

# Use of Innovative Numerical Modelling and Rock Mass Characterization Approaches for Design of Foundations within Weak Coralline Limestones

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**ABSTRACT:** Integration of rock mass characterization and structural building behaviour observations, coupled with extensive use of numerical modelling methods were fundamental to developing an in-depth understanding of settlement issues related to very weak, vuggy, uncemented coralline limestone foundations. Replication of observed building behaviour was achieved through innovative modelling of the weak vuggy openwork fabric of the coralline limestone through use of tessellation approaches within UDEC based on data acquired from a program of detailed diamond core drilling, complete with downhole camera observations of rockmass conditions. For the design of the micropiling and grouting solution developed to improve the conditions of the foundations and remedy the observed distress issues, structure-foundation interaction analyses were undertaken using both geotechnical and structural approaches, with use of FLAC as the principal code for assessment of interaction issues. To verify the original design assumptions and check the veracity of the numerical modelling assessments the data from the drilling and grouting for installation of the micropiling works undertaken to strengthen the foundations was characterized in terms of drillability and grout take and plotted in 3D.

## 1. INTRODUCTION

The Sandy Cove Development is located on the west coast of Barbados between Bridgetown and Hometown in the Parish of St. James. The Phase 1 development includes a six storey luxury condominium complex, including a basement (on the northern half of the building only) and five levels of units, situated 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 Phase 1 building is supported primarily on strip footings (on the southern half of the structure) and a reinforced concrete mat/floor slab locally thickened at load bearing wall/column locations (on the northern half of the structure) all founded on compacted marl fill overlying the coralline rock mass.

The building structure/shell of the Phase 1 development 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. Observation of these initial cracks suggested little change over a couple of 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.

This paper describes the foundation and building investigation, analysis and mechanism assessment, remediation design, construction measures, foundation improvements and subsequent performance of the Phase 1 building at the Sandy Cove Development.

## 2. INVESTIGATION

In April and May 2007, six (6) subsurface investigation boreholes were carried out 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) and 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. In addition, precise levelling points were installed around and within the building and regular precision surveys were carried out to monitor vertical building movement.

### 2.1. Geotechnical Appraisal

The results of the borehole drilling and coring used to develop the geological model, revealed that the dense to very dense marl (coralline) fill immediately below the building foundations overlies a variably vuggy and heterogeneous weak coralline limestone containing numerous voids and subhorizontal and subvertical fissures and joints, and within which zones of more marly/friable limestone are interbedded and intercalated with more coralline and crystalline limestone zones. There was evidence that the weak coralline limestone “rockmass”, prior to building construction, had a locally indurated (or hardened) ‘cap’ present along areas of the shoreline in the crest zones of the cliffs, and also over parts of the top, if not all, of the bedrock within the footprint.

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.

While some areas of real rock-like cap material were recovered in the borehole cores and are observed at exposures around the site, the excavation for construction of Level 0 (Basement) of the Phase 1 building likely removed most of this cap zone in the north part of the building footprint. 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 verbal and photographic information provided by the structural engineer and the building contractor on site, suggests that notching along the coral cliff face locally occurs at sea level and along prominent sub-horizontal weaknesses and that a set of sub-vertical major fissures exists extending landward from the sea (Photo 1).

### 2.2. 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.

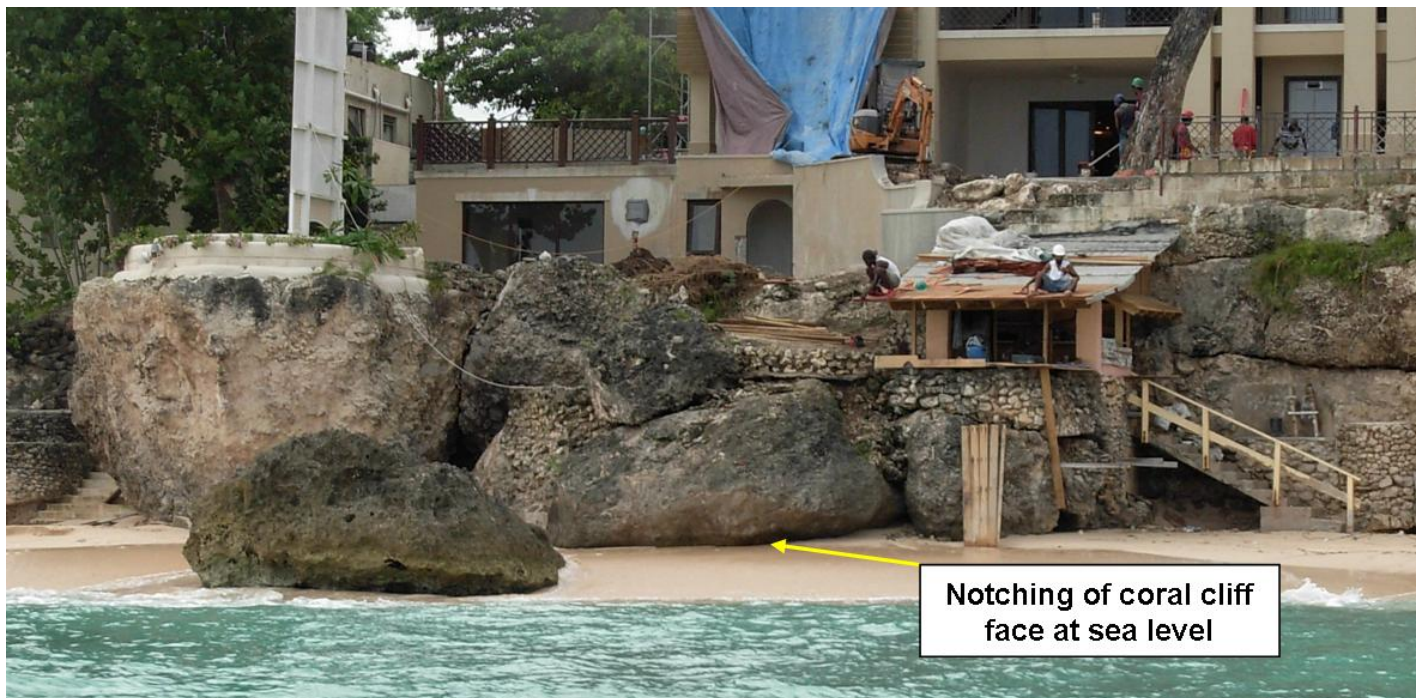


Photo 1 – Notching of coralline cliff face along sub-horizontal weaknesses at sea level.

