FOUNDATION REMEDIATION USING MICROPILES AND LOW MOBILITY GROUTING (LMG) AT SANDY COVE CONDOMINIUM, BARBADOS

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BACKGROUND

The west coast of Barbados is home to platinum white beaches, the sparkling Caribbean Sea, the rich and famous and some extremely challenging foundation conditions. The island itself is covered almost in its entirety by a coralline limestone rockmass, formed in a series of terraces up to 260 feet (79 m) thick. The terraces are the remains of ancient coral reefs. The coralline limestone is technically a rock, but in many places it has strength properties approaching that of a hard soil. The behavior of the coralline as a foundation stratum is further complicated by the presence of relict rock fabric including incipient fracturing, numerous voids, fissures and joints. At or near the ground surface, the rockmass can also have a locally indurated (or hardened) ‘cap’ present including along shorelines in the crest zones of cliffs (Carter et al., 2008).

In 2005, construction started on the Sandy Cove development located on the west coast of the island between Bridgetown and Holetown in the Parish of St. James. The project includes a six-storey luxury condominium complex, with a one level basement (on the northern half of building only) and five levels of above ground units. The building is set-back approximately 50 to 65 feet (15 to 20 m) from the edge of a 10 to 15 foot (3 to 4.5 m) high coral cliff bordering the Caribbean Sea to the west of the building. An approximately 12 foot (3.6 m) deep gully/drainage channel exists immediately adjacent to the north side of the building. The building structure itself is comprised of reinforced concrete and concrete block-wall construction designed to be supported on shallow strip footings founded on engineered fill and/or directly on the native coralline limestone rockmass. Figure 1 provides a view of the condominium from the ocean side, while Figure 2 shows the coral cliff.

Figure 1: Sandy Cove Condominium

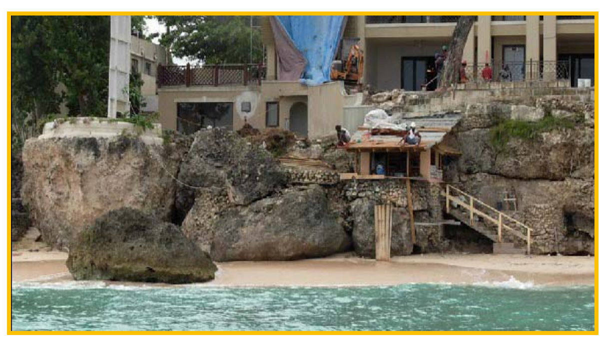

Figure 2: Coral Cliff at Condominium

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During the initial site grading and excavation for the basement and foundations, several small caverns, voids, fractures and zones of very loose material were encountered at the footing level in the coralline foundation stratum. These areas were addressed during construction by one of several methods including localized sub-excavation and replacement with well compacted engineered fill, backfilling of open voids/fractures from surface with high slump concrete and, at one location, the installation of six, 22 foot (6.7 m) long, 1.5 foot (450 mm) diameter, augered piles. In addition, due to the number of anomalies encountered, the foundation design on the northern half of the building (in the basement area) was modified from strip footings to a reinforced mat/raft that was on average about 1 foot (300 mm) thick, but locally thickened to 2 feet (600 mm) at load bearing wall/column locations. The foundations, building structure and exterior shell were substantially completed in April 2006 without incident. Between April and August 2006, the building performed as designed while interior and exterior finishes were in progress. In August 2006, however, following several days of heavy seas, cracking appeared on several walls in the northwest corner of the building, near the intersection of the ocean-side cliff face (to the west) and drainage gully (to the north). Observation of these initial cracks, mostly via crack plates and markings, suggested little change over the next few months and accordingly the cracks were patched and interior finishing was continued. No new cracking or any other observable signs of building movement were noted from this time until early February 2007 when again, following violent sea conditions, the original cracks re-opened and additional sets of cracks appeared and continued to grow.

INVESTIGATION AND MECHANISM ASSESSMENT

Golder Associates Ltd. was contacted by the Owner in February 2007 to evaluate the foundation conditions at the site and the cause(s) of the movement/cracking in the structure and to propose potential remedial solutions. The subsequent investigation ultimately included advancing several boreholes from within and beside the existing building with down-hole video camera survey, geologic surface mapping of the exposed coral features around the site, crack surveys and the installation of monitoring equipment on the building including crack gauges and precise levelling points.

