Forensic Engineering in Applied Civil Engineering and Geo-Domain

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FORENSIC ENGINEERING IN APPLIED CIVIL ENGINEERING AND GEO-DOMAIN

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ABSTRACT

The new discipline that deals with investigation of failures and performance problems in the built environment is known as Forensic Engineering. While forensic civil engineering is a well established science, forensic geotechnical engineering is a relatively new discipline. It involves scientific and jurisprudence related investigations and evaluation to analyze the causes/process of structural distress that originates from geo-domain. Forensics in geo-domain encompasses an extensive array of topics with general emphasis in civil engineering and specific emphasis in geotechnical and related fields having geological, geophysical, geoenvironmental, and structural applications. Mostly it applies to failures after they occur when their application has prevented and/or identified failures prior to their occurrence. Furthermore, cases of analyses and evaluations of selected remedial measures, along with their effectiveness and economy, are normally subjected to judicial scrutiny. Two case histories are presented where forensic engineering was effectively utilized to identify, investigate, and remediate the problems.

The first case history identifies a request received from an adjuster of a national insurance company for a forensic engineering applied review of a reported sinkhole damage claim at a historic church building in north Florida. There had been no collapse and no injury. The insurance claim resulted in discovery of solution activity in Karst formations and identification of subsurface features that are known to aid in the formation of sinkholes.

The second case history illustrates how forensic engineering was applied in investigating a situation where a severe deficiency occurred during placement of concrete for a Steam Turbine Generator (STG) structure of a power plant in west central Florida. Questions were raised regarding the structural integrity, and removal/replacement of the partially completed structure was considered a viable yet costly option. Forensic investigation was conducted to determine whether the damage was surficial and deficiency could be addressed by repairing or removal, and replacement of the structure was, in fact, necessary.

In both of these cases, the application of forensic engineering principles assisted in the identification and proper remediation of the problem.

INTRODUCTION

Forensic engineering is generally defined as the application of engineering principals and methodology to answer questions of fact that may have legal ramifications (Noon, 1992). Forensic geotechnical engineering is a relatively new discipline. It starts from the observational method and concentrates on the identification of hidden clues. Selection of procedures and test parameters are different from the standard ones. The procedures adopted for the analysis and testing should satisfy even legal scrutiny for their validity in-situ.

Everything in our built environment is expected to perform as designed, but when things do not perform as expected, we are reminded that things do not operate continually forever. Failure or alleged failure, is defined as an unacceptable difference between expected and observed performance. The new discipline that deals with investigation of damaged or deteriorated structure, and failures or performance problems in the built environment is known as Forensic Engineering.

In the first case history, the review of reported sinkhole damage and related insurance claim resulted in the discovery of numerous design and construction defects in a 75 plus year old historical church building. Recommendations to repair the damage determined a covered loss by the insurance company.

In the second case history, using forensic engineering techniques, it was possible to determine the condition of the structure. These findings provided a means for assessing potential mitigation techniques. Based upon review of plans, drawings, specifications, and remediation records, it was concluded that the remediated structure was totally restored.

CASE HISTORY ONE – PROJECT DESCRIPTION

In late 2001 or early 2002, a geological investigation indicated a deep-seated problem consisting of weakening and dissolutioning of limestone that created sinkhole related hazardous subsurface conditions under an old historical church structure in northern Florida. Stability of the structure was threatened because of the presence of shallow irregular depth limestone that had experienced significant weakening and the creation of an intricate cavernous system consisting of Karst related features and an internal erosion process. Solution features occur both above and below the water table and these cavities can be air, water, or soil filled in carrying proportions. Additional geophysical resistivity survey indicated anomalous resistivity values suggesting a raveled zone. It was concluded by the consultant that rectification of the solutioned or voided areas was, in fact, necessary. A 3-stage investigation program was carried out.

STAGE 1 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Subsoil conditions beneath the site were evaluated by advancing several test borings to a depth of 50 ft (15.2 m). Site layout and test boring locations are illustrated in Fig. 1. Typical subsurface conditions consisted of up to 20 ft (6.1 m) of predominantly non-plastic soils overlaying a thin highly weathered cap of a plastic material, which in turn overlays a recently cemented calcareous limestone formation and variable depth of limestone with water table.

