Micropile and Low-Mobility Grouting Foundation Remediation for a Coastal Development in Barbados

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ABSTRACT
This case study describes the investigation, design and implementation of foundation improvements comprising micropiles and grouting to remediate the settlement distress of an existing building founded on very weak, vuggy, uncemented coralline limestone located on the coast of the Caribbean Sea in Barbados. The foundation remediation ultimately included the construction of an 88 m long sub-surface 'sea-wall', the installation of 174 micropiles (providing direct and indirect support to the building) and the grouting of voids and interconnected fissures/fractures in the subsurface below the building. An in-depth study of the causes of the settlement, a flexible design and close monitoring of the drilling and grouting during construction were all essential to the success of the project. In addition, contractor procurement and operation based on the 'Alliance' concept resulted in an excellent consultant-contractor team relationship throughout and was the key to the completion of the work within the tight schedule required by the client.

RÉSUMÉ
Cette étude de cas décrit l'investigation, la conception et l'implémentation des améliorations apportées aux fondations d'une bâtisse existante fondée sur des sols très faible, vacuolaires, non-cimentés d'origine calcaire sur la cote des caraïbes au Barbade. Les améliorations apportées pour contrer au tassement comprennent; l'installation de micros pieux et le fonçage des puits par la méthode de cimentation. La réhabilitation des fondations inclue la construction d'un ouvrage longitudinal souterrain de 88 mètres de longueur, l'installation de 174 micros pieux (qui permettaient le support direct et indirect de la bâtisse) et l'injection de ciment dans le but de combler les vides, fractures et fissures interconnectés situés sous la bâtisse. La réussite d'un tel projet découle de l'étude approfondie des causes du tassement, d'un design flexible et d'une surveillance en continu des travaux de construction. De plus, l'élaboration des équipes de travail c'est basée sur un concept de partenariat entre le maître d'œuvre et les sous-traitants, ce qui a permis de réaliser l'ouvrage dans un court laps de temps.

1 INTRODUCTION

The Sandy Cove development is located on the west coast of Barbados between Bridgetown and Holetown in the Parish of St. James. Phase 1 of the project includes a six-storey luxury condominium complex, including a one level basement (on the northern half of the building only) and five levels of above ground units, set-back approximately 15 m to 20 m from the edge of a 3 m to 5 m high coralline cliff bordering the Caribbean Sea to the west of the building. An approximately 4 m deep gully/drainage channel exists immediately adjacent to the north side of the building. The building structure is comprised of reinforced concrete and concrete block-wall construction designed to be supported on shallow strip footings founded on engineered fill and/or the native coralline limestone 'rockmass'.

This paper briefly describes the original building construction and initial distress, the subsequent investigation and mechanism assessment, the remediation design, the contractor selection process, the foundation improvements and subsequent performance of the Phase 1 Sandy Cove Development.

2 ORIGINAL CONSTRUCTION

Construction, involving site grading and excavation for the basement and foundations, commenced on the northern half of the site in March 2005. Between March and May 2005, several small caverns and fractures (some clay in-filled) were encountered. As a result of finding these features, a local geotechnical consultant was retained who performed geophysical surveys at the site. The Ground Penetrating Radar (GPR) and Electrical Resistivity (ER) surveys had a survey penetration depth limited to about 5 m, but the interpreted data suggested a pattern of deep linear features, mostly oriented NW-SE, crossing the southern half of the site. In the northern half of the site, several anomalies/zones of 'disturbed' ground were identified and noted to be potentially either voids, clay filled fractures or very loose pockets of coral rock.

2.1 Localized Ground Treatment and Foundation Design Modifications

Remediation of the majority of the anomalies identified by the geophysical surveys and/or encountered during excavation mainly involved sub-excavation and replacement with a well-graded, limestone or 'marl' fill, placed in lifts not exceeding 200 mm thickness and compacted to at least 98% of the Modified Proctor maximum dry density. This type of ground treatment is considered to be common practise in Barbados.

On the northern side of the structure, in the area of one anomaly considered too deep for sub-excavation and replacement, six (6) augered piles, measuring approximately 0.45 m in diameter and reportedly about 6.7 m in length were installed; no logs of the strata encountered during drilling were maintained and the founding conditions at the base of the piles is unknown.

In a few locations, the foundation excavation encountered vertical fissures and a sub-horizontal void within the coralline rock that extended below the footprint of the building. One vertical fissure, on the western side of the building, reportedly appeared to be connected to a crevice on the ocean-side cliff face as a constant stream of air was observed to be coming up through the void. The remediation of the horizontal void and vertical fissure involved filling with a high slump concrete (by pouring from surface, not tremied) and reportedly required volumes on the order of 11 cubic metres and 16 cubic metres for the horizontal and vertical void, respectively.