In general the cracking typically comprised  $\sim 45^\circ$  oriented flexural shear cracking, however some subvertical ( $\sim 90^\circ$ ) cracking was also observed. Based on the data plotted for the cracking, two different frameworks of cracking were identified. One of the sets of  $45^\circ$  flexural shear cracks dipped towards the sea (to the west) within the east-west building walls, (Figure 1) and the second set dipped towards the gully (to the north), within the north-south structural walls. In addition, some areas existed where cracking could be observed extending through cross-connecting main columns.

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  $\sim 45^\circ$  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,  $\sim 90^\circ$  oriented tensile cracking was observed in the basement and on the first three floors in the northeast corner of the building.

### 3. ANALYSIS

Two orthogonal sections through the northwest end of the building (one in the east-west direction and one in the north-south direction) were analyzed by numerical methods (continuum and discrete element analysis) to further understand and provide additional insight on the mechanisms that resulted in the observed crack patterns.

In addition to the stratigraphic sequence comprised of the marl fill over the vuggy and heterogeneous coralline limestone, various zones of incipient weakness resulting from weathering and degradation along sub-vertical jointing also were noted at the site. 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 geometry of the near surface, pre-construction, excavated areas (i.e. those subsequently backfilled with an engineered marl fill) were also included in the models, and a specific material introduced above the in situ rockmass to reflect the properties of the compacted engineered fill.



Figure 1 – Shear cracking ( $\sim 45^\circ$ ) in east-west building walls.

### 3.1. Material Properties

The properties for the different coralline units of the foundation strata represented in the model were formulated based on precedent experience from similar sites and on laboratory and in situ test results performed in similar materials elsewhere on the island of Barbados.

Based on the inferred geology from the downhole video observations, the coring information and the record of drill rod-drops and other such drilling events, it was inferred in the models that horizons for preferential development of vuggy zones existed, some with large voids/cavities (ranging from about 0.3 m to 0.8 m in diameter) and some with smaller vugs, as observed in the rock core.

The frequency and size of the large voids/cavities was estimated by examining the downhole camera videos and the photographs of the core. A distribution of large voids and/or cavities were therefore discretely included in the distinct element analyses performed using the program UDEC, but not in the continuum analyses performed with FLAC2D. The smaller vugs were not explicitly replicated in either numerical model because it was deemed that their influence was adequately captured in the strength parameters obtained from the laboratory tests conducted on core samples of similar coralline units from other sites in Barbados. For the in situ material it had been found that small vug porosities (as measured in the laboratory samples) ranged from about 10% to 30%, giving rise to overall porosities of >40% once the large vugs were included in the model.

In order to develop realistic input parameters for the modelling a number of coralline rock samples were selected from cores recovered from sites elsewhere on the island and in similar rock units to the stratigraphic sequence encountered at the Sandy Cove site and these were tested in uniaxial and triaxial compression, and in indirect tension (Brazilian tests). Non-linear Hoek-Brown strength envelopes were then estimated for the stratigraphy interpreted to exist at the site – namely an Upper and Lower Vuggy Coralline Limestone with an intermediate Friable Limestone. The Hoek-Brown parameters defining the intact strength envelopes for the three units are shown in Table 1.

Table 1. Intact Laboratory Properties

Layer Name	$\sigma_c$ (MPa)	$E_i$ (GPa)	$\nu$	$M_I$
Upper Coral Rock	9.91	3.80	0.13	8.401
Friable Limestone	10.68	9.22	0.21	6.423
Lower Coral Rock	11.58	4.02	0.22	9.505

Elastic stiffness and strength properties of the rock mass for use in the modeling, were estimated by downgrading

the intact properties according to standard relationships based on rock mass ratings for each stratigraphic unit. Field GSI ratings were then developed based on the core recovery and RQD data taking into account information abstracted from the downhole video inspections of the boreholes. These GSI ratings were also tempered based on comparisons of the downgraded modulus with field moduli, as measured by in situ pressuremeter tests in similar coralline strata at other sites in Barbados. Table 2 lists the derived values of the Hoek-Brown parameters,  $m$  and  $s$ , and the elastic moduli based on the rock mass rating for the stratigraphic sequence at the site.

Table 2. Rock Mass (Field) Properties Used in Analyses

Layer Name	$\sigma_c$ (MPa)	GSI	$E_{rm}$ (MPa)	$E_{press}$ (MPa)	$m$	$s$
Upper Coral Rock	9.91	16	144	141	0.418	0.00009
Friable Limestone	10.68	17	366	-	0.331	0.0001
Lower Coral Rock	11.58	30	327	-	0.780	0.00042

For characterization of the joints, joint zones and the engineered marl fill (placed and compacted below the foundations) Mohr-Coulomb properties were assumed as summarized in Table 3, based on assessment of available laboratory and field characterization data.

Table 3. Joints, Joint Zones and Marl Fill Properties Used in Analysis

Layer Name	$E$ (MPa)	$\nu$	$C'$ (kPa)	$\phi'$
Joint	5	0.3	0	5°
Joint Zone	10	0.3	0	20°
Marl Fill	100	0.3	35	38°

### 3.2. Continuum Analysis - FLAC2D

In order to directly assess the impact on the super-structure, the building was modelled in FLAC2D as a frame consisting of axial-flexural (beam) elements representing the walls and floor slabs. The walls and slabs of the super-structure are 15 cm thick with an elastic modulus of 25 GPa. The space between the flexural members was also modelled to assess the membrane principal stress directions in the walls in the plane of the model. In the model, the rigidity of these walls was pro-rated to their contribution to the overall rigidity of the building. This rigidity was estimated to be equivalent to having the building frame filled with a material with a shear modulus of 500 kPa. This representation of the superstructure was connected to the ground model (continuum) through strip footings and the lower floor slabs (on grade). These foundation (basement) slabs have a thickness of 30 cm which increases to 45 cm under the bearing walls. The ground model was based on the geology interpreted from the boreholes and includes the sub-vertical joints and their

associated altered zones exposed on the cliff face. The cliffs on the west side (sea side) and the north side (gully/drainage channel) were also captured in the model to assess possible mechanisms due to lateral movements.