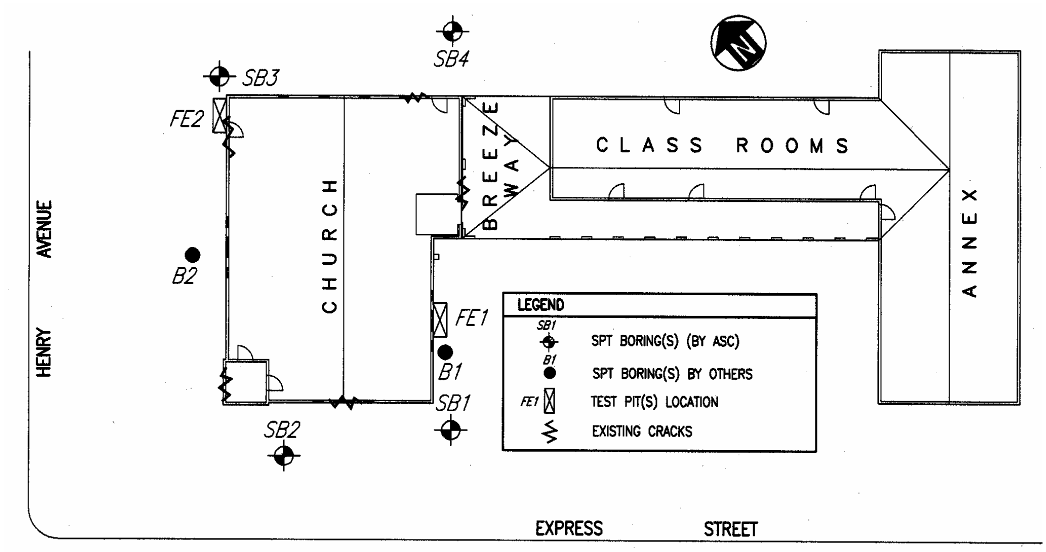

Fig. 1. Project layout and test location plan

Based on these borings supplemented by data from previous exploration (B1 and B2) and knowledge of the area, the chances of an existing solution cavity (or initiation of failure process leading to increased solution activity were increased by the fact that partial loss of circulation between 20 ft (6.1 m) and 30 ft (9.2 m) exist in all of the test borings. Stratigraphy of the project site for the solution cavity areas is illustrated in Fig. 2. Furthermore, depression or absence of the impermeable clay layer, as well as variable limestone elevations, was indicative of subsurface solution activity.

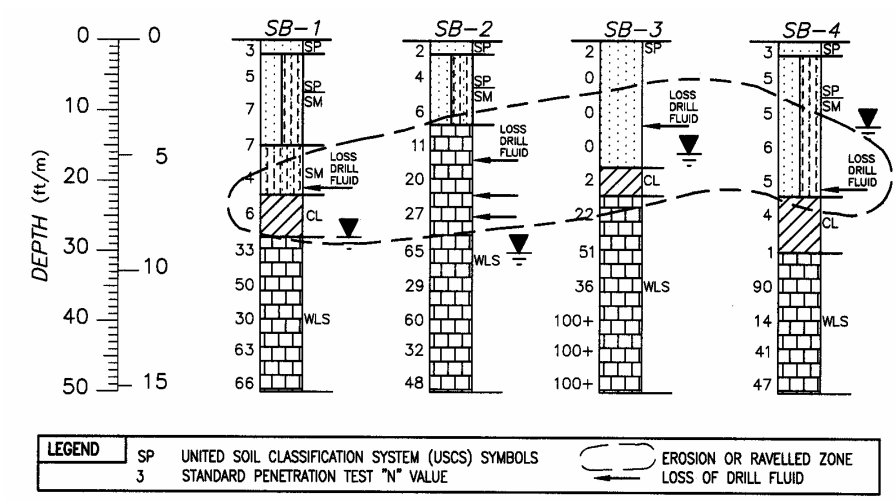

Fig.2. Project site Stratigraphy showing internal erosion and solution features

The findings, comments, conclusions, and recommendations derived from Stage I investigation at the project development site revealed that:

1. the presence of, or potential for, solution channels to transmit soil particles cannot be directly evaluated. The presence of abnormally loose subsoil conditions and loss of drilling fluid circulation in soils, which would normally be impervious to the thick bentonite slurry, is an indicator that conveying seepage passages may exist. This occurrence, coupled with the presence of very loose soils of an erodible texture and composition, would suggest a potential for or evidence of the internal erosion process;
2. the existence of conditions indicative of erosion-type sinkhole activity in a defined area within the exploration area of the building footprint;
3. the anomalous resistivity values recorded by the geophysical resistivity survey suggested the presence of a raveled zone near the base of the sand to silty sand and above the limestone resulting in its connection cavity system in the limestone itself. Evidence of porous zone and/or solution riddled limestone was also detected at about 30 ft (9.1 m) depth; and,
4. remediation was feasible as an acceptable and economical alternate. Development and implementation of a restoration program was, therefore, recommended.

STAGE 2 INVESTIGATION

Based on findings and recommendations from the Stage 1 investigation, a remediation program was developed for the affected portion of the project site. It consisted of elements listed below:

1. Compaction grouting (vertical and angled injection points) under the exterior wall followed by completion of a waiting period and installation of vertical piers or pin piles under the exterior load bearing wall.
2. Remediation program should be a primary grouting program consisting of 17 angled and 13 vertical injection points to 35 ft (10.6 m) foot depth and installation of a total of 57 underpinning piers at locations illustrated in Fig. 3.
3. Underpinning installation to be accomplished by first using a 4-in (10 cm) auger flight and then advancing the 3-in I.D. grout pipe to refusal.
4. Grouting and underpinning to be performed by an experienced specialty contractor experienced in low slump grout.
5. Post grouting test boring to show that all the soft zones, and soil filled voids, have been stabilized through increased standard penetration test value "N" within the proposed remediation area.