The results of the subsurface investigation revealed that the building was founded on a highly variable, vuggy, heterogeneous, weak coralline rockmass containing numerous voids, sub-horizontal and sub-vertical fissures and joints. Numerous voids were encountered in the stratum during the investigation, as evidenced by ‘drops’ in the drill-rods, typically ranging from about 4 inches (100 mm) to 3 feet (900 mm), however at one location, a drop of over 8 feet (2.4 m) was recorded. The boreholes also revealed the presence of a less friable, less voided and generally more competent zone of coralline rock at a depth of about 50 feet (15 m). Figures 3 and 4 show borehole camera images from beneath the structure. The investigation also found that a hardened coralline cap, up to about 10 feet thick, was present around parts of the site including on the remnant coral sea stacks immediately in front of the west side of the building (i.e. on the shore side). This more competent material likely existed over much, if not all, of the rockmass within the building footprint prior to construction. However, excavation for the basement had likely removed most of this cap in the northern half of the building. Mapping of the coral cliff faces adjacent to the building indicated that notching/jointing was present in the coral rock mass near sea level along prominent sub-horizontal weaknesses combined with sub-vertical major fissures extending landward from the sea to below and beyond the building.

Figure 3: Borehole Camera at 13 foot depth

Figure 4: 1 meter high Cave discovered under Structure

The distress cracking in the building appeared on all five levels of the main floors of the building (Level 1 to 5) and also in the basement (Level 0). The majority of the cracking was concentrated in the northwest corner of the building, principally in the basement and on the first three floors. However, cracking was also observed (albeit less severe) in the southwest corner and on the west central side and in the northeast corner of the building. In general the cracking typically comprised about 45° oriented flexural shear cracking on both east-west and north-south structural walls, however some sub-vertical (about 90°) tensile cracking was also observed. Figure 5 shows a map of some of the exterior cracks observed on the structure. Figure 6 is a photograph of typical interior cracking.

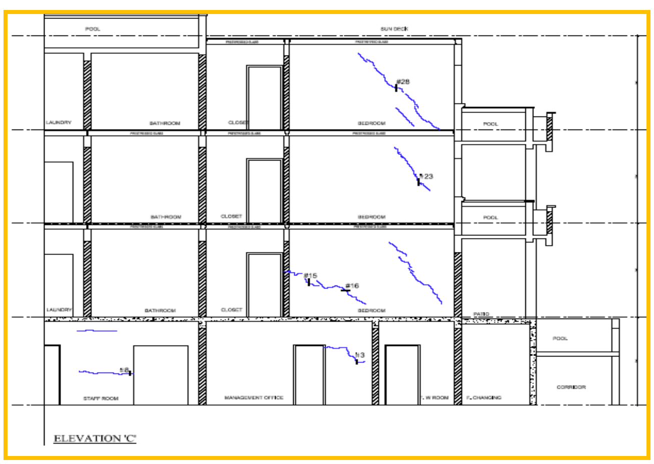

Figure 5: Cracking on Exterior of Building

Figure 6: Interior Cracking

Numerical analysis (continuum, FLAC, and discrete element analysis-UDEC) was carried out on two sections through the northwest of the building to provide insight into the potential mechanisms that resulted in the observed crack patterns in the structure walls. Various vertical zones of weakness along the observed pattern of sub-vertical jointing across the site were included in the models. The models also incorporated the structural modeling of the building shell itself such that vertical displacements and shear and principal stresses within the walls could be calculated and cracking patterns could then be interpreted based on the stress trajectories. By comparing the interpreted crack patterns from the numerical models with the actual cracking observed in the building, an in-depth evaluation of the most likely causes of the cracking patterns was possible thus aiding assessment of the most likely processes controlling the observed building distress.

