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Soil Improvement of D-Wall Platform for road tunnels at the western side of Suez Canal

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ملخص

تهدف الحكومة المصرية في الوقت الراهن إلى التوسع العمراني والصناعي في المناطق الاستراتيجية والمتمثلة في منطقة شرق بورسعيد وقناة السويس وذلك لجذب المزيد من الاستثمارات الأجنبية في مصر وذلك عن طريق إنشاء مناطق صناعية ولوجستية ضمن مشروع محور قناة السويس بالإضافة الي إنشاء العديد من أنفاق السيارات والسكك الحديدية أسفل قناة السويس مثل مشروع أنفاق قناة السويس. وقد تم عمل تحسين للتربة باستخدام طبقات من الاحلال وشبك تسليح التربة من اجل تدعيم قدرتها علي تحمل احمال الماكينات الخاصة بتنفيذ الحوائط اللوحية والمزمع استخدامها بأعمال تنفيذ الانفاق. ومن اجل تقييم النتائج والتأكد من فاعليه مختلف اقتراحات اعمال التحسين فقد تم اجراء اختبار تحميل كامل للتأكد من فاعليه تلك الاعمال وصلاحيتها.

Abstract

Recently the Egyptian governorate have promising plans for urban and industrial expansions in the east side and coastal area around new and old Suez Canal. These projects include mega projects such as industrial zones, terminal ports, tunnels, etc. One of the major geotechnical problem in such area is the existing of soft soil deposits which widespread in the coastal area. The soil have a low strength parameters and high compressibility coefficients leading to low allowable stresses for safety against shear failure and excessive total / differential settlement. In general, for such conditions either soil improvement techniques or deep foundations are required for safe design of embankment and foundations. Indeed, for roads embankments, similar ground improvement methods would be adopted on soft soil to enhance the geotechnical parameters and in turn leads to safe and stable condition against different modes of failure. One of the major projects in east Port Said area is road tunnels passing underneath the old and the new Suez Canal. The construction of the tunnel requires the use of heavy equipment such as pilling rigs and diaphragm wall machines (D-wall). The anticipated stresses due to the D-wall machine were estimated to be much higher than the safe stresses for the existing soft soil deposits. To overcome this problem, a soil improvement using controlled backfilling and geogrid layers (MSL- Mechanically stabilized layers) was adopted to provide a safe working platform for the heavy D-wall equipment. Several full-scale load tests were performed to evaluate the safe construction technique and the effectiveness of the use of MSL to provide safe platform before the start of construction.

Keywords

Controlled Backfill; D-wall platform; Full scale load test; Geogrid; MSL; Port Said; Soft clay; Soil improvement; Tunnel

Introduction

The geotechnical conditions in north coast of Egypt and especially for Port Said deposits are huge challenges for the geotechnical designers (Abdel Rahman, M, 1985). Soft soil deposits extends to large depths reaches about 40m to 50m at the western side of Suez Canal. For the construction of new tunnels under Suez Canal 2015, diaphragm walls were necessary for both lunching and receiving shafts. The diaphragm wall machines exert stresses higher than the allowable stresses for such clay. For such conditions, many soil improvement techniques would be required to reduce the settlement and increase the shear strength parameters (NAVFAC DM-7.1) and in turn provide a safe construction platform to withstand the anticipated stresses with adequate safety factor. One the soil improvement techniques had been used for such condition is the mechanically stabilized layer defined as MSL (GIROUD 1984). The present research presents a summary of the geotechnical condition at project as well as the soil improvement techniques including normal soil replacement and MSL system proposed for the construction of platform. The effectiveness of the two systems had been evaluated throughout full-scale load tests performed to present the actual behavior of the two system. The research also discuss the details and lessons learned from the construction process and results of the tests.

Soil Formation

Massive geotechnical investigations were performed. The results of the analysis show that the formation of the sub soil mainly consists of three main layers below top crust layer, which is medium stiff to stiff clay having thickness in range of 0.5 m.

1. Layer 1: Very soft to medium stiff CLAY: This layer appears from the natural ground level and extends down to depth varies from 40.0m to 43.0m.
2. Layer 2: Silty SAND: this layer appears below the soft clay layer and extends to depth of about 90.0m
3. Layer 3: Silty hard CLAY: this layer appears at different depths with variable thicknesses.

Figure 1 shows the selected design value for undrained cohesion, while figure 2 shows over consolidation ratio (OCR) with depth using sample of cone penetration tests (CPT) performed in the project (Robertson 2009). The figure shows that OCR is in range of 1.