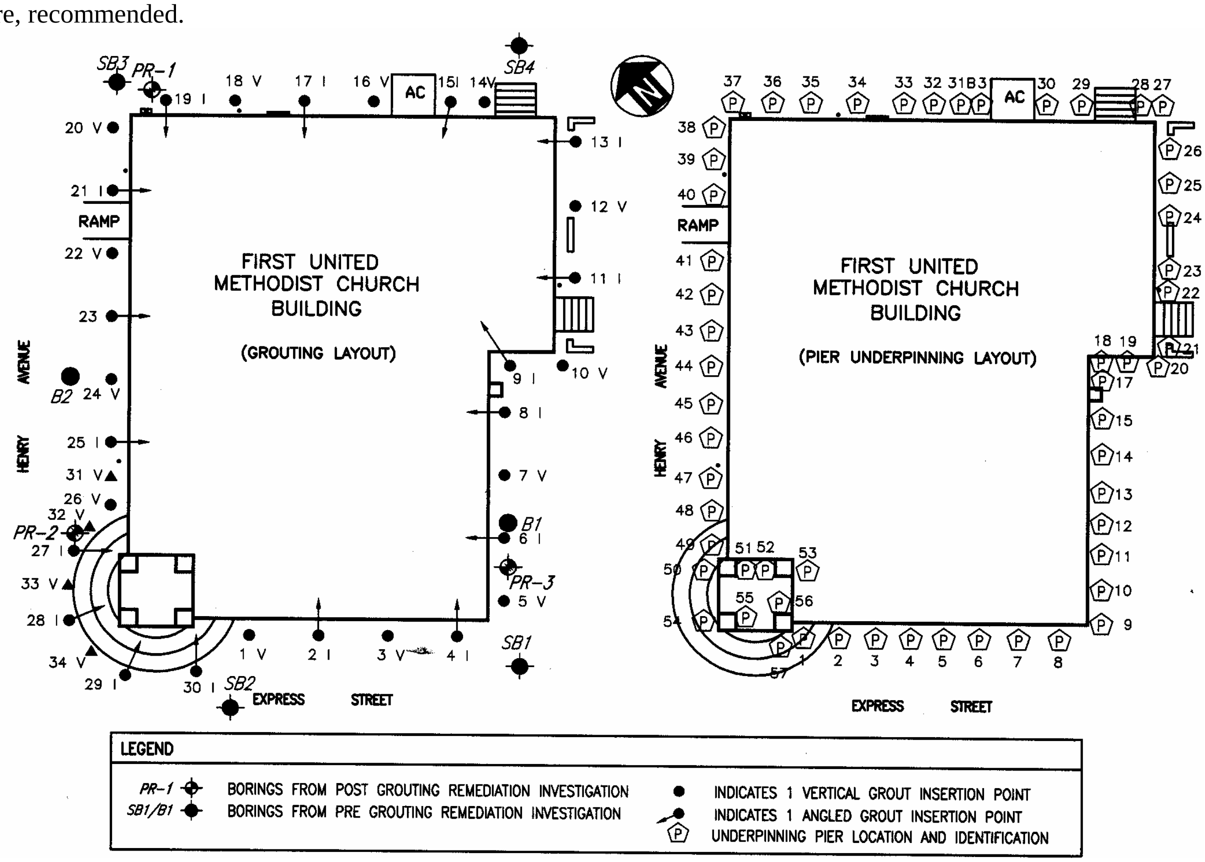

Fig. 3. Plan showing grouting points, underpinning piers, pre-and post-grouting test boring locations

Remediation program monitoring

Following a detailed review of submitted proposals, an experienced specialty remediation and grouting subcontractor, Asset Recovery Foundation Systems, St. Petersburg, Florida, was selected and recommended to the insurance carrier because of their experience, technical content and methodology of their proposal, method of remediation, and warranty ensuring the workmanship and final product.

