Field studies generally offer advantages of working with undisturbed, naturally structured soils on a large scale, and realistic lateral stress/strain boundary conditions. They have the disadvantage, however, of allowing little control of moisture/suction boundary conditions, and continuous measurement is made difficult because of issues of exposure and security of instruments, and some limitations in measurement/monitoring options.

Laboratory studies offer advantages of greater control over applied moisture/suction, albeit coarsely applied in most cases, but the recreation of stress/strain boundary conditions is more difficult and sample disturbance is always problematic.

7. References

[1] Nelson, J. D., and Miller D. J., (1992) *Expansive soils, problems and practice in foundation and pavement engineering*. John Wiley & Sons Inc., New York.

[2] Walsh, P. F., and Cameron, D. A., (1997) *The design of residential slabs and footings*. Standards Australia, SAA HB28-1997.

[3] Meunier A. 2006 Why are clay minerals small? *Clay Minerals*; v. 41; no. 2; p. 551-566

[4] Cardene A, Durand-Vidal S, Turq P and Brendle J. (2005) Study of individual montmorillonite particles size, morphology and apparent charge. *Journal of Colloid Interface Science*, 285, 719-730.

[5] Quirk, J. P. and Murray, R. S. (1991) Towards a model for soil structural behaviour. *Australian Journal of Soil Research*. 29: 829-867.

[6] Oades, J. M. and Waters A. G. (1991) Aggregate hierarchy in soils. *Australian Journal of Soil Research*. 29: 815-828.

[7] Fityus, S. G., Smith, D. W., Imre, E. and Rajkai, K. (2004) Model framework for structure and volume changes in cracking expansive clay soils. In the *Proceedings of the 9th ANZ Geomechanics Conference*, Auckland, NZ.2: 598-604.

[8] Abu-Hejleh, A. N. and Znidarcic, D., (1995) Desiccation theory for soft cohesive soils *Journal of Geotechnical Engineering* 121: 493-502

[9] Chertkov, V.Y. and Ravina, I., (1998) Modelling the crack network of swelling clay soils., *Soil Science* Society of America Journal 62: 1162-1171.

[10] Konrad, J.-M. and Ayad, R., (1997a) An idealized framework for the analysis of cohesive soils undergoing desiccation. *Canadian Geotechnical Journal* 34: 477-488.

[11] Moe, H, and Fityus S. G. Smith, D. W., (2003) `Study of a cracking network in a residual clay soil.' In the *Proceedings of Unsat Asia 2003, the 2nd Asian Unsaturated Soils Conference*, Osaka. pp. 149-154.

[12] Wells, R. R., Dicarlo, D. A., Steenhuis, T. S., Parlange, J. Y., Romkens, M. J., and Prasad, S. N. (2003). Infiltration and surface geometry features of a swelling soil following successive simulated rainstorms. *Soil Science Society of America Journal*, 67, pp. 1344-1351.

[13] White E M (1972) Soil-desiccation features in South Dakota depressions. *Journal of Geology*. 80: 106-111.

[14] Schwartz, G L (1966) Modification of the cracking pattern in a black earth of the Darling Downs, Queensland. *Queensland Journal of Agriculture and Animal Science*. 23: 275-289.

[15] Slatter, E., Fityus, S. and Smith, D.W. (2005) Measuring Lateral Pressures during suction-controlled one-dimensional consolidation. *Experus 2005. Advanced Experimental Unsaturated Soil Mechanics*. Balkema. 117-123.

[16] Ng, C.W.W, Zhan, L. T. and Cui Y. J. (2002) A new simple system for measuring volume changes in unsaturated soils *Canadian Geotechnical Journal* 39(3): 757–764

[17] Fityus, S.G., Smith, D.W. and Allman, M.A. (2004) An expansive soil test site near Newcastle., ASCE *Journal of Geotechnical and Geoenvironmental Engineering* Vol. 130, No. 7,686-695.

[18] Wells, T, Fityus, S., Smith, D.W. and Moe, H. (2006) The indirect determination of hydraulic conductivity of soils, using measurements of gas permeability. I Laboratory testing with dry granular soils. *Australian Journal of Soil Research* Vol. 44 No.7, 719-725.