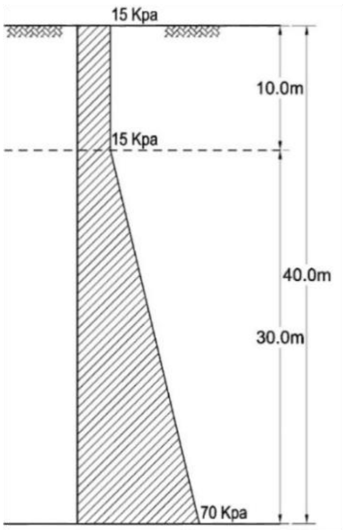

Figure 1: The selected design value for the undrained cohesion

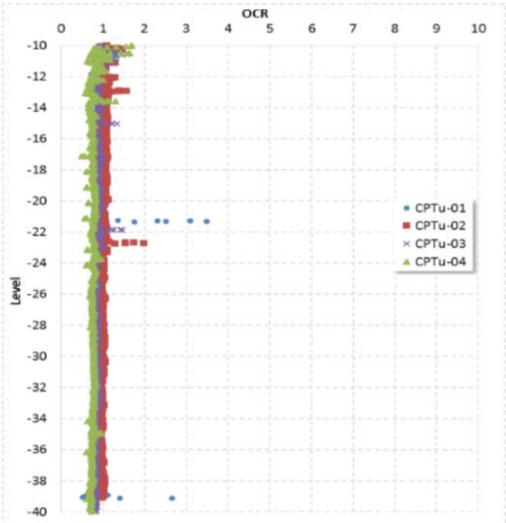

Figure 2: The OCR ratio with depth using sample of CPT tests

Diaphragm wall platform

The location of the proposed D-wall platform is shown in figure 3

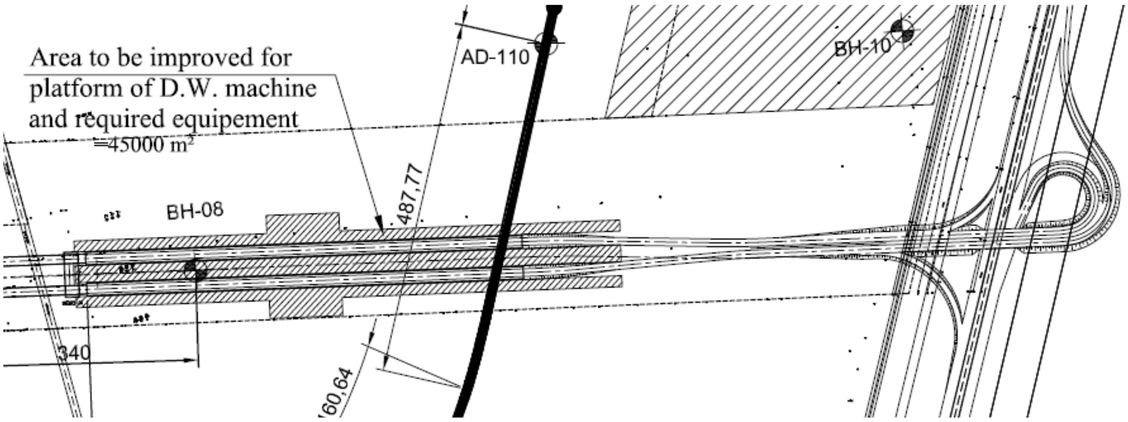

Figure 3: Location of the proposed D-wall platform

The D-wall platform had width of 15 m on the outside area of the D-wall, while the internal area had a width of 25 m as shown in figure 4. The length of the platform under study was about 700 m. The platform was necessary for the movement of D-wall machine, tower cranes, concrete pumps, and other heavy equipments. In the same time, an area of platform was used for installation of the tunnel-boring machine TBM. The information about the maximum stresses and loading area shows that D-wall machine would extents a maximum pressure of about 250 kN/m^2 (2.50 kg/cm^2) with width of the loaded area and equipment 1.2 m and length about 5.5 m. The loading areas are shown in figure 4

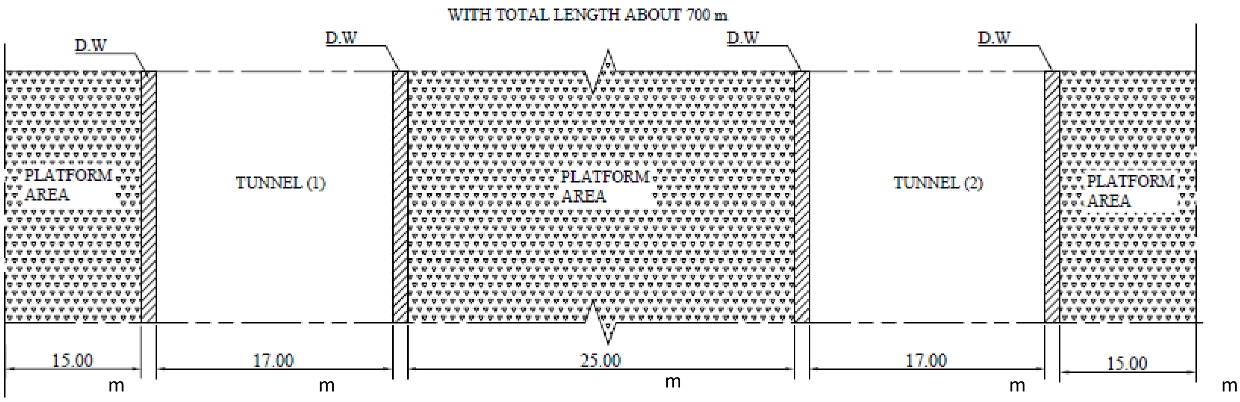

Figure 4: Platform location and dimensions (plan view)

Initial selection of the ground improvement system

The allowable bearing capacity had been estimated for such soil with a design undrained cohesion (15 kN/m^2 for upper 10m) for a factor of safety of 2 and strip loading with shape factor of 1.3 which represents the actual condition of the D-wall base. The estimated allowable bearing capacity was in range of 66 kN/m^2. This value is much lower than the actual stresses.

Several alternatives for ground improvement had been studied including vertical drains and preloading, stone columns and mechanically stabilized layers (MSL) and finally MSL had been selected from cost and schedule points of view. Indeed this value of allowable bearing capacity was much lower than required value as per anticipated loading.

Jenner 2000 had described in details the mechanism of MSL system, which increase the bearing capacity and unify the settlement, which is mainly the interlocking mechanism.
For stabilized layers to be effective it must have the ability to distribute load through 360 degrees. To ensure optimum performance, the geogrid in a mechanically stabilized layer should have a high radial stiffness throughout the full 360 degrees. Geogrid can solve stabilization problems because it interlocks very efficiently with granular materials. When granular materials are compacted over the geogrid. This means that the interlock is the mechanism by which the geogrid and aggregates interact under an applied load. This mechanism results in the confinement and lateral resistance of granular materials. The above is illustrated in figure 5.

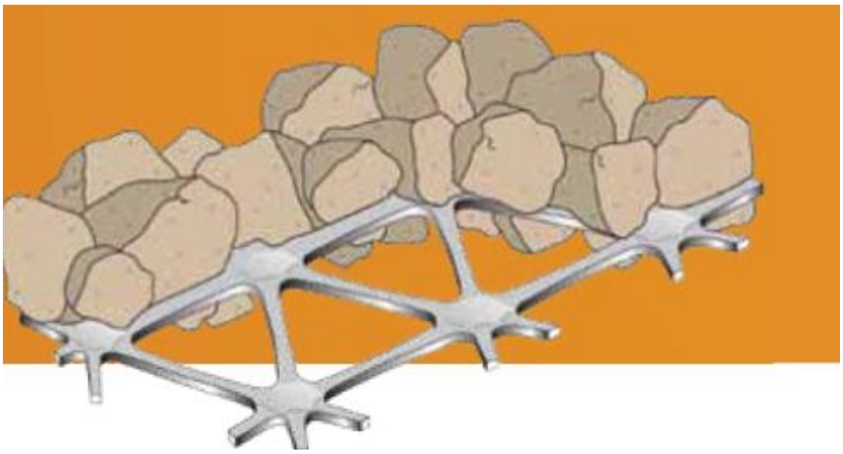

Figure 5: Interlocking mechanism

Initial Unsuccessful Full Scale Load Test

As no direct design, producers are available to date for design of MSL and usually empirical methods are used for design of replacement layer provided by MSL. In most of these methods, the backfilling layer shall be provided with multiple layer of MSL to distribute the loads by an angle of 1 horizontal to 1 vertical due to the interlocking and confinement with the backfilled soil as shown in figure 6 (Jenner 2007).

