wall is the rear wall of the auditorium. The walls observed are in excellent condition with minimal cracking, no spalling and no signs of distress. The CH2MHill report documents 8 inch, concrete walls throughout the facility and recommends that they be reinforced if supporting floor loads, and that the perimeter wall foundations are undersized and need to be underpinned with piling, due to the rock bearing capacity of 4,000 psf. There are no signs of distress in the perimeter concrete walls and they have performed adequately for nearly one hundred (100) years supporting gravity and lateral loads. I see no reason that they cannot continue to support gravity and lateral loads. With their actual thickness of 11inches, they are significantly stronger and more durable than reported in the CH2MHill report. If the recommendation of the AMEC, rock bearing capacity is in the 6,000 psf to 8,000 psf range, the existing wall foundations are adequate and will not require underpinning with deep foundations. In the CH2MHill report, augercast piles, pile caps and grade beams had a combined cost of $398,500.00 and if the existing foundations are adequate and new foundations can bear directly on, or in the rock, augercast piles, pile caps and grade beams will not be required. If only conventional shallow foundations bearing on, or in the rock are needed, this will bring significant savings to the project.

The historic proscenium beam at the auditorium is a 5'-2" deep, wood truss and is in excellent condition (see photographs 10 and 11). This truss is not documented in the CH2MHill report. The auditorium roof consists of roof sheathing on 1- 5/8" x 7-1/2" roof joists at 2'-0" on center, which bear on four rows of two, 1- 5/8" x 11-1/2" wood beams, supported by 6'-6" deep steel trusses in which the bottom chord drops below the historic ceiling and created a coffered auditorium ceiling. The ceiling joists consist of 1-5/8" x 5-1/2" joists at 2'-0" on center, supported by four rows of two, 1-5/8" x 9-1/2" wood beams, also supported by the steel trusses. The CH2MHill report presents 1-5/8" x 5-1/2" roof joists at 2'-0" on center, supported by five rows of wood trusses, supported by 48" deep steel trusses.

At the auditorium roof interface with the second floor, there is an area with an active roof leak and deteriorated roof sheathing (see photograph 12). In this area, there are termite damaged ceiling joists. The auditorium roof beams are in excellent condition and the ceiling beams are in good condition with areas of termite damage (see photograph 13). At the proscenium beam, a diagonal from the ceiling beam to the roof beam has been cut to accommodate ductwork (see photograph 14) and no distress is apparent. It appears that these verticals and diagonals were installed for ease of construction and may not be acting as a truss. The roof and ceiling joists are fire cut into the 11 inch concrete walls and the concrete walls are in excellent condition (see Photographs 15 and 16). The steel trusses, roof and ceiling beam connections to the trusses are in excellent condition with minimal surficial rust (see photographs 17, 18 and 19). The truss bearings are placed integral with the concrete walls and are totally encapsulated in the walls (see photographs 20 and 21). There was one area of corrosion noted in one of the trusses, but this corrosion is surficial and can easily be cleaned and coated (see photograph 22).

The historic stage was a thrust stage with angled end rafters for foot lights (see photograph 23) and the historic stage has been enlarged to its present size. The CH2MHill framing in this area does not correctly depict the actual framing in this area. The stage framing is in excellent condition. The main auditorium floor framing at the stage could not be reviewed due to low crawl room. Its framing and condition could not be confirmed.

At the connecting breezeways, between Buildings A and B, there is significant wood rot and termite damage with active termites (see photographs 24 and 25). The breezeway wood roof framing is supported by concrete columns that are in excellent condition. The CH2MHill report documents these columns as wood columns.

The roofing for both buildings is in poor condition with active leaks. The concrete parapets and cornices are in good condition with no signs of distress other than random cracking in the cornice cement wash (see photographs 26 and 27).

