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*The paper was published in the proceedings of the 11<sup>th</sup> Australia New Zealand Conference on Geomechanics and was edited by Prof. Guillermo Narsilio, Prof. Arul Arulrajah and Prof. Jayantha Kodikara. The conference was held in Melbourne, Australia, 15-18 July 2012.*

# Ground Improvement in Sabkha, Abu Dhabi Emirate, UAE

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## ABSTRACT

MASDAR city occupies some 556 hectares of fairly flat land to the east of Abu Dhabi, and lies within a wide and continuous low lying coastal sabkha plain. Here, the sabkha deposits comprise interbedded layers of clays, silts and sands with variable shell and organic content where a common feature is a variable level of cementation by various salts. The deposition is recent (Quaternary) and there is a tendency for loose and weakly cemented grain structures which result in low unit weight, low shear strength and high compressibility. Via a collaborative initiative between the Promoter and regional ground improvement contractors, a series of large scale ground improvement trials were undertaken at MASDAR in order to ascertain the field performance of a range of ground improvement techniques using columns. The findings from the trials are presented and discussed as a rare example of a parallel comparison of performance within a common ground condition. Back analysis of the load tests allows for the development of a common ground model which is supported by an advanced ground investigation comprising *in situ* tests such as CPTu, cyclic Menard Pressuremeter, and shear wave velocity profiling, and routine laboratory testing. From the model, observations regarding underlying geotechnical characteristics, cross correlations for the sabkha and settlement without ground treatment are made.

*Keywords:* sabkha, ground improvement, grouted stone columns, Young's Modulus, primary compression index

## 1 GROUND INVESTIGATION STRATEGY

A direct visual assessment of the fabric and structure of the sabkha layer was undertaken at open excavations in the site. The sabkha layer comprises a series of impersistent layers of silty clay, silt, and silty sands and variable degrees of cementation.

Accordingly, the ground investigation was planned as clusters (investigatory holes in close proximity to one another) so as to aid in the development of parameter cross correlation. The following *in situ* testing was important for the overall characterisation of the sabkha layer and underlying sands:

- Seismic CPT profiling and cyclic Menard Pressuremeter testing for evaluation of *in situ* soil stiffness and shear strength.
- CPTu profile for detailed examination of soil layering and classification of soil types within the sabkha, and for detailing changes in soil strength and stiffness.

The accompanying laboratory test programme included routine soil classification tests, shear strength and consolidation tests.

## 2 GROUND PROFILE AT TRIAL SITES

There were three predominant soil layers common to all trial sites, namely Aeolian Sand, Sabkha and Marine Sand. The underlying bedrock comprised siltstone, mudstone, sandstone, and gypsum layers, and was encountered at depths vary between 5.5 m and 7.5 m below original ground level. Wider investigations across the site revealed some instances of localised shallow height voiding at the interface of the bedrock and the overlying superficial soils, however these were not encountered at the test sites. The geotechnical characteristics of the soil layers are summarised in Table 1.

Table 1: Typical Parameters for Key Geotechnical Units

Geological Units / Characteristic	Thickness (m)	Carbonate Content (%)	Friction Angle $\Phi'$ ( $^{\circ}$ )	Cohesion $c'$ (kPa)	SCPT - Shear Wave Velocity (m/s)	Average Menard Pressure-meter Modulus- $E_m$ (MPa)
<b>Unit 1a - Aeolian Sand</b>	1.0	30 to 40	32 -35	0	180 - 300	23.0
<b>Unit 1b – Sabkha</b> PI = 10% $\pm$ 4 m/c = 31 to 40 Organic matter= 1%	1.5 - 2.0	60 to 70	27 - 32	0	100 - 150	8.20
<b>Unit 1c - Marine Sand</b>	2.5 – 5.0	25 to 50	32 -36	0	250 - 500	20.0

**Aeolian Sand.** This layer was typically encountered as dense silty sand and is most likely a made ground layer of around 20 years in age, placed for facilitation of agriculture.

**Sabkha.** The sabkha comprises interbedded layers of impersistent bands of silts, sands, and clays, typically between 100 mm and 300 mm in thickness (Figure 1). There was occasional growth of salts and a variable degree of weak cementation. There was very little visible organic matter except for occasional seams and as indicated by spikes in the friction ratio of CPT data.