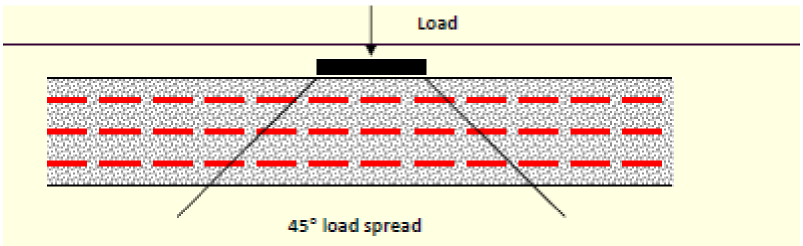

Figure 6: Stress distribution for backfill provided with MSL

The initial assessment had been carried out got the MSL system with a total thickness of 2.0 m and with three layers of type TX 160 (Triaxial geogrid).
In order to assess the safety and the feasibility of the proposed system it was decided to perform full scale load test (mockup area). The planed dimensions of the mockup area had been selected as 14*14 m, which would represent the actual construction activity. The test had been carried out with an actual tested area had been selected as 11*11m with less footprint from the loaded area (figures 7 and 8).

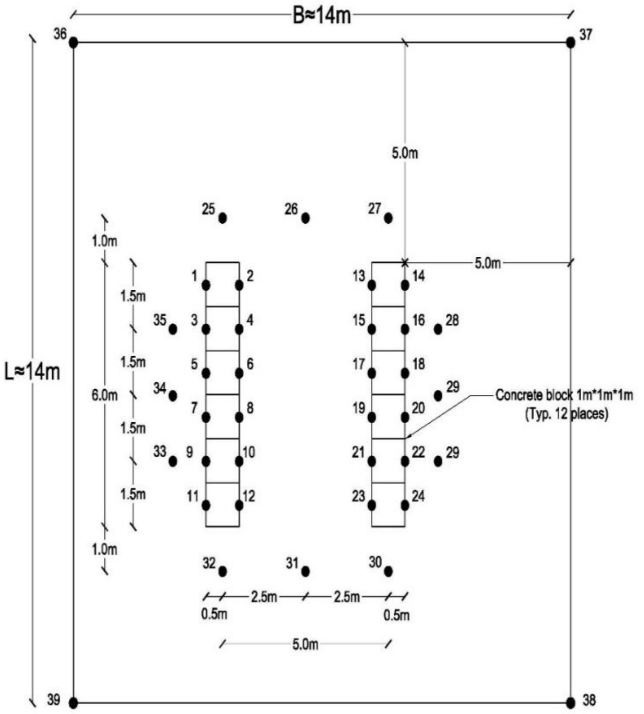

Figure 7: Planned dimensions for mockup area

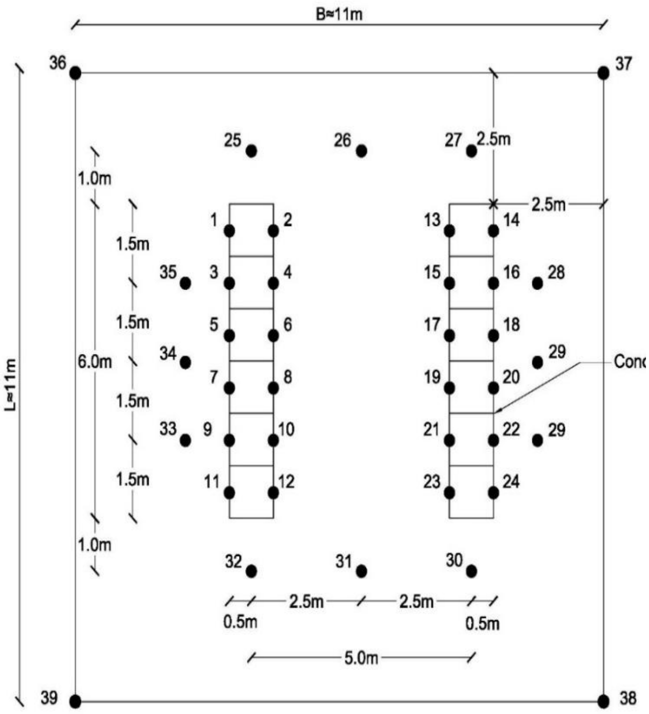

Figure 8: As built dimensions for mockup area

The planned total test stress below each row was about 300 kN/m^2 exerted from kentledge system. The kentledge system comprises of three (3) layers of blocks. The arrangement of these blocks shall be 64 blocks for the first layer, 49 blocks for the second layer and 36 blocks for the third layer as shown in figure 9.

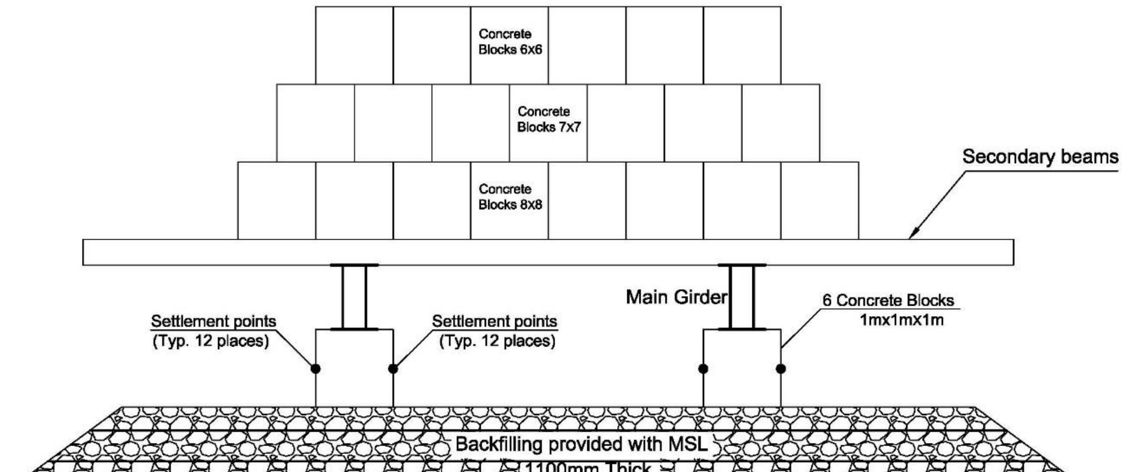

Figure 9: The arrangement of initial unsuccessful test

A monitoring system included the construction included elevation reference points (ERP'S) erected on the contact concrete blocks and improved platform with a total number of 39 points. Figure 10 shows the as built of the test and the monitoring points.