The FLAC 2D analyses of the east-west section through the building predicted higher differential settlements occurring in the central bays of the structure, possibly due to the higher loads in the inner bearing walls. The resulting differential settlement causes shear loads in the walls of the inner bays. The principal stresses in the walls are oriented diagonally with the tensile direction dipping to the east in the highest loaded walls. This results in potential crack patterns which have the cracks dipping to the west (towards the beach) and similar to those patterns observed on the structure.

By contrast, as shown on Figure 2, the results of the analyses of the north-south section show more severe differential settlements and shear loading on the walls than east-west section. Based on close examination of the results and comparison with the observed crack patterns it is inferred that the higher differential settlements, which appear to preferentially occur towards the north end of the building, occur as a result of higher loads in the inner bearing walls in association with the poorer ground in this area. This differential settlement results in shear loads in the walls of the three northernmost bays. The principal stresses in the walls are oriented diagonally with the tensile direction dipping to the south in the highest loaded walls. This result in potential crack patterns which have the cracks dipping to the north (towards the gully) and similar to those patterns observed on the structure.

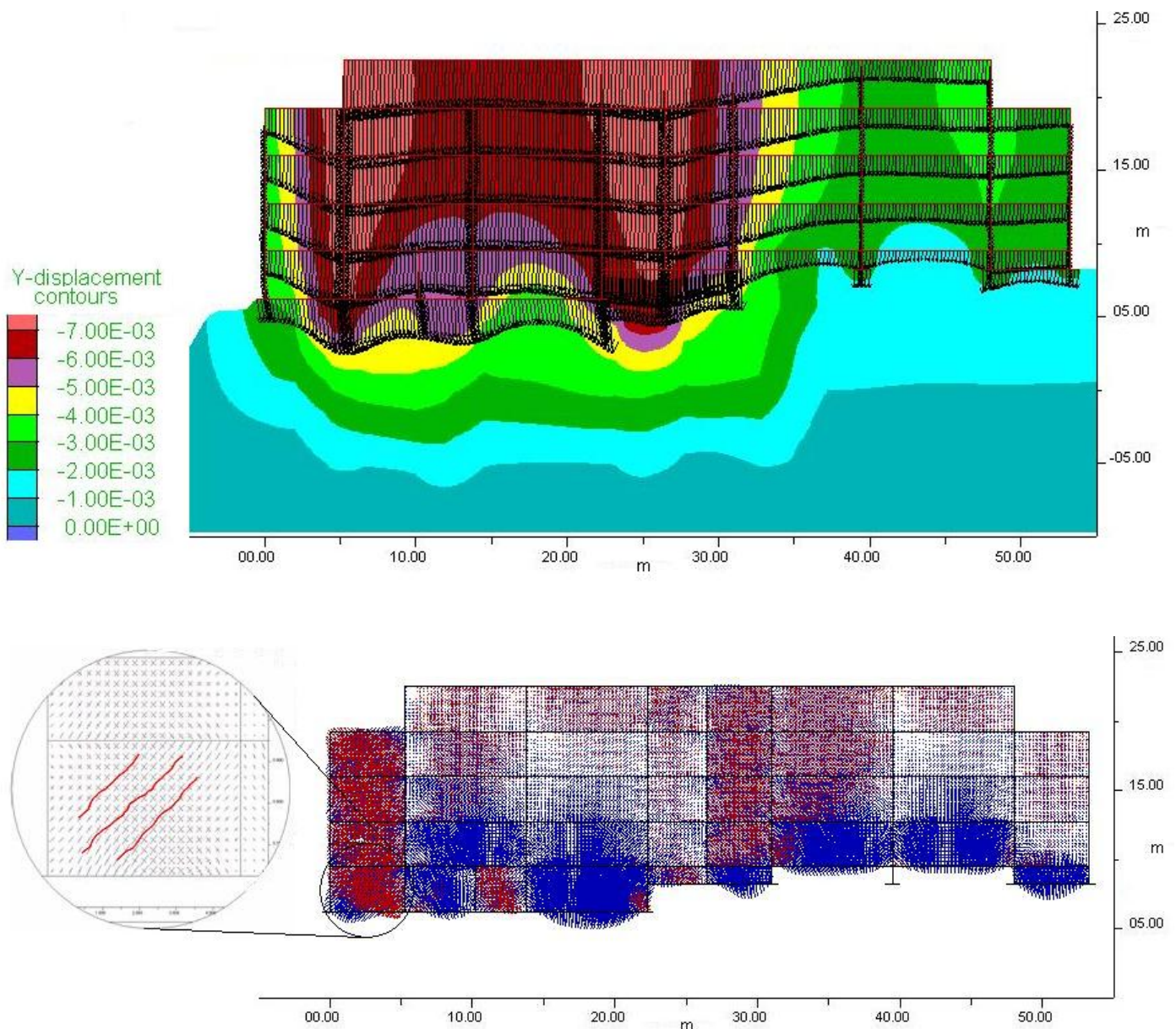


Figure 2 – Vertical displacements and principal stress directions – walls of first bay from the North end experience highest shear stress, consistent with the differential displacements.

### 3.3. *Distinct Element Analysis - UDEC*

In order to assess the possible failure mechanisms associated with the building structure founded on the weak coralline limestone, the northern half of the building and rock mass was modeled using the 2D distinct element code UDEC. The model considered the building with design loads on the foundation footings, founded on the compacted marl fill base overtop of the coralline limestone units.