1. Following acceptance of the primary grouting program layout consisting of 17 angled and 13 vertical injection points prepared by the specialty subcontractor, the part I primary grouting program was performed. The total amount of grout pumped was approximately 370 cubic yards for a total drilled footage of 991 ft (302 m). It was drilled at 30 locations, as illustrated in Fig. 3, and was evaluated by the consultant.
2. Engineering representatives of the consultant also monitored and documented this grouting program. Documentation included locations, drill depth, grout pumped, and date of drilling and pumping along with any unusual field observations.
3. Post grouting test borings (PR1, PR2, and PR3) were performed to 40.0 ft (12.5 m) depth. Borings indicated that while grouting had been effective in filling the raveled zones, in most of the affected areas a localized area near boring PR1 (traversed by injection points 26-1 through 28-1) near the bell-tower entrance area, had not been effectively remedied. This was evidenced by loss of drilling fluid circulation in the area. Accordingly, a secondary set of four (4) vertical grouting points identified between locations 25-1, 26-1, 27-1, 28-1, and 29-1 were completed prior to initiating pier installation underpinning program during Phase 2.
4. Post grouting borings were located near the original SPT borings (B1,B2, SB1, and SB2) in the non-grouted area as illustrated in Fig. 3.
5. Pre and Post-grouting profiles, illustrated in Fig. 4, show that the injection grouting operations performed by the specialty subcontractor have been effective and were determined to be acceptable to allow for the underpinning program to proceed.

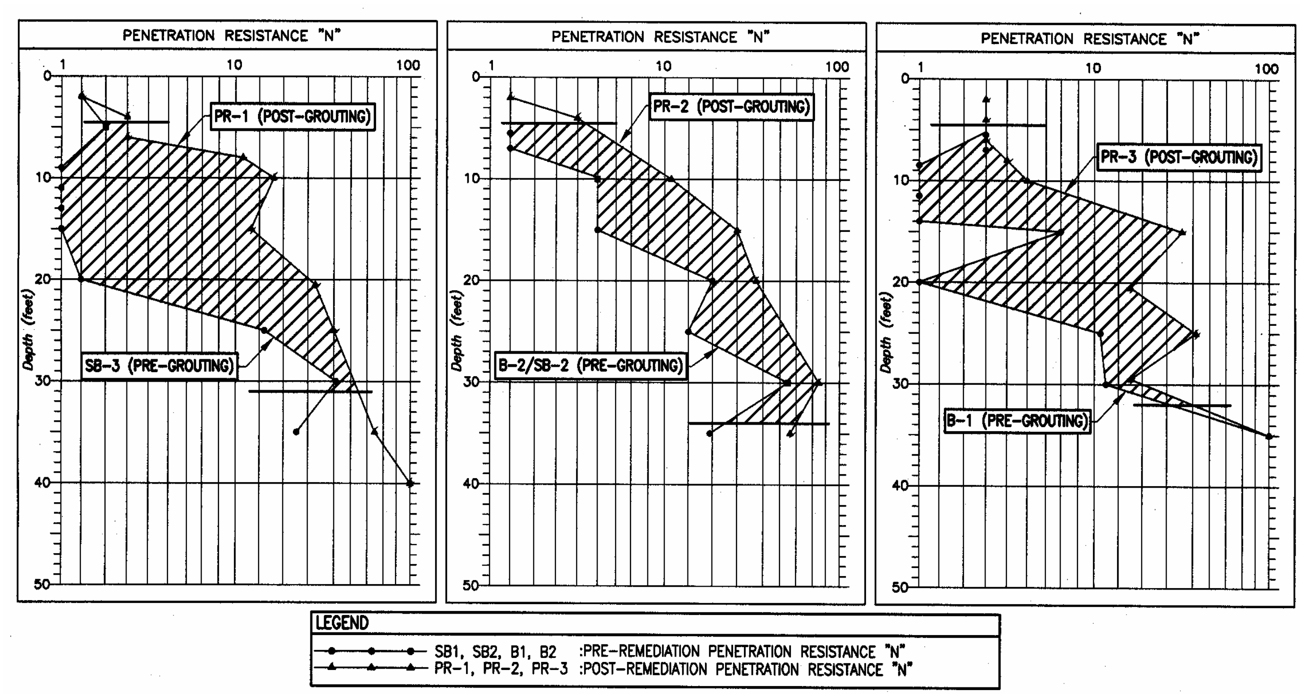

Fig. 4. Pre-and post-remediation resistance "N" value profile

6. Part II remediation program included pier layout preparation by the subcontractor and approval by the consultant. It consisted of installing a total of 57 underpinning piers at locations illustrated in Fig. 3.
7. These piers were installed at 4.0 ft to 4 ft 6-in (1.2 to 1.4 m) centers and were loaded to an adequate capacity to completely stabilize the structure without performing lift.
8. These are ATLAS Pier model AP2, 2-piece standard, and at each location a piece of structural steel (1/2-in x 8-in x 3 ft) was placed over the piers and under the wall footing to distribute the wall load and provide for a uniform load distribution over the load points.
9. Engineering representatives from the consultant monitored the stabilization and loading of the piers

and documented locations, pier length, approximately vertical lifts, and PSI at lift lock.

10. Specific locations are illustrated in Fig. 3 with lengths ranging from 12.0 ft (3.7 m) to 28.0 ft (8.5 m).

STAGE 3 INVESTIGATION

Subsequent to completion of compaction grouting operations by the specialty subcontractor, and prior to the commencement of the part two underpinning program, the grouted area was evaluated by performing SPT borings half way through the 30-day waiting period. These post-grouting test borings were performed adjacent to the pre-remediation boring locations. Pre and post grouting profiles are illustrated in Fig. 3.