The sand bands were found to be of very low dry density ( $1.1 \text{ Mg/m}^3$ ) whilst that of the cohesive bands lies between  $1.43 \text{ Mg/m}^3$  and  $1.57 \text{ Mg/m}^3$ . The silt and clay bands were of normal activity and classify as CL or ML, with a liquidity index of around 0.75 to 1.0. The friction angle of the sabkha is found to lie between 27 and 35 Degrees, depending upon the relative amounts of sand and clay (Figure 2).

The undrained shear strength profile was established by a combination of laboratory tests, and the Menard pressuremeter tests (Figure 3). The profile is indicative of a degree of over-consolidation which may relate to a combination of cementation, former higher sea level, removal of an agricultural sand layer, and ongoing site dewatering. The measurement of the primary compression index ( $C_c$ ) was also found to reduce with depth and in proportion to the density (Figure 4.). The over-consolidation ratio is estimated at 1.8. The consistency, degree of over-consolidation, and primary compression index of the cohesive part bears a striking similarity to that reported for upper levels of coastal sabkha in Saudi Arabia (Al-Sharmrani, 2004).

From a total of 8 number Menard Pressuremeter Tests, the Menard Modulus  $E_M$  was typically found to range between 10 MPa (top) and 3 MPa (base). A typical empirical conversion would give rise to an equivalent drained Young's Modulus,  $E'$ , about 1.5 times higher (say 15 MPa to 5 MPa).

From the dynamic shear wave velocity from SCPT, the dynamic Young's Modulus ( $E_d$  for very small seismic shear strain level) is around 50 MPa. Based upon typical modulus degradation relationships, Young's Modulus of silts and clays at normal engineering operational strain levels could be expected to be around 1/10 of that for the dynamic strain level. Accordingly, the Young Modulus for normal strain levels as deduced from both pressuremeter and SCPT is rather comparable, with an  $E/S_u$  ratio of about 500.

**Marine SAND** comprises medium dense and dense silty sand with occasional loose silt layers. SPT N values were found to correlate well with CPT tip resistance, with  $q_c/SPT N = 4$ , a ratio close to the median line as published by (Robertson et al.1983).There was also a reasonably good correlation between normalised seismic shear wave velocity and normalised cone resistance, following the relationship proposed by (Robertson and Fear 1995). The sands characterise as aged and compressible.

When normalised by the dynamic Young's Modulus ( $E_d$  from SCPT), the unload-reload Young's Modulus,  $E_M$  from the pressuremeter test correlated closely with the operational cavity strain range during each test loop (Figure 5).