The coralline limestone units were modeled as discrete polygonal blocks ranging in size from 0.3 m near the bedrock surface to 1.0 m in size at depth. The rock fabric incorporated Voronoi Tessellation for the generation of the polygonal blocks in order to replicate the natural near honeycombed texture of the rockmass as observed in the drillhole cores and from field exposure. To properly model the behaviour of the coralline rockmass material strength parameters were defined for the contacts between the blocks, and laboratory measured (intact) elastic moduli were applied to the blocks themselves. In this representation of the vuggy coralline limestone, use of the Voronoi discretization into a blocky model configuration can be thought of in somewhat the same way as a *mélange* material (something akin to a concrete mixture) wherein the more competent blocks constitute the aggregate of the rock mass, and block to block contact strength properties reflects the cementitious binder.

To reflect the internal variation within the general rock mass, the fabric was also zoned according to the broad litho-stratigraphy established within the coralline rock units, based on the drilling data. Vertical joints within the rock mass were then incorporated into the broad stratigraphy based on field defined spacing and through application of lower moduli and contact strengths in the defined jointed zones. A further refinement to the model was made to replicate the voided zone determined from the field drilling to exist at and around tide level, presumed to have developed due to loss of material via washout from storm activity. This voided zone was modelled by staged deletion of random blocks within the rock mass in and around these critical elevations. The last refinement to the model was to incorporate an undercut along the sea/gully side of the cliff to replicate the as-observed wave-eroded cliff notch zone generated by constant marine action. Continued undercutting by the same mechanism was also modelled by softening and

weakening a seam of rock extending across the model from the perceived cliff undercut zone at about sea level.

As can be observed on Figure 3, the results from the modelling show that significantly aggravated differential settlements develop towards the northwest corner of the building as a result of washout/marine flushing (replicated in the model by block deletion). With increasing void creation coincident increased differential settlements and plastic failure develop within the structure, until a critical state is reached at approximately 12% rock mass voids (ie., 40-45% total voids) wherein caving failure of the rock mass occurs associated with considerable settlement and structural damage. At this stage the highest differential settlement develops at the vertical jointed zones.

When a significant undercut is allowed to progressively develop in the model, as shown on Figure 4, failure becomes more of a toppling type mechanism, with the more competent rock above the undercut seam tipping and slumping towards the sea. This model response behaviour of toppling and overturning is absolutely characteristic of the pattern of sea cliff and cliff notch recession processes that are clearly evident around the island.

The excellent replication behaviour from the distinct element model simulations of the failure mechanisms governing cliff recession within these weak coralline limestone units (as illustrated by the results on Figures 5 through 7) was key to developing a proper remediation approach, as the modelling results gave confidence for examining the spacial, temporal and energy dependant degradation of the rockmass from an engineering perspective. As both differential settlement due to void creation, as well as undercutting of the cliff from wave action, could be conceivable controls on the settlement, this modelling insight was of enormous value as it allowed in-depth evaluation of the cracking patterns, thus aiding assessment of the most likely process controlling the observed building distress. This in turn aided decisions on the design of remediation approaches. Without this modelling insight there had been much uncertainty as to the most important controlling mechanism and what might be the best remediation. Based on the modeling, as void creation in conjunction with the vertical jointing showed the more convincing settlement results on the building structure, a micropiling and grouting solution seemed viable.

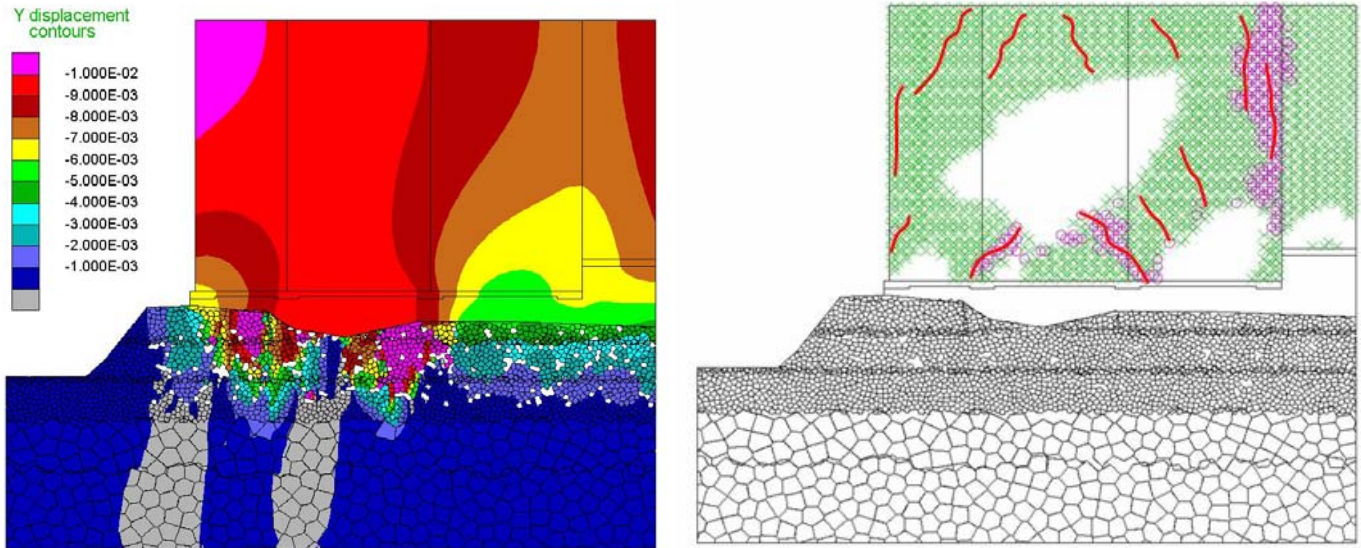


Figure 3 – Vertical displacement contours and interpreted cracking patterns based on stress trajectories after 12% void creation.