Based upon review of field monitoring logs of compaction grouting provided by the specialty subcontractor, post-improvement testing, and review/evaluation of the SPT borings, it was concluded that the proposed remediation area was generally stabilized. Approval and acceptance of the compaction grouting remediation and commencement of part two of the program for the project site was, therefore, recommended. The part two underpinning program was allowed to proceed.

Engineering representatives of the consultant monitored the stabilization and loading of the piers and documented locations, pier length, approximate vertical lifts, and the pressure at lift lock. View of a typical model AP2, 2-piece standard resistance ATLAS pier (as installed) is illustrated in Fig. 5. Because of the delicate nature of the footings, (brick and mortar) and the point loading provided by the piers, special steel beam support was determined to be necessary. A steel angle beam spanning between the piers was rigged up to distribute the footing support load and alleviate stress concentration and possible cracking.

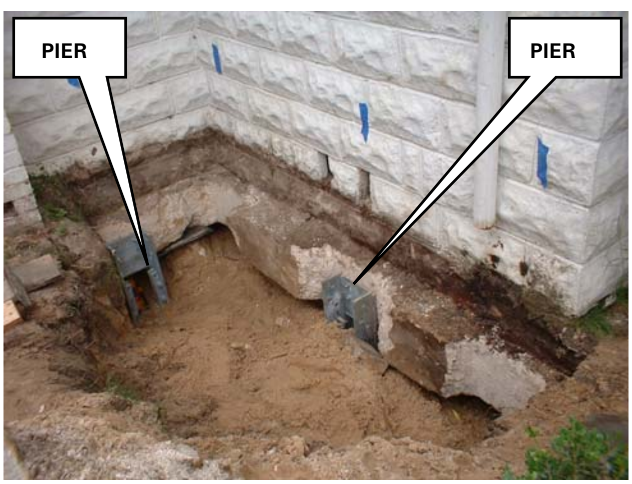

Fig 5. View of underpinning showing model AP2 ATLAS piers

Based upon field monitoring of the original and supplemental grouting operations as well as field monitoring of pier installation operations, post stabilization improvement testing and review/evaluation of the profiles, it was determined that the methods and procedures utilized and installed by the specialty contractor had been effectively and satisfactory.

It was further determined that the three part remediation program had resulted in the stabilization of the sinkhole related subsurface features and effective restoration of the church structure. Exterior and interior cosmetic repairs have since been completed by another contractor.

Conclusions

This case history substantiates the fact that the compaction grouting and/or pin pile underpinning program is a viable remedial technique and that it can be both technically and economically feasible for rectifying sinkhole problems in Karst areas.

CASE HISTORY TWO – PROJECT DESCRIPTION

In May 1993 an appreciable amount of concrete was poured into a total of 8 columns and interconnecting overhead beams of the Steam Turbine Generator (STG) structure within a power plant complex located in west central Florida. Upon removal of forms, a detailed visual examination revealed objectionable conditions within a major portion of the approximate 20 ft (6.1 m) structure height in most of the columns and connecting beams. These objectionable conditions appeared more severe in the lower 8.0 to 12.0 ft (2.4 to 3.6 m) and included extensive areas of deep/shallow honeycombing, segregated aggregate, and rock pockets. The factors resulting in these conditions were identified as i) exceeding the specified lift o deposited layer thickness during placement; and, ii) lack of consolidation of placed concrete by a smaller inadequate vibrator.

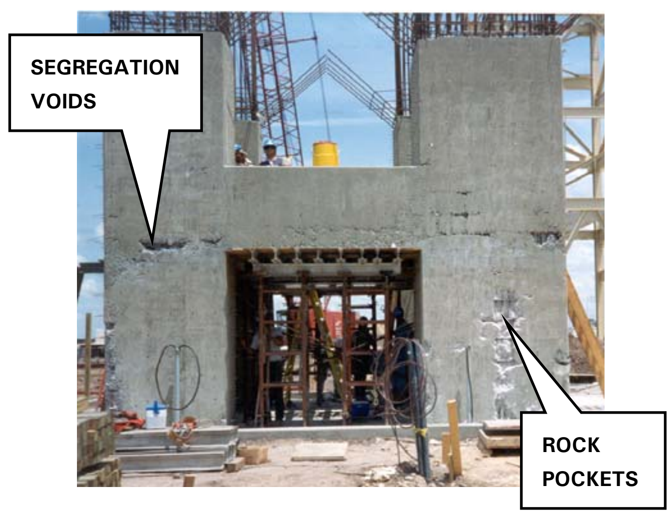

Fig. 6. View of structure upon removal of forms. Segregation, voids, and rock pockets are visible

Improper placement of concrete for a structure resulted in a visual appearance of segregated concrete and honeycombing, as illustrated in Fig. 6. Questions were raised regarding the structural integrity, and removal/replacement of the partially completed STG structure was considered a viable yet costly option. A detailed investigation was performed to determine whether damage was surficial and if the extent of any deficiency could be addressed by repairing the shallow and deep voided areas or removal and replacement of the structure was, in fact, necessary. Failure or alleged failure was defined as an unacceptable difference between expected and observed performance. A 3-stage investigation was conducted.