[19] Wells, T, Fityus, S. and Smith D.W. (2007) The use of in situ measurements of air flow to determine the intrinsic permeability of structured clay soils as a function of depth. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*. Vol 133: No. 12, 1557-1586.

[20] Buzzi O., Fityus S., Sloan S.W. The use of expanding polyurethane resin to remediate expansive soil foundations. Accepted (2009) for publication in *Canadian Geotechnical Journal*.

[21] Fityus S. G. and Smith D. W., (2003) Behaviour of a model footing on expansive clay. In the *Proceedings of Unsat Asia 2003, the 2nd Asian Unsaturated Soils Conference*, Osaka, pp. 181-186.

[22] Fityus S, Buzzi O, Holt M & Gunther T. (2010) The effect of ripping clay soil on swell behaviour To appear in the *Proceedings of the Fifth International Conference on Unsaturated Soils*, Barcelona. September, 2010.

DYNAMIC REPLACEMENT FOR CONSTRUCTING EMBANKMENTS AND WALLS ON SOFT SOIL

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Serge Varaksin, Menard, France
Hamid Nikraz, Curtin University of Technology, Australia

Abstract

The new J W Marriott Hotel in Abu Dhabi includes 90,000 m^2 of roads, manmade hills (embankments), and 100 chalets that have been built on them. The hills are up to 8 m high and retained from one side by mechanically stabilised earth (MSE) walls. The initial ground conditions of this the site was very unfavourable with loose saturated soft silty and clayey soils located at depths extending down to 5 m below ground level and exceptionally as deep as 6 m.
Dynamic Replacement (DR) and pre-excavated DR were used to improve the ground conditions and to ensure that the numerous geotechnical problems including bearing capacity, total and differential settlements, wall stability and primary consolidation period were within project specifications. Improvement phases had to be broken into sub-phases to allow the pore pressures to dissipate. Special design considerations were utilized in the wall sections by pre-excavating and backfilling the wall foundations in strips and trenches with a mixture of demolished concrete and sand.
Menard Pressuremeter Tests (PMT) were used to verify bearing capacity and to predict settlements. Actual ground settlements were measured during and after the construction of the hills by using settlement plates, and the consolidation ratio was determined using Asaoka's method.

Keywords: ground improvement, soil improvement, dynamic replacement, soft soil, pressuremeter

1. Introduction

The new J W Marriott Hotel is a luxurious resort constructed on the southern coastlands of Abu Dhabi, UAE. It is located on 210,000 m^2 of land inclusive of the previous site of the now demolished Gulf Hotel and the virgin plot of land that was situated on its southern border. An 11 storey hotel with an underground parking is located in the middle of the resort. This structure is encompassed on the north and south by two man-made hills gradually elevating to about 8 m above street level. 100 single storey chalets are spread out throughout the hills and the original ground. The hills are retained on the outer perimeter with mechanically stabilised earth (MSE) walls and slope inwards towards a man-made lagoon that is excavated in front and around the hotel. Figure 1 shows the project's layout.

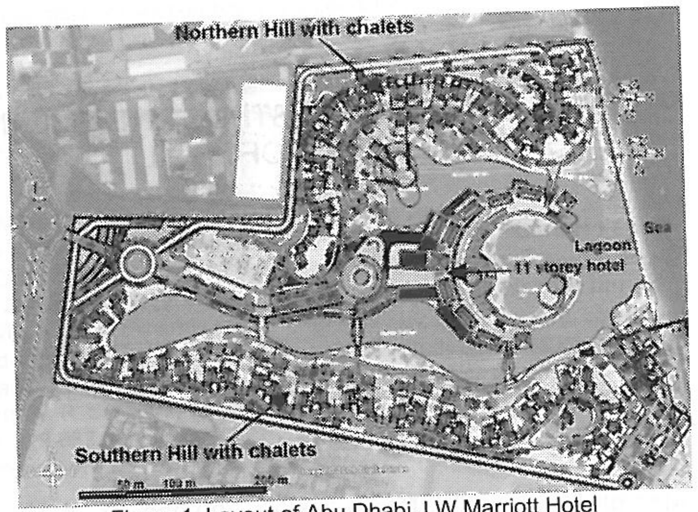


Figure 1. Layout of Abu Dhabi J W Marriott Hotel

1.1. Initial Ground Conditions

Noting that the site was located in a part of the city with a known history of soft soils, and considering that the two storey buildings of the original Gulf Hotel were built on piles, it could have been expected that the project was situated on problematic soils. This speculation was confirmed by the preliminary soil investigation.