In addition to the localized ground treatment described above, the foundation design on the northern half of the building (in the basement area) was modified from strip footings to a reinforced mat/raft nominally 0.3 m thick, locally thickened at load bearing wall/column locations to up to 0.55 m thick.

2.2 Structure Shell Completion and Initial Distress

The building structure and exterior shell was 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, following several days of heavy seas, it is reported that cracking appeared on several walls in the northwest corner of the building, near the confluence 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, again following violent sea conditions, when these original cracks re-opened and additional sets of cracks appeared.

3 INVESTIGATION

In April and May 2007, six (6) boreholes were advanced at the site to investigate the foundation conditions beneath and adjacent to the Phase 1 building and to help understand the cause of the cracking patterns observed on the walls of the structure. Three boreholes were vertically oriented while the other three were drilled to cross the two prevailing joint sets. All of the boreholes were advanced using rotary coring techniques using a triple-tube core barrel system (HQ3) combined with various flush methods aimed at improving recovery from the very weak substrata. Upon completion, all boreholes were examined using a downhole video camera.

During this same time period, qualitative crack mapping surveys were initiated and crack gauges were positioned on various key cracks to quantitatively assess rates of movement across the existing cracks. Precise levelling points were installed around and within the building and regular precision surveys were carried out to monitor vertical building movement. In addition, off-shore wave height and local rainfall data were sourced for the period of time since building construction.

3.1 Geotechnical Subsurface Conditions

The results of the borehole drilling and coring revealed that the engineered marl fill immediately below the building foundations overlies a variably vuggy and heterogeneous, weak coralline limestone ‘rockmass’ containing numerous voids and subhorizontal and subvertical fissures and joints. The coralline limestone stratum contains zones of marly/friable limestone that are interbedded with more crystalline limestone zones.

Although technically a rock, the term rock is a bit of a misnomer for much of the foundation zone, as in many zones the rockmass strength is so low that the material has properties approaching that of a soil, with relict rock fabric and incipient fracturing.

3.1.1 Engineered Fill

The engineered fill below the footings and floor slab is described as dense to very dense, coralline sand and gravel (marl) fill. Standard Penetration Testing (SPT) was not carried out in the boreholes, however, during subsequent test pitting into the marl fill to expose the top of the exterior strip footings it was found that the marl fill had a very dense relative density and required a jackhammer for excavation.

3.1.2 Cap

Based on the conditions encountered in some of the boreholes and from geological mapping exercises carried out at and adjacent to the site, there is evidence that the weak coralline limestone rockmass, has a locally indurated (or hardened) ‘cap’ present along areas of the shoreline in the crest zones of the cliffs.

Evidence from the remnant coral sea stacks immediately in front of the west side of the building (i.e. on the shore side), and from anecdotal and photographic information, suggests that notching along the hardened coral cliff face locally occurs near sea level and along prominent sub-horizontal weaknesses. In addition, there is evidence that a set of sub-vertical major fissures exists extending landward from the sea through the cap and into the underlying coralline stratum.

The cap is described as moderately weathered, medium bedded, amorphous to reefal, weak (R2), porous, fine to very fine grained, cream to white, coralline limestone with some small vugs. This more competent material was likely present (up to 3 m in thickness) over parts of the top, if not all, of the rockmass within the building footprint prior to construction. However, the excavation for construction of the basement of the building likely removed most of this cap zone in the north part of the building footprint.

3.1.3 Coralline Stratum

Below the cap, the coralline stratum is described as moderately to slightly weathered, medium bedded, amorphous to reefal, weak to very weak (R1 to R2), very porous and vuggy, fine to very fine grained, white to cream, friable coralline limestone with many voids and large non-interconnected vugs. Numerous voids were encountered in the coralline stratum as evidenced by 'rod-drops' during the borehole drilling. The voids typically ranged from about 0.1 m to 1.0 m in interpreted size, however, at one location, a rod-drop of greater than 2.5 m was recorded.

The boreholes also revealed a less friable, less voided and generally more competent zone of coralline rock exists at a depth of about 16 m below the basement floor slab. Although this zone was not explored to depth, all of the boreholes indicate that it is at least 3 m thick.

3.2 Structural Evaluation

The distress cracking that appeared in the building was generally of several metres in lateral extent and in configurations of structural significance. The cracking appeared on all five levels of the main floors of the building (Level 1 to 5) and also in the basement (Level 0). However, the majority of the cracking was concentrated in the northwest corner of the building, principally in the basement and on the first, second and third floors.