STAGE 1 INVESTIGATION

Stage I consisted of a Pre-Remediation Survey (PreRS) encompassing an external and internal condition reconnaissance.

External condition survey

The initial phase of the PreRS consisted of visual inspection, logging, photographs, and documentation of each column of the 8 to evaluate the surficial damage and to detail the depth of voiding and honeycombing. The exterior honeycombing and extensive voiding which, in many cases, exposed rebar was seen to be surficial. The column located in the extreme northeast portion of the STG Structure, showed extensive honeycombing across the base. In addition, the base of the crossbeam in the northwest portion of the STG structure, spanning between the 2 columns, showed honeycombing and voiding. An internal condition survey was carried out through a field program that consisted of these elements.

External condition survey

Field coring and laboratory evaluation program. A second phase of the PreRS consisted of coring horizontally at 22 strategic locations (bottom, middle, and top sections of columns and 1 overhead beam spanning between columns). A series of 2.75 in(70 mm) diameter cores were retrieved from each borehole in 1-ft (305 mm) sections. A total of 4 to 5 core runs were retrieved per borehole. These cores were removed from the core barrel, identified for location, wrapped in plastic to preserve their natural moisture condition, and secured in core boxes for shipment to the testing laboratory.

A total of 66 cores were sawed, trimmed, capped, and cured. Post curing cores were subjected to compressive strength tests and revealed 3,670 psi (25.33 N/mm^2), 4,340 psi (29.9 N/mm^2), and 4,960 psi (34.2 N/mm^2) at 7.21, and 28 days respectively.

In addition, these 2.75-in (70-mm) diameter cores were removed and subjected to a petrographic examination using a petrographic microscope to determine their integrity, quality, and constituents. The petrographic examination showed their past/aggregate quality ratio and bonding characteristics and their-void system in the paste. Petrographic examination revealed the quality of the concrete to be relatively good, bonding to be intact in major portions of core with the exception of the surface where honeycombing was apparent. In general, the paste/aggregate ration ranged from 35/65 to 60/40 with the average being 50/50. The water-cement ratio was interpreted at 0.45 to 0.50. Small spherical voids, typical of entrained air, were also present in the concrete. The concrete appeared to be fairly well consolidated and exhibited no evidence of detrimental internal past-aggregate reaction.

TV video examination. The third part of the PreRS was to carry out a video examination on the interior surfaces of each borehole to confirm the concrete quality and to identify any nonconforming features of past/aggregate. This Borehole Camera Survey (BCS) was carried out by utilizing a 1.5-in (38-mm) diameter Reese borehole video camera. It was equipped with a 90° side viewing lens capable of rotating 360° and having a 7X magnification. Borehole inspection was carried out by moving the camera slowly through each of the 22 boreholes to allow examination of the inside surface for any unusual characteristics within the concrete.

Packer tests. A total of 6 strategic locations were selected for the performance of Packer tests. These 6 locations were cored and cores were removed. The purpose of these Packer tests was to determine competency of in-place concrete as related to internal voices or large honeycombing which might affect the structural soundness.

The Packer test was performed by sealing one or both ends of the cored hole into or through the structure, and forcing water pressure into the core hole. Water was forced through a device comprised of a water meter capable of reading to 1/100th of a gallon, and a pressure gauge capable of reading the pressure build-up, with a shut off valve to close off and maintain pressure.

The pressure testing for this project consisted of forcing water under pressure through the device using the on-site water pressure, allowing the valve to remain open with a constant pressure of 30 to 50 psi (2.1 to 3.5 kg/cm2), and measuring the amount of water, if any, required to maintain the pressure in the cored hole at the maximum pressure achieved. The STG structure layout identifying columns, overhead beams, core TV video, as well as Packer test locations is shown in Fig. 7.

