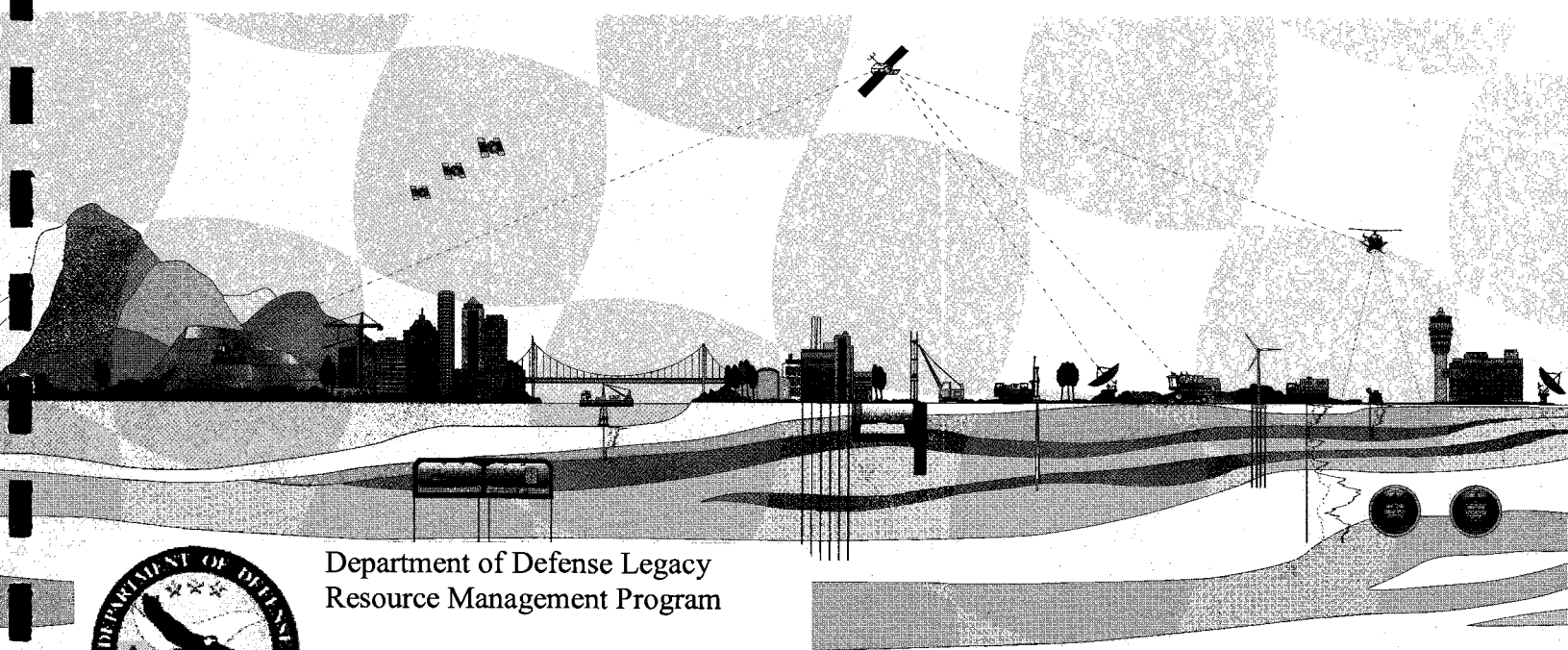




FUGRO CONSULTANTS LP

**GEOTECHNICAL STUDY
FORENSIC EVALUATION AND TESTING
OF VARIOUS STRUCTURES
FT. SAM HOUSTON, TEXAS**

3D/INTERNATIONAL, INC.
San Antonio, Texas

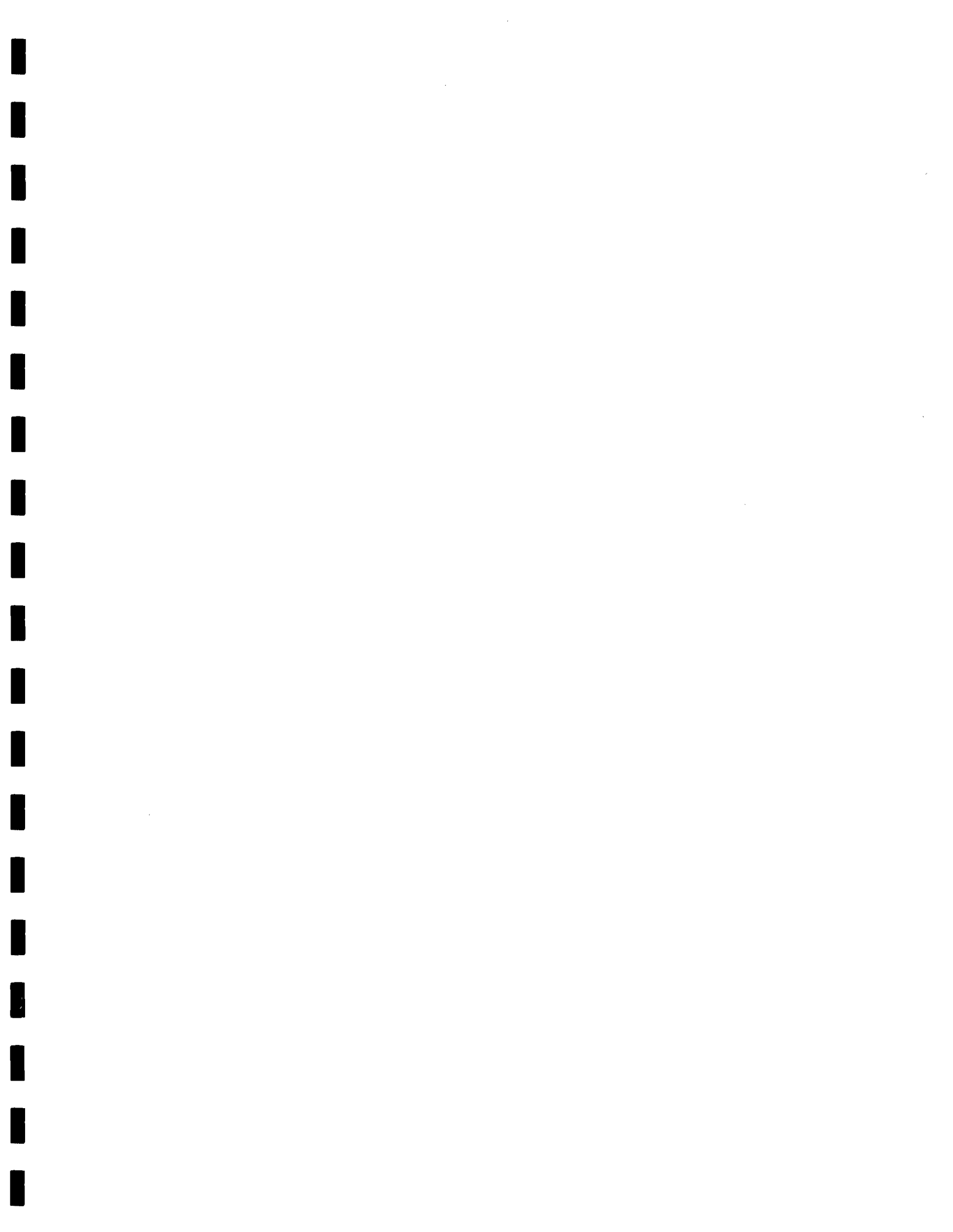


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REPORT ON GEOTECHNICAL STUDY,
FORENSIC EVALUATION, AND TESTING
OF VARIOUS HISTORIC STRUCTURES
FT. SAM HOUSTON, TEXAS

Report to:

3D/INTERNATIONAL, INC.
San Antonio, Texas

Submitted by:

FUGRO CONSULTANTS LP
February 2005

FUGRO CONSULTANTS LP



Report No. 1004-0170
February 14, 2005

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3D/International, Inc.
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Attention: Mr. Robert D. Morris

Report on Geotechnical Study, Forensic Evaluation, and Testing of Various Historic Structures Ft. Sam Houston, Texas

Fugro Consultants LP is pleased to present the results of our investigation for the above referenced project. This work was performed in general accordance with Fugro's Proposal, dated October 7, 2004, and was authorized with 3D/I Consultant Agreement dated October 19, 2004.

This report contains an evaluation of the existing foundation conditions, a limited masonry strength evaluation, and general guidelines to assist in preparing a stabilization plan. The information obtained during the field and laboratory investigation of the study is also included.

Fugro Consultants LP appreciates the opportunity to be of service to 3D/International, Inc. on this project. Please call if you have any questions, or if we may be of any additional assistance.

Sincerely,

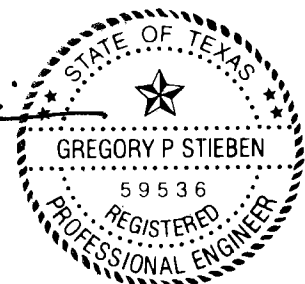
FUGRO CONSULTANTS LP

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Marcus W. Horner, E.I.T.
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Mr. Bob Morris, 3D/I (6)
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Appendix A: Elevation Survey Performed by Stephen G. Cook Engineering.

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SUMMARY

Fugro Consultants LP performed a forensic geotechnical study to evaluate the foundation movement and associated distress occurring at various historic buildings at Ft. Sam Houston, Texas. Our study included specialized testing to evaluate the structural masonry in the buildings. This report documents the study. It contains a brief synopsis of the project, field exploration and testing and laboratory tests results, and a discussion of the significance of the results. This summary provides an overview of the report and is not intended to present all pertinent information.

Subsurface soil conditions were explored by drilling nine soil borings. Laboratory tests were performed to measure the pertinent index and engineering properties of the site soils.

The principal findings developed as part of this geotechnical study are summarized below:

1. Ft. Sam Houston is located in Northeast Central San Antonio, Texas. We understand the buildings included in this study were constructed between 1876 and 1918 and are typically founded on unreinforced foundations of bonded limestone laid on unreinforced rubble concrete. Many of the buildings are undergoing foundation movements due to shrinking and swelling of expansive soils. The movements are a nuisance, and in some cases have projected cracks up into the structural brick walls.
2. Stephen G. Cook Engineering performed elevation surveys of nine buildings to document the existing floor elevations. The survey report is included in Appendix A.
3. Subsurface conditions at the site were explored by a total of nine borings. The borings encountered primarily lean to fat clays with intermittent gravel layers. The clays would be expected to have a high to very high shrink/swell potential.
4. The borings were advanced using dry auger drilling techniques. Free water was not encountered at the boring locations at this site.
5. The results of this study indicate the soils in the vicinity of the structures are expansive and affected by variations in moisture condition. Fugro recommends moisture barriers be installed along the perimeter of the buildings to help reduce moisture variations beneath the building foundations.
6. Flatjack testing and concrete core compression testing were performed to evaluate the structural properties of the construction materials. The flatjack method provides an in-situ measure of compressive stress versus deformability and, in some cases, compressive strength.



INTRODUCTION

Fugro Consultants LP is submitting this report on our study performed at Ft. Sam Houston, Texas. This work was performed in general accordance with Fugro's Proposal, dated October 7, 2004, and was authorized with 3D/I Consultant Agreement dated October 19, 2004. 3D/International, Inc., in turn, is providing an overall report concerning the structures to the United States Army Corps of Engineers.

PROJECT DESCRIPTION

General

3D/International is evaluating the condition of various historic structures located at Fort Sam Houston, Texas. A vicinity map showing the approximate location is presented on Plate 1. Lundy & Franke Engineering is providing structural engineering services on the project. We understand the buildings included in this study were constructed between 1876 and 1918 and are typically founded on unreinforced foundations of bonded limestone laid on unreinforced rubble concrete. Many of the buildings are undergoing foundation movements due to shrinking and swelling of expansive soils. Lundy & Franke is analyzing the existing construction of the buildings and the soil conditions to develop a conceptual stabilization and repair plans. 3D/International has requested Fugro provide geotechnical and materials testing for the project.

Buildings Evaluated

The study area includes about thirty-eight (38) buildings. There are about eight to nine different building types; each structure type has a similar building footprint and most are characterized by similar construction methods. Our investigation included the detailed evaluation of nine of the buildings:

- Building No. 123
- Building No. 131
- Building No. 135
- Building No. 145
- Building No. 147
- Building No. 151
- Building No. 156
- Building No. 197
- Building No. 238

The investigated buildings were selected by Mr. David Brigham with Fort Sam Houston, and are identified on Plate 2. Eight of the nine buildings we investigated are characterized by structural masonry load bearing walls supported on bonded limestone block basement walls bearing below the existing grade. Building 197 was constructed of wood framing with precast panels on the interior and exterior surfaces; the foundation of that buildings consisted of cast-in-place concrete basement walls.



Ongoing Foundation Movements

The structures have experienced differential movements resulting in various degrees of cracking in the foundation elements and structural brick walls. Fugro was contracted to perform a field exploration and laboratory testing program to explore soil stratigraphy near the structures. In addition Fugro provided a limited evaluation of the construction materials to assist Lundy & Franke with their structural evaluation of the buildings. Fugro's investigation included flatjack testing on the structural masonry of selected buildings to explore various properties of the brick and mortar elements.

PURPOSE AND SCOPE

The purposes of this study were to: perform elevation surveys of the floors; provide an evaluation of the existing subsurface conditions; perform a limited masonry strength evaluation; and develop general guidelines to assist in preparing a stabilization plan. These purposes were accomplished by performing the following scope of work:

- (1) providing a floor elevation survey of selected buildings;
- (2) drilling and sampling nine exploratory borings to explore subsurface conditions and obtain samples for laboratory testing;
- (3) performing laboratory tests on selected soil samples from the borings to evaluate the pertinent physical properties;
- (4) performing flatjack testing on selected masonry walls; and
- (5) presentation of field and laboratory data and engineering recommendations in a report.

Field sampling, laboratory testing, and soil classifications and descriptions were in general accordance with methods, procedures, and practices set forth by the American Society for Testing and Materials, 2003 Annual Book of ASTM Standards, where applicable.

ELEVATION SURVEY

Stephen G. Cook Engineering was contracted to perform a limited elevation survey of the various buildings previously identified. The survey was performed between November 15 and 22, 2004. The survey generally consisted of a limited number of survey points in areas accessible to the survey crew. The findings of the survey are presented in Appendix A. Electronic files in 'pdf' format and AutoCad 2005 drawing file formats are included in the CD appended to this report.

GEOTECHNICAL STUDY

Field Investigation

Subsurface Exploration. Subsurface exploration was performed on December 13, 2004. The field program consisted of a total of nine subsurface exploration borings advanced and sampled to 20 ft. Total footage explored was 180 ft. The borings were drilled throughout the site at the approximate locations illustrated on Plate 2.

The boring locations were selected by Fugro's project engineer to provide general coverage of the site. The borings were staked by Fugro's field crew measuring from known landmarks at the site.

Boring Method and Sampling. The sample borings were drilled to their completion depths using dry auger drilling techniques. The borings were drilled with a truck-mounted rig equipped with the following sampling tools: (1) continuous flight augers for advancing the holes dry and recovering disturbed samples; (2) thin-walled tubes for obtaining undisturbed samples of cohesive strata (ASTM D 1587), and; (3) split-barrel samplers and drive weight assembly for obtaining representative samples and measuring the penetration resistance (N-values) of non-cohesive soil strata (ASTM D 1586).

Soil samples were recovered in general accordance with applicable American Society for Testing and Materials (ASTM) standards. In general, soil samples were obtained at about 2-ft intervals to 10-ft depth, and then at 5-ft intervals thereafter to 20-ft depth. Thin-walled tubes were used for obtaining undisturbed samples of cohesive strata (ASTM D 1587), and a split-barrel sampler and drive weight assembly was used to obtain representative samples and measure the Standard Penetration Test (SPT) N-values of noncohesive and very hard strata (ASTM D 1586).

The SPT N-value is the number of blows of a 140-lb drop hammer falling about 30 inches required to drive the SPT sampler the final 12 inches of an 18-inch sampling interval. The blows required for the first 6 inches of sampler penetration (seating) are usually not considered representative of in situ densities due to the possible presence of loose material or cuttings from the drilling operations. Where very dense material is encountered, the actual penetration after the initial 6 inches seating of the sampler is recorded for a maximum total of 50 blows.

Sample Handling. After recovery, each sample was removed from the sampler, examined, and visually classified by our geotechnical technician. The samples were sealed in plastic bags to preserve the moisture content and then transported to Fugro's San Antonio laboratory for further examination and testing. Prior to laboratory testing, the samples were maintained in a climate-controlled environment to reduce the potential for moisture changes.

A record of field observations was maintained in the form of field logs visually describing the subsurface materials encountered, and other pertinent field data. Hand penetrometer values are also included on the boring logs. The hand penetrometer provides a measure of the compressive strength in tsf, which is equivalent to the undrained shear strength in kips per square foot (ksf). These logs were later edited to incorporate information obtained from laboratory examination and testing. The final logs for Borings 1 through 9 are presented on Plates 3 through 11, respectively. Keys to symbols and terms used on the logs are presented on Plate 12.

Depth to Water. The borings were advanced without the use of drilling fluids. At the completion of the field exploration, the boreholes were sounded for groundwater using a weighted measuring tape. Any depth to water measurements are recorded on the boring logs.

Laboratory Testing

General. The laboratory testing program was directed toward identification and classification of the foundation soils and evaluation of the unconfined shear strength. To aid in soil classification, liquid and plastic limits, collectively termed Atterberg limits, were performed on selected cohesive soil samples. The Atterberg limits are used to classify the soil types according to the Unified Soil Classification System. The Atterberg limits may be used to provide a general indication of the materials' potential for volumetric change (shrink/swell) with moisture variations.

Water content measurements were made on many of the samples to help establish the moisture content profile for each boring. The undrained shear strengths of selected samples were measured by performing unconfined compression tests; moisture content and unit dry weight were measured as routine portions of the compression tests. The results of the laboratory classification and strength tests are presented on the individual boring logs on Plates 3 through 11.