The original ground level (OGL) in the undeveloped southern plot was mostly from +1.6 to +2.1 m RL (reduced level= Abu Dhabi Datum); however the level in the eastern portion of this area was even lower at +0.9 m RL. OGL in the Gulf Hotel area was about +2 m RL. Ground water depth was recorded to be from 0.5 to 1.8 m below testing borehole levels with ground water level (GWL) being from +0.4 to +1.30 m RL.

During the preliminary geotechnical investigation 23 SPT boreholes were drilled down to the depths of 15 and 30 m. It was observed that the upper 1 to 2 m of soil was loose silty sand with fines content from 5 to 25%, and generally in the range of less than 15%.

The upper sandy layer was underlain by a very soft silty to clayey sand to sandy silt layer, 3 to 4 m thick, and with fines content from 35 to 70%. The bottom of that layer was generally at the depth of 4 to 5 m and exceptionally as deep as 6 m. SPT blow counts were consistently low and quite often as low as 1.

The very soft layer was followed by a medium to very dense silty to very silty sand layer with SPT blow counts commencing from at least 18 and rapidly increasing to 30 and even more than 50. The bottom of this layer was at the depth of 12 m.

Mudstone with occasional layers of crystalline gypsum or sandstone was encountered down to the bottom of all boreholes. Figure 2 shows the profiles of four SPT boreholes in a section through the site.

Pressuremter Tests (PMT) carried out during later phases of the project as part of a supplementary geotechnical investigation also indicated the presence of a very soft layer with PMT limit pressure, Pl, as low as 110 kPa. The ground profile is summarized in Table 1.

Description	Bottom Depth (m)	thickness (m)	Fines content (%)	N_{SPT}	Pl (kPa)
Silty sand	1-2	1-2	5-25%	5-10	500-1000
Silty sand/sandy silt	5	3-4	35-70	1-3	110-230
Silty sand	12			18-50	2500-4000
Mudstone	-	-	-	-	-

Table 1: Summary of ground profile

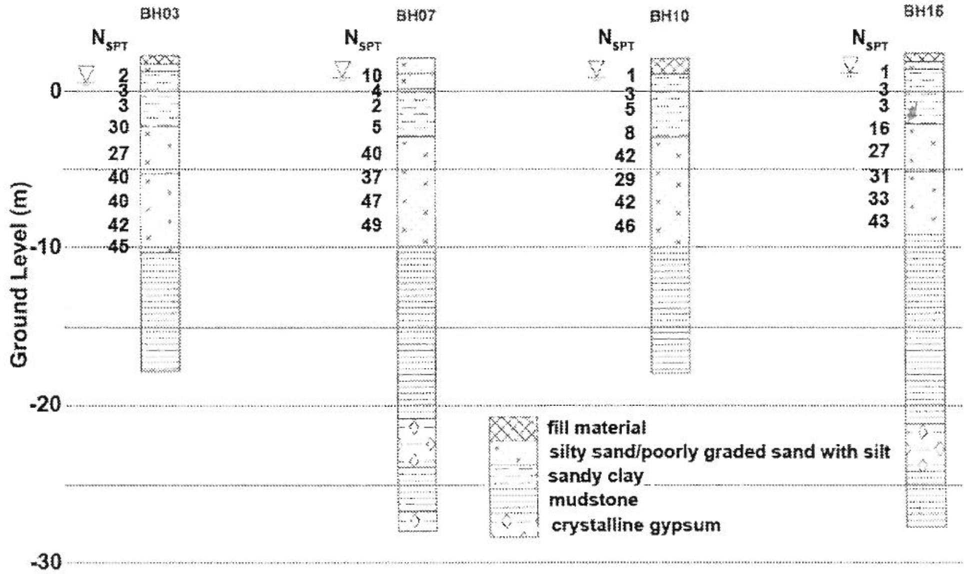


Figure 2. The profiles of four SPT boreholes in a section through the site

The construction of the original Gulf Hotel on piles, the local failures of the ground under the load of the demolition equipment, the poor results of the soil investigation, and the preliminary calculations all indicated that the site was situated on problematic soil and unable to support the loads that the new project were going to realize.