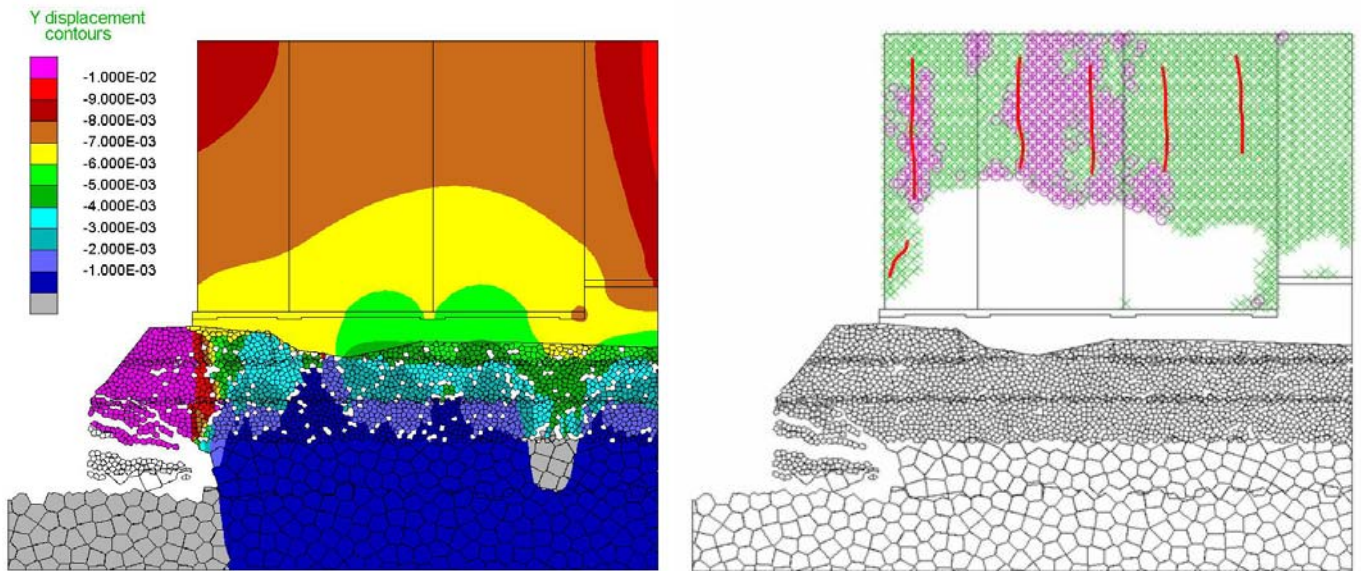


Figure 4 – Vertical displacement contours and interpreted cracking patterns based on stress trajectories after 12% void creation.

#### 4. BUILDING MOVEMENT MECHANISMS

The information gathered from the geotechnical investigation and structural mapping as well as the results of the numerical modeling pointed to differential building settlement issues, especially in the northwestern area of the building, related to weak, vuggy/voidy foundation conditions and specific marine wave and tide state effects (specifically storm conditions on the west coast of the island) as being the primary causative reasons for the building movements.

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 however likely complicated building movements, as 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, which in turn potentially gave rise to the migration of fines from the engineered marl fill that was placed below the foundation level 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 5, 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. Because the excavations undertaken for the basement level construction likely removed any harder, indurated and more competent coralline cap-rock material that would have originally existed on the surface of the site in this area, 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.

## 5. REMEDIATION CONCEPTS AND DESIGN

Given the mechanisms outlined above, as tested and evaluated by the modelling, a comprehensive remediation program was designed to improve the subsurface conditions and minimize the potential for additional building movement. The remediation comprised three main components:

- (i) creation of an effective barrier to further marine intervention into the subsurface zone beneath the building; (i.e. by creating a buried, sub-surface seawall/grouted curtain),
- (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 subsurface strata below the interior of the northern half of building.

The foundation improvement measures that were implemented incorporated a grouting and micropile installation program that was targeted around and within the affected areas to not only enhance the condition of