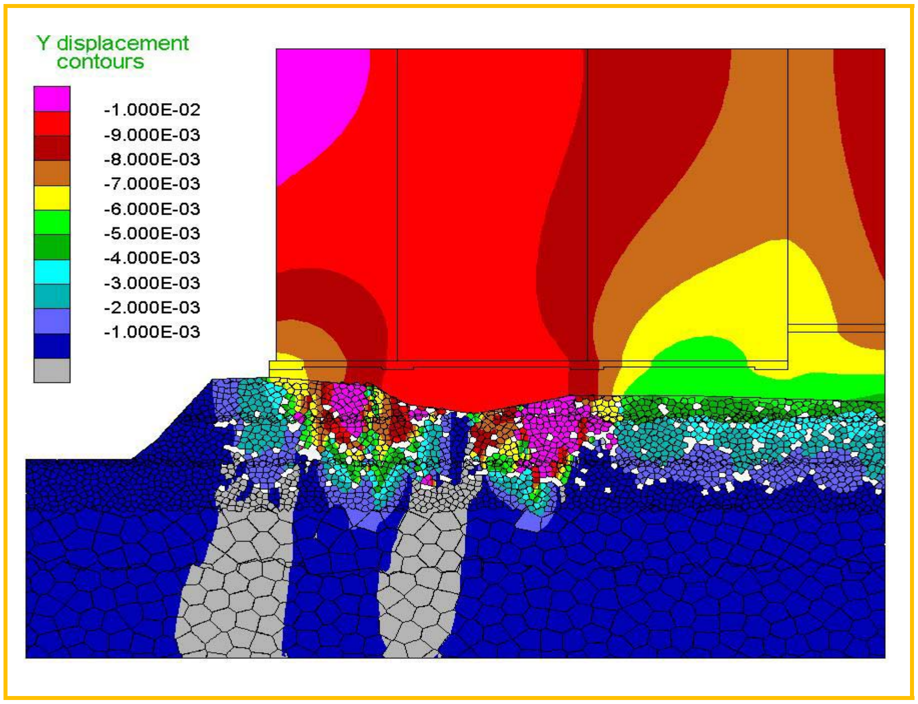

Figure 7: Model of Zones of Vertical Weakness with Displacements (meters)

Based on the modeling, void creation as well as undercutting of the cliff face (from wave action), in conjunction with a weakened rock mass along the sub-vertical jointing, showed the most convincing settlement and interpreted cracking patterns in the building structure. Foundation degradation (i.e. progressive undermining, loss of support and associated building movement) was likely exacerbated by migration of fines from natural fissures and void zones within the coralline rock mass during violent sea conditions. In addition, the fact that the northern half of the structure was constructed with a lower foundation (due to the inclusion of a basement level) was likely a key factor in the observed building behavior. It was concluded that the removal of the more competent coralline cap material in this area, higher foundation loads as a result of the additional level and a founding level in closer proximity to the weak subsurface conditions resulted in the observed building distress. These findings were the basis for the design of remediation approaches.