2. Development of the Foundation Solution

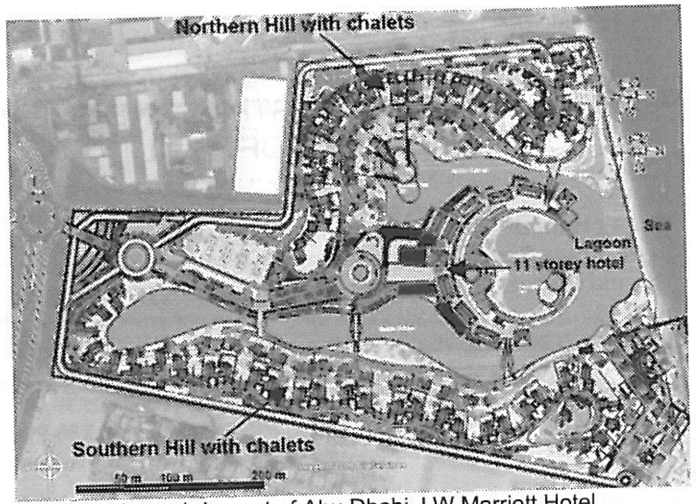
It was decided that heavy structures such as the 11 storey hotel and the underground (under the hill) car parks to be built on piles; however piling did not seem to be a solution of interest for an area of 90,000 m^2 covering the hills, MSE walls, chalets and roads as it would take too much time to execute and would be equally very costly.

Ground improvement was deemed as an economical solution for treating the ground under the mentioned above.

As a first step it was required to develop an appropriate design criteria based on the special requirements and needs of the embankments, walls, and chalets. The stipulated design criteria were:

1. Allowable Bearing capacity
   a. Chalets: 100 kPa under raft footings located 1 m below finished floor level (variable) of the structure.
   b. Man-made landscape hills: To support fill compacted to 95% proctor dry density (assumed to have a unit density equal to 18 kN/m^2) for a slope less than 2V:3H.
   c. Retaining walls: To support 1.3 times the fill weight within a strip behind the wall with a width equal to 70% of the wall height.
2. Total settlement
   a. Chalets: 25 mm under actual footing loads (on average 20 kPa per unit area of chalet).
3. Differential settlement
   a. Chalets: 1/500 between adjacent columns under footing loads
   b. Added fill: 1/500 after 90% consolidation for two points located 10 m apart under fill load
4. Consolidation ratio: 90% after 90 days of constructing the full fill height.

As can be observed although the design criteria could have been further optimized, it has still been able to well address each of the designer's concerns. The required allowable bearing capacity of the chalet does not appear to be in line with the actual 20 kPa load of the chalet; however as this bearing can be usually achieved under a raft footing without extra work and cost, it is acceptable. On the other hand relating settlement to a 100 kPa load would be very irrational because that load will never happen in a one storey chalet, but can be expected in a several storey building. Relating settlements to a load equivalent to the bearing capacity would needlessly increase construction outer perimeter with mechanically stabilised earth (MSE) walls and slope inwards towards a man-made lagoon that is excavated in front and around the hotel. Figure 1 shows the project's layout.