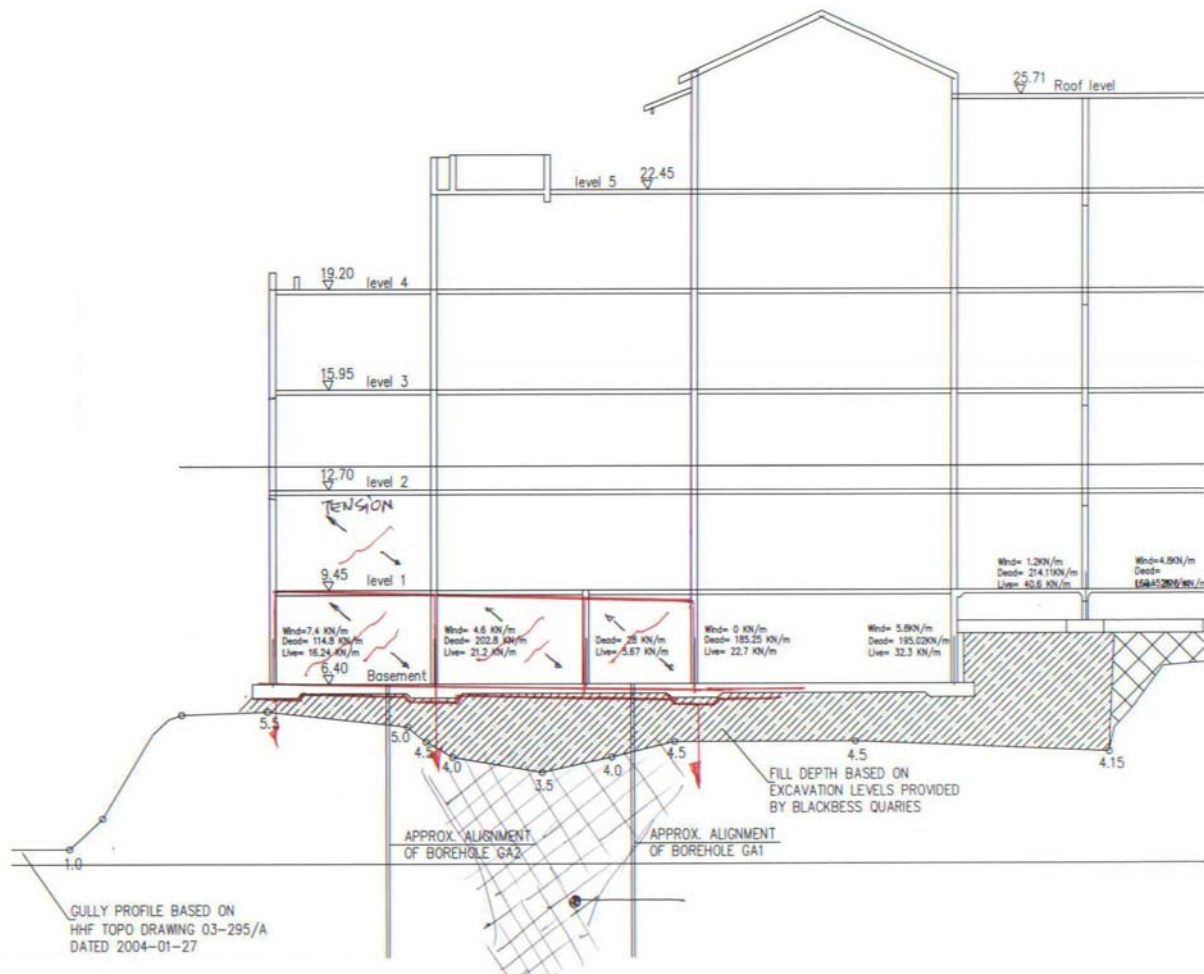


Figure 5 – Northern half of the building with basement level.



the weak coralline limestone foundation under the building, but also to create a subsurface seawall aimed at inhibiting wave and marine energy influx into the foundation zone. The remediation measures were thus specifically targeted at reducing future foundation distress by controlling the direct causes of instability deemed though the detailed modelling to have been responsible for the building movements.

The micropile wall was designed to be comprised of two rows of 140 mm (5.5") diameter micropiles; one row of near vertical micropiles extending down into the more competent coralline rock at 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 future vertical settlement.

As shown on Figure 6, the micropiles in each row were laid out on an approximately 1.2 m (4 foot) spacing in an alternating pattern. The outer row of near vertical micropiles were designed to be installed on a 15° 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° from the vertical (along most of the southern and western sides – extending approximately 10 m horizontally below the building), to alternating between 30° and 45° (along the northern side – extending approximately 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 (65 feet) in length) and then half length piles (approximately 10 m (32 feet) 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 (65 feet)) 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° to 55° from the vertical) to reach further below the building in this area.

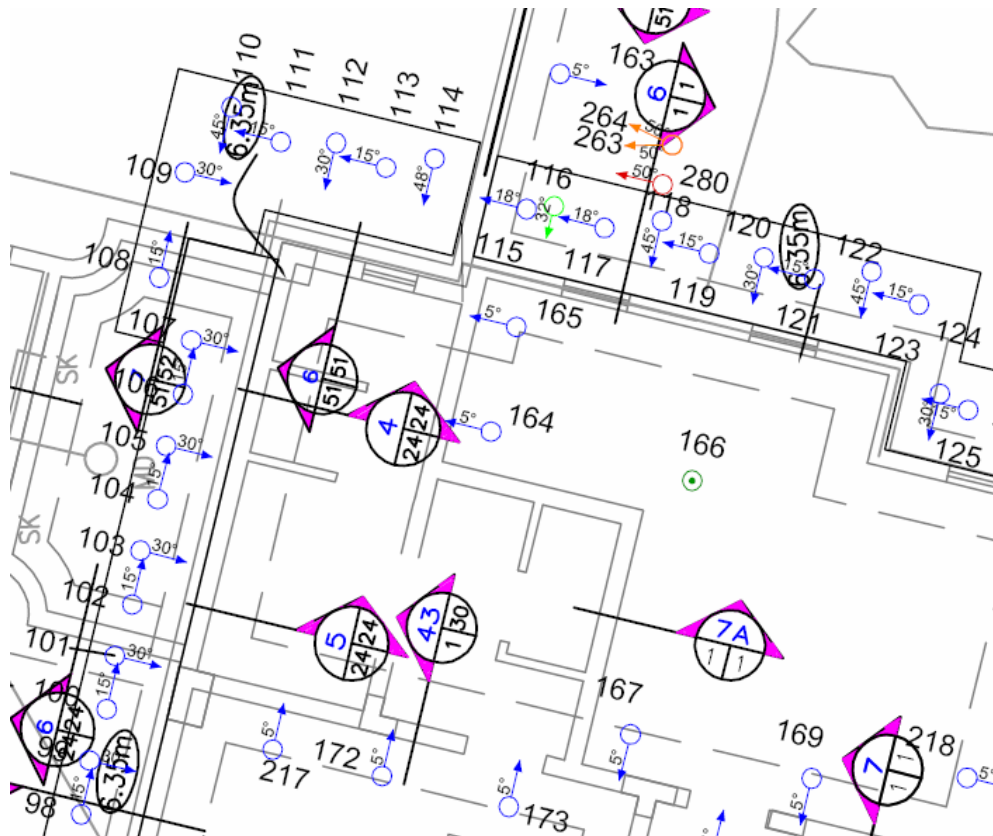


Figure 6 – Micropile layout in northwest corner of building.

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 installed first, so that the “sea-wall” concept was created as efficiently as was feasible. Drilling and installation of this outer row was followed by installing the inner row of battered (or inclined) piles that extended below the building. Wherever possible split spaced grouting closure principles were 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) were controlled and preferentially restricted to the higher order holes; with smaller grout takes generally occurring in the Quaternary and Quinary locations as closure (and tightening of the ground) started to occur.

## 6. 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 simulations 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 were 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 7 shows a typical key view from the 3-D grout-take model.