In general the cracking typically comprised ~45° oriented flexural shear cracking, however some sub-vertical (~90°) cracking was also observed. Based on the data plotted for the cracking, two different frameworks of cracking were identified. One of the sets of 45° flexural shear cracks dipped towards the sea (to the west) within the east-west building walls, (as shown on Figure 1) and the second set dipped towards the gully (to the north), within the north-south structural walls.

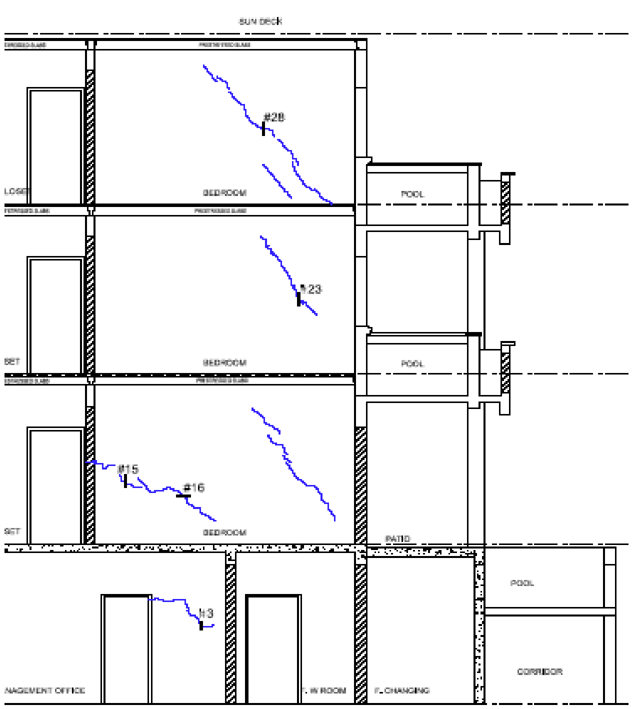

Figure 1. Shear cracking the east-west building walls.

The overall pattern of cracking indicated that the most distressed area occurred in the northwest quadrant of the building, with most noticeable cracking occurring close to the northern and western margins of the building footprint. However, several ~45° oriented flexural shear cracks were also observed on the first three floors in the southwest corner of the building and on the first two floors on the west central side of the building. In addition, ~90° oriented tensile cracking was observed in the basement and on the first three floors in the northeast corner of the building.

3.3 Sea States Preceding Crack Initiation

The two periods of structural distress (August 2006 and February 2007), observed as cracking developing in the interior panel walls, correspond to times during or following several days of abnormally heavy sea conditions. Data on the offshore sea state conditions at Holetown (approx. 4 km north of the site) as recorded at a buoy moored approximately 250 m offshore indicate wave heights that exceeded 1.4 m above datum during the August storm event.

The wave height is predicted to approximately double as the waves shoal. In addition, the energy impacted to the building foundation system increases significantly if air is entrapped in any caves or clefts in the rockmass as a wave impacts the shoreline. This condition is believed to likely exist at this site based on the reported evidence of a constant stream of air observed to be coming up through the void uncovered on the western side of the building during foundation excavation.

4 NUMERICAL ANALYSIS

Numerical analysis (continuum, FLAC, and discrete element analysis, UDEC) was carried out on two sections through the northwest end of the building to provide additional insight into the potential mechanisms that resulted in the observed crack patterns in the structural walls. In addition to the stratigraphic subsurface sequence comprised of the engineered marl fill over the vuggy and heterogeneous coralline limestone (as described above), various vertical zones of weakness resulting from weathering and degradation along the observed pattern of sub-vertical jointing across the site were included in the models. In order to model these zones to best reflect the fact that the rockmass adjacent to these structures had undergone fairly deep weathering, vertical zones of increased porosity and reduced strength were included in the models to simulate these sub-vertical major features. The models also incorporated the structural modelling of the building shell itself so that vertical displacements, shear and principal stresses within the walls could be calculated and so that 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 process controlling the observed building distress. The details of the material properties, types and results of the numerical analysis are beyond the scope of this paper and are described in detail in Carter et al. (2008). However, in summary, 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. These findings were the basis for the design of remediation approaches.

5 KEY MECHANISMS CONTROLLING DISTRESS

The information gathered from the geotechnical investigation and structural mapping along with the results of the numerical modeling indicate differential building settlement, primarily in the northwestern area of the building, related to weak, vuggy and voided foundation conditions and specific marine wave and tide state effects (specifically storm conditions on the west coast of the island) as being the primary mechanisms for the observed building distress.