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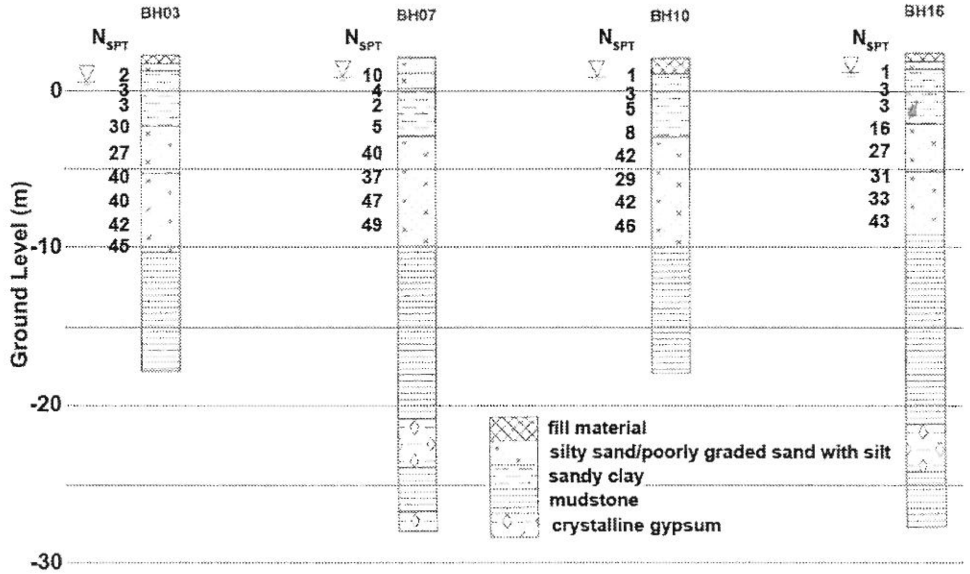

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The design and construct (D & C) ground improvement tender was based on achieving the mentioned design criteria, and any method was deemed as acceptable. In the opinion of the authors this is the preferred method of performing ground improvement projects as specialist contractors are given an equal opportunity to propose any technique that will meet the requirements with the lowest cost. Furthermore, soil improvement is a technology that is usually driven by specialist contractors who are continuously developing enhancements to their methods to improve performance. It can be assumed that such specialists may be able to provide smart and affordable solutions while accepting responsibility of both design and construction.

The project was awarded to a specialist contractor who had based the ground treatment on the Dynamic Replacement (DR) method.

3. The Application of Dynamic Replacement

Dynamic Replacement is a soil improvement technique that was invented and patented by the late French engineer, Louis Menard, in 1975 [1].

DR is a very effective and affordable technique for treating saturated high fines content soils. The main idea behind DR is to penetrate and replace the loose and soft soil with dense granular material.

DR is applied by dropping a heavy pounder, punching through the soft zones and continuously filling the crater with new materials. Suitable material for the DR columns can be a wide range of granular material such as sand, gravel, crushed stones or demolished concrete.

Application of DR will be optimized with the existence of a granular transition layer over the DR columns. This layer will help to redistribute the loads using the arching effect [2].

If necessary and possible, material may be pre-excavated prior to soil improvement operations at the DR impact location (print). This may be due to the existence of a stiff superficial layer or to facilitate deeper penetrations. In this project due to the availability of redundant sand in the lagoon area and affordable sand resources in the vicinity of the project it was decided to pre-excavate the DR prints and to backfill them with sand.

In order to provide a minimum thickness of granular material as the working platform that was sufficiently above the groundwater level (GWL), the lowlands and areas with thin layers of the surface sand area were initially backfilled with sand in such a way that the thickness of the granular layer was at least 1.5 m and the platform level was at least 1.3 m above GWL.

High energy impacts such as what is experienced in Dynamic Replacement increase the pore water pressure. In cohesive soils that do not allow the rapid dissipation of pore water pressure, each consecutive blow increases the pressure to the point where the soil liquefies. In some soils only one blow is enough to liquefy the soil and to prohibit the application of additional blows. In such a case, additional blows can only be applied when the pore water pressure has sufficiently decreased. As can be seen in Figure 3, built up of pore pressure and its release in the form of sand boiling was even observed in prints surrounding the impact location during the soil improvement works.

Application of pre-excavation and backfilling of DR prints with sand was able to accelerate the pore water pressure distribution in the soil; however even the implementation of this technique was not able to prevent the liquefaction of soil during the first few poundings, and work had to be continued in sub-phases once the pore water pressure had reduced sufficiently.

Experience gained during the works suggests that the liquefaction of the soil can be retarded by minimizing the time interval between pre-excavation and backfilling and the pounding. This may be somewhat expected as increasing the time interval between the different stages of the work process will allow the backfill material's water content to increase.