The concrete walls and foundations at Glynn Archer Elementary School are in excellent condition with little cracking and no observed spalling. The minimum concrete wall thickness is 11 inches throughout except at the rear of the auditorium, where it is 8 inches thick. The walls have performed adequately for nearly one hundred (100) years supporting gravity and lateral loads and I see no reason, that they cannot continue to support gravity and lateral loads. With their actual thickness of 11inches, they are significantly stronger and more durable than reported in the CH2MHill report. The existing foundations bear on and in the rock and I am anticipating that the forthcoming AMEC foundation recommendations will recommend a higher rock bearing capacity than recommended in the Nutting report, allowing the existing foundations to be used without being underpinned and new foundations not requiring piles, which will bring significant savings to the project. The auditorium roof and floor structure is in excellent condition except for isolated areas of active roof leaks with deteriorated roof sheathing and termite damage. There are discrepancies in the roof and floor framing as presented in the CH2MHill report and the observed discrepancies are noted above. The discrepancies are not minor and if portions of the existing framing are to be reused, the framing in those areas should be surveyed and verified. The breezeway roof framing between Buildings A and B are in poor condition with significant wood rot and termite damage, along with active termites.

It has been a pleasure serving you as a consulting structural engineer. Please contact our office if there are any questions regarding this correspondence, or if you need any additional information or assistance.

Very truly yours,
ATLANTIC ENGINEERING SERVICES OF JACKSONVILLE
FLORIDA CERTIFICATE OF AUTHORIZATION #791

Mark J. Keister, P.E.
Principal

MARK J. KEISTER ★ LICENSE No. 37435 ★ STATE OF FLORIDA ★ PROFESSIONAL ENGINEER

MJK/drg

PHOTOGRAPH 1

PHOTOGRAPH 2

PHOTOGRAPH 3

PHOTOGRAPH 4

PHOTOGRAPH 5

PHOTOGRAPH 6

PHOTOGRAPH 7

PHOTOGRAPH 8

PHOTOGRAPH 9

PHOTOGRAPH 10

PHOTOGRAPH 11

PHOTOGRAPH 12

PHOTOGRAPH 13

PHOTOGRAPH 14

PHOTOGRAPH 15

PHOTOGRAPH 16

PHOTOGRAPH 17

PHOTOGRAPH 18

PHOTOGRAPH 19

PHOTOGRAPH 20

PHOTOGRAPH 21

PHOTOGRAPH 22

PHOTOGRAPH 23

PHOTOGRAPH 24

PHOTOGRAPH 25

PHOTOGRAPH 26

PHOTOGRAPH 27

APPENDIX A

EXISTING STRUCTURAL CONDITIONS EVALUATION CRITERIA

Atlantic Engineering Services
6501 Arlington Expressway, Building B, Suite 201
Jacksonville, FL 32211
Phone: 904.743.4633 Fax: 904.725.9295
E-mail: jax@aespj.com

EXISTING STRUCTURAL CONDITIONS
EVALUATION CRITERIA

EXCELLENT	**Meets or exceeds current structural code requirements.** Capable of safely carrying proposed occupancies. No significant vibrations, cracking or deflections. No structural reinforcement or repairs required. Very minor, if any, maintenance required.
GOOD	**Meets current structural code requirements.** Capable of safely carrying proposed occupancies. Deflections, cracking, vibrations may be observable. No structural reinforcement required. Minor structural repairs required. Some significant maintenance repairs required.
FAIR	**Majority of structure meets structural code requirements.** Portions of structure are not capable of carrying proposed occupancies. Deflections, cracking, vibrations, structural distress is observable. Structural reinforcement required in limited portions of the structure. Structural repairs required generally. Many significant maintenance repairs required.
POOR	**Majority of structure does not meet structural code requirements.** Much of the building is not capable of carrying proposed occupancies. Deflections, cracking, vibrations, structural distress commonly observable throughout the structure. Major reinforcement or reconstruction of the structure is required. Major maintenance repairs are required.
EXTREMELY POOR	**Collapse of structure is imminent.** Structure exhibits significant deflections, cracking, vibrations, structural distress. Structure requires extensive reinforcement or reconstruction of impractical scope.

NOTE: Some parts of each definition may not apply