The proximity of the building to the ocean on the west side and to the gully on the north edge combined with natural sea-cliff recession and development of tensile fracture zones likely complicated building movements. The sea cliffs not only constituted a free face for lateral movement and/or for potential rainfall washout of fines as per the gully, but also would have been subject to additional clapotis-induced high suction forces from breaking waves under high sea states. Under these conditions, foundation degradation (and associated building movement) was likely exacerbated by a winnowing and migration of fines from natural fissures and void zones (possibly even from interconnected vertical fissures) within the coralline rock mass. This in turn potentially gave rise to the migration of fines from the engineered marl fill that was placed below the foundations as part of construction. This migration of fine materials within the subsurface below the building likely then progressively led to a subsequent undermining and loss of foundation support.

As shown on Figure 2, the fact that the northern half of the building was constructed with a lower foundation level than the southern half as a result of the basement may well have locally complicated the building response and been a key factor in the building behaviour. As described previously, the excavations undertaken for the basement level construction likely removed any of the harder and more competent coralline cap that would have originally existed on the surface of the site in this area, and this may have exacerbated the settlement response. Further, in this area, because of the basement, an additional floor level was created resulting in higher foundation loads in the northern half of building. This and the fact that because of the lower founding elevation, higher loads were transferred to the weaker, vuggy/voidy foundation conditions at depth further complicated building response. Finally, the reinforced concrete mat/floor slab foundation in the northern half of the building would have resulted in load spreading and distribution to a greater depth (into the weaker and more voided coralline strata at depth) than would have been experienced below the narrow strip footings (perched high in the relatively more competent coralline cap) below the southern half of the building.

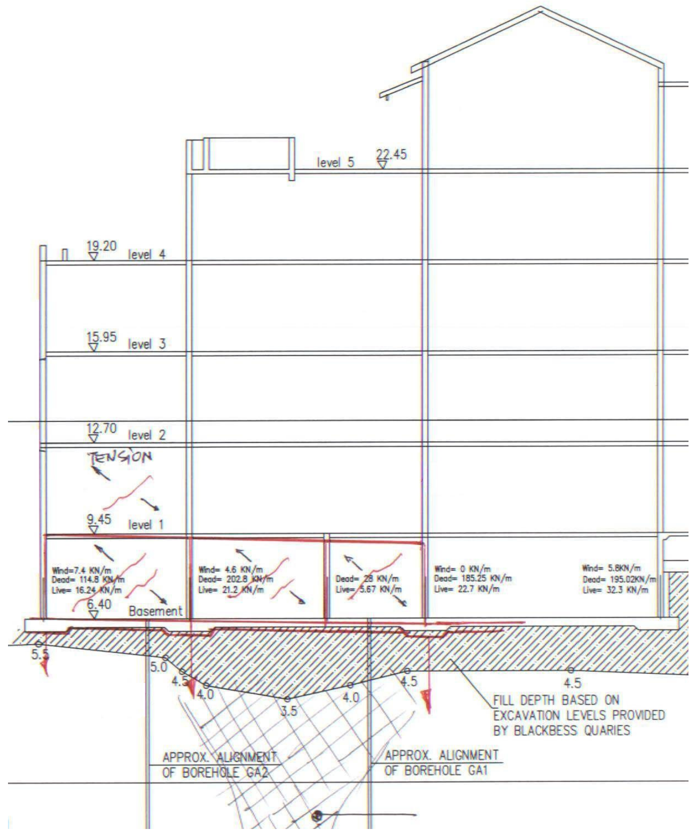

Figure 2. Northern half of building with basement.

6 REMEDIATION CONCEPTS AND DESIGN

Given the mechanisms described above, a remediation program was designed to improve the subsurface conditions below the building and minimize the potential for additional building movement. The remediation comprised 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 foundation improvement measures incorporated a grouting and micropile installation program that was targeted around and within the affected areas. The remediation measures were designed to reduce future foundation distress by controlling the direct causes of instability deemed, though the detailed modeling, to have been responsible for the building movements.

The sub-surface seawall was designed to be comprised of two rows of 140 mm diameter micropiles; one row of near vertical micropiles extending down into the more competent coralline rock below 16 m depth and one row of battered micropiles extending below the existing building. The top of the micropiles were formed into a concrete cap/grade beam that was structurally connected to the existing building footings and/or to the foundation wall. The approach was that the combination of steel and grout in the micropiles would provide additional axial support to the building foundations in compression while the steel on its own would satisfy lateral and rotational movement concerns by providing tensile resistance via the battered piles. 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 so as to stiffen the in situ rockmass, reduce void porosity and hence minimize potential for future vertical settlement.