While design calculations indicated that execution of Dynamic Replacement columns would be able to satisfy the design criteria in most areas, the wall stability analyses suggested that additional measures may be required.

MSE walls were 3, 5, 5.5 and 8 m above finished road level (at +2 m RL). Calculations using Talren software demonstrated that in order to have a safety factor of 1.5 (see Figure 4), in lieu of DR columns, DR trenches had to be performed. For walls higher than 7.5 m, the trench would be 5.25 m behind the wall and extend 2 m in front of the wall. For shorter walls, the treatment zone required only 50% replacement, hence while maintaining the original band width, the excavation was done in alternative strips within the trench area with a total coverage of 50%.

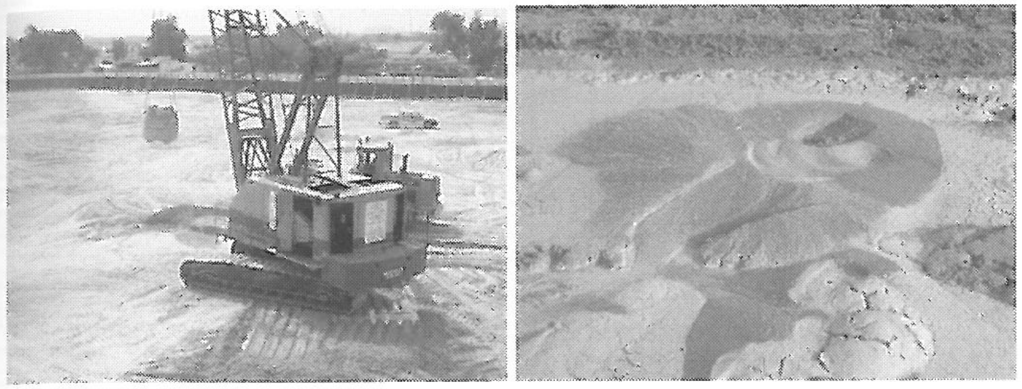

Figure 3. Dissipation of pore water pressure in the form of surface sand boiling (behind the rig) in J W Marriott Hotel Project