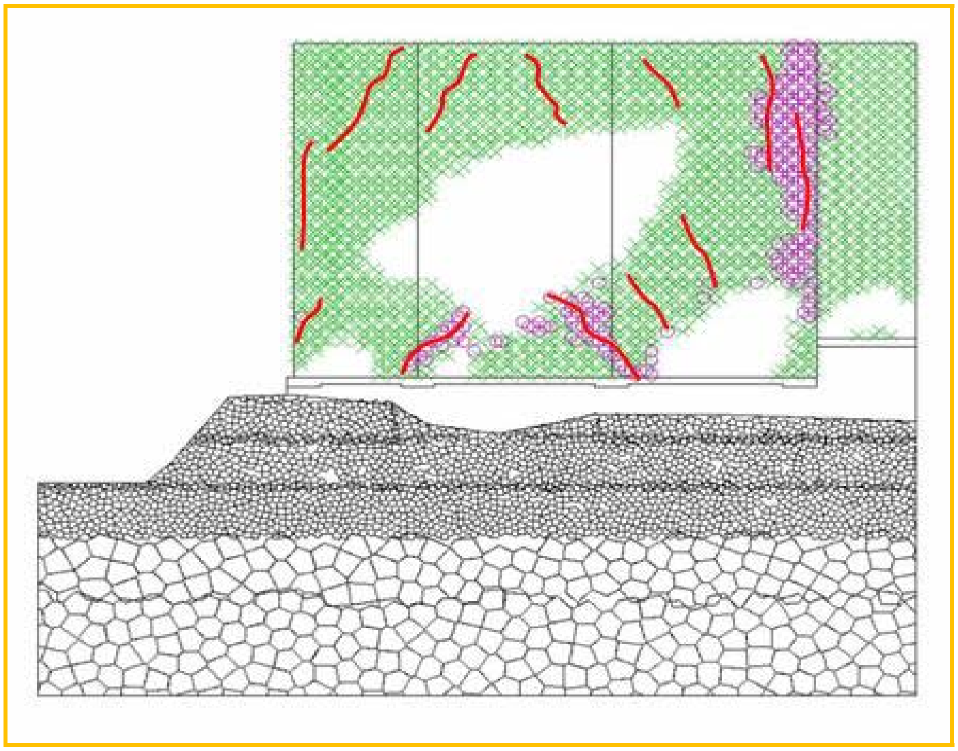

Figure 8: Interpreted Crack Patterns which Matched Observed Cracking

REMEDIATION CONCEPTS AND DESIGN

Considering the subsurface conditions at the site and the fact that any foundation remediation would have to be constructed from within and around the existing building, a combination of micropiles and grouting was conceived as a probable solution. As such, in March 2007, Geosystems L.P. was asked to join the consultant team to provide expertise on micropiles and grouting to refine the remediation design and to guide contractor selection for the project.

Given the mechanisms believed to be causing the building movement, the remediation was conceptualized to comprise three main components:

(i) creation of a barrier (i.e. a buried, sub-surface seawall/grouted curtain) to prevent further marine intervention/energy influx into the subsurface zone beneath the building;
(ii) provision of additional direct support to the foundation on three sides of the perimeter of the building; and
(iii) improvement of the load-bearing capacity of the existing weak coralline subsurface strata below the interior of the northern half of building.

The subsurface seawall was designed to be comprised of two rows of 5 1/2 inch (140 mm) diameter micropiles. The outer row of near vertical micropiles was designed on a 15 degree inclination from the vertical, parallel to the sides of the building and extended down into the more competent coralline rock at a depth of 50 foot (15 m). The inner row of battered micropiles was designed to be installed perpendicular to the sides of the building at inclinations alternating between 30° and 45° from the vertical and depths varying from 30 feet to 60 feet to extend below the existing building. The tops of the micropiles were designed to be formed into a concrete cap/grade beam that was structurally connected to the existing building footings and/or to the foundation wall (See Figures 8 and 9). The simultaneous grouting, carried out as part of the micropile installation and via supplemental grout-only holes, was laid out to essentially back-fill the washed out zones and any open and interconnected fissures and fractures to stiffen the in situ rockmass, reduce void porosity and hence minimize potential for future vertical settlement. In addition to the micropile wall on the exterior of the building, the design also included a number of near vertical micropiles to be installed within the interior of the building through the basement foundation slab in the areas of highest wall loads and largest measured vertical movement to date. These interior micropiles were supplemented by a series of grout-only holes to provide additional void filling and foundation stiffening at key locations on the interior to minimize future vertical differential settlements.