The micropiles in each row were laid out on a approximately 1.2 m spacing in an alternating pattern. The outer row of near vertical micropiles were designed to be installed on a 15^{o} inclination (from the vertical) parallel to the sides of the building in order to intersect as many near vertical joint features as possible in the subsurface. The inner row of battered micropiles was designed to be installed perpendicular to the sides of the building at inclinations varying from 30^{o} from the vertical (along most of the southern and western sides – extending about 10 m horizontally below the building), to alternating between 30^{o} and 45^{o} (along the northern side – extending about 10 m to 14 m horizontally below the building). Additionally, on the western side of the building (away from the area that experienced the greatest distress) the inner row of battered micropiles was designed to be comprised of alternating installations of full length piles (approximately 20 m in length) and then half length piles (approximately 10 m in length). However, in the northwest corner and on the northern side of the building, all of the battered micropiles were designed to be full length (approximately 20 m) installations. At the northwest corner of the building, an extra row of five (5) grout only holes was included to be installed at a low angle (between about 50^{o} to 55^{o} from the vertical) to reach further below the building in this area.

In addition to the micropile wall on the exterior of the building, the design also included the requirement for 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.

Where it was possible to do so, within each of the construction work areas, the outer row of near vertical micropiles was to be installed first, so that the “sea-wall” concept was created as efficiently as was feasible. Drilling and installation of this outer row was to be followed by installing the inner row of battered (or inclined) piles that extended below the building. Wherever possible, split-spaced grouting closure principles were to be adopted in each row such that the micropile installation followed a Primary, Secondary, Tertiary, Quaternary, Quinary (or PSTQN) sequence or pattern of installation. In this manner, larger grout takes (which used a low-mobility grout) would be controlled and preferentially restricted to the higher order holes; with smaller grout takes expected to occur in the Quaternary and Quinary locations as closure (and tightening of the ground) started to occur.

7 CONTRACTOR SELECTION

The project had extremely challenging aspects, beyond those purely technical. The overall schedule was extremely compressed, given the need to have the remediation at least largely completed before the onset of the hurricane season. In practice, 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 had to be made.

Given the above, the project was bid on at most a 25% design, and the contractor had to commit to shipping his equipment and materials before all the details of the commercial contract with the owner could be fully agreed. 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 fashion most responsive to the reaction of both the foundation and the structure itself. This meant, of course, that the contractor's expertise and experience would be invaluable as an integral part of engineering the solution in real time. 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 subcontractor, the project management team and, of course, the owner himself.

Such a combination of factors strongly favours the creation of an "Alliance," as opposed to the more conventional owner-engineer-contractor arrangement (Carter and Bruce, 2005). At the Sandy Cove project, key elements of alliancing were implemented to assure selection of the "correct" contractor, and to maintain excellent communications, problem resolution mechanisms, compliance to schedule, and equable cost management structures throughout the project's duration.

The contractor procurement process may be taken as the example. The engineer compiled a data summary and a conceptual design which was circulated to a small group of 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 teams 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 a contractor was selected immediately after the interview period was over, and his commitment was given to mobilize as promptly as possible. A timeline was set between him and the owner to conclude agreeing the commercial contract. This contract contained a financial risk sharing feature ("pain share - gain share") which would incentivize both sides to be as efficient as possible and to protect their respective financial exposure, bearing in mind the somewhat indeterminate nature of the work at that time.

The authors have absolutely no doubt that the procurement of the most appropriate contractor, and the innovative financial vehicles, were key factors in the excellent quality and pace of work which was conducted, and the extremely functional and efficient communication framework under which it proceeded.

8 MONITORING DURING 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 remediation design phase and formulated into the numerical modelling was adjusted and refined as construction proceeded. Refinements to design understanding and layouts were undertaken in near real-time as additional subsurface information was obtained during the remediation construction. Records were updated daily and the grout-take data was tracked using 2-D and 3-D graphical models so that the poorest conditions (i.e. most voided) in the subsurface could be readily identified. These areas where then targeted with additional grout-only holes during the course of the production work and then ultimately with a series of closure holes at key locations in the perimeter/cut-off wall. Figure 3 shows a typical key view from the 3-D grout-take model.

At the completion of the works, data had been acquired from the drilling and grouting of 174 micropiles, during which 750 m^3 (1000 yd^3) of low-mobility grout was injected into the voided areas of the foundation around the perimeter and below the interior of the building.