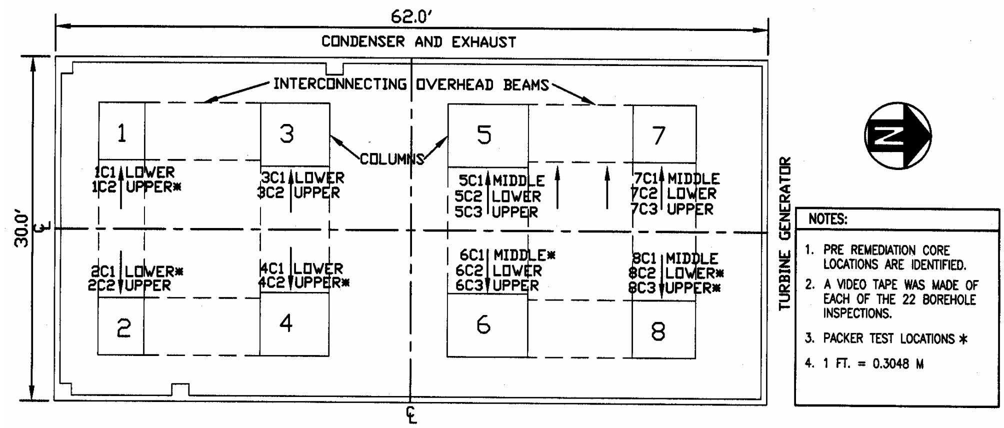

Fig. 7. STG structure layout identifying columns, overhead beams, core locations, and Packer test locations

Conclusions and remarks

Forensic investigation consisting of a visual, video, and petrographic examination, as well as results of compressive strength tests performed on cores revealed that:

- structural soundness and competency of the concrete in the 8 columns and beams was intact;
- the columns consisted of satisfactory quality concrete and retained the mass and integrity for which they were intended;
- the exterior honeycombing and extensive voiding, which in many cases exposed the rebar, was generally surficial; and,
- development and implementation of a restoration program was also recommended that would repair the external honeycombing and voiding, following a complete removal of all loose and non-intact paste and aggregate from the affected areas.

It was, therefore, concluded that an extensive restoration is feasible and should be pursued as an acceptable and economic alternate.

STAGE 2 INVESTIGATION

Based on findings and recommendations from the Stage I investigation, the retained consultant developed a remediation and restoration program to rehabilitate the STG structure to its originally intended design. This remediation and restoration program consisted of:

1. preparation of guideline specifications as well as a remediation and restoration program.
2. chipping and hydroblasting of honeycombed and void areas;
3. spraying or coating hydroblasted and dried surface using a SIKA bonding agent to create a bonding badge;
4. forming, pumping, and pouring a high-strength grout mixture of SIKA-monotop;
5. using vibrators during pours to ensure complete filling or all honeycombed and voided areas;
6. hand-toweling into deeper voids to ensure complete filling; and,
7. reviewing proposals from various specialty subcontractors for the remediation and restoration program that would effectively restore shallow and deep voided areas as well as the surficial honeycombing.

During all stages of the Phase II remediation and restoration program, the retained consultant provided inspection services for various operations, quality of the grout, forming, and the application procedures. Any defects in workmanship or grouting quality were immediately noted and corrected by the specialty subcontractor. Overhead beam areas were determined to be lacking in bond as determined by visual separation in core specimens). These areas revealed insufficient bonding, and the specialty subcontractor was directed to chip the grout out for inspection and was further directed to re-perform the repairs. A typical photograph showing sections of a pre- and post-remediated column is illustrated in Figure 8.

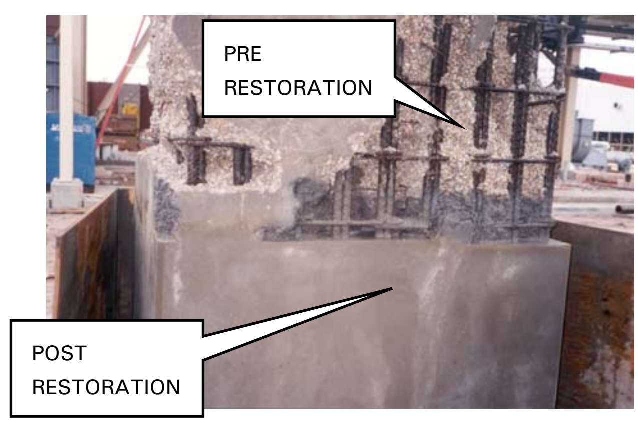

Fig. 8. View of column with pre- and post-restoration areas

STAGE 3 INVESTIGATION

The purpose of the Stage II Post-Remediation Survey (PostRS) was to assure integrity and competency of the restored concrete columns in the STG area. The scope of work included:

1. a detailed visual inspection of the chipped and hydroblasted portions of the honeycombed and voided areas and a selection of a number of areas for coring;
2. monitoring of repair procedures and surveying repaired/restored areas;
3. evaluation of the remediation program by recovering and testing cores drilled through repaired grout materials and original concrete of each of the columns at 14 locations;
4. performing microscopic petrographic examination and then trimming for compressive strength tests; and,
5. evaluating the bonding characteristics of the original and grouted portions by performing tensile strength tests as well as a microscopic evaluation.

The initial phase of the PostRS consisted of coring horizontally a total of 14 cores through the repaired grout/concrete bond in the upper and lower sections of each of the columns (refer to Fig. 8). These 2.75-in (70-mm) diameter cores were removed and examined by petrographic analysis using microscopic examination. The strength testing of concrete revealed the quality of the concrete to be excellent, and bonding characteristics to be intact in a major portion of the cores.