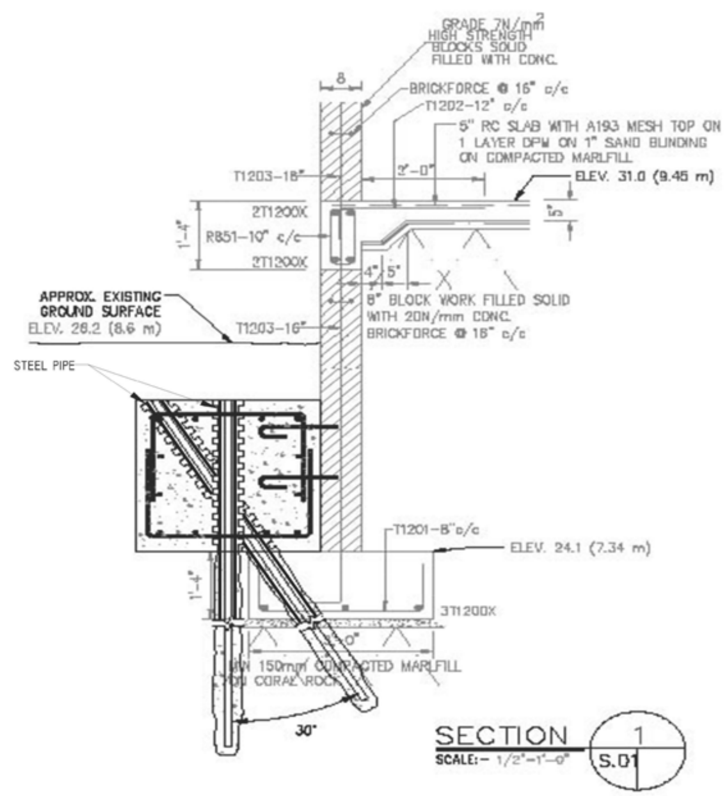

Figure 8: Typical Micropile Grade Beam Attachment

Figure 9: New Grade Beam around Foundation

CONTRACTOR SELECTION

Given the need to have the remediation largely completed before the onset of the hurricane season, the overall schedule was extremely compressed. This meant that the site assessment and preliminary remedial design had to progress during the same period when the contractor was selected, and a fast mobilization to the island was required. Furthermore, the precise scope of the remediation, and the selection of the most appropriate means and methods could only be determined when the work got underway, given the need to implement the remediation in a responsive fashion while observing the foundation and the structure itself. This meant that the contractor's expertise and experience would be invaluable as an integral part of engineering the solution in real time and at all times on this project such input was sought, and given. Overall, the fast track nature of the work would tend to place severe interpersonal strains between the respective groups of personalities represented on site, including several sets of specialist consultants, a general contractor, the specialty foundation/grouting subcontractor, the project management team and, of course, the owner himself. Such a combination of factors strongly favors the creation of an ‘Alliance,’ (Carter and Bruce, 2005) and at the Sandy Cove project, key elements of alliancing were implemented to assure selection of the correct specialty contractor, and to maintain excellent communications, problem resolution mechanisms, compliance to schedule, and cost management structures throughout the project duration.

During contractor procurement, the engineer compiled a data summary and a conceptual design which was circulated to a small group of international specialty contractors believed to have the requisite resources and experience. These contractors then submitted a preliminary assessment report - including statements of commitment regarding their ability to meet the schedule, and their commitment to working within the Alliance framework. A short-list of three potential bidders was then prepared by the owner-engineer team and these three companies were invited to the island for individual rounds of site visits and facilitated technical meetings and interviews. Of special significance to the evaluation team was the ability and willingness of the respective potential bidders to make suggestions regarding the design and construction which would significantly benefit the project if they were successful. The outcome of this process was that the contractor, Hayward Baker Inc, was selected immediately after the interview period was over, and their commitment was given to mobilize as promptly as possible. There is no doubt that the procurement of the most appropriate contractor was a key factor in the excellent quality and pace of work which was conducted, and the extremely functional and efficient communication framework under which it proceeded.

The structural design of the micropiles was prepared by Golder and Hayward Baker and reviewed by Mueser Rutledge. The pile consisted of a full-length grade 75 ksi (517 MPa) all-thread bar from DSI and included a short steel pipe in the upper zone of the micropiles. A total of four tension load tests were performed on sacrificial piles near the building (Figure 10). Two tests were completed with LMG grout and two with High Mobility Grout (HMG) (Figure 11). A sample performance of an LMG test is shown on Figure 12.