Free swell tests were performed on selected samples as a method to evaluate the shrink/swell characteristics of the soils at the site. In the free swell test, a specimen is placed in a consolidometer and loaded to a nominal seating pressure, which is approximately equal to the overburden pressure. The specimen is inundated, permitted to absorb moisture, and the amount of swell is then recorded. The swell test results are discussed later in this report.

Review. Descriptions of strata made in the field at the time the borings were drilled were modified in accordance with results of laboratory tests and visual examination in the laboratory. All recovered soil samples were examined, classified and described in accordance with ASTM D 2487, ASTM D 2488 and Unified Soil Classification procedures. Classifications of the soils and finalized descriptions of soil strata are shown on the attached boring logs.

Generalized Site Conditions

Site Description. The project consists of about thirty-eight (38) historic structures located on Fort Sam Houston, Texas. Fort Sam Houston is located in northeast central San Antonio, Texas. We understand the buildings included in this study were constructed between 1876 and 1918 and are typically founded on unreinforced foundations of bonded limestone laid on unreinforced rubble concrete. The buildings are generally located along Stanley Road west of North New Braunfels.

There are about eight to nine different building types; each structure type has a similar building footprint and is characterized by similar construction methods. The site topography generally drains to the south and west. The existing buildings vary in size and shape and are typically surrounded by landscaped areas vegetated with grasses.

Site Geology. A review of available geologic information,¹ indicates that the project site is located on Alluvial soils over gravels of the Uvalde Gravel Formation overlying clays of the Navarro formation.

The alluvium soils are floodplain deposits and consist primarily of clays containing various amounts of silt, sand, and gravel. The Uvalde Gravel consists primarily of gravel and cobble-sized particles of chert, quartz, limestone, and igneous rock. The material is often cemented with calcium deposits and is typically quite dense. Oftentimes, the gravel is water bearing.

Navarro Group clays generally consist of the lower part of the formation and are composed of dominantly montmorillonitic, greenish-gray to brownish-gray clay, which weathers to a black clay. The clays are generally fairly strong, but can exhibit a high shrink/swell potential. The deeper unweathered portions of the Navarro consist of gray clay shale. The clay shale is strong and often exhibits a natural petroleum odor.

Stratigraphy. Subsurface conditions were explored at the site by nine borings. The subsurface conditions have been generalized into the following major strata:

<u>Stratum</u>	<u>Description</u>
I	Hard Dark Brown Fat Clay (Alluvium)
II	Dense Sandy Gravel (Uvalde Gravel)
III	Hard Tan and Gray Lean Clay (Weathered Navarro)
IV	Hard Tan and Gray Fat Clay (Navarro)

¹ The University of Texas at Austin Bureau of Economic Geology, (1983), "Geologic Atlas of Texas, San Antonio Sheet".

Stratum I. Stratum I is an alluvial deposit composed of a dark brown fat clay. At three locations (Borings 2, 5, and 8), the surficial soils consisted of fill material. This material is highly-plastic (CH) based on measured liquid limits ranging from 59 to 67 and plasticity indices (PI's) of 39 to 48. Based on pocket penetrometer readings and compression tests, the clay is generally very stiff to hard in consistency. The plasticity characteristics would suggest the near surface clays would be expected to have a very high potential for volume change (shrink/swell) resulting from moisture fluctuations².

Stratum II. Stratum II consists of Uvalde Gravel. The material was encountered at 2- to 4-ft depth and varied in thickness from 2 to 6 ft. The material is sandy and is very dense in condition. At the boring locations the gravel was not cemented, but it should be noted that cementation of this formation is common and can vary significantly over short distances. This gravel is often water bearing.

Stratum III. A tan and gray low plasticity clay (CL) was encountered below the Stratum II gravels. The soil contained numerous calcareous deposits and pockets. The material has measured liquid limits ranging from 30 to 50 and plasticity indices (PI's) of 13 to 36. The clay is generally hard in consistency based on the pocket penetrometer estimates and compression test results. Strength test results indicated strengths ranging from 1.3 to 1.9 ksf; the low strengths are likely due to failure along the calcareous zones. Based on correlations with the plasticity characteristics, the Stratum III soils would be expected to have a medium potential for volume change (shrink/swell) resulting from moisture fluctuations.

Stratum IV. The deepest stratum encountered at the site was clay of the Navarro Group. The material is tan and gray in color and is hard in consistency. Strength test results indicated shear strengths ranging from 2.2 to 9.9 ksf. The clay is highly-plastic (CH) based a measured liquid limits of 50 to 69 and plasticity indices of 33 to 50. The fat clays would be expected to have a very high shrink/swell potential; however, the overburden stress will tend to suppress such movements.

Shrink/Swell Potential. To further evaluate the swelling potential of the site soils under their current moisture conditions, a series of free swell tests were performed on selected samples from the borings. Given a free source of water and under the estimated in situ overburden stress, the following swell values were measured:

² Peck, R.B., Hanson, W.E., and Thornburn, T.H., (1974) Foundation Engineering, Second Edition, John Wiley & Sons, Inc., New York, Pg. 337.

Boring	Depth, ft	PI	Moisture Content, %		Liquidity Index		Free Swell, percent
			Initial	Final	Initial	Final	
1	0 to 2	39	28	31	+0.21	+0.28	0.2
2	13 to 15	44	20	21	+0.07	+0.09	0.9
4	4 to 6	33	26	27	+0.36	+0.39	0.2
4	8 to 10	45	17	24	+0.02	+0.18	4.0
5	6 to 8	13	18	19	+0.08	+0.15	0.0
6	0 to 2	48	27	29	+0.17	+0.21	0.1
8	6 to 8	44	20	24	+0.09	+0.18	0.0

As indicated above, the tested samples typically displayed little to no swell. However, the 8- to 10-ft sample from Boring 4 indicated a higher swell value of 4 percent.

Groundwater. The borings were advanced without the use of drilling fluids. During the drilling and sampling operations, the borings were observed for signs of groundwater. No free groundwater was observed in the borings during the drilling of the borings. It should be noted that fluctuations in groundwater level may occur, and the groundwater level may rise during extended periods of precipitation.

Variations in Subsurface Conditions. Subsurface conditions have been explored at the boring locations only. Sound geotechnical practice requires that some mention be given to the fact that since some variation was found in subsurface conditions, all parties should recognize that even more variation might be possible at other locations. In addition, the soil stratigraphy described above is based on interpretation of our technician's observations during sampling, and classification of the soil samples. The boundaries between soil layers are approximate, and transitions between soil types may be gradual.

Discussion of Study

Foundation Types. We understand the buildings included in this study were constructed between 1876 and 1918. Eight of the nine buildings we evaluated consisted of structural masonry walls founded on bonded limestone block walls. The limestone blocks extended below grade and also serve as basement walls. Lundy & Franke Engineering subcontracted an excavation contractor to expose the base of the foundations. We understand Lundy & Franke will be reporting the findings and observations of the excavations by as part of their site evaluation.

Foundation Movements. The foundation walls were founded at relatively shallow depths. The age of the buildings would indicate the foundations have undergone numerous wetting and drying seasonal moisture fluctuations. This would likely include the extreme range from drought conditions and prolonged wet periods. The moisture fluctuation in the foundation soils would likely

have resulted in their shrinking and swelling, accompanied by cracking of the foundation elements and brick walls.

The two most common causes of foundation movement in the central and south Texas areas are: 1) settlement of the underlying soils (typically due to inadequately-compacted fill material used to raise site grade during construction) and 2) shrinkage or swelling of the underlying soils resulting from moisture changes. Since settlement would have occurred far in the past, shrinkage or swelling of the underlying soils resulting from moisture changes is likely the cause of the movements.

Shrink/Swell Movements. Highly plastic clays normally have a tendency to shrink and swell with variations in soil moisture.³ Swelling of highly plastic clays is due to the clay minerals' affinity for moisture; if free water is available, the water will be adsorbed to the minerals, thereby increasing the soil volume. Low plasticity clays are generally not as susceptible to shrink/swell due to differences in soil mineralogy; granular materials (silt, sand, gravel) are not subject to shrink/swell changes.

A comparison of the soil moisture content with the Atterberg limits can provide an approximate indication of whether the highly-plastic clays are in a shrunken or in a swelled state. If the moisture content is below or near the plastic limit (liquidity index near or below zero), the soils are probably in a low state of swell. However, moisture contents significantly greater than the plastic limit (higher liquidity index) may indicate a swelled soil.⁴

The liquidity index (LI) is defined as the difference between the moisture content and the plastic limit, divided by the plasticity index. The liquidity index provides a measure of the moisture content relative to the Atterberg limits of the sample. For the same type of soil, the higher the moisture content, the higher the liquidity index. For example, an LI of zero indicates the moisture content is equal to the plastic limit; an LI of +0.50 indicates the moisture content is halfway between the plastic limit and liquid limit. A negative LI indicates that the moisture content is less than the plastic limit.

Our experience in the San Antonio area has been that liquidity index values typically display an overall range between about -0.20 and +0.30. That is, the soil is probably in a non-swelled state if its LI is close to -0.20, and it is likely in a fully-swelled condition if its LI is on the order of +0.30.

The amount of foundation movement associated with soils changing from a non-charged to a charged state depends on: the clay mineralogy, the moisture content of the soil at the time of construction, the unit weight of the soil, the thickness of the active zone, the overburden provided

³ Peck, et al.

⁴ Sowers, G.F. and Kennedy, C.M., (1967) "High Volume Change Clays of the Southeastern Coastal Plain", *Third Pan-American Congress of Soil Mechanics and Foundation Engineering*, Caracas, Venezuela, Volume II.

by the structure, and the amount of free water available. It is difficult to accurately estimate potential slab movement resulting from shrinkage or swelling, but for highly-plastic soils, movement on the order of several inches is possible.⁵ Lean clays would be expected to produce less movement.

Evaluation of Shrink/Swell Condition. The computed LI values have been plotted versus depth on Plate 13. The computed liquidity index values of the tested samples ranged from -0.38 to $+0.35$, which indicates highly varying states of swell.

In general, the LI values were higher in the upper 10 ft and typically ranged from $+0.08$ to $+0.35$. The clay samples near the gravel interfaces also typically had higher LI's. These values would suggest the samples are in moderate to high state of swell near the surface.

The lower plasticity samples and the granular soils had negative LI values; high variation of LI's is common in low plasticity clays. Also, the samples tested for plasticity at the Boring 9 location and somewhat lower LI values of -0.12 and -0.14 ; those values indicating the clays at the Boring 9 location are in a low state of swell.

Vegetation Effects. Soil moisture can also be affected by vegetation.^{6,7} Trees and shrubs draw moisture from the soil through their root systems, causing localized drier areas in their vicinity. The fast growing varieties generally create the greatest demand on soil moisture. The radius of effect of a tree is related primarily to the lateral extent of its root system, which has some relationship to tree height and the spread of its branches. In general, it may be assumed that a root system has a significant effect out at least as far as the drip line (extent of branches). If the moisture withdrawn by vegetation is not replenished by precipitation or irrigation, highly plastic clays will dry and shrink. Foundations built over the affected area may lose support and settle; severe cracking can result. Examples of such movement can be seen in the "roller coaster" effect of curbs and paving along tree-lined streets and in concentric crack patterns in street pavement near large trees.

5 Terzaghi, K. and Peck, R.B., (1967) Soil Mechanics in Engineering Practice, Second Edition, John Wiley & Sons, Inc., New York, Pg. 146.

6 Perkins, R.L. and Elsbury, B.R., (1981) "Structural Damage due to Soil Volume Change", Reprint from Soundings, McClelland Engineers, Inc., Houston, winter edition.

7 Peck, et al, pg. 343.

Recommendations

Several of the buildings have undergone significant foundation movements. The moisture condition of the soils at the time of this study would suggest the near-surface site soils were in a high state of swell; the soils are susceptible to undergo additional shrinkage movements with a decrease in moisture. We have developed our recommendations to account for these potential movements.

Fugro recommends Fort Sam Houston:

1. continue periodic floor elevation surveys to identify areas subject to seasonal moisture variations;
2. install moisture protection barriers around selected areas of the buildings to stabilize the soil moisture levels, and;
3. perform cosmetic repairs after foundation movements have been normalized.

Floor Elevation Surveys. Fugro recommends the owners continue monitoring the floor elevations. The observed foundation movements at the site are likely due to shrinkage or swelling of the underlying soils resulting from moisture changes.

The shrinking and swelling of the soils will continue with seasonal moisture fluctuations. The amount of foundation movements will depend on various factors, including the climatic environment, building locations, site-specific soil conditions, and groundwater flows, among others.

The purpose of Fugro's monitoring program is to identify the areas that are most susceptible to shrink/swell movements. Mitigating moisture migration and maintaining a uniform soil moisture condition under the foundations are likely the most effective remedial measures to improve the foundation conditions and reduce future shrink/swell movements.

Perimeter Moisture Protection. The near-surface foundation soils are currently in a high state of swell. Future foundation movements due to shrinking and swelling of the soils can be reduced by protecting the soils from future moisture fluctuations. Fugro's repair design methodology is based on maintaining the existing moisture conditions of the subgrade soils.

The moisture levels beneath the buildings can be controlled by installing moisture barriers around the building perimeters to reduce future moisture variations beneath the buildings. The site improvements should include maintaining proper drainage to prevent ponding in the vicinity of the buildings. Additionally, trees and other vegetation capable of withdrawing moisture from the soil should be kept at a distance from the buildings and foundations equal to at least three-quarters of their ultimate height.

The results of our geotechnical investigation indicated the soils in the vicinity of the structures are expansive and affected by variations in moisture condition. A moisture barrier is recommended along the perimeter of the buildings to help reduce moisture variations beneath the building foundations. The barrier may be provided by either a vertical or lateral system. A vertical barrier may be achieved with a slit trench backfilled with a grout, lean concrete, or plastic sheeting.

A vertical barrier should be installed to at least 4 ft below deepest adjacent foundation element. A lateral barrier may consist of flatwork and/or paving extending a minimum distance of 8 ft beyond the building; the flatwork should be installed to ensure positive drainage away from the building and to allow for some future movements. Plates 14 and 15 provide general details of two configurations of moisture barriers.

Foundation Repair. The foundation movements observed at the site will are likely due to seasonal moisture variations. The movements are, more likely than not, a result of shrinking and swelling of the supporting soils and will likely not warrant extreme repairs and renovations.

Typical repair methods include jacking or lifting to re-level areas with excessive floor slopes. The foundations consist of un-reinforced block walls and are not favorable for underpinning. Considerable structural reinforcements will need to be evaluated to even consider foundation leveling. If the structural engineer deems that structural repairs are required, Fugro recommends repairs be done only after it has been determined that the moisture levels in the foundation soils have been stabilized. A detailed remedial design can be prepared at a later date.

MASONRY TESTING

General

Flatjack testing was performed on the brick walls at one location on each of the eight masonry buildings. The test locations are indicated on Plate 2. At Building 197, which was not constructed of brick, three concrete cores were recovered from the basement wall for strength testing. This section provides a discussion of the field testing and a discussion of the significance of the test results.

Flatjack Test Method

We performed flatjack testing on the exterior masonry walls at one location on each of the eight masonry buildings to evaluate the condition of the masonry in the structures. The flatjack method can be used to provide an in-situ measure of the compressive stress, deformability, and possibly compressive strength. Testing was performed in general accordance with ASTM C 1197.

Different test setups are used to determine in-situ compressive stress and compressive stress versus deformability and strength. The testing procedures used in this study was directed towards establishing deformability and strength parameters. We began by installing a series of brass pins in two brick courses about five courses apart. Horizontal slots were then saw cut into the mortar joints above and below the reference pins. A flatjack was placed in each slot. (The actual flatjacks consist of two thin stainless steel plates welded around their perimeters to provide a hydraulic seal. A flatjack has a pressurization port and a bleed port to allow the space between the plates be pressurized with hydraulic fluid.) The two flatjacks were simultaneously pressurized; the pressure readings and the spacing of the reference points were recorded.

Curves presenting the pressure versus computed strain in the "in situ prism" of bricks are presented on Plates 16 through 23. The compression moduli were approximated by fitting a line to the stress-strain curves produced during testing. ASTM suggests that the modulus produced by flatjack testing is as much as 15% higher than compression modulus determined with prism samples.⁸

If the pressurization of the flatjacks is carried to higher stress, eventually the "in situ prism" will fail, giving the strength of the masonry, f'_m . We limited the applied stress in an attempt to reduce damage to the wall and flatjacks. Mr. Shawn Franke, P.E. of Lundy & Franke Engineers specified a max-applied load of 600 psi to evaluate the condition of the masonry. In some cases we stopped the pressurization prior to rupture as the deformation curve appeared to undergo plastic deformation.

Repair of Test Areas

The work areas were cleaned prior to our field crew's departure. We understand the test areas were to be repaired by Curtis Hunt Restorations. The repairs generally consisted of re-pointing the mortar slots and the reference point holes. Also, the brick damaged at three places during testing were to be repaired.

Flatjack Test Results

General. The results of the testing are presented on Plates 16 through 23. The resulting slope of the load curves provided the deformability (modulus) of the masonry. In most cases, we limited the applied stress in an attempt to reduce damage to the wall and flatjacks. General failure type, approximated compression modulus, and the maximum pressure applied are indicated in the following table:

⁸ American Society for Testing and Materials, Annual Book of ASTM Standards, "Chemical-Resistant Nonmetallic Materials; Vitrified Clay Pipe; Concrete Pipe; Fiber-Reinforced Cement Products; Mortars and Grouts; Masonry", Volume 04.05, 2002, West Conshohocken, PA., pg. 769.