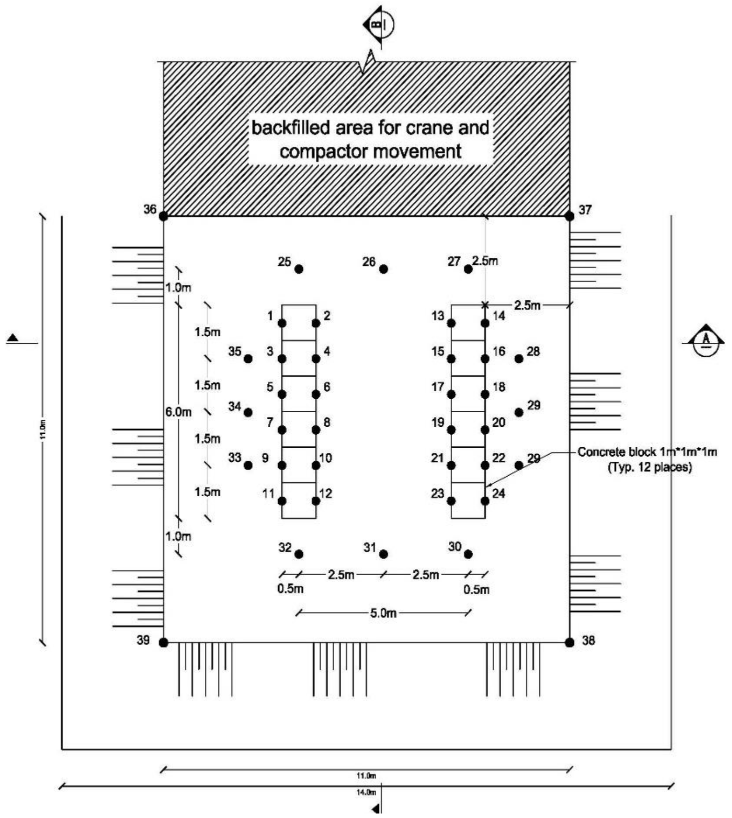

Figure 10: The as built of the test and the monitoring point for initial test

The readings of the tests had been used to plot relation between the time elapsed and settlement for selected points as shown in figure 11.

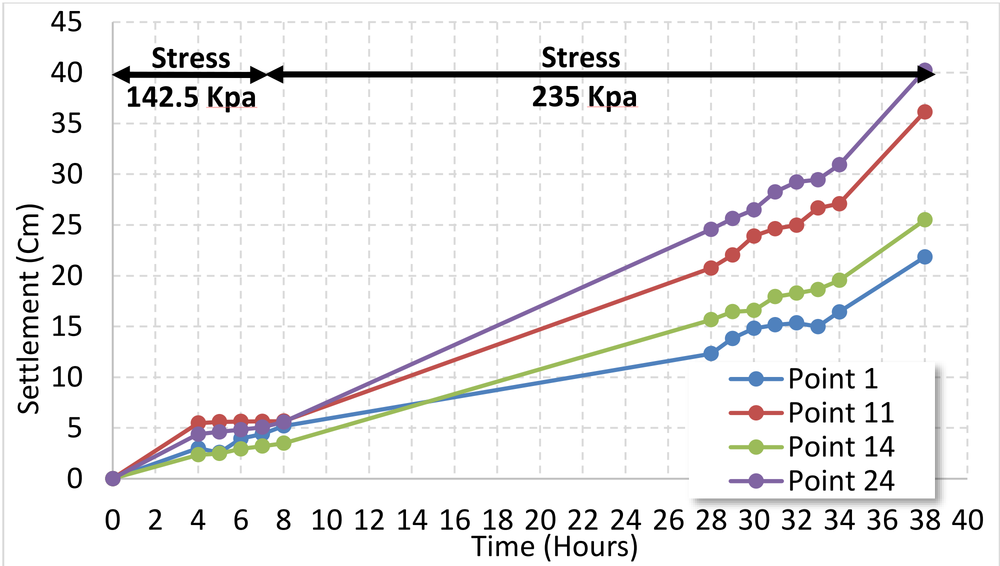

Figure 11: The relation between the time elapsed and settlement for initial test

From this figure, it can be noticed that sudden increase in settlement of all points occurred. In addition settlement of points 1, 14 are much less than that of 11 and 24. The two points 1 and 14 are on one side while the other two points are on the opposite side. This means that movement direction is as shown in figure 12. From the above, it is concluded that failure stress is in range of 230 kN/m^2, which is the maximum reached stress.

With respect to failure mode, it is concluded that the failure mode occurred is mainly slope failure (deep seated failure) rather than bearing capacity failure. This is in light of the following longitudinal crack observed between the two rows of the blocks (as per figure 12) and the direction of movement where minimum movement occurred for the area where backfilling behind it had taken place and the maximum settlement is for other area where no backfilling took place.

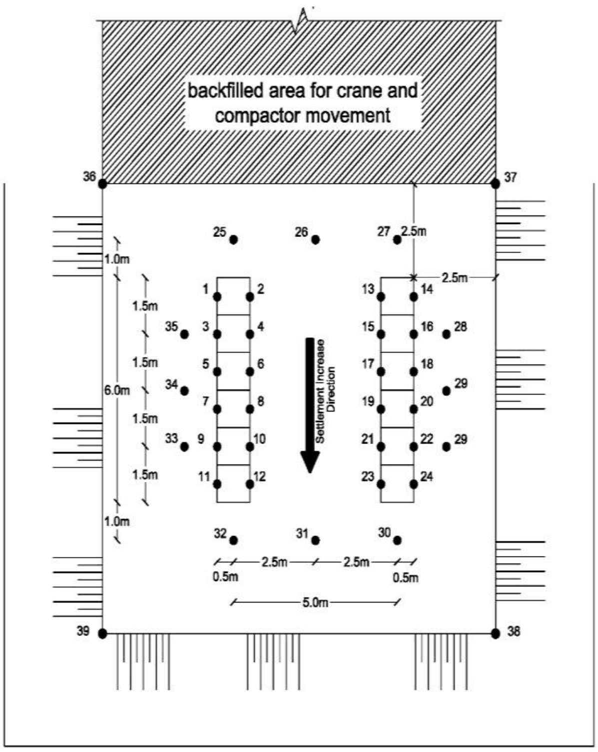

Figure 11: The settlement increase direction

Figure 12: Cracks noticed during test

In addition, during installation of backfilling layers and the mechanically stabilized layers (MSL), many observations had been recorded would be contributed to the failure. First, the gradation of the backfilling material did not satisfy the required design gradation included necessary for backfilling. Second, the grade of MSL layers used in the full-scale load test is not the same included in the initial. Third, the top relatively stiff crust assumed in the original design does not exist. Finally, the dimensions of the constructed area were less than the planned area. This results in changing the edge distance to 2.5 m instead of 5.0 m.