Figure 10: Micropile Tension Testing

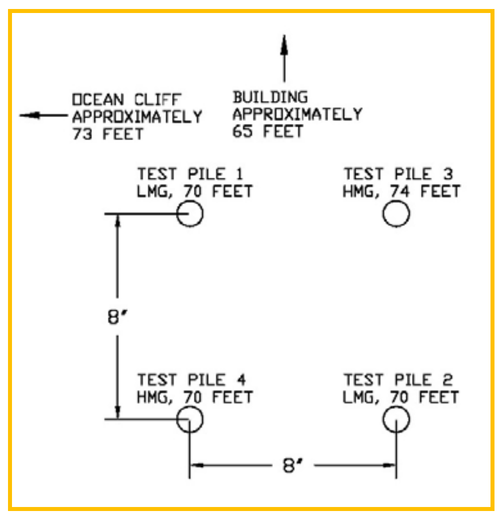

Figure 11: Test Pile Layout

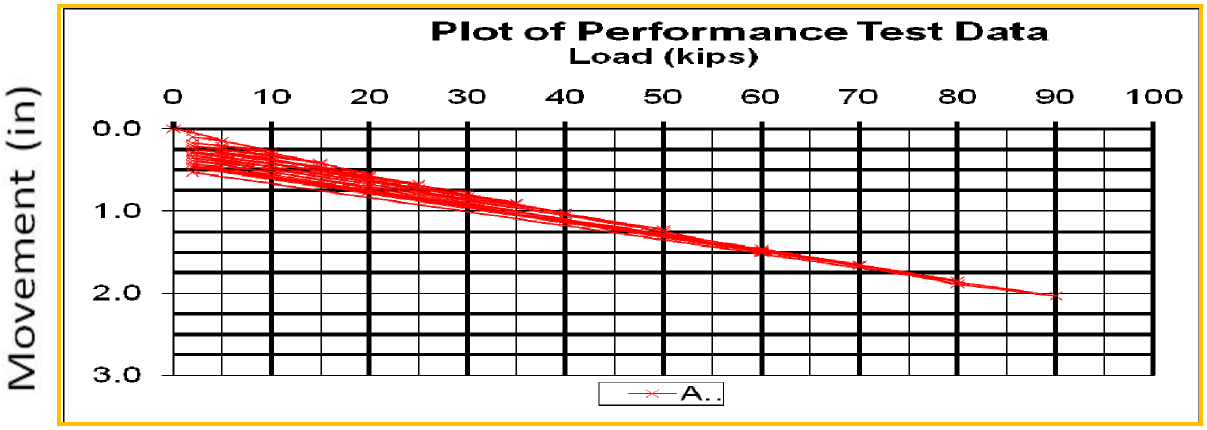

Figure 12: Tension Test Result from 74 foot long LMG Micropile (60 ft free length) Monitoring During and Following Construction

During the course of the remediation work, the conditions encountered during drilling and the volume of grout injected (or 'take') at discrete depth intervals in each hole was carefully recorded. In this manner, the geological model developed as part of the design phase and formulated into the numerical modelling was adjusted and refined as construction proceeded. Refinements to the design and borehole layouts were undertaken in near real-time as additional subsurface information was obtained during the construction. Records were updated daily and the grout-take data was tracked using 2-D and 3-D graphical models so that the weakest conditions (i.e. most voided) in the subsurface could be readily identified. Figure 13 shows the 3-D model, where higher grout take volumes are shown with larger spheres. These areas were then targeted with additional grout-only holes during the course of the production work. At the completion of the works, data had been acquired from the drilling and grouting of 174 micropiles, during which 1000 yd^3 (750 m^3) of low-mobility grout was injected into the voided areas of the foundation around the perimeter and below the interior of the building.

In addition to documenting the drilling and grout-takes during the remediation, the building was regularly monitored for settlement or heave, tilt and crack propagation throughout the period of construction and after completion. The building monitoring instrumentation included a suite of electro-levels, tilt meters, crack gauges, precise leveling points and prisms. The instrumentation data showed the building responding to the construction by initial downward (i.e. settlement) movement as a result of the disturbance to the weak soils during drilling and flushing for the micropiling operations followed by upward (i.e. heave) movement as a result of the pressure grouting. In general a trend of increasing stabilization was observed in the instrumentation throughout the remediation program, as each area of the building was underpinned and grouted.