<u>Building No.</u>	<u>Compression Modulus (psi)</u>	<u>Compressive Stress* (psi)</u>	<u>Failure Notes</u>
123	257,143	250	No Failure, Plastic Strain
131	618,182	500	Yes, Failure Above Test Section
135	461,538	484	No Failure, insufficient overburden
145	844,444	600	No Failure, Capacity achieved
147	550,000	600	No Failure, Capacity achieved
151	240,000	400	No Failure, insufficient overburden
156	200,000	350	Failure, Below Test Section
238	333,333	300**	Failure, Test section failed

* Pressure corrected for flatjack stiffness

** Ultimate strength achieved

Discussion. The following provides discussion of the flat jack test results:

- At two of the test locations (Buildings No. 135, 151), testing was stopped when peripheral observations indicated that damage to the wall was impending. In some cases separation in the mortar above the test area indicated that the test section was lifting a large section of the wall, which implies that the section being tested is capable of withstanding much higher loads than they were bearing in situ (prior to testing).
- The testing prism failed at the Building 238 location. At that particular test location, the test provided a compressive strength (f_m) of 300 psi.
- Although the in-situ prism did not rupture, the masonry above or below the test section spalled at two of the locations (Buildings No. 131 and 156).
- In one location (Building 123) loading was stopped when the stress strain curve appeared to enter the plastic range.
- The maximum target load provided by Lundy and Franke was achieved at two of the test locations (Buildings 145 and 147).

Concrete Coring

Building 197 was constructed of wood framing with precast panels on the interior and exterior surfaces; the foundation of that building consisted of cast-in-place concrete basement walls. Fugro sampled concrete cores of the basement wall to investigate the compressive strength of the concrete. Three samples were cored and transported to our laboratory for compression testing. Testing was performed in accordance with ASTM C-42. The test results indicated compressive strengths ranging from 2,000 to 2,210 psi and averaging 2,120 psi. Results of the concrete compression tests are presented on Plate 24.



CONDITIONS

Since some variation was found in subsurface conditions at boring locations, all parties involved should take notice that even more variation may be encountered between boring locations. Statements in the report as to subsurface variation over given areas are intended only as estimations from the data obtained at specific boring locations.

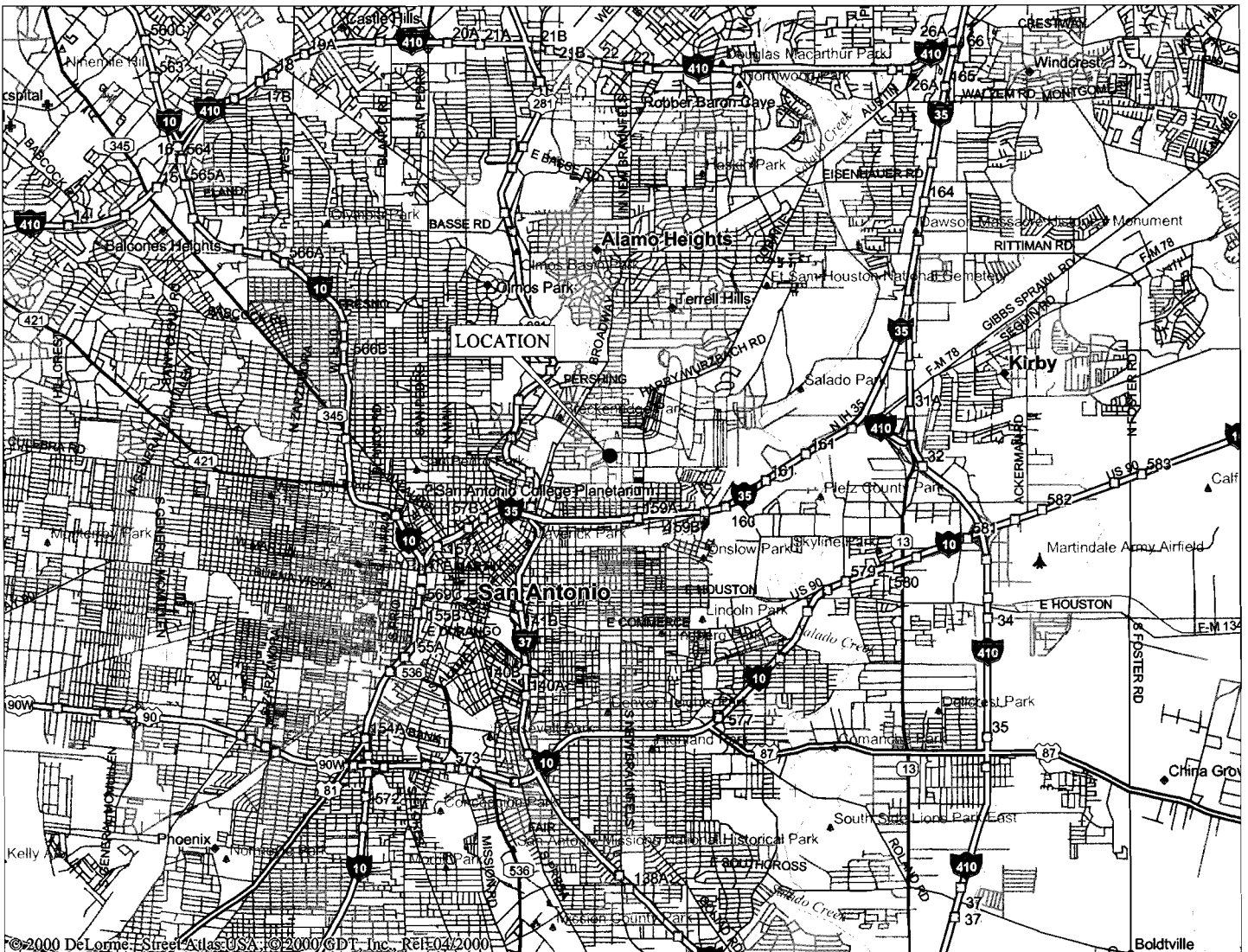
The professional services that form the basis for this report have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in the same locality. No other warranty, expressed or implied, is made as the professional advice set forth.

The results, conclusions, and recommendations contained in this report are directed at, and intended to be utilized within, the scope of work contained in the agreement executed by Fugro Consultants LP and client. This report is not intended to be used for any other purposes. Fugro Consultants LP makes no claim or representation concerning any activity or condition falling outside the specified purposes to which this report is directed, said purposes being specifically limited to the scope of work as defined in said agreement. Inquiries as to said scope of work or concerning any activity or condition not specifically contained therein should be directed to Fugro Consultants LP, for a determination and, if necessary, further investigation.

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PLATES

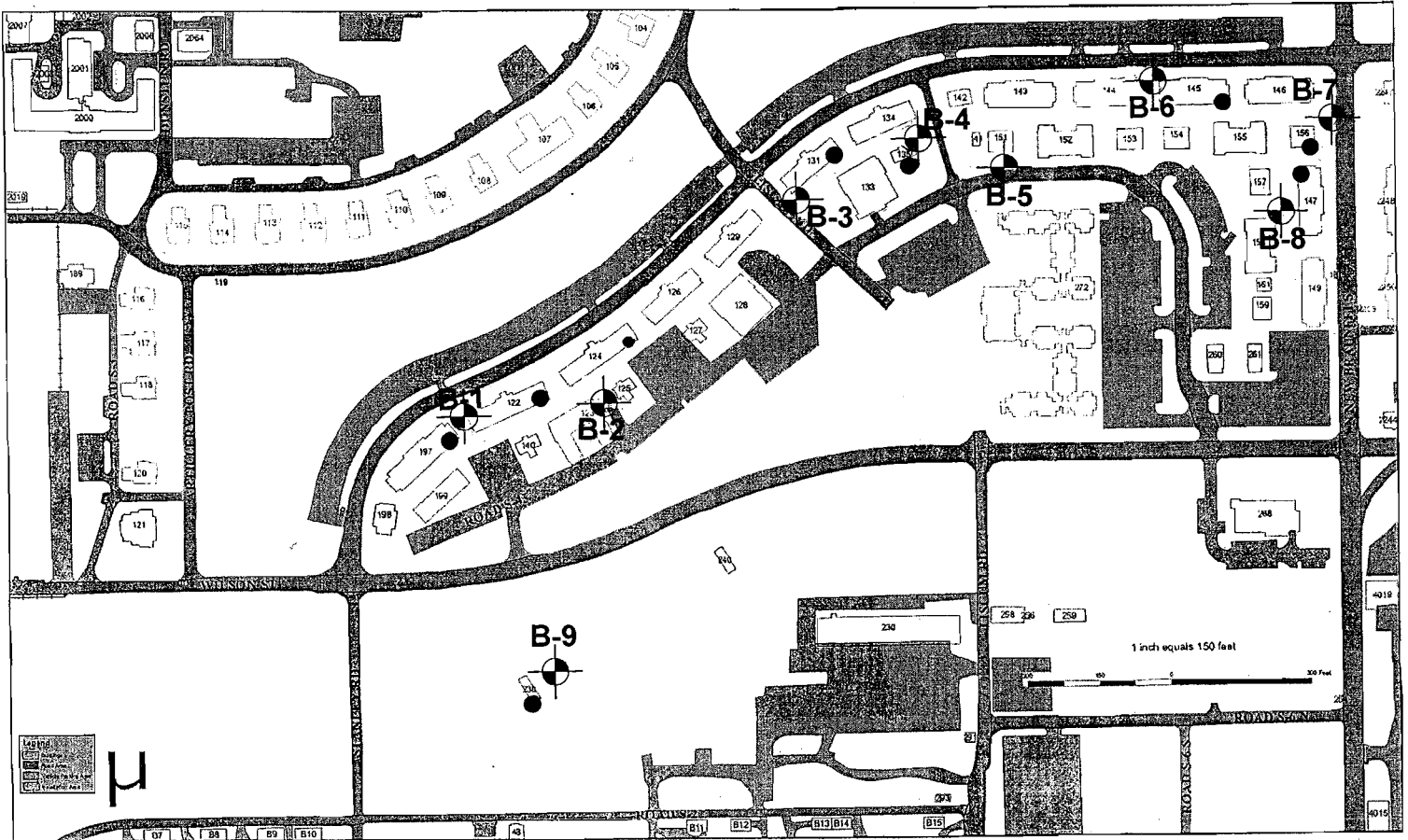


Vicinity map taken from Delorme Street Atlas USA® (800) 452-5931

Scale 1 inch = 2 miles



**VICINITY MAP
FORENSIC EVALUATION AND TESTING
OF VARIOUS STRUCTURES
FT. SAM HOUSTON, TEXAS**





Not to Scale

Legend

-  Borings
-  Flat Jack Tests

**PLAN OF TEST LOCATIONS
FORENSIC EVALUATION AND TESTING
OF VARIOUS STRUCTURES
FT. SAM HOUSTON, TEXAS**



LOG OF BORING NO. 1
FORT SAM HOUSTON HISTORIC BUILDING EVALUATION
FORT SAM HOUSTON, TEXAS

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(RQD), %	STRATUM DESCRIPTION	DEPTH, ft	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, pcf	LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf
				CLAY (CH), brown, fat, hard, with roots		29	59	20	39			+0.23	
			47	GRAVEL (GC), brown and tan, dense, clayey, with chert	2.0	22							
5				CLAY (CL), tan and light gray, lean, very stiff, with ferrous and many calcareous deposits -hard below 6'	4.0	25	50	17	33			+0.24	3.5 (P)
													4.5+ (P)
10						19	40	14	26			+0.19	4.5+ (P)
15				CLAY (CL), tan and light gray, lean, hard, with calcareous deposits	14.0	19					111		4.0 (U)
20						14	46	14	32			0.00	4.5 (P)
					20.0								
				Note: No free groundwater was observed.									
25													
30													

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
 Q = Unconsolidated-Undrained Triaxial

P = Pocket Penetrometer
 T = Torvane



LOG OF BORING NO. 2

FORT SAM HOUSTON HISTORIC BUILDING EVALUATION

FORT SAM HOUSTON, TEXAS

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(RQD), %	STRATUM DESCRIPTION	DEPTH, ft	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, pcf	LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf
			22	GRAVEL (GC), brown, medium dense, clayey (possibly fill)		19	48	16	32			+0.09	
			50	-dark brown below 2'		27							
5			24	CLAY (CL), tan, lean, very stiff, with many calcareous deposits	3.0								
				CLAY (CL), tan and light gray, lean, very stiff, with calcareous deposits	6.0	21	48	15	33			+0.18	2.8 (P)
						25					104		1.6 (U)
10				CLAY (CH), tan and light gray, fat, hard, with ferrous deposits	10.0								
						20	61	17	44			+0.07	4.5+ (P)
15													
													4.5+ (P)
20					20.0								
				Note: No free groundwater was observed.									
25													
30													

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
Q = Unconsolidated-
Undrained Triaxial

P = Pocket Penetrometer
T = Torvane

LOG OF BORING NO. 3

FORT SAM HOUSTON HISTORIC BUILDING EVALUATION

FORT SAM HOUSTON, TEXAS

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(RGD), %	STRATUM DESCRIPTION	DEPTH, ft	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, pcf	LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf
				CLAY (CH), brown, fat, hard, with gravel -with roots to 2'		27	66	22	44			+0.11	4.5+ (P)
			19	GRAVEL (GC), tan and brown, medium dense, clayey, with chert	2.0	19							
5				CLAY (CH), tan and light gray, fat, very stiff, with ferrous and many calcareous deposits	4.0	20					112		1.9 (U)
						20	55	14	41			+0.15	3.3 (P)
						22					105		3.1 (U)
10				CLAY (CH), tan and light gray, fat, very stiff, with calcareous deposits	9.0								
				-hard below 13'		21	61	16	45			+0.11	4.5+ (P)
15													
						21					107		5.3 (U)
20					20.0								
				Note: No free groundwater was observed.									
25													
30													

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
Q = Unconsolidated-
Undrained Triaxial

P = Pocket Penetrometer
T = Torvane



LOG OF BORING NO. 4

FORT SAM HOUSTON HISTORIC BUILDING EVALUATION

FORT SAM HOUSTON, TEXAS

TYPE: Flight Auger

LOCATION: See Plate 2

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(ROD), %	STRATUM DESCRIPTION	DEPTH, ft	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, pcf	LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf
	[Diagonal Hatching]			CLAY (CH), brown, fat, very stiff, with gravel		30							
	[Stippled]		19	GRAVEL (GC), tan and brown, medium dense, clayey, with chert	2.0	31							
5	[Diagonal Hatching]			CLAY (CL), tan, lean, hard, with many calcareous deposits	4.0	26							4.5+ (P)
						14	47	14	33			0.00	4.5+ (P)
				-fat (CH) below 8'	8.0	18	61	16	45			+0.04	4.5+ (P)
10	[Diagonal Hatching]			CLAY (CH), tan and light gray, fat, hard, with calcareous deposits	10.0								
													4.5+ (P)
15	[Diagonal Hatching]			-with ferrous deposits at 15'									
20	[Diagonal Hatching]				20.0	19	54	15	39			+0.10	4.5+ (P)
				Note: No free groundwater was observed.									

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
 Q = Unconsolidated-Undrained Triaxial

P = Pocket Penetrometer
 T = Torvane



LOG OF BORING NO. 5
FORT SAM HOUSTON HISTORIC BUILDING EVALUATION
FORT SAM HOUSTON, TEXAS

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(ROD), %	STRATUM DESCRIPTION	DEPTH, ft														
						WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, pcf	LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf						
				CLAY (CL), tan, lean, hard (possibly fill)															
			15	-tan, very stiff, clayey gravel at 4'		9	27	14	13										
			10	CLAY (CH), dark brown, fat, stiff	4.0	19													
5				CLAY (CL), tan and light gray, lean, hard, with many calcareous deposits	6.0	33	61	18	43	75		+0.35							
				CLAY (CH), tan and light gray, fat, hard	10.0	18	30	17	13			+0.08	3.3 (P)						
10						21					109		1.9 (U)						
						24	67	17	50			+0.14	4.5+ (P)						
15																			
20					20.0													4.5+ (P)	
				Note: No free groundwater was observed.															
25																			
30																			

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
 Q= Unconsolidated-
 Undrained Triaxial

P = Pocket Penetrometer
 T = Torvane



LOG OF BORING NO. 6
FORT SAM HOUSTON HISTORIC BUILDING EVALUATION
FORT SAM HOUSTON, TEXAS

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(RQD), %	STRATUM DESCRIPTION	DEPTH, ft	SOIL TEST RESULTS				LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf
						WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %		
0 - 15	[Symbol: Diagonal lines]	15		CLAY (CH), brown, fat, very stiff, with gravel	0 - 15	30	67	19	48	+0.23	3.0 (P)
15 - 40	[Symbol: Cross-hatch]	40		GRAVEL (GC), tan, dense, clayey	15 - 40	10	69	24	45	-0.31	
40 - 7.0	[Symbol: Stippled]	21		CLAY (CL), tan and light gray, lean, very stiff, with many calcareous deposits	40 - 7.0	23	41	15	26	+0.31	3.8 (P)
7.0 - 10.0	[Symbol: Horizontal lines]			CLAY (CH), tan and light gray, fat, hard, with calcareous deposits	7.0 - 10.0	20				107	2.2 (U)
10.0 - 20.0	[Symbol: Diagonal lines]				10.0 - 20.0	24	69	19	50	+0.10	4.5+ (P)
20.0 - 30.0	[Symbol: Diagonal lines]				20.0 - 30.0						

Note: No free groundwater was observed.