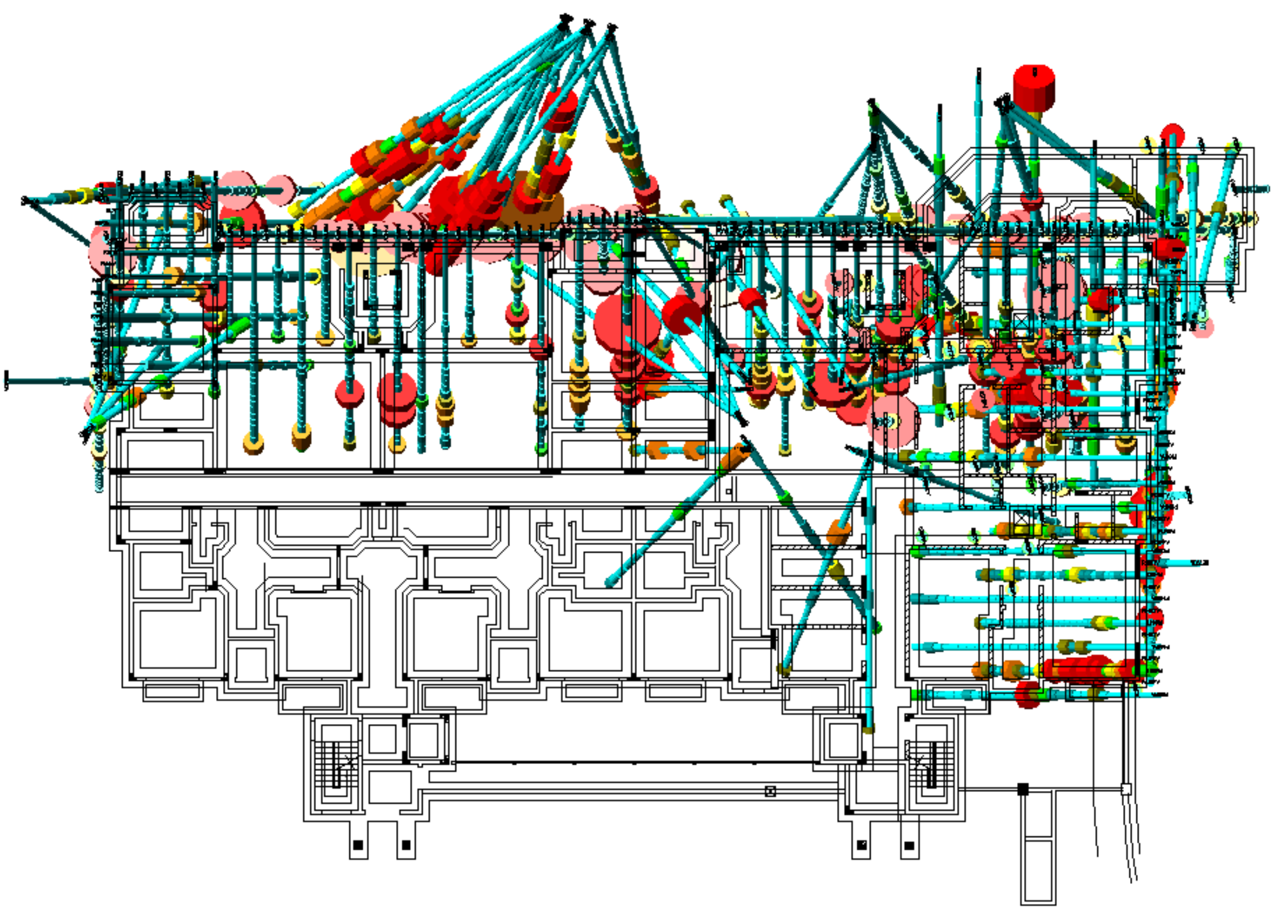

Figure 3. Plan view from 3-D grout take model.

9 BUILDING MONITORING AND POST-REMEDIATION PERFORMANCE

In addition to monitoring the drilling and grout-takes during the remediation, prior to the start and throughout the period of construction, the building was regularly monitored for settlement, tilt and crack spreading. The building monitoring instrumentation included a suite of electrolevels, tiltmeters, crack gauges, precise leveling points and prisms. The electrolevels and tiltmeters were set-up to monitor and record data in near-real time (every 15 minutes) during construction. The precise level points and prisms were also surveyed three times a week during the construction while the crack gauges were measured on average about once every two weeks.