In order to assess the reasons of failure, a finite element model using Plaxis software had been implemented to represent section near the smallest edge distance. All soil layers had been modeled using MC model while geogrid had been modeled using Geogrid element (Plaxis reference manual). The undrained cohesion of the clay had been selected 20 kN/m^2 as per original design. The model had been used to represent actual applied stresses in order to investigate the actual failure mode. For such case an applied stress of 100 kN/m^2 had been implanted and second step involves the calculations of safety factor using C-ϕ reduction method. The failure stress concluded in this model slope failure is 225 kN/m^2 as per figure 13, which is close to the measured value.

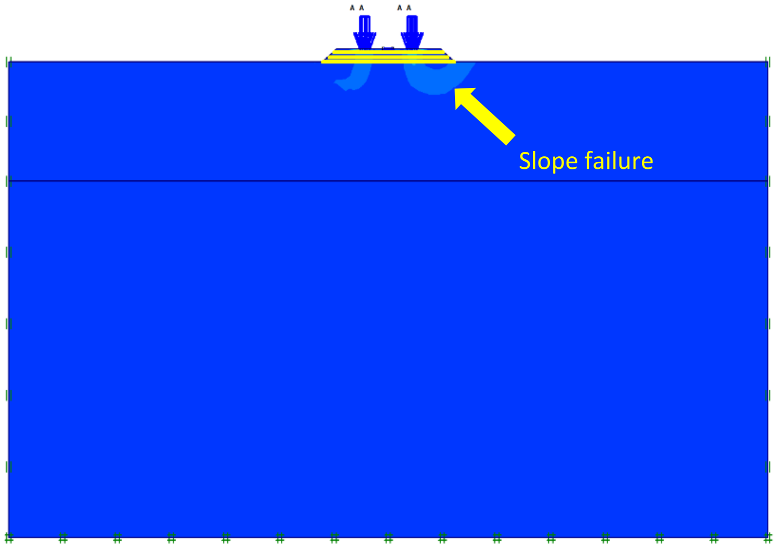

Figure 13: Slope failure in Plaxis model

Lessons Learned from the Unsuccessful Test

The lesson learned from the test can be summarized as follows:

1- The design must be based on neglecting of the top crust.

2- The materials used either geogrid or backfilling must be within the design and the specifications requirements.

3- The edge distance between the applied stresses and the improved area must be not less than 5.0 m.

4- All quality control tests must be performed on the backfilling and the degree of compaction must be not less than 95%.

Modified Soil Improvement Alternatives based on lessons learned

Based on the first unsuccessful load test and the lesson learned from this test, the soil improvement issue had been revisited and two alternatives had been proposed for final selection and evaluation.

-Alternative 1- Backfilling with 2.5m replacement soil. In this alternative controlled backfill shall place over the existing level.

-Alternative 2- 1.0m backfilling provided with MSL (mechanically stabilized layers) and 1.5m replacement soil (sand backfilling).

The configuration of the two alternatives and the therotical stress distbrbution are shown in figure 14 and 15.

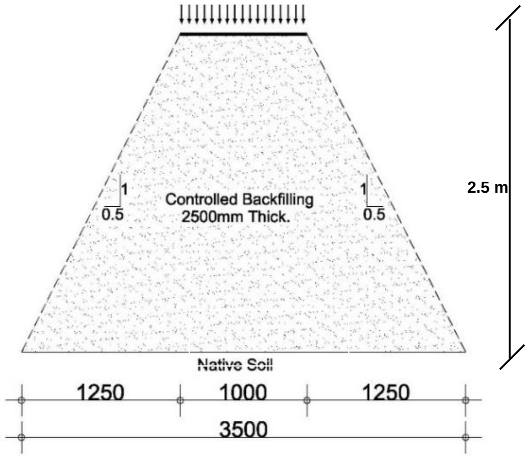

Figure 14: Stress distribution in case of using replacement backfilling (embankment)

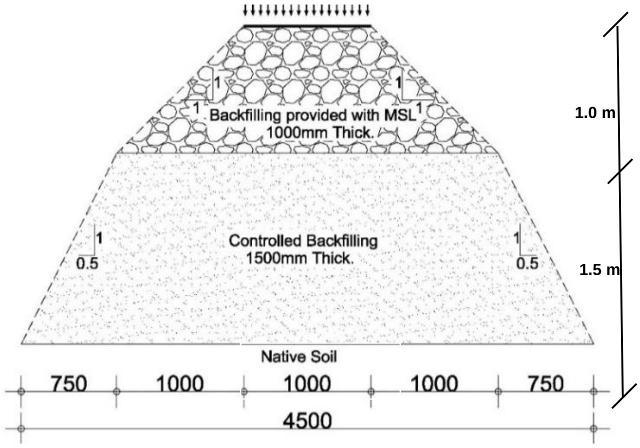

Figure 15: Stress distribution in case of using replacement backfilling with MSL

Arrangement of Full Scale Load Test on selected alternatives

The test arrangement was prepared to model the anticipated stress exerted from the D-wall equipment. The test included two (2) loaded rows of blocks with a total test stress below each row of about 400 kN/m^2 exerted from kentledge system. All quality control tests for replacement had been carried out and all dimensions and design data had been considered.

The kentledge system comprises of six (6) layers of blocks with total number of 199 blocks. The arrangement of these blocks shall be as per figure 16.