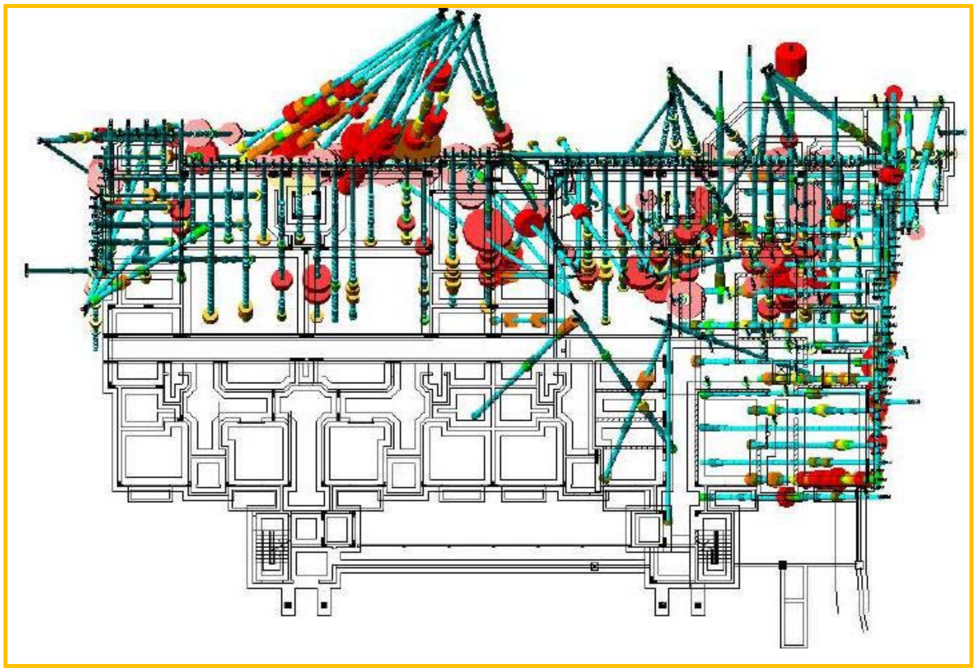

Figure 13: 3-Dimensional Illustration of Grout Locations and Quantities

The robustness of the remediation fix has been tested by both marine and non-marine dynamic stresses. During the one year post-construction monitoring period, heavy seas with recorded offshore wave heights equal to or even greater than those recorded during the periods of the original crack initiation occurred. In addition, the structure was subjected to a magnitude 7.4 earthquake (which occurred in the eastern Caribbean with an epicenter just north of Martinique on November 29, 2007). It was estimated that the quake measured about magnitude 3.0 in Barbados (www.caribbean360.com). There was no settlement or crack development as a result of these events.

CONCLUSIONS

The remediation and improvement of the building foundation ultimately included the following works:

- Installation of a 290 foot (88 m) long sub-surface 'sea-wall' barrier/grouted cut-off curtain around three sides of the building;
- Direct support by 137 – approximately 70 feet long (21 m), 5 1/2 inch (140 mm) diameter micropiles underpinning the edges of three sides of the building (north, south and west);
- Indirect support by 37 – approximately 65 feet (20 m) long, 5 1/2 inch (140 mm) diameter micropiles installed along heavily loaded walls below the interior of the northern portion of the building; and
- Grouting of voids and interconnected fissures/fractures in the subsurface below the building.

The micropiling and infill grouting program achieved its two main design objectives of:

- Creating a 'sub-surface sea-wall' to prevent further wave-induced flushing and migration and loss of fine material from the subsurface below the building; and,
- Providing enhanced consolidation and improvement of the foundation rockmass to affect an overall stiffening of the subsurface to improve the load-bearing capacity of the originally weak and voided, coralline rockmass.

No damage (or even reactivation of earlier patterns of adverse cracking) occurred to the structure in response to the passage of Hurricane Dean (in August 2007 toward the completion of the remediation works). Also, the structure was unscathed after a large earthquake (magnitude 7.4) occurred shortly following completion of the remediation in November 2007. These events demonstrate the effectiveness of the micropiling and grouting completed on the project, which has stabilized the structure from any further movement to date.

ACKNOWLEDGEMENTS

Trevor Carter and Joseph Carvalho were technical leaders for Golder Associates for geological and structural engineering for the project and helped in the writing of this paper. Frank Arland from Mueser Rutledge performed a peer review of the design and his independent assessment was key to the project success. Donald Bruce of Geosytstems, L.P. was the micropile, grouting and alliance consultant. Steve Buckner was the project manager for Hayward Baker. Ryan Smith was the field engineer, Ed Lehman was the superintendent and Tom Finn was the technical drilling and grouting specialist for Hayward Baker. The owner was James Burdess of Cliff Tops, Ltd. Monir was the instrumentation consultant.

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