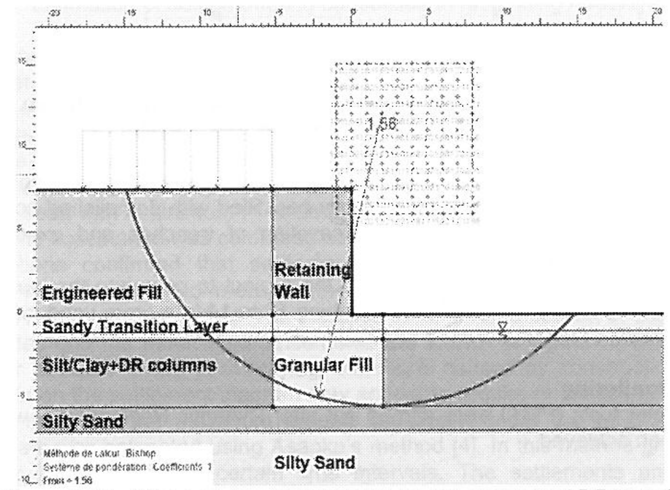

Figure 4. Stability analysis of MSE Wall on pre-excavated DR trench

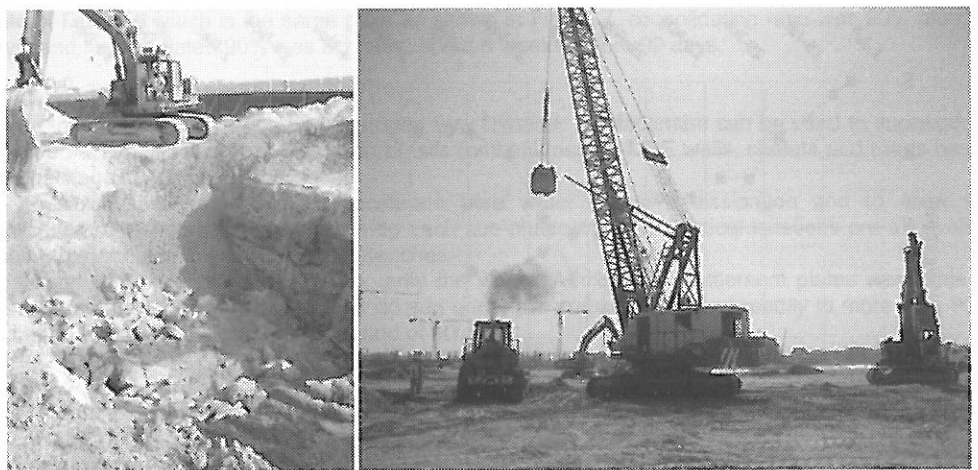

Figure 5. Excavation of DR trenches, backfilling with demolished concrete and sand, and execution of Dynamic Replacement

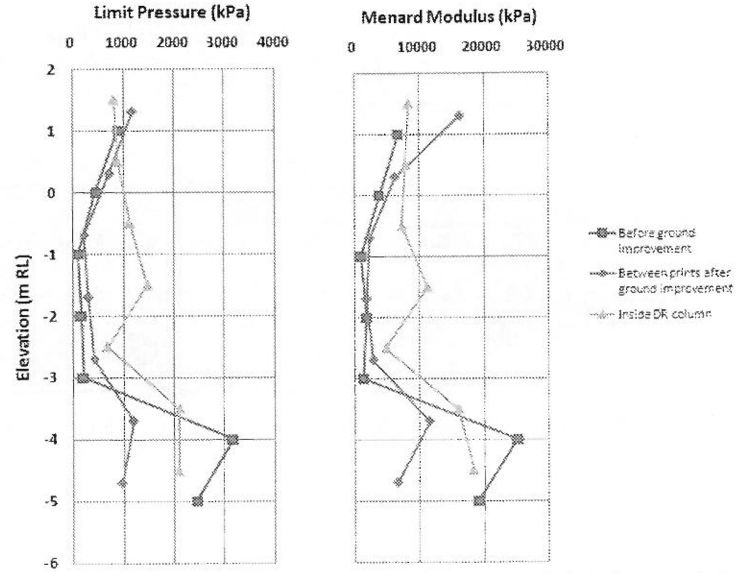

Figure 6. Comparison of PMT parameters before ground improvement, in between prints after ground improvement and inside DR columns after ground Improvement

In order to ensure that the bottom of the trenches or strips would have a high friction angle for stability assessment, the excavations were originally backfilled with demolished concrete pieces and then with sand similar to the DR columns. Excavation of trenches and execution of Dynamic Replacement for the walls is shown in Figure 5.

Two specially modified rigs were allocated to the project to complete the works within the 150 days that the contract specified. DR pounders weighing 12 to 14 tons were used for the works. Based on the ground condition, drop heights were variable from 5 to 15 m.

4. Testing and Monitoring

22 pressuremeter tests (PMT) were carried out after Dynamic Replacement works to verify that acceptance had been achieved.

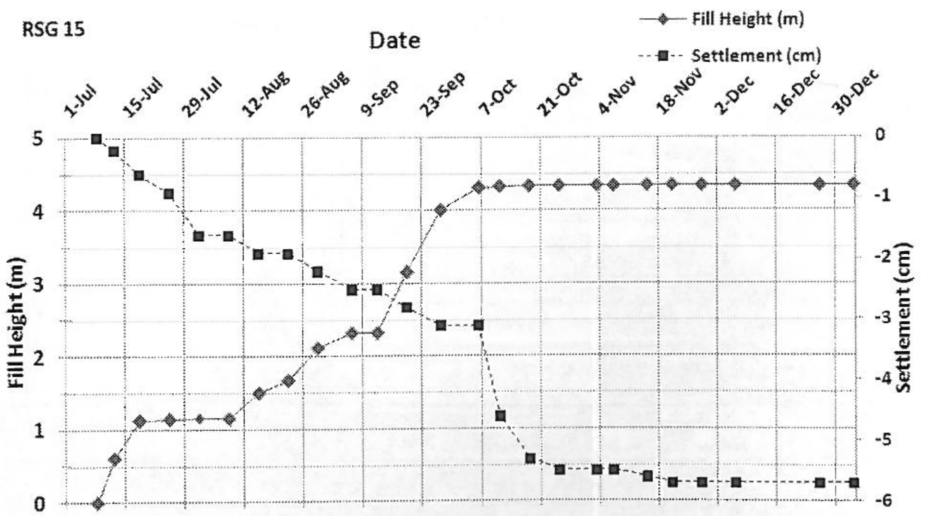

Figure 7. The measurements of fill height and ground settlement under the fill during a time interval