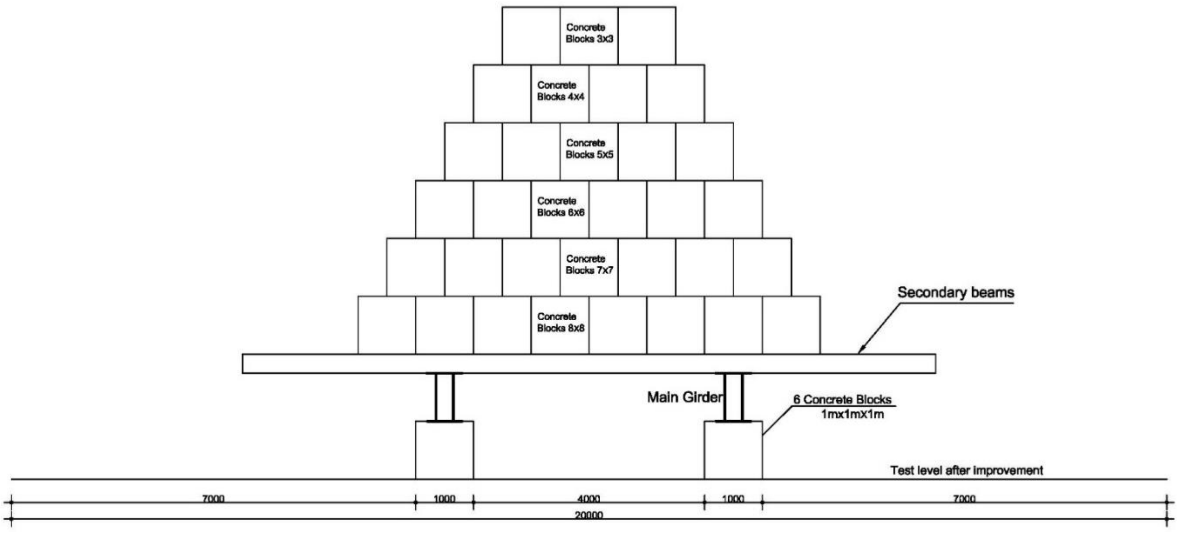

Figure 16: Kentledge system arrangement

The full-scale load tests on the proposed alternatives was tested to stress of about 400 kN/m^2, which represents 158% of the working stress (254 kN/m^2). The anticipated stress from each row of blocks considering the bock density of 22.5 kN/m^3 can be summarized as follows:

Stage	Blocks	Anticipated stress (kN/m^2)	Total percent % of test stress
Stage 1	Contact blocks (6 blocks)	22.500	5.70%
Stage 2	First layer (64 blocks)	142.500	36.00%
Stage 3	Second layer (49 blocks)	234.375	59.25%
Stage 4	Third layer (36 blocks)	301.875	76.30%
Stage 5	Fourth layer (25 blocks)	348.750	88.16%
Stage 6	Fifth layer (16 blocks)	378.750	95.70%
Stage 7	Sixth layer (9 blocks)	395.600	100%

The dimensions of the tested areas after placement of the improvement system was 20*20 m, which represents the actual construction activity. Figure 17 shows the planned layout of the mockup areas for the proposed tests while Figure 18 shows the location of the monitoring points.

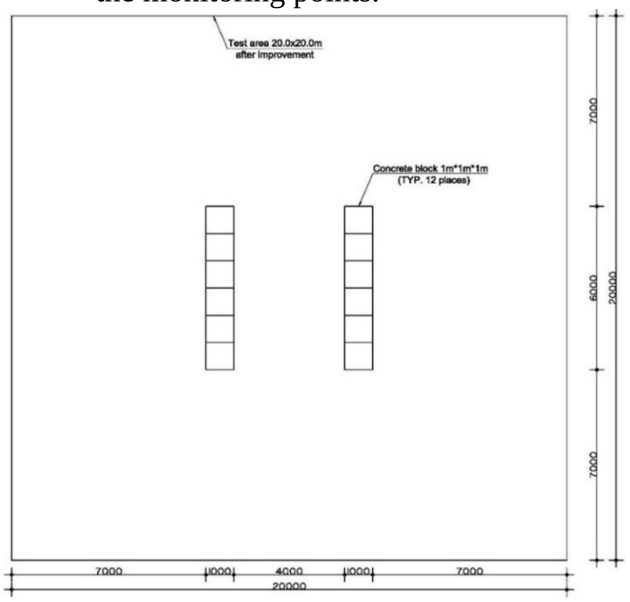

Figure 17: Plan for test area after improvement

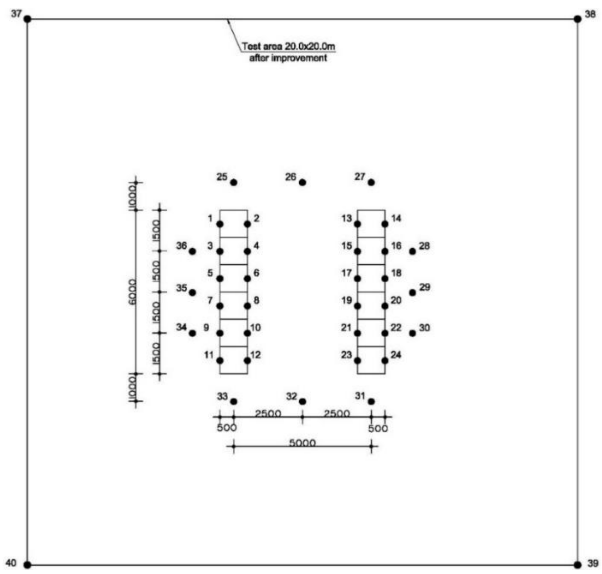

Figure 18: Location of the monitoring points

Results of the Full Scale Load Test

For test no (1) performed using controlled back filling with 2.5m replacement soil, after adding the fourth layers of blocks with an applied stress at this stage of about 348.75 kN/m^2 which represents about 88.16% of the planned test load (400 kN/m^2). It was noticed that excessive settlement occurred ranges between 316mm and 797mm with an average differential settlement between he contact blocks of about 340mm. In addition, the contact blocks was embedded in the improvement platform, which is a sign of bearing capacity failure. Based on the above, a decision of finishing the test took place. The stress-settlement curve for selected point of test no.1 is shown in figure 19

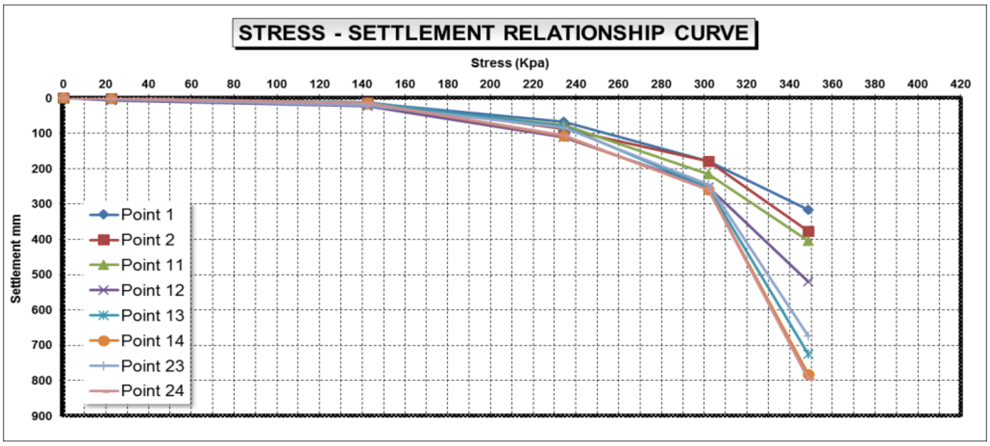

Figure 19: Stress-settlement curve for test No. 1 (2.5 m sand)