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.

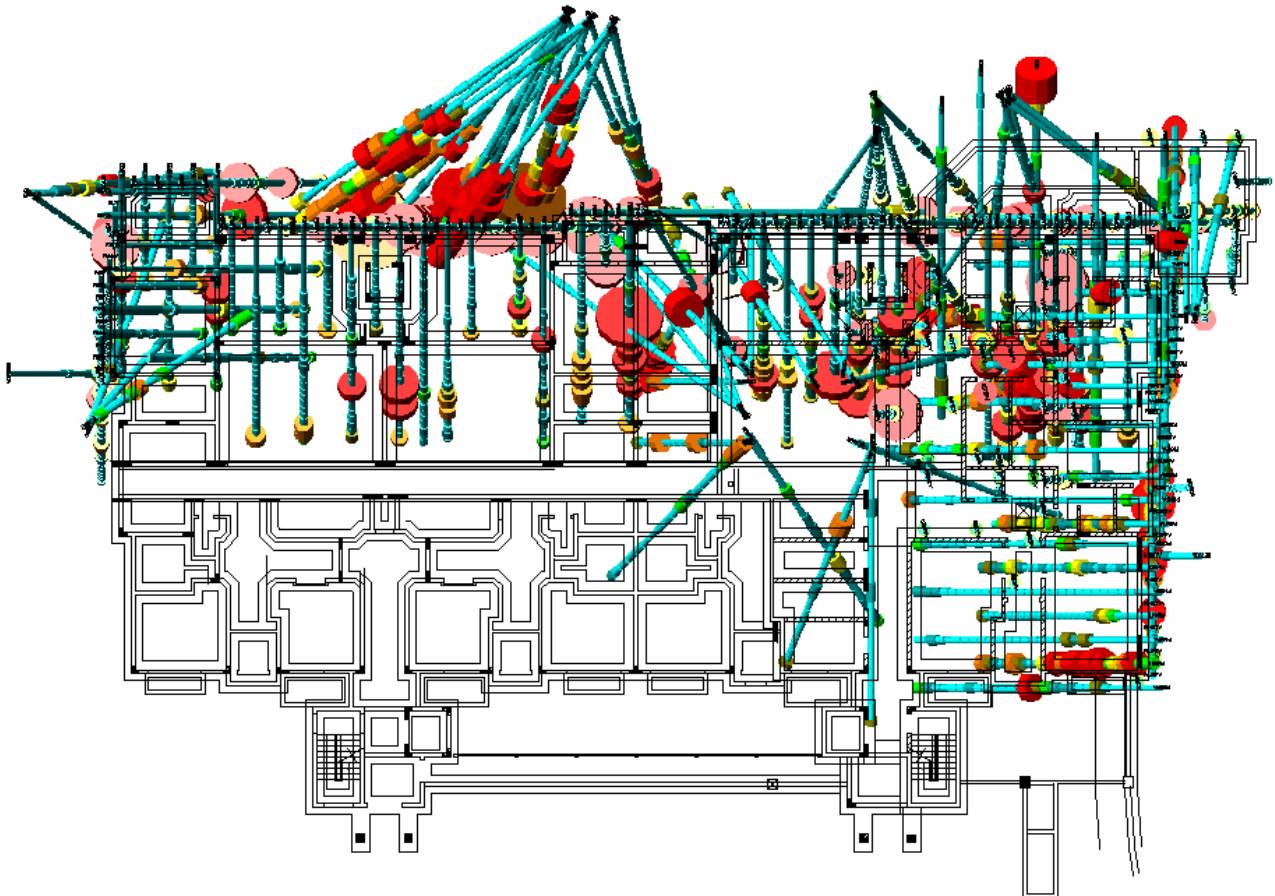


Figure 7 – Plan view from 3-D grout take model.

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.

Upon completion of the foundation remediation, a selected number of the electrolevels and precise levelling points were left within the structure to allow continued monitoring to assess the post-construction and long-term performance of the building.

### 7. VERIFICATION OF MODELLING OF FOUNDATION IMPROVEMENT

The modelling undertaken as part of the design process was refined in the light of the grouting data being aquired during the initial phases of the remediation works, and during the remediation was verified by means of the 3D visualization of grouting behaviour and takes which essentially confirmed the voided geometry assumed for the models.

By the time the works were completed, data had been acquired from the drilling and grouting of 174 micropiles, during which some 1000 yd<sup>3</sup> (757 m<sup>3</sup>) of low-mobility grout was injected into the voided areas of the foundation around the perimeter and below the interior of the Phase 1 building.

The instrumentation monitoring of the building behaviour during the course of construction confirmed the modelling assessments. The monitoring data (as seen in the typical electrolevel data plot on Figure 8) 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, the geometry of the voided zones assumed in the modelling were matched by the zones of higher takes from the grouting and the expected behaviour of the building remediation well replicated, with increasing stabilization being achieved throughout the remediation program (see Figure 8), as each area of the building was underpinned and grouted, with completion of the improvement in the north zone of the building achieved in September 2007 right at the end of the remediation construction program.