COMPLETION DEPTH: 20.0 #
 DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
 Q = Unconsolidated-
 Undrained Triaxial

P = Pocket Penetrometer
 T = Torvane



LOG OF BORING NO. 7

FORT SAM HOUSTON HISTORIC BUILDING EVALUATION FORT SAM HOUSTON, TEXAS

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(ROD), %	STRATUM DESCRIPTION	DEPTH, ft	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, pcf	LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf
				CLAY (CH), dark brown, fat, hard, with gravel		26					97		4.2 (U)
				GRAVEL (GC), tan and brown, dense, clayey, with chert	2.0								
5			38	CLAY (CL), tan, lean, hard, with many calcareous deposits -very stiff below 6'	4.0	20	47	15	32			+0.16	4.5+ (P)
						24					104		1.8 (U)
						20	43	14	29			+0.21	
10				CLAY (CL), tan and light gray, lean, hard, with calcareous deposits	10.0								
						16					115		9.9 (U)
15													
						16	49	13	36			+0.08	3.4 (U)
20				Note: No free groundwater was observed.	20.0								
25													
30													

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
Q = Unconsolidated-
Undrained Triaxial

P = Pocket Penetrometer
T = Torvane



**LOG OF BORING NO. 8
FORT SAM HOUSTON HISTORIC BUILDING EVALUATION
FORT SAM HOUSTON, TEXAS**

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(RQD), %	STRATUM DESCRIPTION	Layer Elev./ Depth	WATER	LIQUID	PLASTIC	PLASTICITY	PASSING NO.	UNIT DRY	UNDRAINED
						CONTENT, %	LIMIT, %	LIMIT, %	INDEX (PI), %	200 SIEVE, %	WEIGHT, pcf	SHEAR STRENGTH ksf
			18	SAND (SC), brown and tan, medium dense, clayey, with gravel (possibly fill)		26						
			25	GRAVEL (GC), brown and tan, medium dense, clayey	2.0	22				18		
5				CLAY (CL), tan and light gray, lean, hard, with many calcareous deposits	4.0	21	33	16	17			4.5+ (P)
				-fat (CH), 6' to 8'	6.0	20	60	16	44			4.0 (P)
					8.0	20	32	16	16			1.3 (U)
10				CLAY (CH), tan and light gray, fat, hard	10.0							
												4.5+ (P)
15												
20						23					107	3.4 (U)
					20.0							
				Note: No free groundwater was observed.								
25												
30												

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
Q = Unconsolidated-Undrained Triaxial

P = Pocket Penetrometer
T = Torvane



LOG OF BORING NO. 9
FORT SAM HOUSTON HISTORIC BUILDING EVALUATION
FORT SAM HOUSTON, TEXAS

TYPE: *Flight Auger*

LOCATION: *See Plate 2*

DEPTH, ft	SYMBOL	SAMPLES	BLOWS PER FOOT OR REC/(RQD), %	STRATUM DESCRIPTION	DEPTH, ft	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, pcf	LIQUIDITY INDEX (LI)	UNDRAINED SHEAR STRENGTH ksf	
			19	GRAVEL (GC), tan, medium dense, with clay -very dense, below 2'		13								
			50				24			28				
5			40				5							
				CLAY (CL), tan, lean, hard, with many calcareous deposits	6.0	9	50	14	36			-0.14	4.5+ (P)	
							11				116		4.4 (U)	
10				CLAY (CH), tan and light gray, fat, hard	10.0									
													4.5+ (P)	
15														
						14	51	18	33			-0.12	4.5+ (P)	
20				Note: No free groundwater was observed.	20.0									
30														

COMPLETION DEPTH: 20.0 ft

DATE: 12-13-05

PROJECT NO. 1004-0170

U = Unconfined
 Q = Unconsolidated-Undrained Triaxial

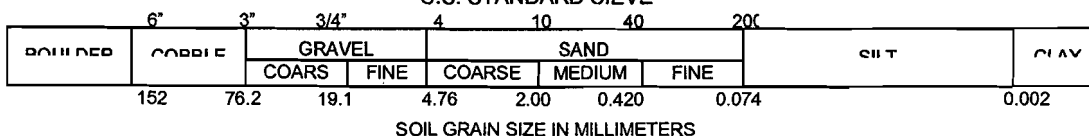
P = Pocket Penetrometer
 T = Torvane

TERMS AND SYMBOLS USED ON BORING LOGS FOR SOIL

SOIL TYPES

	CH, fat clays		SC, clayey sands		GC, clayey gravels		CL, lean clays
	SM, silty sands		GM, silty gravels		ML, silts		SW, well-graded sands
	GW, well-graded gravels		Fill, unclassified		SP, poorly-graded sands		GP, poorly-graded gravels

SOIL GRAIN SIZE U.S. STANDARD SIEVE



CONSISTENCY OF COHESIVE SOILS ⁽²⁾

CONSISTENCY	UNDRAINED SHEAR STRENGTH
Very Soft	Less Than 0.25
Soft	0.25 to 0.50
Firm	0.5 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

CONDITION OF GRANULAR SOILS ⁽²⁾

NUMBER OF BLOWS PER FT.. N	RELATIVE DENSITY
0-4	Very Loose
4-10	Loose
10-30	Medium
30-50	Dense
Over 50	Very Dense

STRUCTURE ⁽¹⁾

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 6 mm thick.
Laminated	Alternating layers of varying material or color with the layers less than 6 mm thick.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusions of small pockets of different soils.

MOISTURE

Dry	-No water evident in sample; fines less than plastic limit.
Moist	-Sample feels damp; fines near the plastic limit
Very Moist	-Water visible on sample; fines greater than plastic limit and less than liquid limit
Wet	-Sample bears free water; fines greater than liquid limit

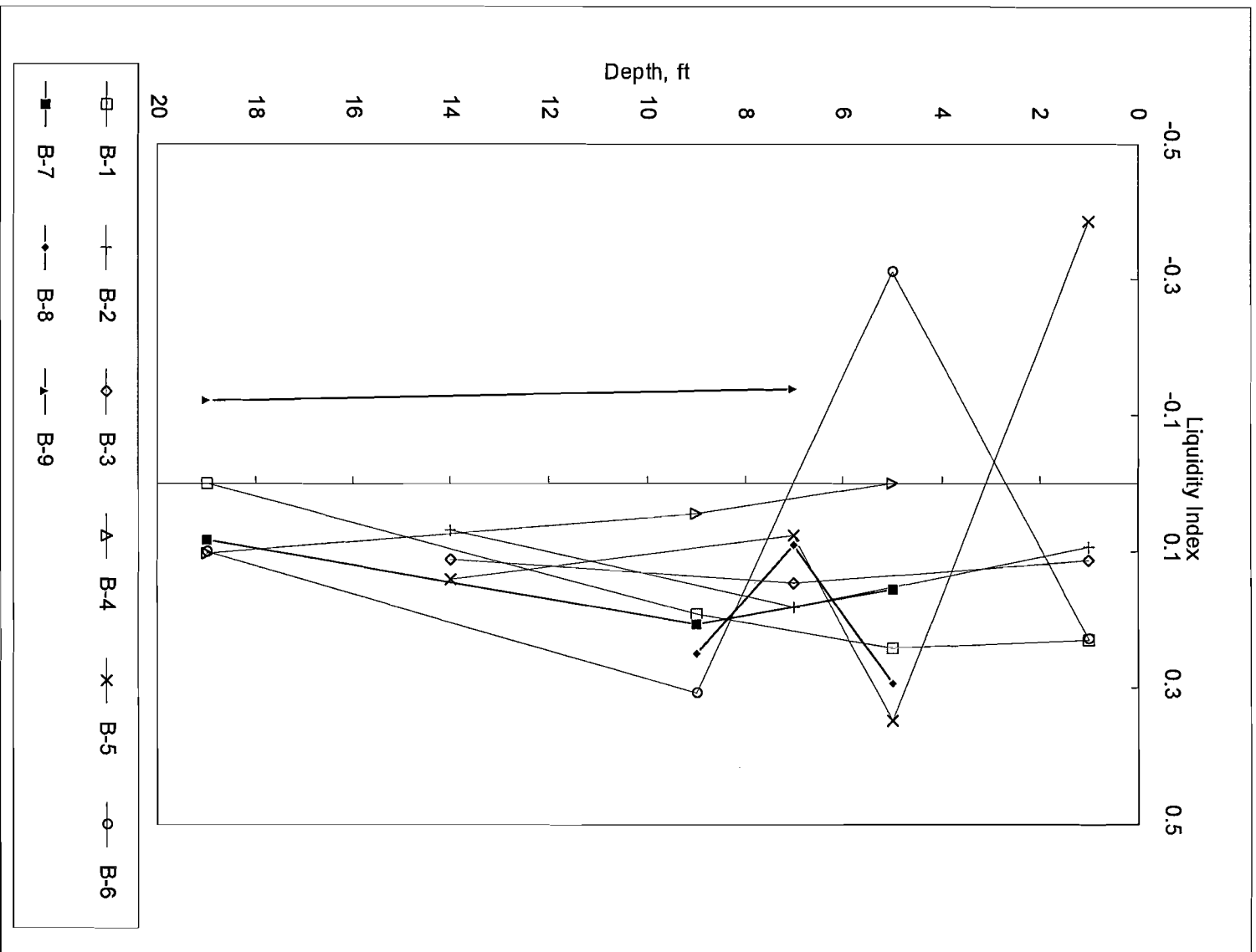
INCLUSIONS ⁽¹⁾

Parting	-Inclusion <1/8" thick extending through sample.
Seam	-Inclusion 1/8" to 3" thick extending through sample.
Layer	-Inclusion >3" thick extending through sample.
Trace	-<5% of sample.
Few	-5% to 10% of sample.
Little	-10 to 25 % of sample.
Some	-30% to 45% of sample.

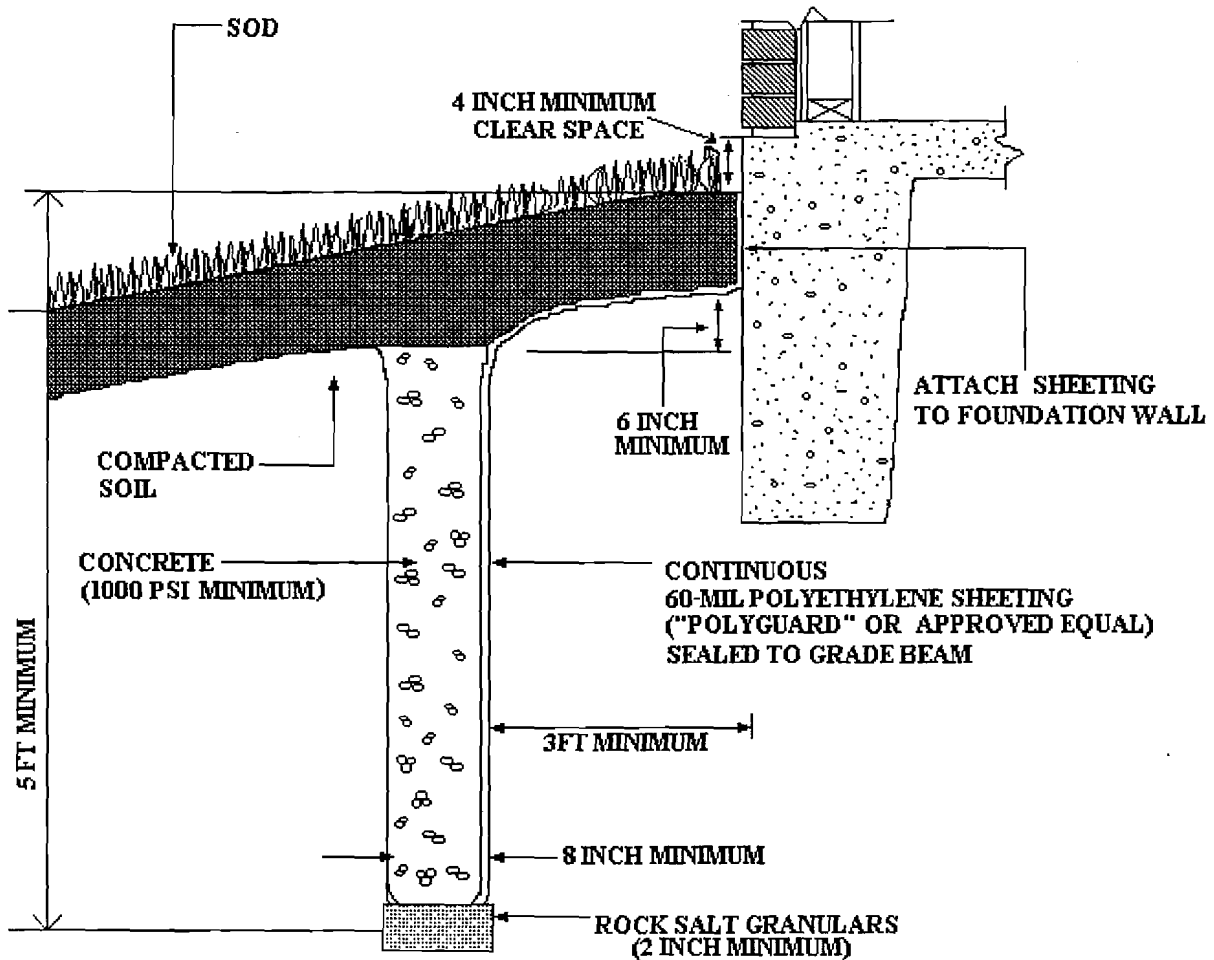
REFERENCES:

- 1) ASTM D 2488
- 2) Peck, Hanson, and Thornburn, (1974), Foundation Engineering.

Information on each boring log is a compilation of subsurface conditions and soil and rock classifications obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the times and places indicated, and may vary with time, geologic condition or construction activity.



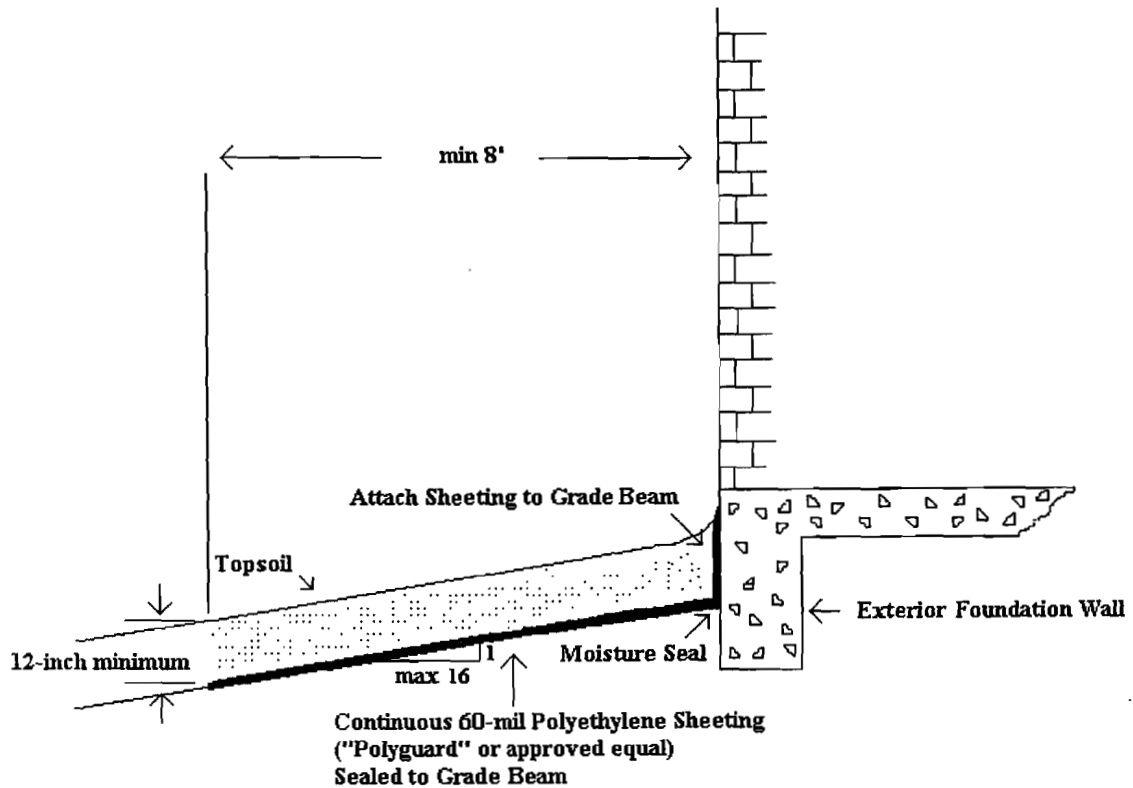
LIQUIDITY INDEX vs. DEPTH
FT. SAM HOUSTON HISTORIC BUILDING EVALUATION
FORT SAM HOUSTON, TEXAS



Notes:

- 1) Landscaping plan should be developed and maintained throughout the life of structure to prevent penetration of, or any other damage to moisture barrier.
- 2) Vegetation type should be limited to prevent root growth through moisture barrier.
- 3) Any penetrations through the moisture barrier should be properly sealed.
- 4) Irrigation line should be properly tested upon installation in order to detect the presence of leaks
- 5) Seal Moisture Barrier to Foundation Wall.