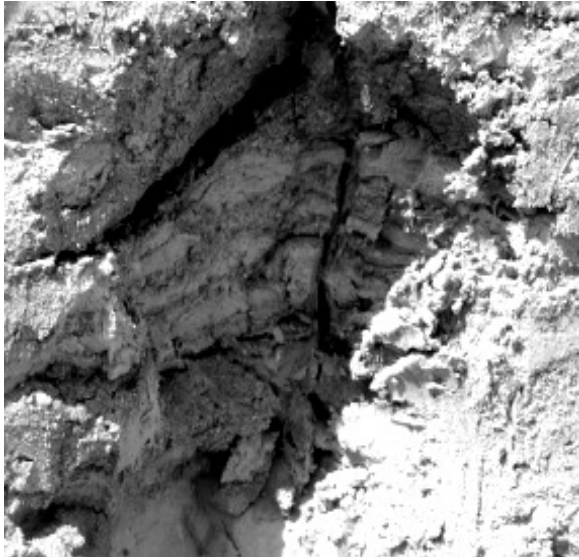


Figure 1. Cut face in the sabkha layer

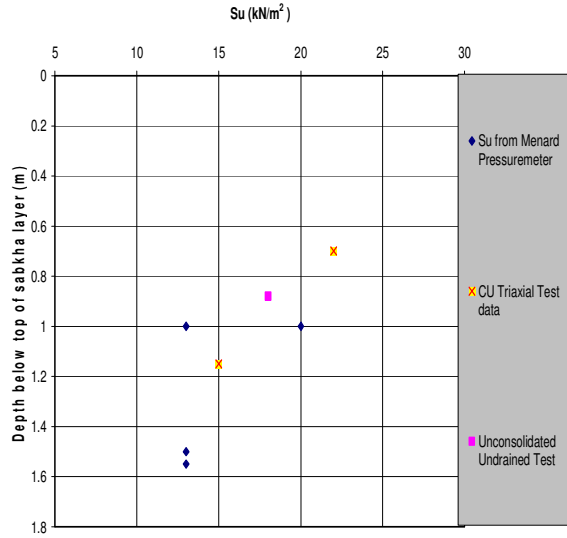


Figure 3. Sabkha - undrained shear strength profile

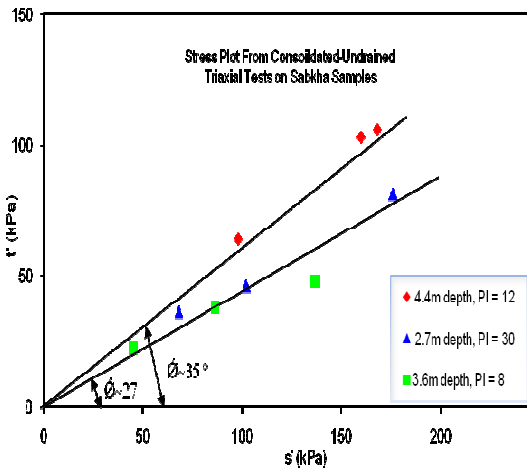


Figure 2. Sabkha - angle of friction

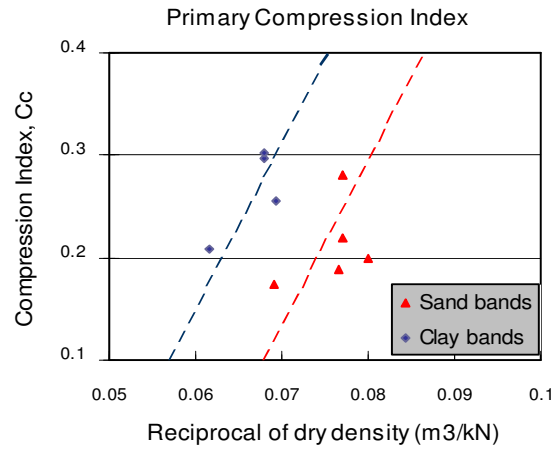


Figure 4. Sabkha - variation of  $C_c$  with reciprocal of dry density

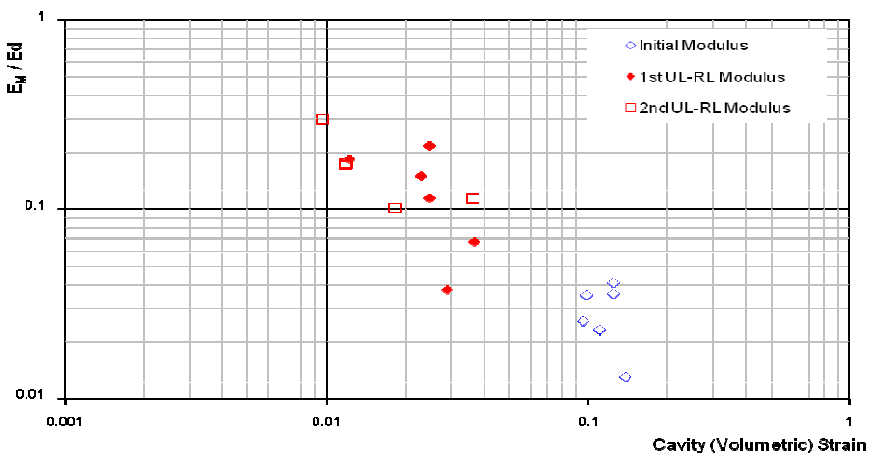


Figure 5. Marine sand - modulus degradation curve

### 3 ZONE LOAD TESTS

The contractors were allocated individual test sites referred to herein as A, B, C and D. In total nine zone load tests were carried out, as detailed in Table 2. CPT testing across all test sites revealed a common ground condition, albeit that the thickness of the sabkha layer varied between about 1 m and 2 m in thickness. The groundwater table was encountered at a depth of close to 2.0 m at all sites.