A total of 14 core specimens were sawed, trimmed, capped, and cured. Following completion of curing, thee core specimens were subjected to compressive strength and direct tensile strength tests at 7 and 28 day time intervals after molding. The compressive strength average for the 14 core specimens at 7 days was 4,400 psi (30.3 N/mm^2) and at 27 days was 5,800 psi (40.0 N/mm^2).

The core specimens subjected to direct tensile strength tests exhibited failure within the original concrete with the failure mode to be through paste and aggregate. In addition, the average direct tensile strength values of the core specimens that were subjected to remediation or restoration compared favorably (i.e. equal or greater) with that of the original concrete core specimens. Failure characteristics were noted to be normal.

The STG structure layout showing areas exhibiting deficiencies and requiring remediation as well as the PostRS core locations is shown in Fig. 9 below.

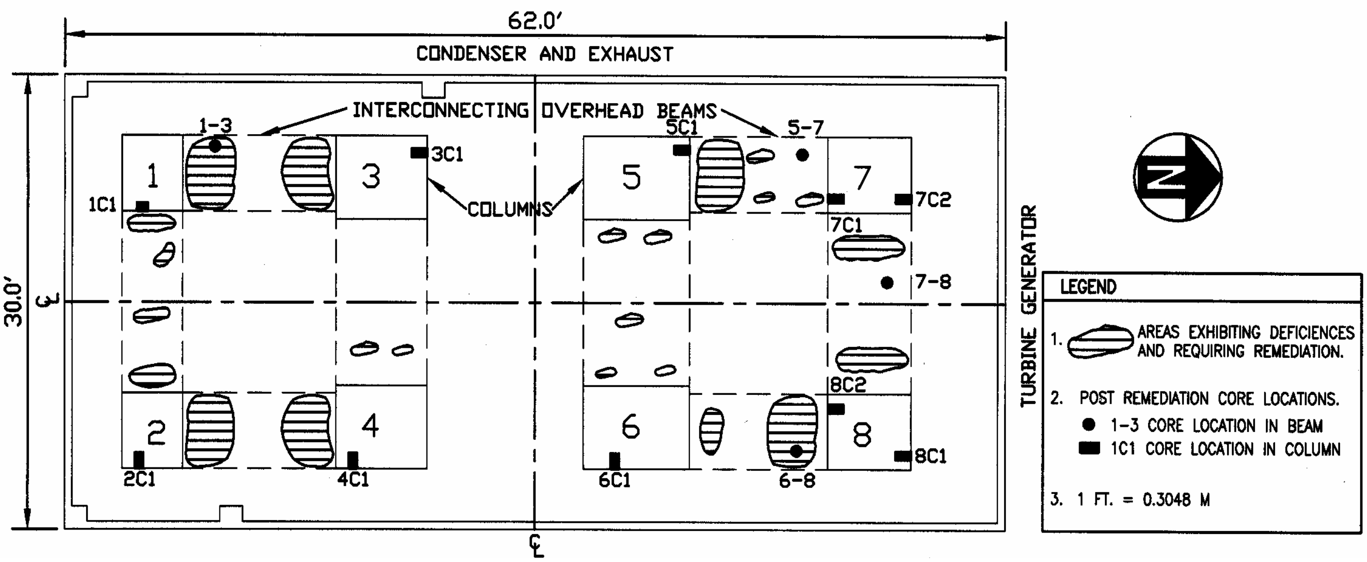

Fig. 9. STG structure layout showing deficient area and post remediation core locations

Concluding remarks

- Following the close coordination between the consultant and the owner's representative, completion of the Stage II investigation, and based upon results of Stage III investigation, it was determined that the repair of 8 columns and overhead beams had been achieved satisfactorily.
- The final repair resulted in restoration of the STG structure to its originally planned and designed dimensions and load conditions, as illustrated in Fig. 10.
- Application of forensic engineering principles was a key factor in restoration of the STG structure.
- As a result of the monitored and satisfactory remediation program, the consultant recommended acceptance of the restored structure.

Fig, 10. View of restored STG structure.

ACKNOWLEDGEMENTS

The opportunity to perform the services described herein provided an interesting exercise in the planning and execution of this unique project. The information herein is from projects where the author and his firm, ASC geosciences, inc. were involved as the geotechnical engineering and testing consultant.

ASC expresses its appreciation to the other project team members: For Case History 1: United Methodist Church, Branford, Florida, Owner; and, Asset Recovery Foundation Systems, St. Petersburg, Florida, Specialty Contractor. For Case History 2: Zurn/Nepco, Portland, Maine, General Contractor; Preservation Services, Inc., Tampa, Florida, Specialty Subcontractor; and, Tom Carmichael, P.G., Petrography Consultant,

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