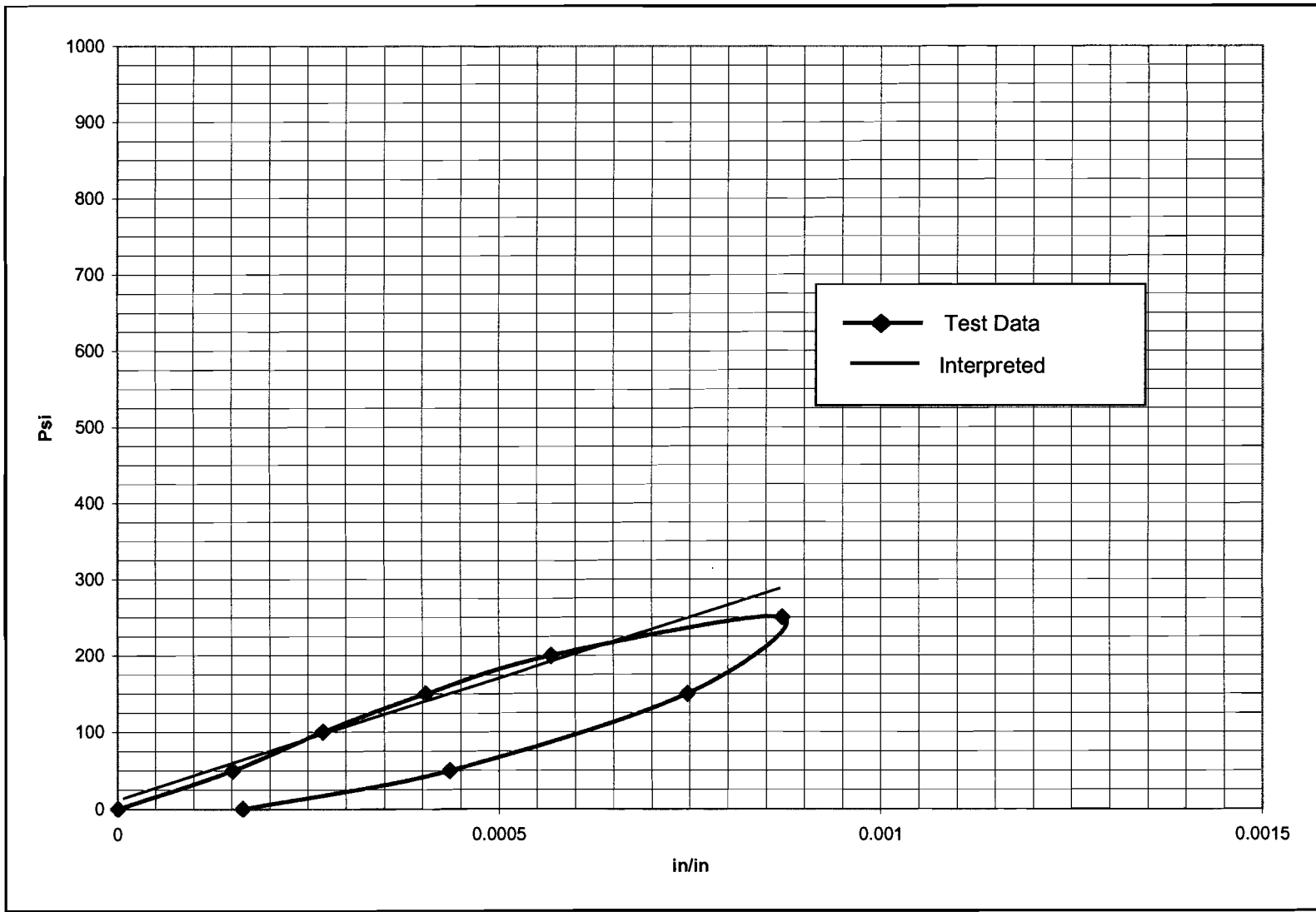
**VERTICAL MOISTURE BARRIER DETAIL
FORENSIC EVALUATION AND TESTING
OF VARIOUS HISTORIC STRUCTURES
FT. SAM HOUSTON, TEXAS**



Notes:

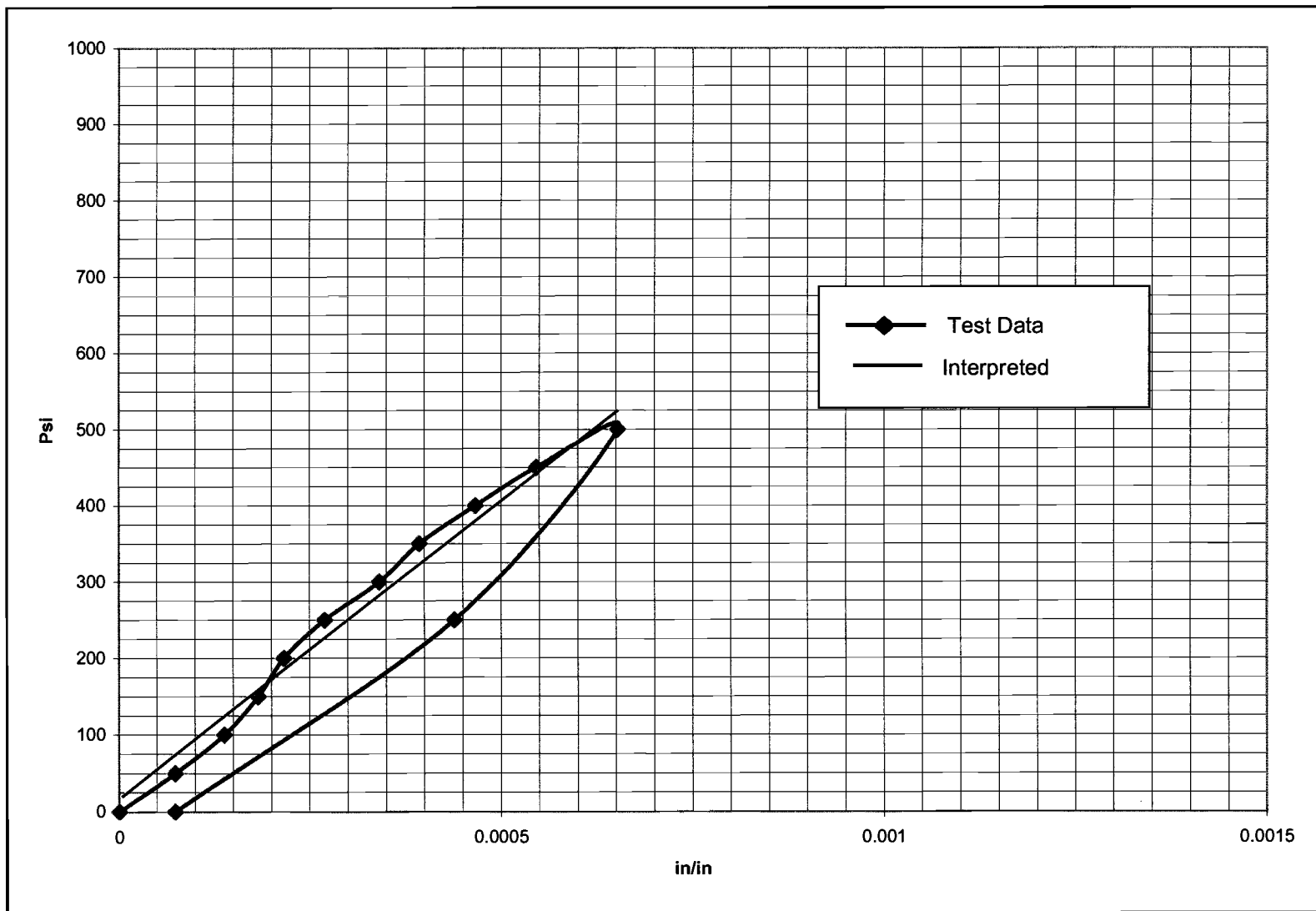
- 1) Landscaping plan should be developed and maintained throughout the life of structure to prevent penetration of, or any other damage to moisture barrier.
- 2) Vegetation type should be limited to prevent root growth through moisture barrier.
- 3) Any penetrations through the moisture barrier should be properly sealed.
- 4) Irrigation line should be properly tested upon installation in order to detect the presence of leaks.
- 5) Seal Moisture Barrier to Foundation Wall.

**HORIZONTAL MOISTURE BARRIER DETAIL
FORENSIC EVALUATION AND TESTING
OF VARIOUS HISTORIC STRUCTURES
FT. SAM HOUSTON, TEXAS**



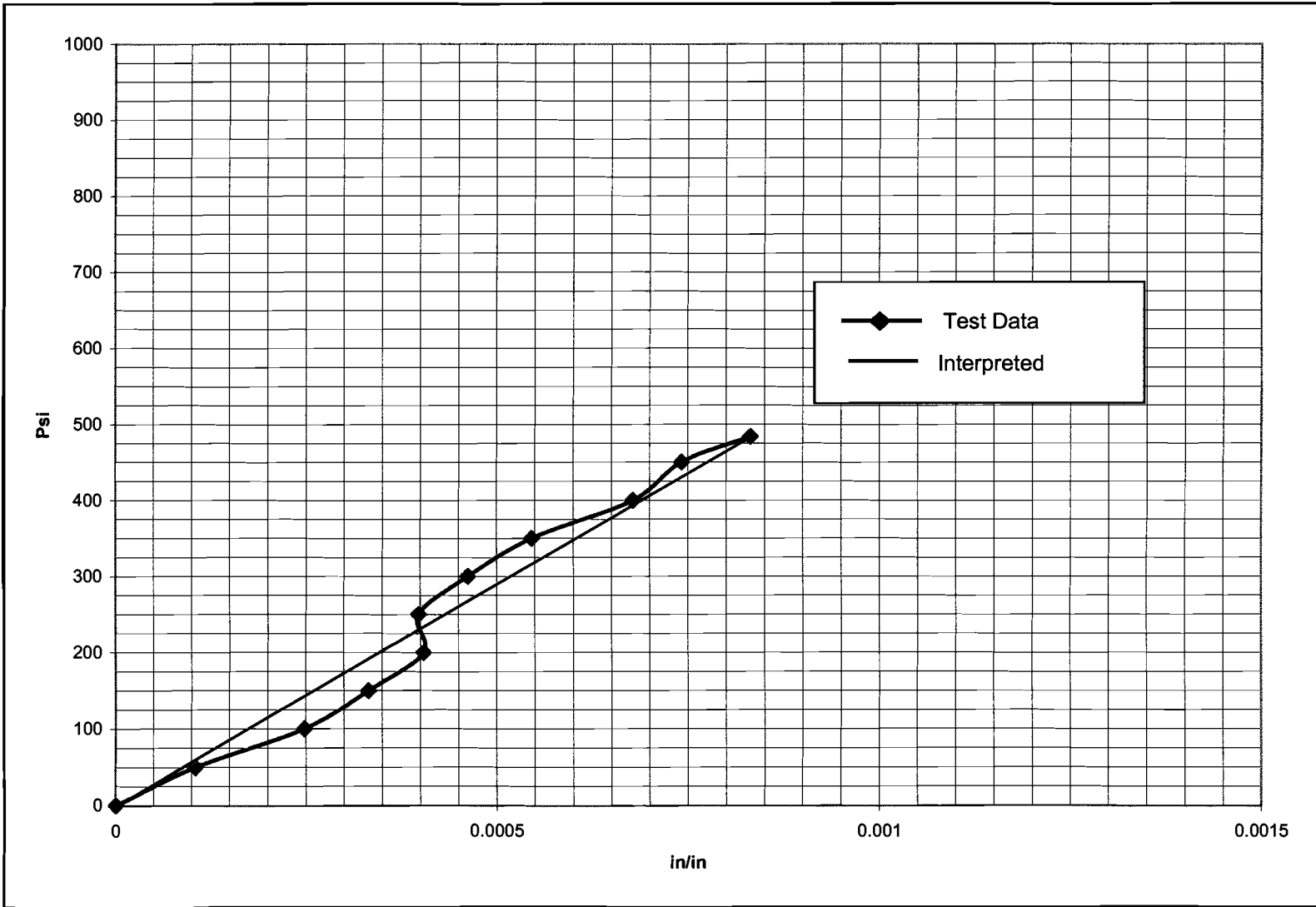
STRESS vs. STRAIN
BUILDING 123
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS



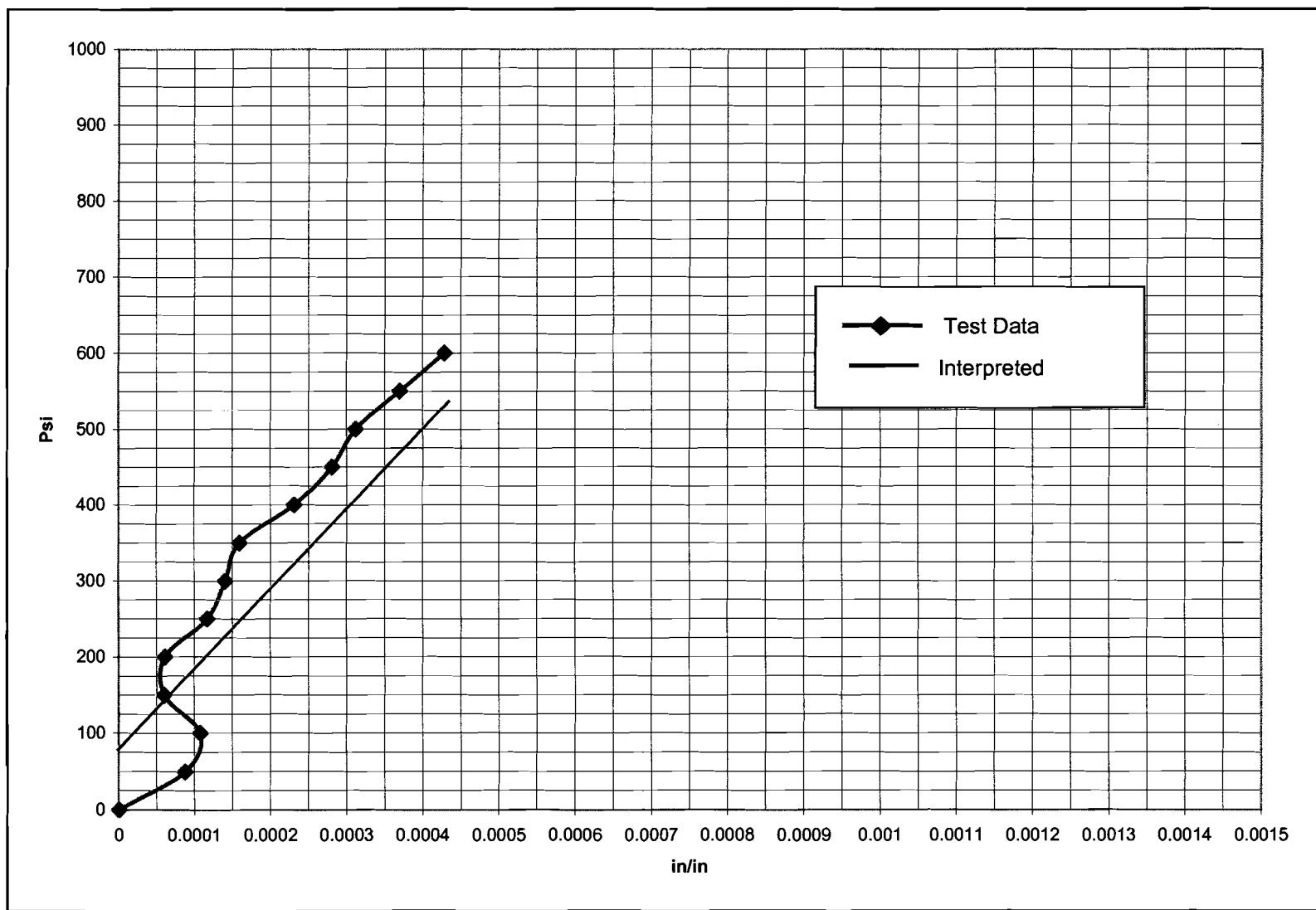


STRESS vs. STRAIN
BUILDING 131
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS

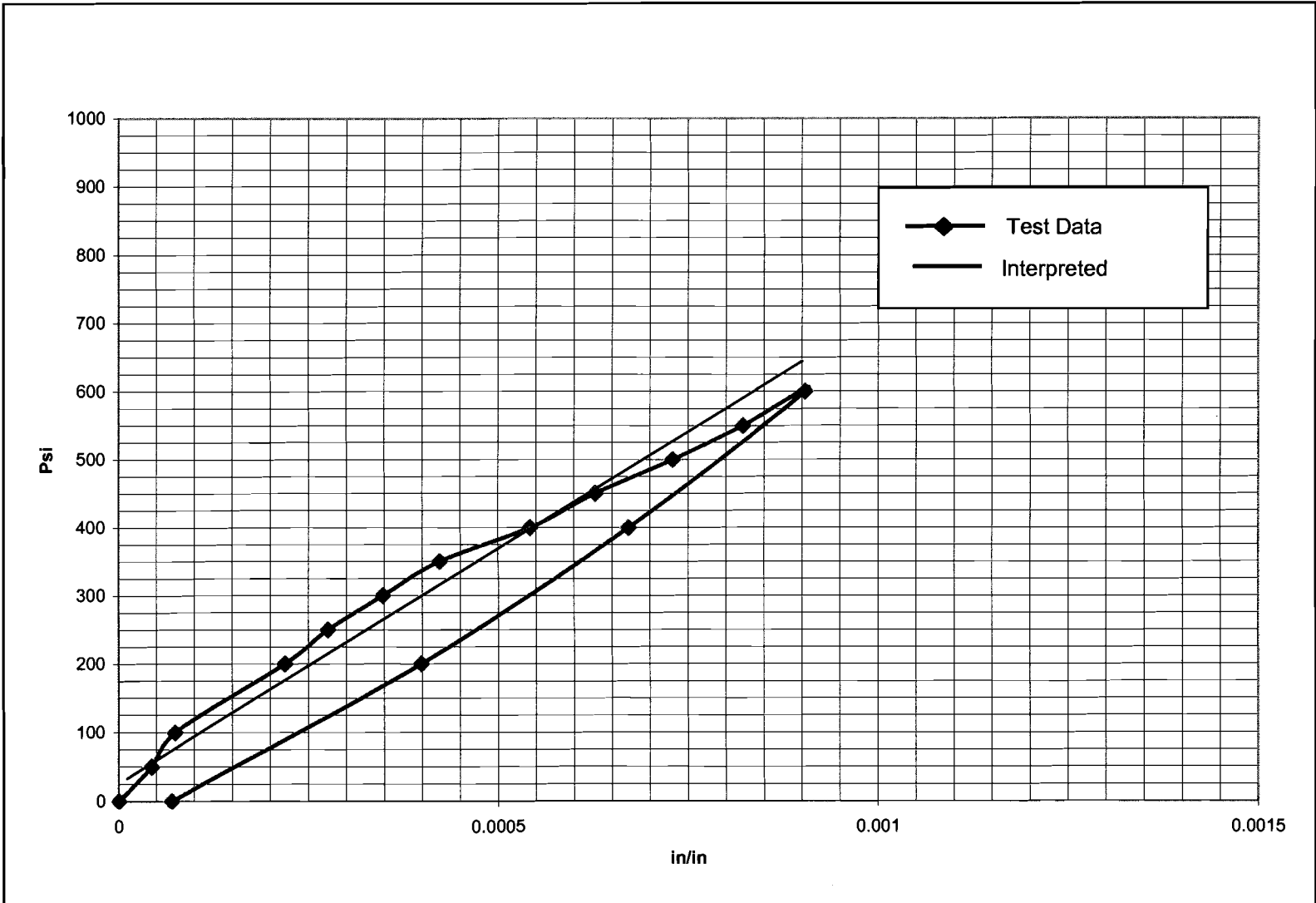




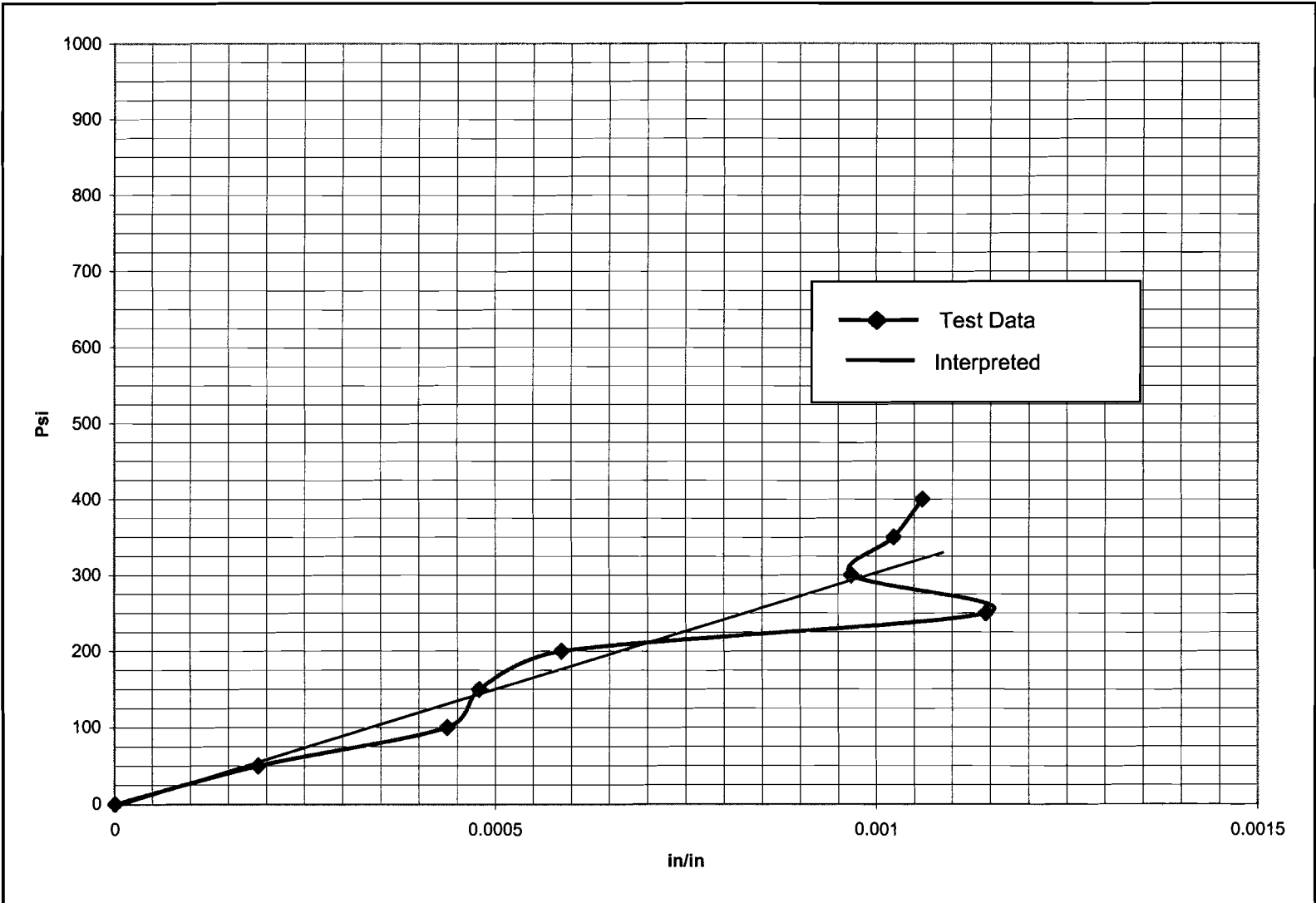
STRESS vs. STRAIN
BUILDING 135
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS



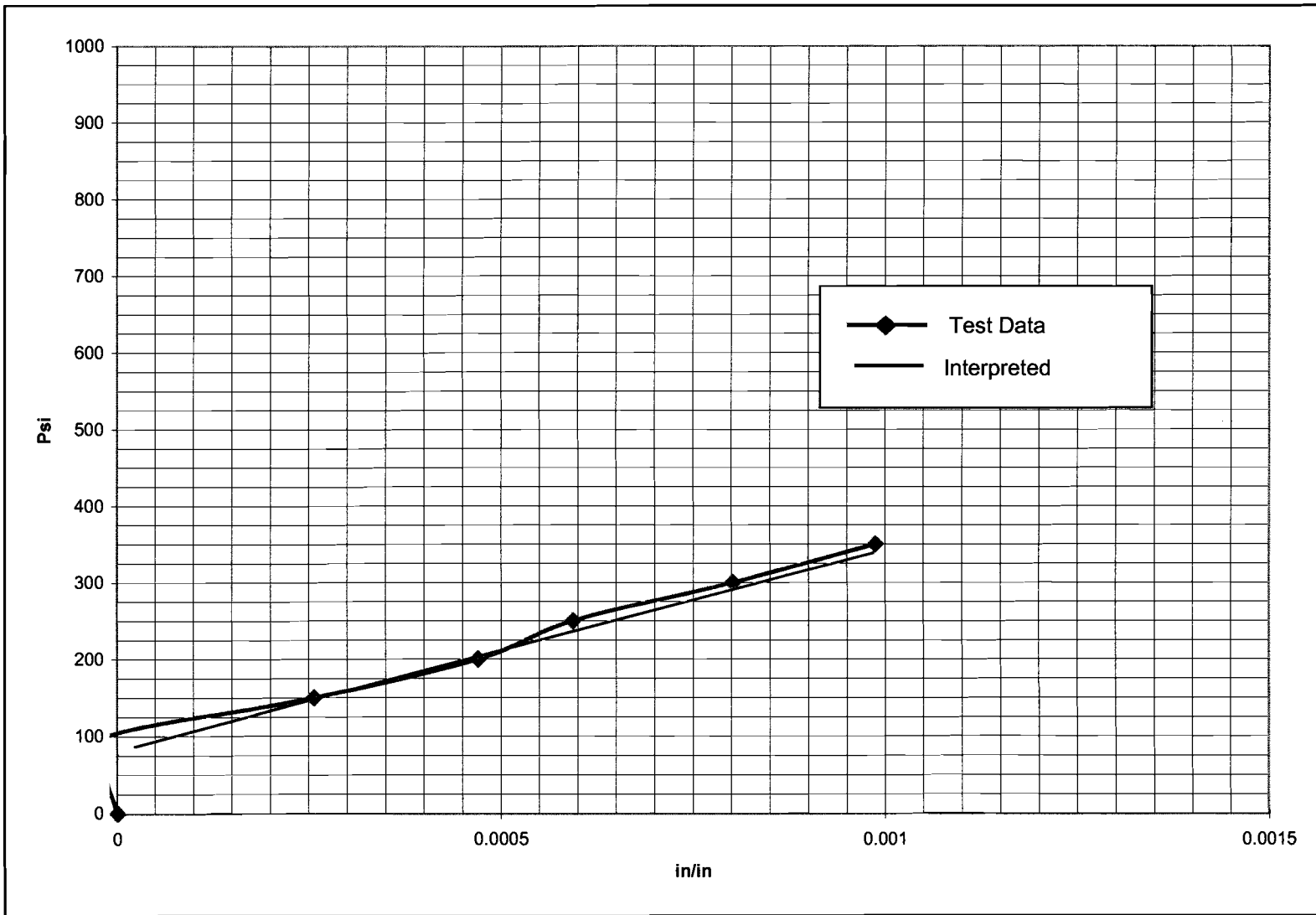
STRESS vs. STRAIN
BUILDING 145
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS



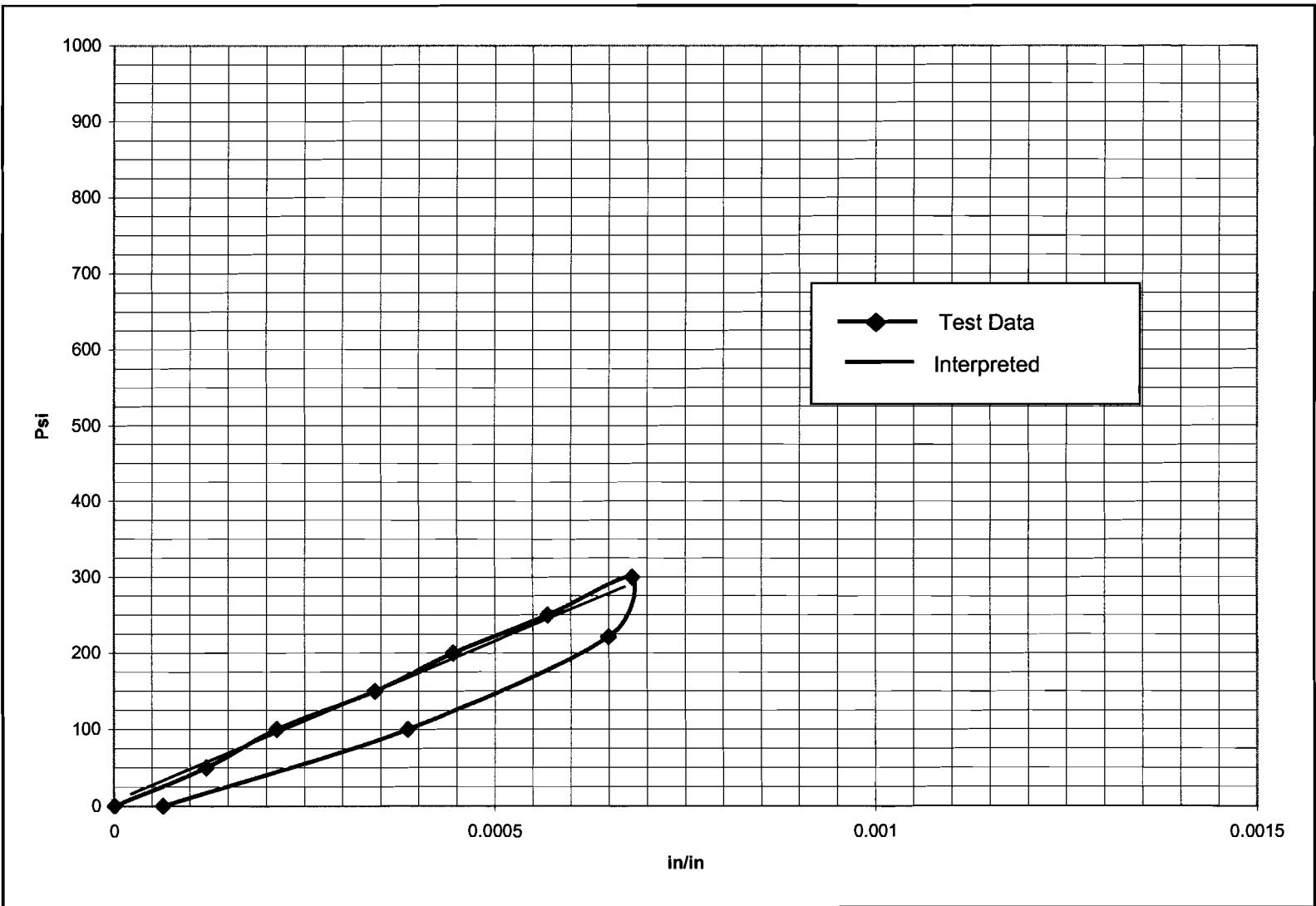
STRESS vs. STRAIN
BUILDING 147
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS



STRESS vs. STRAIN
BUILDING 151
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS



STRESS vs. STRAIN
BUILDING 156
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS



STRESS vs. STRAIN
BUILDING 238
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS

CORE NO.	BREAK DATE	CORE DIAMETER inches	CORE LENGTH inches	FAILURE LOAD pounds	CORR. FACTOR	UNCORRECTED COMPRESSIVE STRENGTH psi	CORRECTED* COMPRESSIVE STRENGTH psi
A	2/4/2005	3.70	7.50	21,580	0.995	2010	2000
B	2/4/2005	3.70	7.48	23,860	0.996	2220	2210
C	2/4/2005	3.70	7.47	23,160	0.998	2150	2150

Average: 2120

Testing was performed in accordance with ASTM C-42.

CONCRETE CORE BREAK RESULTS
BUILDING 197
FORENSIC EVALUATION AND TESTING OF VARIOUS HISTORIC STRUCTURES
FORT SAM HOUSTON, TEXAS

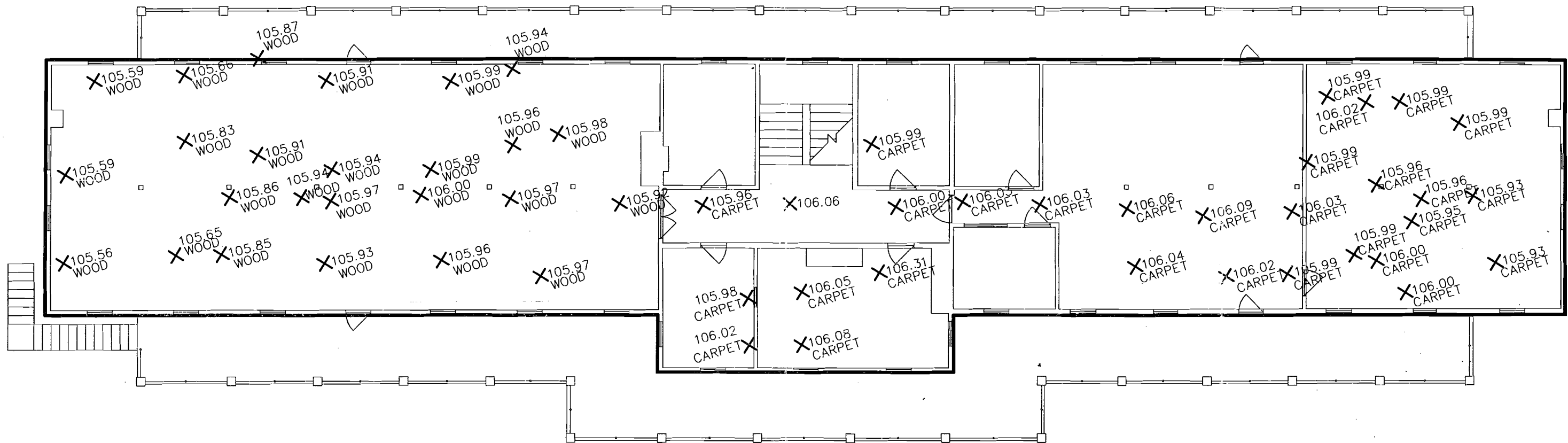




APPENDIX A:

Topographic Elevation Survey performed by Stephen G. Cook Engineering

BUILDING NO. #197

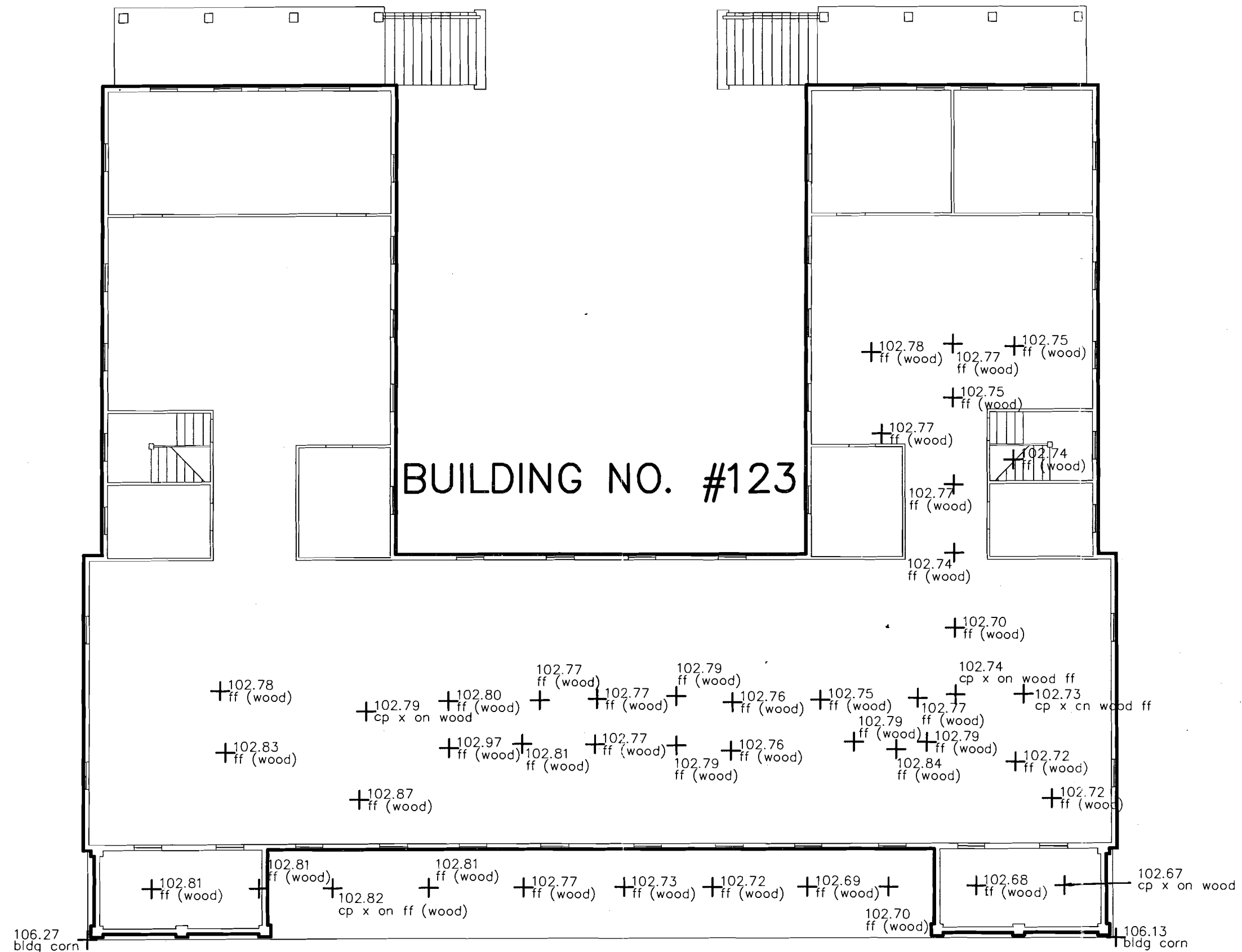


X100.00
BENCHMARK

JOB #911-001
DATE OF SURVEY: 09 FEB 05
DRAWN: XXX
CHECKED: XXX
SCALE: 1"=10'
SHEET 1 OF 2

Stephen G. Cook Engineering, Inc.
12000 Starcrest Suite 107
San Antonio, TX 78247
Phone (210) 481-2533
Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
ELEVATION SHOTS
BUILDING NO. # 197