The monitoring instrumentation data (as seen in the typical electrolevel data plot on Figure 4) showed the building responding to the grouting by initial downward (i.e. settlement) movement as a result of the drilling/injection/flushing/disturbance to the poor subsoils by the micropiling operations, followed by upward (i.e. heave) movement as a result of the pressure grouting operations. 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.

Upon completion of the foundation remediation, a selected number of the electrolevels (including Electrolevel EL#4 shown on Figure 4) and precise levelling points (including PLP-BB shown on Figure 5) were left within the structure to allow continued monitoring to assess the post-construction and long-term performance of the building.

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 on the order of 1.0 m to 1.75 m (equal to or up to 75% greater than those recorded during the periods of the original crack initiation), occurred in August 2007 (with the passing of Hurricane Dean part way through the remediation), and then in March, April, September and October 2008. 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). As can been seen on Figure 4, there was a slight response on some of the electrolevels but virtually no tilt or rotational displacement. As can be seen on Figure 5, there was no increase in settlement as a result of either of these events. Further, no crack development occurred in the building in response to these events.

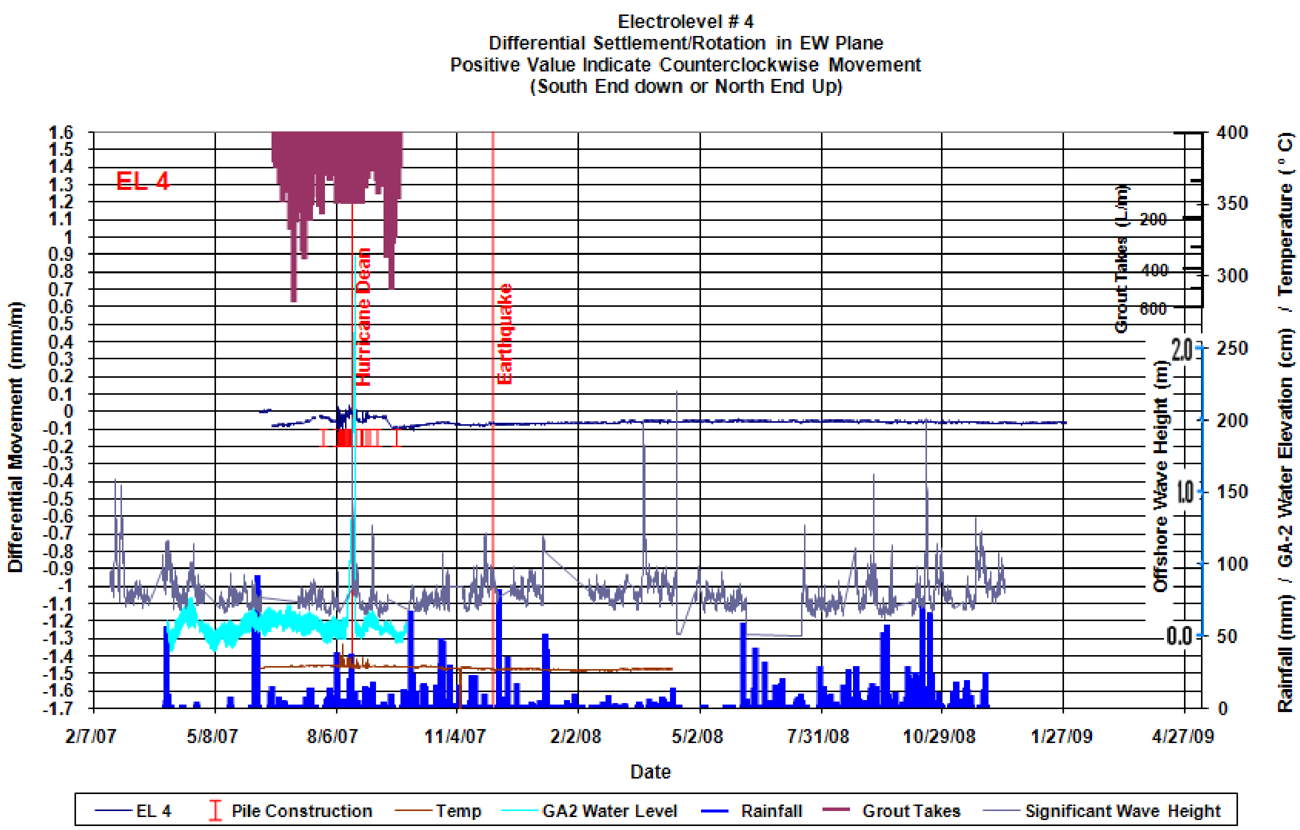

Figure 4. Building monitoring data from electrolevel.