A zone test typically comprised an area of treated ground upon which a 4 m square and 0.5 m deep concrete raft was cast. The raft was then loaded with kentledge so as to generate a foundation bearing pressure of up to 250 kPa. At site D, the treated area was much larger and the area was loaded directly with a 31m square earth embankment so as to generate an equivalent foundation load of 100 kPa. Unlike the other trials, the ground treatment did not penetrate to the top of bedrock, but stopped in the upper level of the Marine Sand. The recorded raft settlements are presented in Table 2 for a common foundation load of 130 kPa (sites A to C). At higher test loads of between 200 kPa and 250 kPa, various creep effects started to appear in the settlement record for certain tests.

Table 2: Zone Load Tests - degree of Improvement for foundation pressure of 130 kPa.

Test	Type of Treatment	Average Actual Diameter and Length (m)	Reciprocal of Area Replacement Ratio	Estimate of Settlement without treatment (mm)	Test Settlement (at 130 kPa)	Improvement Ratio
A1	VR-Wet top feed stone columns	1.25 x 7.0	2.61	50	13.8	3.6
A2	VR-Dry bottom feed stone columns	0.88 x 7.3	5.26	54	24.5	2.2
A3	VR-Dry bottom feed stone columns	0.88 x 7.3	3.76	54	18.0	3.0
A4	Grout mix stone columns	0.65 x 7.0	11.14	52	10.3	5.0
B1	VR-Dry bottom feed stone columns	0.88 x 7.7	6.58	32	16.0	2.0
B2	Vibro-compaction on 3.5 square grid with a stone column at grid centre- dry bottom feed	0.88 x 6.8	31.85	32	30.0	1.1
C1	VR-Dry top feed stone columns	0.85 x 5.50	8.77	42	20.0	2.1
C2	Sand Cement Column	0.40 x 6.70	12.80	42	10.0	4.2
D1	Dynamic replacement	1.90 square x 5.0	6.50	43	28.0	1.5

The area replacement ratio is the area attributed to a column divided by the cross sectional area of that column. In the case of the raft based load tests, the ratio is computed as the total area of the raft divided by the number of columns contributing to the support of the raft. The performance of the tests is compared in Figure 6.

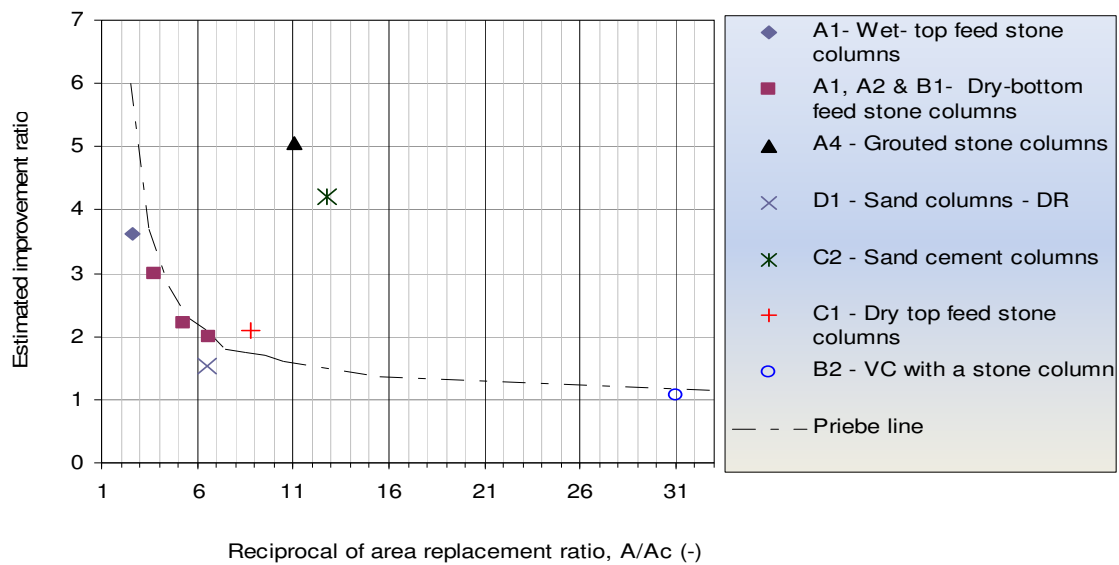


Figure 6. Predicted improvement ratios for various techniques.