For test (2) performed using 1.0m backfilling provided with MSL (mechanically stabilized layers) above 1.5m replacement soil, all loading stages was performed without any signs of bearing capacity failure had been noticed. The stress-settlement curve for selected point of test no.2 is shown in figure 20.

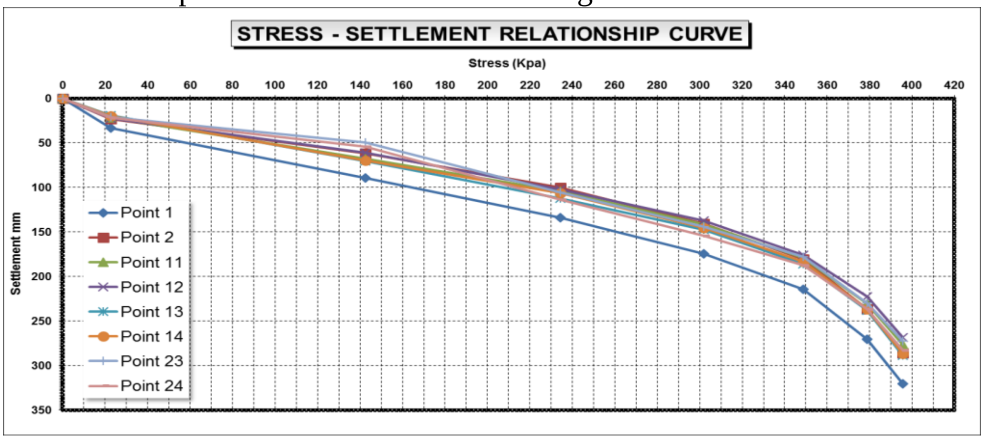

Figure 20: Stress-settlement curve for test no. 2 (1.5 m sand +1.0 m MSL)

In order to have better understanding for the difference in behavior between the results of the two tests, the representative stress-settlement curve for the two tests are plotted on the same graph as shown in figure 21.

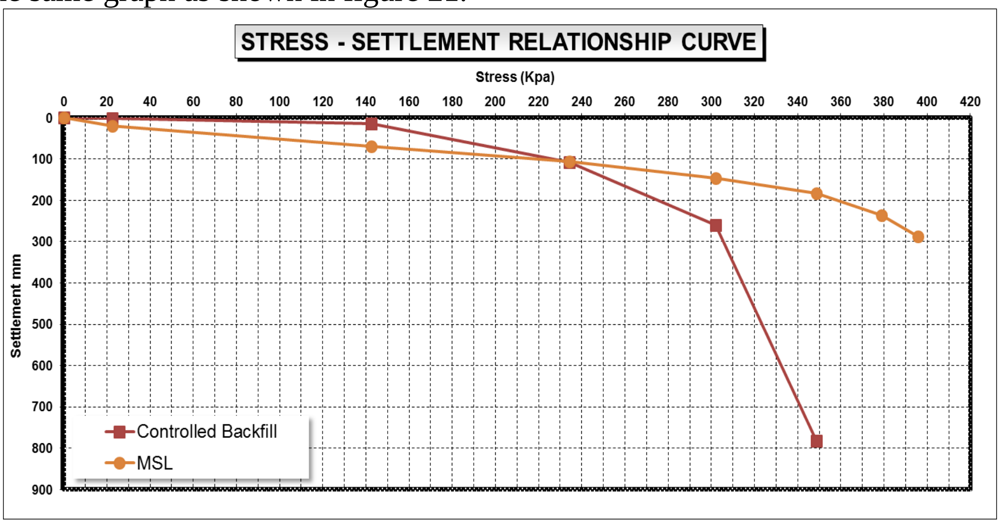

Figure 21: Representative Stress-settlement curve for test no. 1 and 2

From figures 19 to 21, the following can concluded:

1- The test no. 1 shows signs of bearing capacity failure while test no.2 did not show signs of bearing capacity failure.
2- The settlement of test no.1 is non-uniform along the tested area especially at higher stresses (stress of about 340 kN/m^2). The difference between the maximum and minimum settlement is about 500 mm.
3- The settlement of test no.2 the settlement is almost uniform along the tested area for lower and higher stresses up to stress of 400 kN/m^2. Except point 1, all points have the same settlement. Even considering point 1, the difference between the maximum and minimum settlement is about 50 mm.
4- The use of MSL caused considerable improvement in settlement values and settlement uniformity especially for higher stresses.

The ultimate bearing capacity for the two tests was determined by the two different methods, which are slope tangent method (Uneo 1998) and log-log plot (Egyptian code for soil Mechanics part 4). Selected settlement trend line was used to determine the ultimate bearing capacity. The results of the two tests using the two proposed methods are shown in figures 22 and 23 .These figures show that for test no. 1, the ultimate bearing capacity is ranging between 230 and 290 kN/m^2 , while for test no.2 using MSL, the ultimate bearing capacity is ranging between 370 and 380 kN/m^2

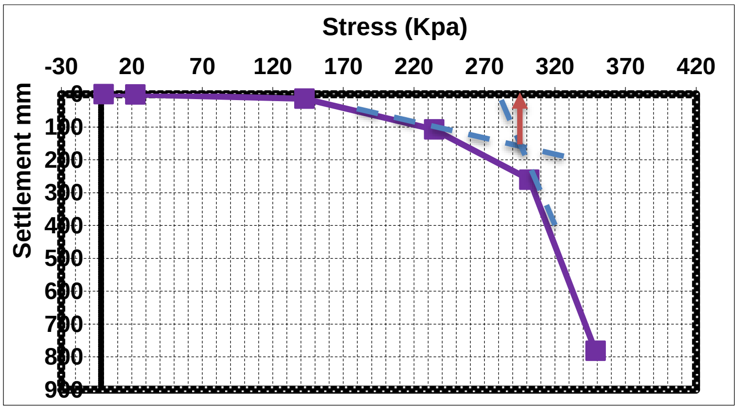

Figure 22-a: Ultimate bearing capacity for test 1 considering slope tangent method