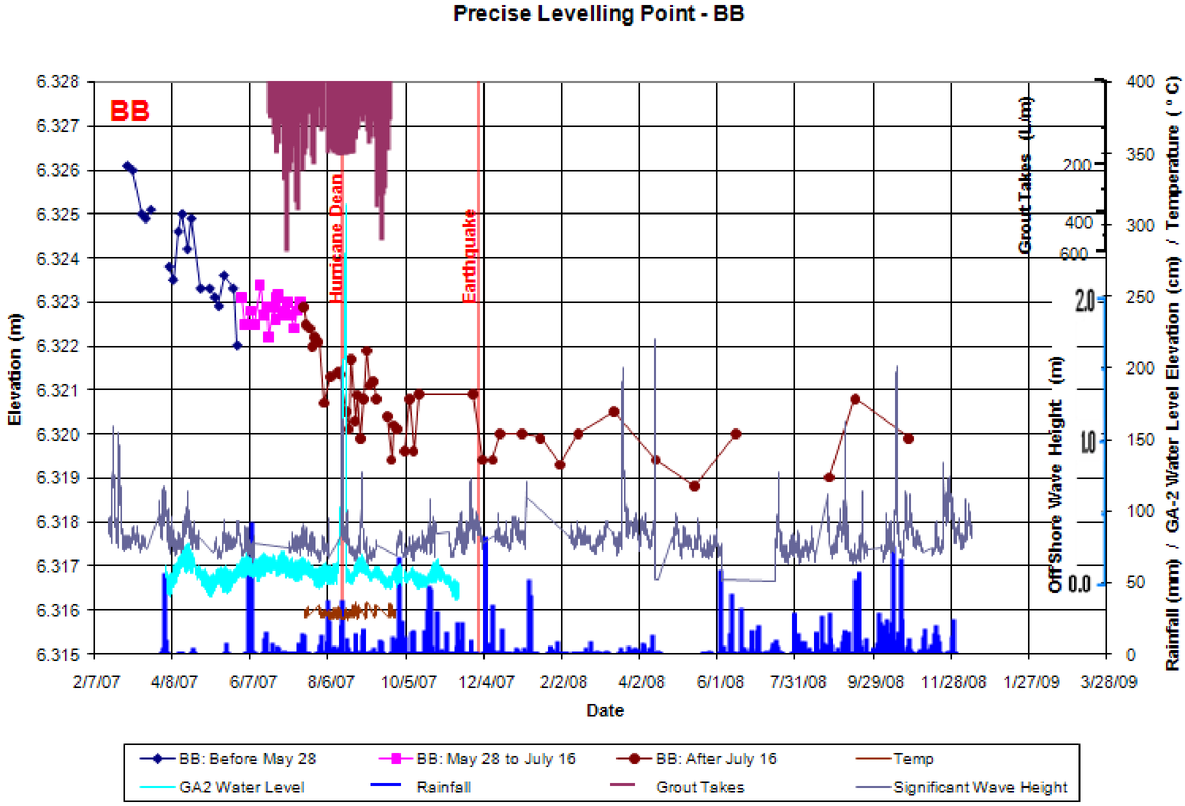

Figure 5. Building monitoring data from precise survey.

10 SUMMARY AND CONCLUSIONS

To completion of the improvement of the building foundation, the following works had been accomplished:

- Installation of an 88 m (290 feet) long sub-surface ‘sea-wall’ barrier/grouted cut-off curtain around three sides of the building;
- Direct support by 137 – approximately 21 m (70 feet) long, 140 mm (5.5”) diameter micropiles underpinning the edges of three sides of the building (north, south and west);
- Indirect support by 37 – approximately 20 m (65 feet) long, 140 mm (5.5”) 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.

It is considered that 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 effect an overall stiffening of the subsurface to improve the load-bearing capacity of the originally weak and voidy, coralline rockmass.

The fact that no damage (or even re-activation of earlier patterns of adverse cracking) occurred in response to the passage of Hurricane Dean (in August 2007 toward the completion of the remediation works) or in response to the earthquake shortly following completion of the remediation in November 2007 clearly demonstrates the effectiveness of the grouting and micropiling. In addition, the distinct improvement in the foundation and building behaviour under the impact of pounding waves and adverse sea states during March, April, September and October 2008 that had recorded off-shore wave heights up to 75% greater than those that occurred in August 2006 and February 2007 (at the initiation of the cracking and severe damage effects) provides proof of the effectiveness of the remediation works.

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