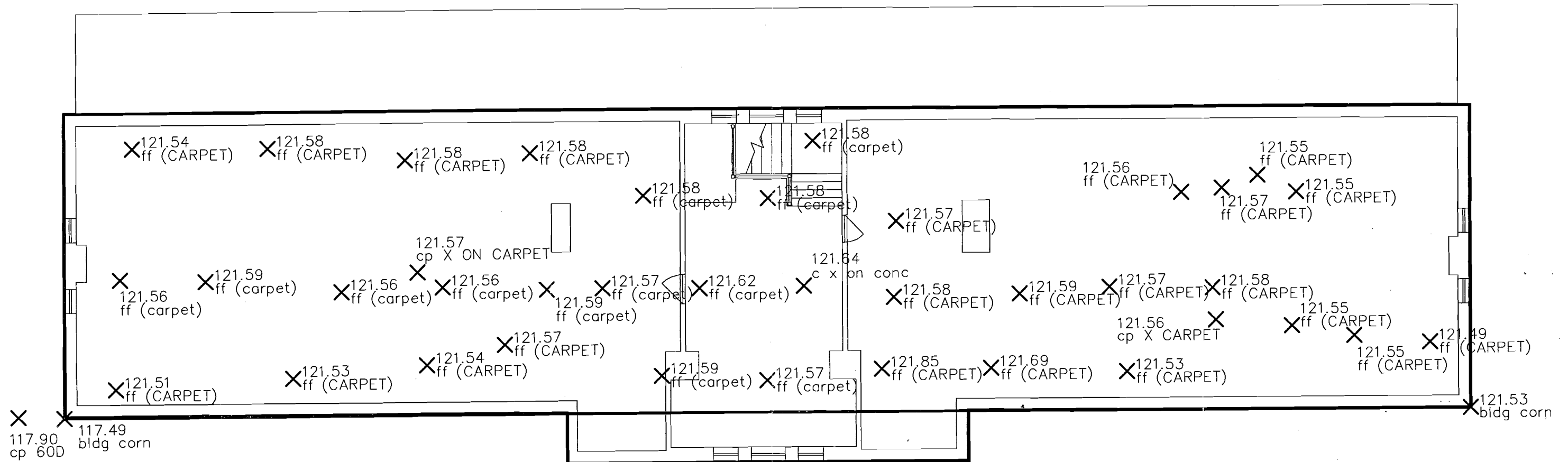
BUILDING NO. #123

JOB #911-001
 DATE: 09 FEB 05
 DATE OF SURVEY: 30 NOV 05
 DRAWN: XXX
 CHECKED: XXX
 SCALE: N.T.S.
 SHEET 2 OF 9

Stephen G. Cook Engineering, Inc.
 12000 Starcrest Suite 107
 San Antonio, TX 78247
 Phone (210) 481-2533
 Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
 ELEVATION SHOTS
 BUILDING NO. # 123

BUILDING NO. #131



X 100.00
BENCHMARK
60 DNAIL

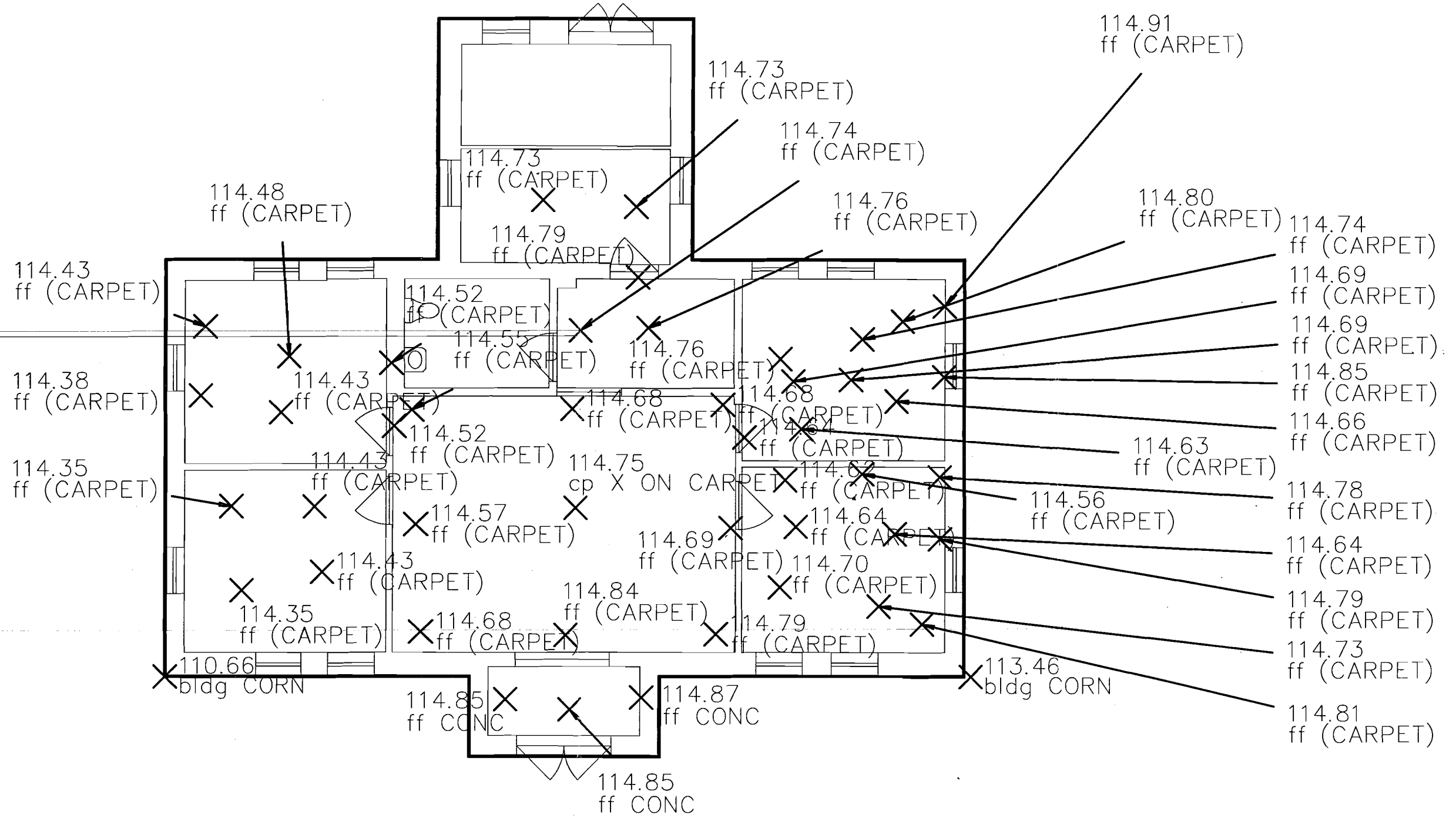
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DATE: 09 FEB 05
DATE OF SURVEY: 30 NOV 05
DRAWN: XXX
CHECKED: XXX
SCALE: N.T.S.
SHEET 3 OF 9

Stephen G. Cook Engineering, Inc.
12000 Starcrest Suite 107
San Antonio, TX 78247
Phone (210) 481-2533
Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
ELEVATION SHOTS
BUILDING NO. # 131

C:\Projects\911-001\Drawings\911-001-03.dwg 11/30/05 11:54:00 AM

BUILDING NO. #135

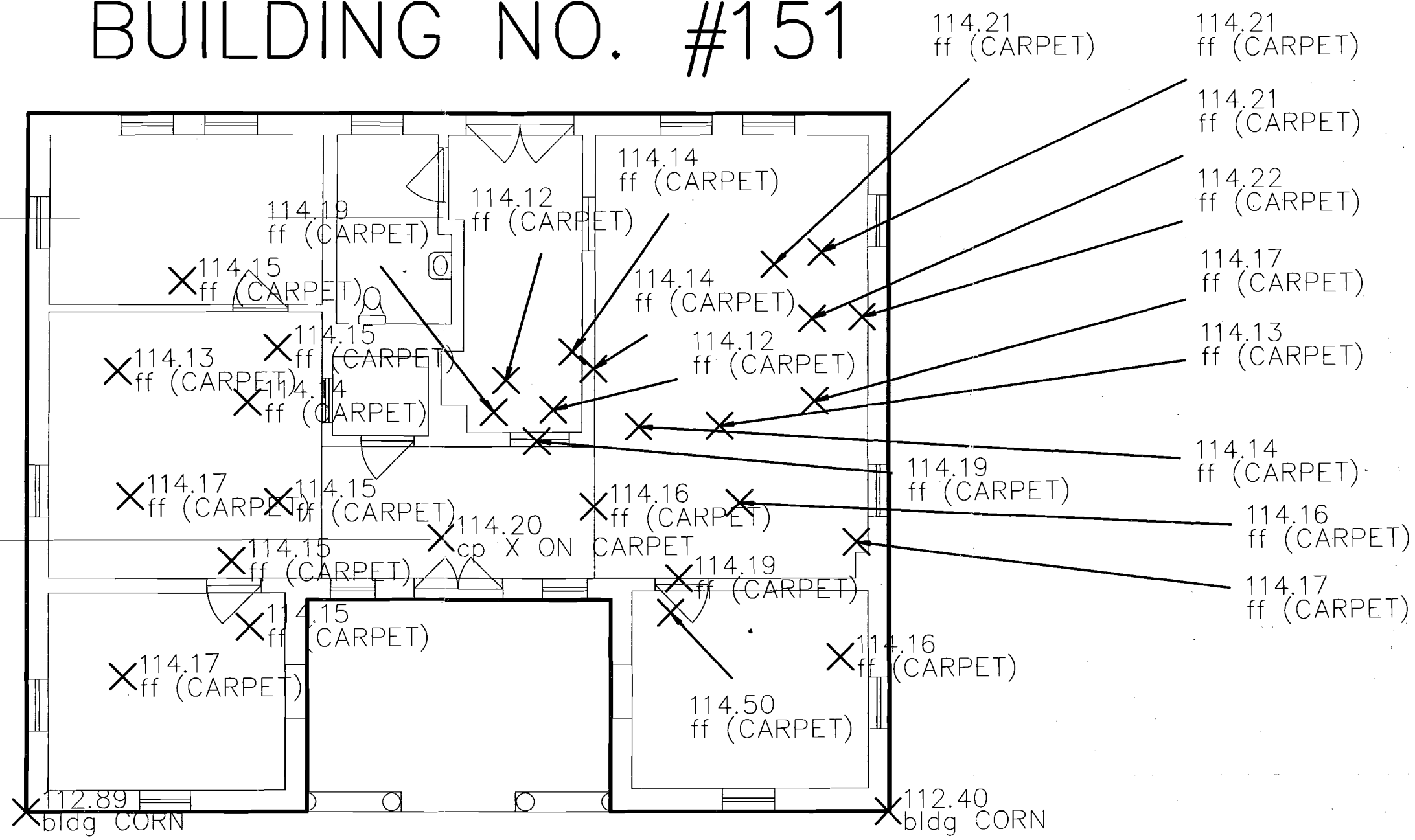


JOB #911-001
DATE: 10 FEB 05
DATE OF SURVEY: 30 NOV 05
DRAWN: XXX
CHECKED: XXX
SCALE: N.T.S.
SHEET 4 OF 9

Stephen G. Cook Engineering, Inc.
12000 Starcrest Suite 107
San Antonio, TX 78247
Phone (210) 481-2533
Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
ELEVATION SHOTS
BUILDING NO. # 135

BUILDING NO. #151



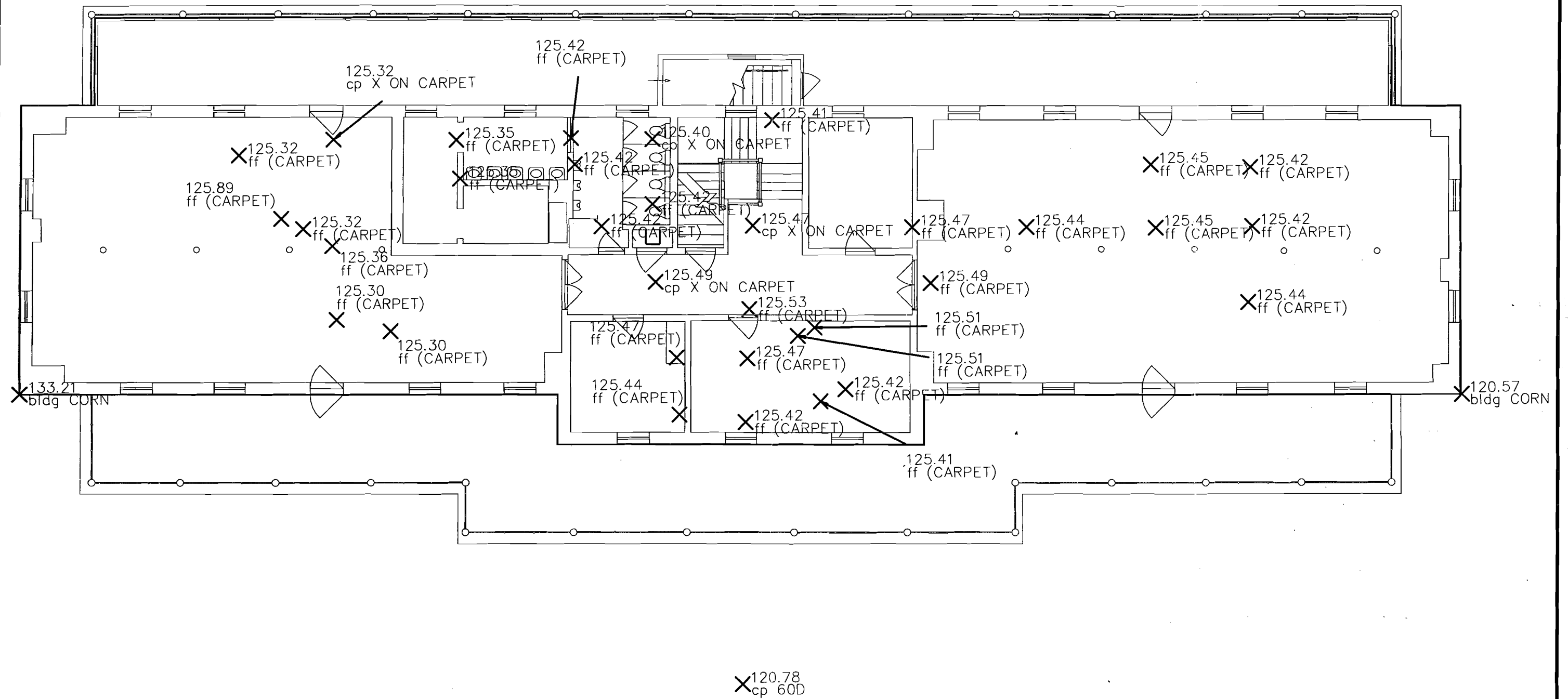
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- 114.21 ff (CARPET)
- 114.22 ff (CARPET)
- 114.22 ff (CARPET)
- 114.17 ff (CARPET)
- 114.17 ff (CARPET)
- 114.13 ff (CARPET)
- 114.14 ff (CARPET)
- 114.16 ff (CARPET)
- 114.16 ff (CARPET)
- 114.17 ff (CARPET)

JOB #911-001
 DATE: 09 FEB 05
 DATE OF SURVEY: 30 NOV 05
 DRAWN: XXX
 CHECKED: XXX
 SCALE: N.T.S.
 SHEET 5 OF 9

Stephen G. Cook Engineering, Inc.
 12000 Starcrest Suite 107
 San Antonio, TX 78247
 Phone (210) 481-2533
 Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
ELEVATION SHOTS
 BUILDING NO. # 151