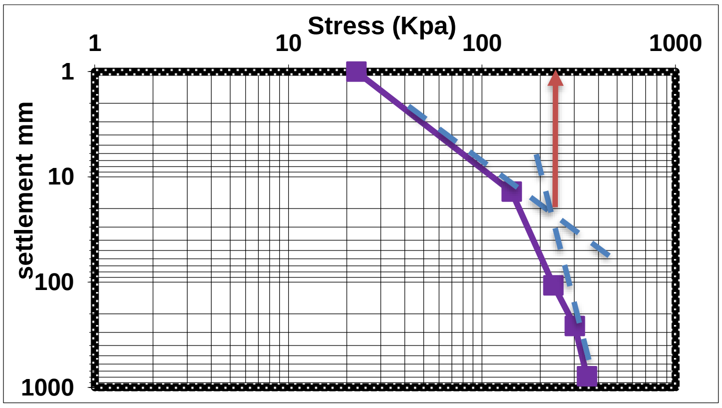

Figure 22-b: Ultimate bearing capacity for test 1 considering Log-log method

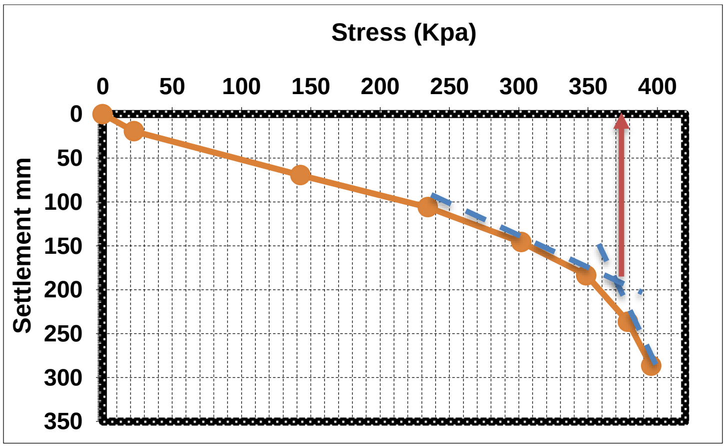

Figure 23-a: Ultimate bearing capacity for test 2 considering slope tangent method

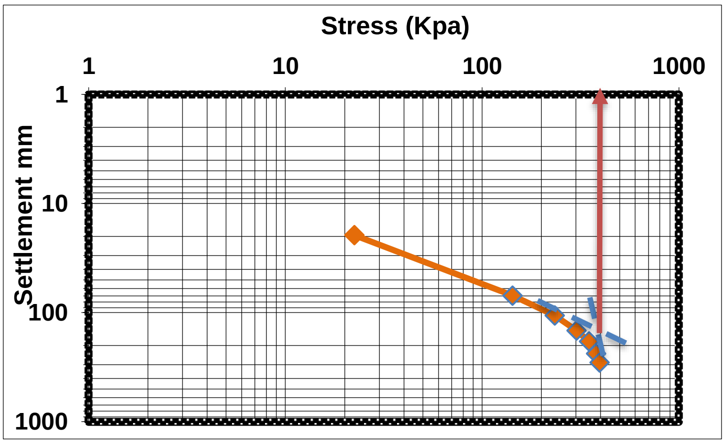

Figure 23-b: Ultimate bearing capacity for test 2 considering Log-log method

Based on the above test results, the design final section had been selected as shown in figure 24. The construction had took place and finished successfully

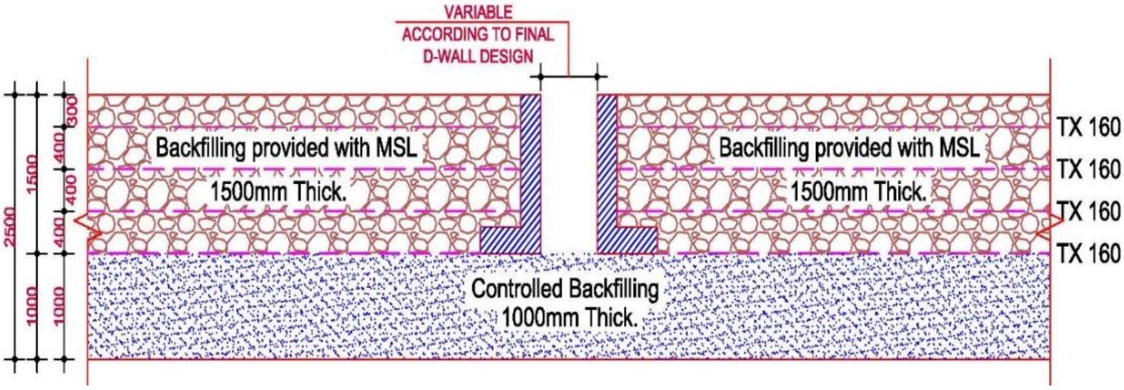

Figure 24: Final selected section for soil improvement

Conclusions

The conclusions and findings of the conducted data and analysis included in this research can be summarized in the following points:

- The use of full scale load tests for MSL system designed for heavy loaded platforms and embankments is essential to judge both the empirical design methods and also the construction methodology
- The MSL is considered a very effective way of soil improvement for heavy loaded platforms founded on deep deposits of soft clays in terms of improving the bearing capacity compared to normal controlled backfilling.
- The use of 1.0 m of MSL over topping 1.5 m of controlled backfilling showed an ultimate bearing capacity of 370 kN/m2 compared to an ultimate value of 230 kN/m2 with the use of 2.5 m of controlled backfilling
- . The use of MSL resulted in a considerable uniformity of the settlement along the loaded areas especially at high stresses.
- The interlocking and confinement of the MSL with the backfilled soil showed very clear improvement results.
- The empirical methods used for design of replacement layer provided by MSL and distributes the loads by an angle of 1 horizontal to 1 vertical is considered a good approach for estimations.
- The quality control and the edge distance play a very important rule in the behavior of mechanically stabilized layers.
- Although top crust appears in Port Said area shows higher strength compared to underlying soft soil, the strength of this layer would be neglected in design of platforms and road embankments.
- The soil improvement would be a good and cost effectiveness solution for heavy platforms and road embankments founded on soft soil deposits in Egypt like Port Said and Suez Canal area.

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