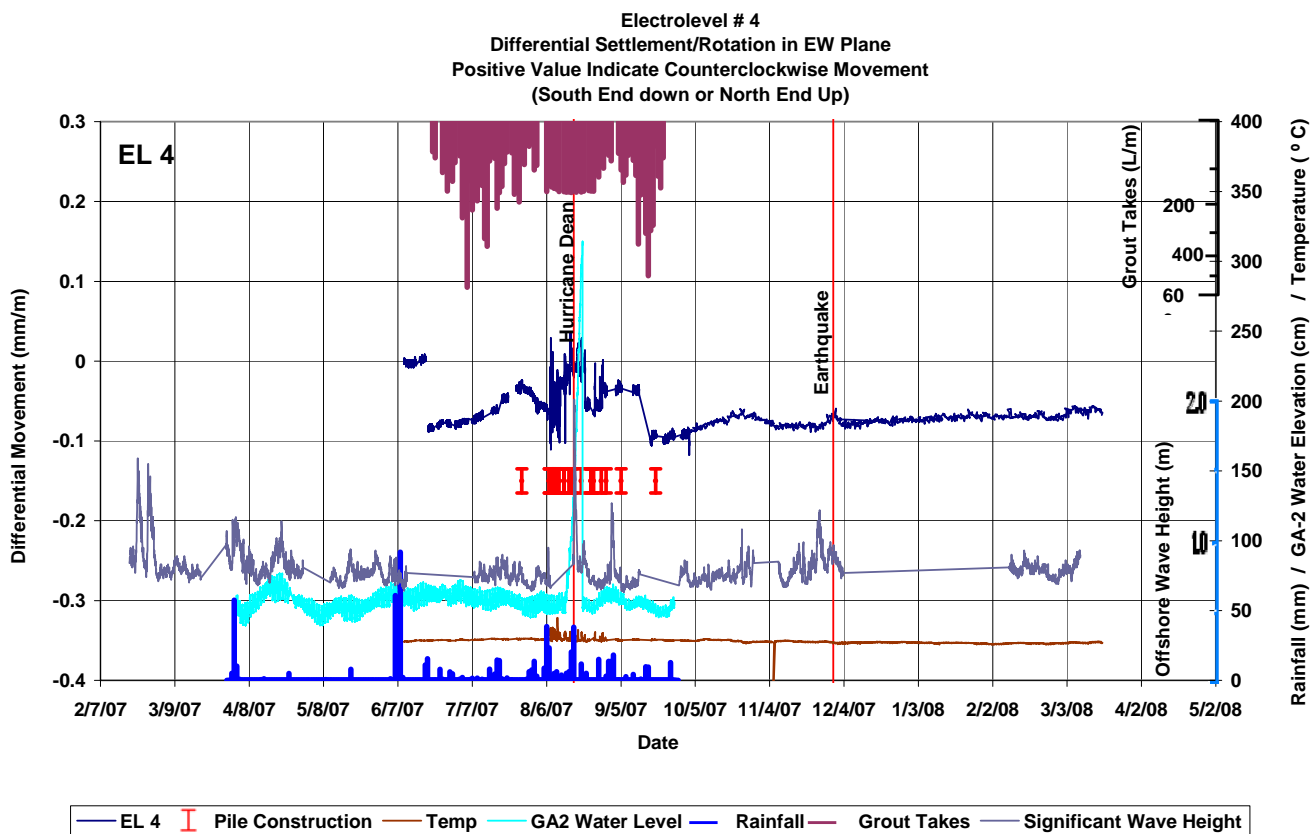


Figure 8 – Monitoring data plot from electrolevel during and after completion of construction.

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.

## 8. CONCLUSIONS

This case record of the application of prescriptive detailed downhole data collection coupled with carefully calibrated numerical modelling to achieve good matching between movement (settlement and tilt) behaviour and the various stages through and subsequent to completion of the remediation works has demonstrated the effectiveness of using advanced numerical modelling approaches for characterizing the behaviour of even weak settlement sensitive soil-like rockmasses.

The fact that all movement monitoring data collected during construction and over a six month period after completion of the remediation works showed behaviour consistent with the model predictions also demonstrates the effectiveness of the remediation design approach.

The structure-foundation interaction modelling with FLAC and the distinct element tessellation modelling with UDEC (of the vuggy voided zones) that was used to evaluate probable foundation response to building loading and cracking, significantly added focus to the micropile design and layout.

The dramatic difference in building response behaviour pre- and post-remediation further confirms that the modelling and foundation characterization accurately replicated reality. The fact that the wave energy and wave height conditions of the original August 2006 storm that triggered the initial cracking were equaled by the passage of Hurricane Dean in August 2007 but with no damage when only the higher priority segments of the remediation works on the sea-cliff side of the building had been completed is further testament to the accuracy of the ground characterization and building-structure numerical modelling assessments. This ability to develop prioritization schedules for the remediation works areas actually benefited significantly from the

understanding gained from the behavioural modelling and ground characterization. The distinct improvement in foundation and building behaviour under the impact of pounding waves and adverse sea states during January and February 2008 as compared with the February 2007 severe damage effects provides further proof of the effectiveness of the remediation works.

The fact that no damage (or even re-activation of earlier patterns of adverse cracking) occurred in response to the passage of the hurricane or in response to the severe wave pounding experienced in January and February 2008 clearly substantiates the model prediction assisted decisions made to reinforce the weak vuggy foundations by grouting and micropiling effectively constructing a sub-surface sea-wall to inhibit further flushing of fines and mitigate any tendency for renewed collapse of voided zones.

The insight gained from the modelling assessments significantly assisted with the development of a robust remediation scheme that has effectively arrested building movement by implementing sufficient stiffening of the foundation by the combination of closely spaced raked micropiling and infill low mobility grouting of the main voided zones to recreate the originally desired competent rockmass foundation. The scheme has also controlled further wave energy induced damage by construction of the sub-surface in place grouted 'sea-wall'.

The robustness of the remediation fix has in addition also been tested by non-marine dynamic stresses, as during the 6 months monitoring period, the structure has been 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 be seen on Figure 8, there was a slight response on the electrolevels but virtually no tilt or rotational displacement, and no crack development.

The modelling and ground characterization contributed significantly to the success of the remediation program to successfully stabilize the building and arrest further foundation distress.

It is considered that the micropiling and infill grouting program achieved its two main design objectives of:

- (a) 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 (i.e. undermining) that would otherwise cause continual downward movement and settlement of the structure; and,
- (b) providing enhanced consolidation and improvement of the foundation rockmass to effect an overall stiffening of the subsurface below the building foundations to improve the

load-bearing capacity of the originally weak and voidy, coralline rockmass.

## REFERENCES

1. Itasca 2004. UDEC Version 4.0, Itasca Consulting Group Inc., Minneapolis, MN.
2. Itasca 2005. FLAC Version 5.0, Itasca Consulting Group Inc., Minneapolis, MN.