BUILDING NO. #145

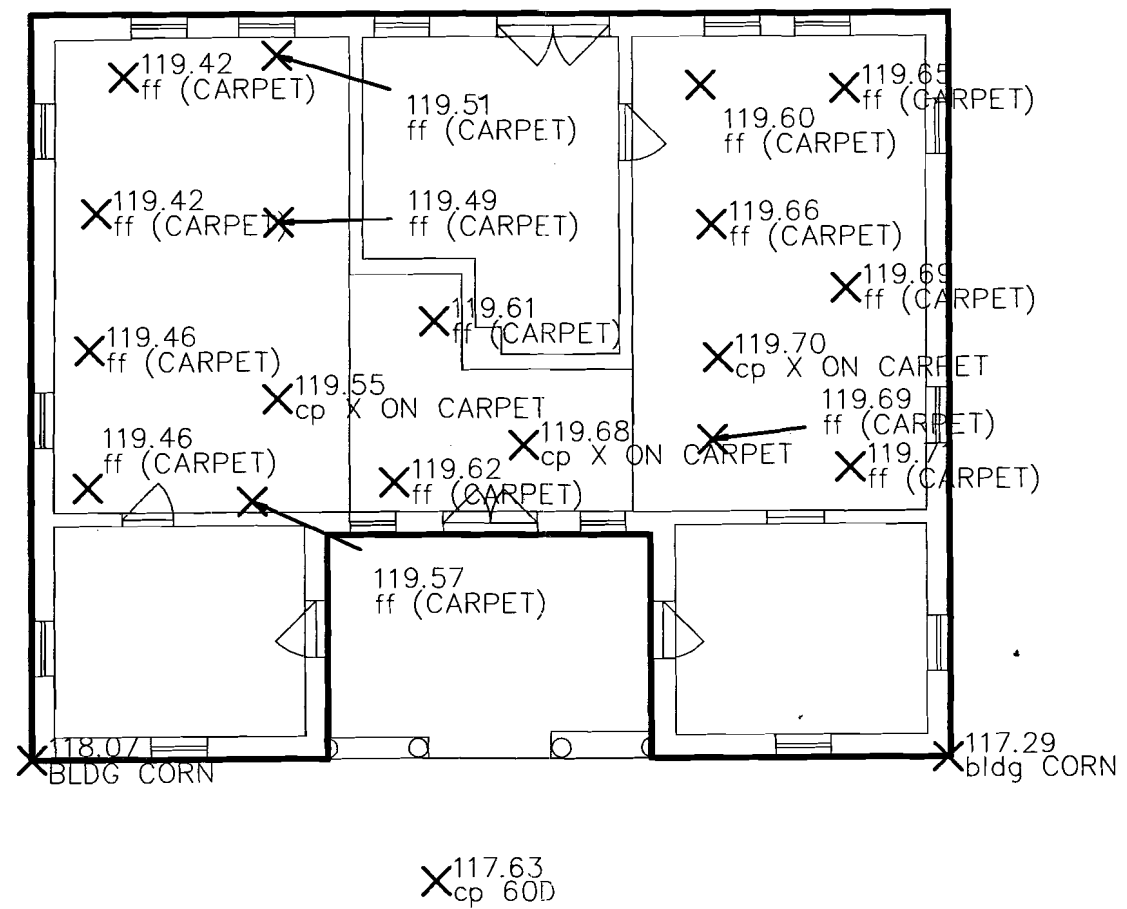


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DATE: 11 FEB 05
DATE OF SURVEY: 30 NOV 05
DRAWN: XXX
CHECKED: XXX
SCALE: 1" = 10'
SHEET 6 OF 9

Stephen G. Cook Engineering, Inc.
12000 Starcrest Suite 107
San Antonio, TX 78247
Phone (210) 481-2533
Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
ELEVATION SHOTS
BUILDING NO. # 145

BUILDING NO. #156



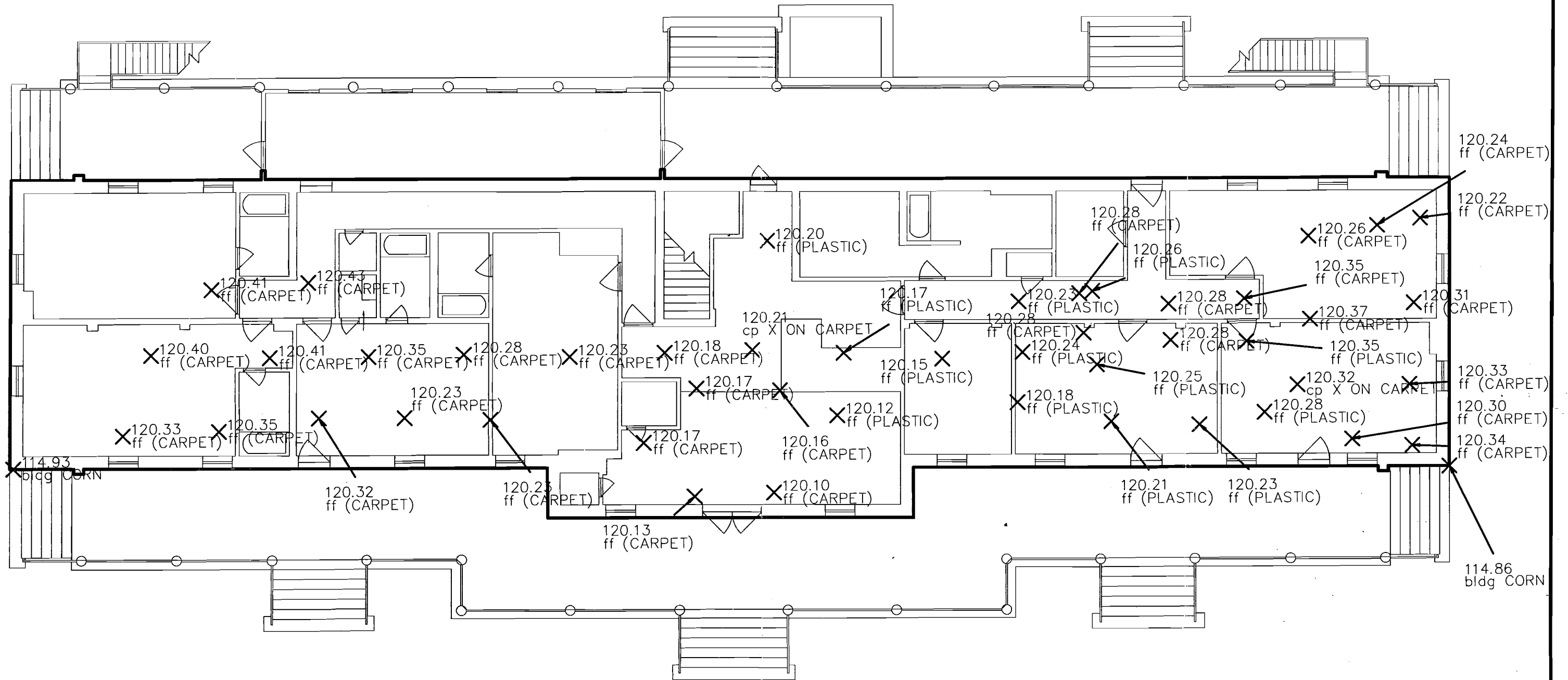
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 DATE OF SURVEY: 30 NOV 05
 DRAWN: XXX
 CHECKED: XXX
 SCALE: 1" = 10'
 SHEET 2 OF 9

Stephen G. Cook Engineering, Inc.
 12000 Starcrest Suite 107
 San Antonio, TX 78247
 Phone (210) 481-2533
 Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
 ELEVATION SHOTS
 BUILDING NO. # 156

S:\Engineering\Drawings\911-001.dwg\110\STEP20805_156.dwg

BUILDING NO. #147



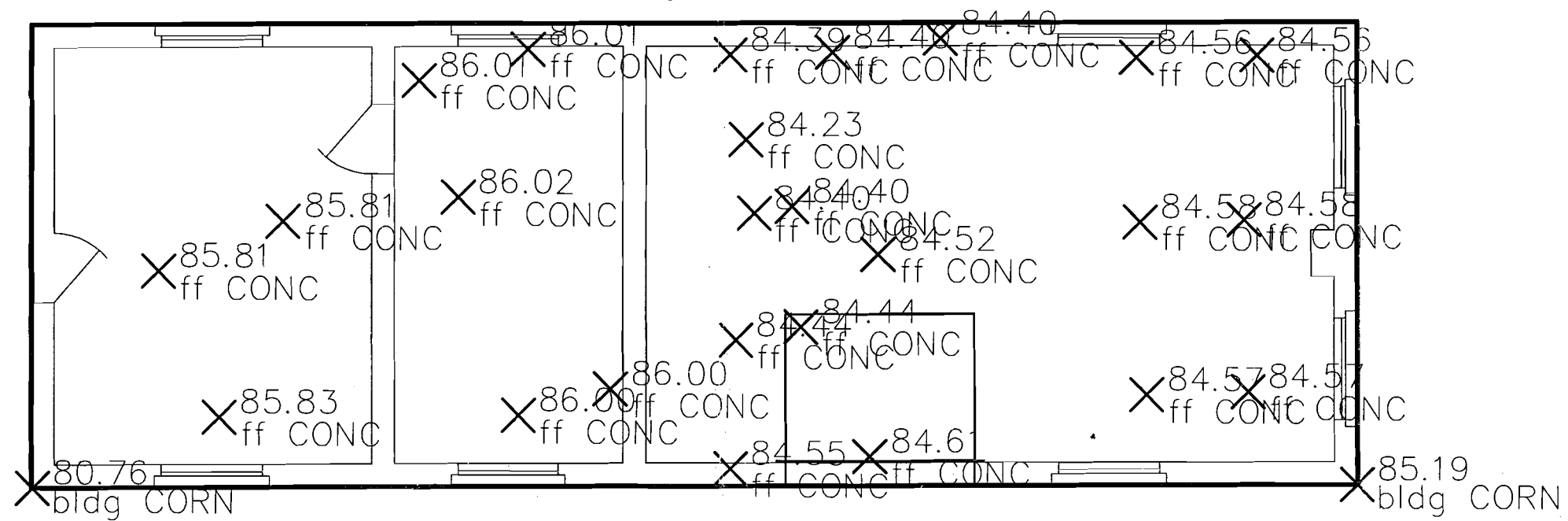
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JOB #911-001
DATE: 14 FEB 05
DATE OF SURVEY: 30 NOV 05
DRAWN: XXX
CHECKED: XXX
SCALE: 1" = 10'
SHEET 8 OF 9

Stephen G. Cook Engineering, Inc.
12000 Starcrest Suite 107
San Antonio, TX 78247
Phone (210) 481-2533
Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
ELEVATION SHOTS
BUILDING NO. # 147

BUILDING NO. #238



JOB #911-001
DATE: 15 FEB 05
DATE OF SURVEY: 30 NOV 05
DRAWN: XXX
CHECKED: XXX
SCALE: N.T.S.
SHEET 2 OF 2

Stephen G. Cook Engineering, Inc.
12000 Starcrest Suite 107
San Antonio, TX 78247
Phone (210) 481-2533
Fax (210) 481-2150

FT. SAM HOUSTON - SAN ANTONIO, TX
ELEVATION SHOTS
BUILDING NO. # 238



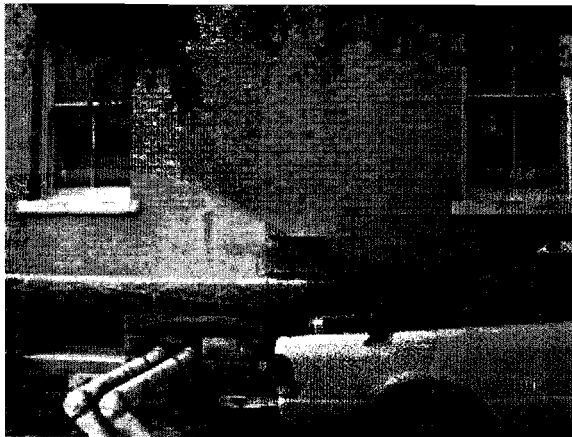
APPENDIX B:

Photographs of Flatjack Testing



Building 123

Stopped when the stress strain curve appeared to enter the plastic range

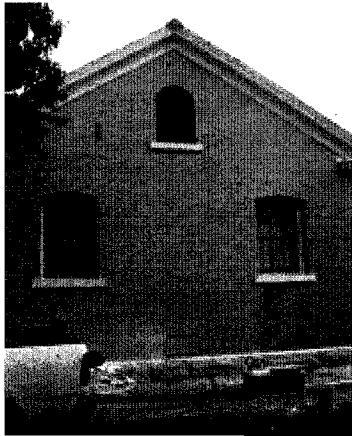


Building 131

Prism did not rupture, the masonry above the test section spalled

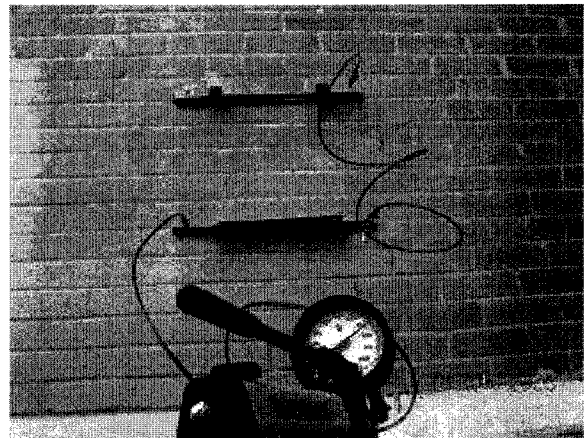
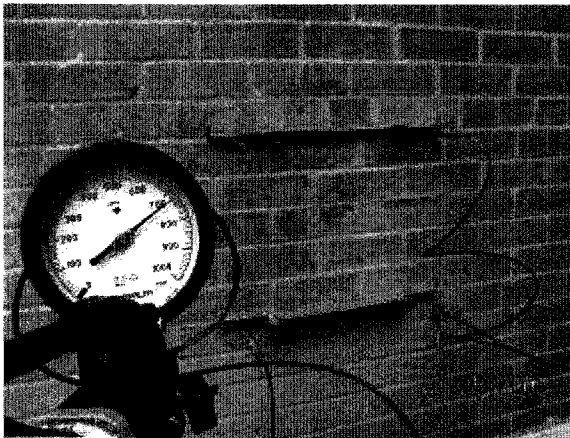
**PHOTOGRAPHS OF TEST LOCATIONS
FT. SAM HOUSTON, TEXAS**





Building 135

Peripheral observations indicated the wall above the test section was lifting and that damage to the wall was impending

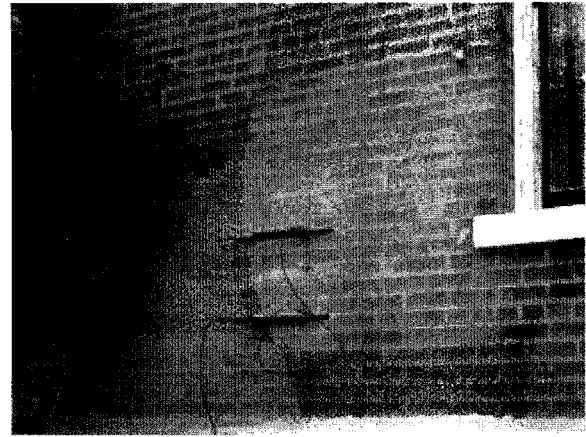


Building 145

The maximum target load provided by Lundy and Franke was achieved

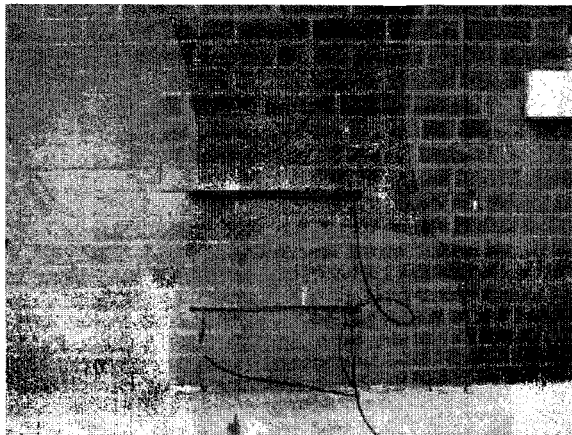
**PHOTOGRAPHS OF TEST LOCATIONS
FT. SAM HOUSTON, TEXAS**





Building 147

The maximum target load provided by Lundy and Franke was achieved

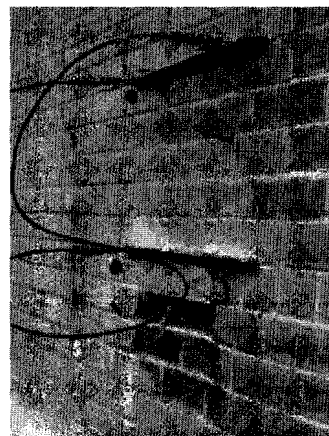
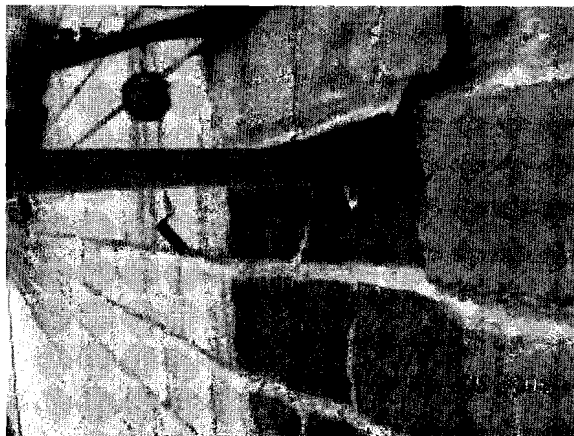


Building 151

Peripheral observations indicated the wall above the test section was lifting and that damage to the wall was impending

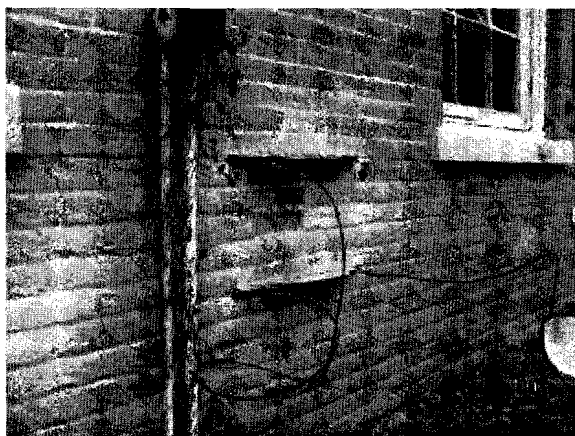
**PHOTOGRAPHS OF TEST LOCATIONS
FT. SAM HOUSTON, TEXAS**





Building 156

Prism did not rupture, the masonry below the test section spalled



Building 238

Testing prism failed, the test provided a compressive strength (f_m) of 300 psi

**PHOTOGRAPHS OF TEST LOCATIONS
FT. SAM HOUSTON, TEXAS**



Important Information About Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply the report for any purpose or project except the one originally contemplated.*

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that

have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.



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e-mail: info@asfe.org www.asfe.org

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