It is evident that there is a reasonably good agreement between the computed improvement factors for the sand and stone columns and the theoretical line of (Priebe, 1995) for nominal friction angle of  $40^\circ$  in Figure 6. Where cement or grout is introduced, the improvement factor is significantly enhanced. The wet method of stone column installation appears to result in a somewhat reduced relative performance compared to the dry instillation method.

The reduced performance at site D (sand columns) can be attributed to the termination of the sand columns in the Marine Sand, a significantly larger scale and depth of influence (embankment loading) compared to the raft zone tests and a reduced friction angle for a sand column.

#### 4 INTERPRETATION OF GROUND MODEL

A back analysis of each zone load test was undertaken using the Priebe method in order to find a common geotechnical model that could support the settlement records as one set of results. In this model, the soil layer thicknesses are varied but the underlying geotechnical relationships for the stiffness of the layers are kept essentially the same. In the case of ground improvement by soil cement and or grouted columns, it is found that back analysis based upon the equivalent composite modulus approach is more suitable. The stiffness for the columns is based upon plate load tests, pressuremeter tests, and compression tests. It is found that there is a consistent model between back analysed properties and key geotechnical relationships as summarised in Table 3.

Table 3: Geotechnical Model from back analysis

Unit	Back Analysed Compressibility / Young's Modulus MPa	Measured Column Stiffness MPa	Key geotechnical properties / relationships
Aeolian Sand	E = 35 – 50	Stone : 100 Sand : 60 Sand cement: 3800 Grouted Stone: 250	E = 35
Sabkha	C <sub>c</sub> = 0.18 (layer average) C <sub>r</sub> = 0.06 (layer average) p' <sub>c</sub> = 80 kPa. E' = 3.0 to 3.5		Clayey bands: C <sub>c</sub> = 0.25 Silty bands: C <sub>c</sub> = 0.17 Sandy bands: C <sub>c</sub> = 0.10
Marine Sand	E = 25 to 70		E = 5qc

## 5 CONCLUSIONS

Based upon a detailed study of the ground investigation data, cross correlations, and the derivation of a common back analysed soil model, it is found that even though soil carbonate contents are quite high, the compressibility behaviour of the soils within normal loading levels is very much in line with standard publications for common empirical derivations of soil stiffness, such as Young's Modulus from CPT cone resistance as derived for non-carbonate soils. However, when the soils are under shear (such as for the measurement of  $q_c$  during CPT testing), the fundamental behaviour is that of compressible soil grains.

Whilst there may be some light degree of cementation in the sabkha layer, it does not appear to significantly influence the compressibility of the soils – they appear to behave much like uncemented silica soils at the scale of the zone load tests. The back analysed compression indices are in line with those that can be routinely measured in laboratory Oedometer tests. The layering within the sabkha needs to be taken into account in order to derive a composite compressibility, and this can best be resolved by means of electric cone penetration testing.

All uncemented / ungrouted columns provide a degree of improvement proportional to the area replacement ratio and column stiffness in line with Priebe's formula. When interpreting zone load tests on uncemented columns, the area ratio that most appropriately matches the performance is that relative to the raft footprint, and not that of the column repeat pattern. When transferring the results of zone load tests into design / or in design verification, it is important to adjust the improvement factor so as to be compatible with column area ratio at the same scale as that of the intended application.

Grouted stone columns and sand cement columns can offer significantly higher degrees of ground improvement with only modest additions of cement and/or grout. These are best analysed as a composite elastic system.

## 6 ACKNOWLEDGEMENTS

The authors would like to thank Mr Yousef Baselaib, the Director of Masdar City Operations of A Mubadala Company for hosting this research and Dr. Jimmy Wehr of Keller Grundbau GmbH for his valuable comments. The authors would also like to thank Keller Grundbau GmbH, Soletanche Bachy, Freyssinet Middle East L.L.C., (Menard) and Al Ghurair Arabian Foundation Engineering for agreeing to release the data of their ground improvement tests.

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