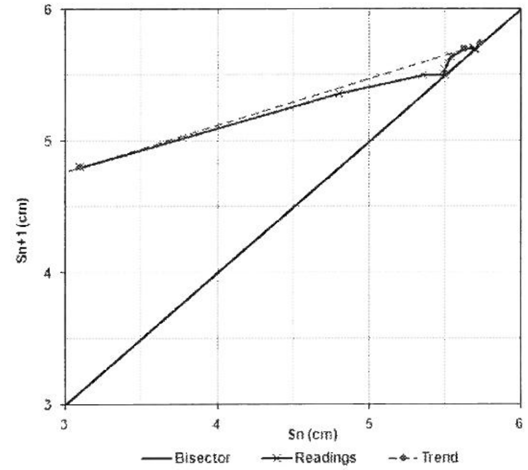

Figure 8. Estimation of settlement and consolidation ratio using Asaoka's method

PMT parameters (Pl and Menard Modulus) before ground improvement, after ground improvement in between the prints and after ground improvement in the DR column are shown in Figure 6. It can be observed that after ground improvement while the limit pressure in between the DR columns also increased by about 100% (from an original low of 110 kPa to 220 kPa), yet this improvement alone was not sufficient to reach acceptance, and it was the substantial increase of strength in the DR columns that made acceptance possible. In Figure 6, the PMT results of a typical DR column can be seen with a minimum Pl value of about 700 kPa.

Bearing capacity acceptance was confirmed using the method proposed by Menard [3].

While calculations confirmed that settlement requirements had been met, additionally 15 settlement plates were installed to measure the ground subsidence during construction of the embankments and for a period after that. The increase in height of a section of the hill around one of the settlement plates and the associated settlements are shown in Figure 7. This diagram has not been corrected for possible errors in surveying, vibrations caused by construction equipment, etc; hence certain points on the settlement diagram may appear to require re-assessment.

Consolidation ratio and the maximum ground settlement under the weight of the hill at the location of each plate was estimated using Asaoka's method [4]. In this method ground settlement at a specific location is measured at certain time intervals. The settlements are plotted on both coordinates; however for each settlement, S_n, on the x-axis, the next settlement, S_{n+1}, is plotted on the y-axis. At 100% consolidation the settlement plot must intersect with the bisector. In the settlement plate of Figure 8 which is the same plate as shown in Figure 7, consolidation ratio was 99% after 90 days, and the stipulated 90% was achieved about 6 weeks before 90 days.

5. Conclusion

In this project it has been demonstrated how Dynamic Replacement can be used to successfully improve the ground for the construction of hills (embankments), MSE walls, chalets and roads based on a package of design criteria.

Pre-excavation was used to accelerate pore water pressure dissipation and to allow the application of more pounder blows during each sub-phase of DR. At critical locations pre-excavation was further adopted in the form of DR trenches.

Pressuremter tests were used to verify the works. Additionally, settlement plates were able to demonstrate that the ground had consolidated under the embankment load readily to more than 90% and up to 99% after the contractual period of 90 days.

6. Acknowledgement

The authors wish to express their gratitude to Menard for providing the paper's data.

7. References

1. Menard Soltraitement, (no date) *Soil Improvement Specialists - Design, Construct ... Perform (Brochure)*

2. Hamidi, B., Nikraz, H. & Varaksin, S. (2009) Arching in Ground Improvement. *Australian Geomechanics Journal,* 44, 99-108.
3. Menard, L (1975). "The Menard Pressuremeter: Interpretation and Application of Pressuremeter Test Results to Foundation Design, D.60.AN" *Sols Soils,* 26, 5-43.
4. Asaoka, A. (1978) Observational Procedure of Settlement Prediction. *Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering,* 18, 4, 87-101.