Cavity remediation for pylon foundation of the Transrhumel Viaduct in Constantine

Résolution des problèmes de cavité sous les fondations du Viaduc Trans-Rhumel de Constantine

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ABSTRACT: The geology of Constantine, Algeria is highly influenced by previous seismic activities and the erosional feature by the river Rhumel. The Transrhumel Viaduct is a new river crossing featuring a 749 m long cable stayed bridge with a main span of 259 m and 80 m clearance. The soil/rock deposits are dominated by Marls of different strength underlain by Marlstone and very competent Limestone. However, exploratory boreholes for the foundation piles of Pylon P4 indicated the presence of possible cavities of up to 1 m high within the Limestone at the base of the upper weathered zone. To save 15 m of time consuming drilling of the 14 Ø2 m foundation piles into competent Limestone it was decided to end the foundation piles above the possible cavity feature in the Marlstone. The paper describes the innovative remedial measures carried out to safeguard the capacity of the individual foundation piles and ensure acceptable individual and overall displacements. The cavity feature was pressure grouted and transfer of axial load across the cavity into Limestone was facilitated by insertion of grouted steel reinforcement assemblies. The success of the remedial measures was proven by carrying out an O-cell load test on the pile positioned over the maximum recorded depth of the cavity.

RÉSUMÉ : Les conditions géologiques de la ville de Constantine en Algérie sont pour une grande part le résultat d'évènements sismiques passés et de l'érosion de l'oued Rhumel. Le viaduc Trans-Rhumel est un nouvel ouvrage qui traverse le court d'eau comprenant un pont à hauban de 749 m avec une travée principale de 259 m placée à une hauteur de 80 m. Les sols de fondation sont majoritairement composés de marne de compétence variable supportée par des roches marno-calcaire et roches calcaires très compétentes. Les sondages par forages effectués au droit du pylône P4 ont cependant révélé la présence de cavités d'une hauteur jusqu'à 1 m dans le calcaire à la base de la zone érodée supérieure. Afin d'économiser 15 m de forage onéreux pour les 14 pieux de 2 m de diamètre dans la roche calcaire, il a été décidé de stopper les pieux de fondation au-dessus de la zone de présence probable des cavités des roches marneuses. Cet article décrit les mesure innovantes mises en place pour assurer la capacité portante de chaque pieu de fondation et garantir des déplacements globaux et individuels acceptables. Un coulis de ciment a été injecté dans la cavité et le transfert des efforts axiaux au travers de la cavité fut assuré par l'insertion d'assemblages d'armatures placées dans le ciment. Le succès de cette méthode a été démontré en exécutant des essais de charges O-cell sur le pieu fiché à la hauteur maximale de la cavité.

KEYWORDS: natural hazards, cavity, bored piles, pile load test, pressure grouting, Marls, Limestone.

1 BRIEF PROJECT DESCRIPTION

The Constantine Viaduct (Figure 1) is a new bridge, crossing the River Rhumel in Constantine, Algeria. Constantine is situated on a plateau at 640 metres above sea level. The city is framed by a deep ravine and has a dramatic appearance with a number of bridges and a viaduct crossing the ravine.

Figure 1. Artist's impression of completed cable stayed bridge (Pylon P4 to the left).

The main bridge features a cable-stayed bridge as well as access and ramp bridges. The project includes very extensive road works of approximately 10 km, with 13 over and underpasses, 1 km of up to 45 m deep steep cuts stabilised with ground anchors and soil nails, 4 km retaining walls of which 1 km embedded retaining walls were stabilised with ground anchors (up to 32 m retained height).

The total length of the bridge between the main abutments is 749 m with an 80 m clearance over the river bed.

The Owner is Direction des Travaux Publics de la Wilaya de Constantine – DTP and the contractor is Andrade Gutierrez - AG from Brazil with COWI A/S Denmark as designer.

2 GEOLOGY WITH FOCUS ON PYLON P4

The geology of the site is highly influenced by previous seismic activities (medium severity) and the erosional feature by the River Rhumel.

The soil/rock deposits are dominated by Marls of different strength underlain by Marlstone and very competent Limestone. At the original position of Pylon P4 a very dramatic ancient fault was discovered where the Limestone surface dips near vertically and hence the Pylon was moved 14 m to the north, thus enabling the foundation to be completely based on bored piles with the toe well into the Limestone formation (Figure 2).

The borehole B4/03 at the location of Pile P4/3 indicates a soil profile as summarized in Table 1.

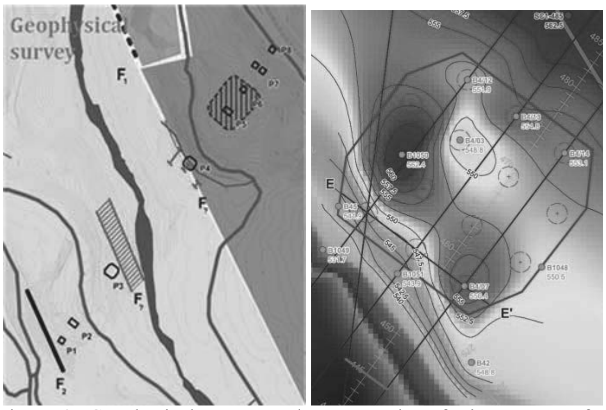

Figure 2. Geophysical survey and contour plot of Limestone surface based on boreholes (F_1 denotes the main fault at P4 location)

Table 1. Soil profile at borehole B4/03 (at pile 4/3) and assumed lower bound characteristic soil parameters (τ = 0.37 c_u assumed for design in soils and formula (4.25) from Fleming et al. 2009 in rock).

Soil/rock layer	Top level [1)] (m)	Undrained shear strength c_u (kPa)	Shaft friction τ (kPa)	Toe bearing capacity (MPa)
Fill from ground	+586.6	61	23	
Marl 1	+579.9	141	52	
Marl 2	+570.4	335	124	
Marlstone	+565.3	6750	1520	11
Pile toe	+559.5			
Weathered Limestone	+557.8	3500		
Limestone (intact)	+548.3	9000		

[1)] Datum in Fuse 32

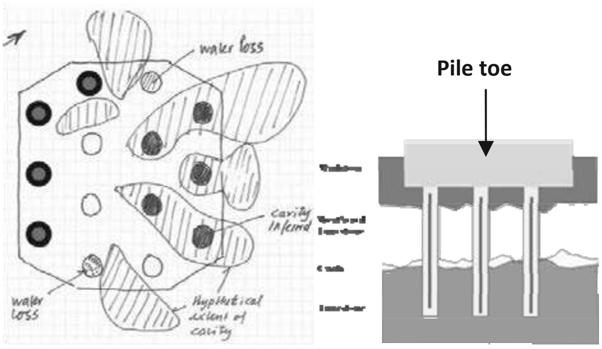

Figure 3. (a) Hypothetical extent of cavity feature (piles with bold outline were drilled into Limestone); (b) Schematic cross section at pile toe in Marlstone with pre-drilling holes below pile toe.

The unconfined compressive strength in the weathered Limestone (5 to 9 m layer between Marlstone and top of intact Limestone) had to be re-assessed from dried out samples and hence the average value of σ_c = 13.1 MPa (7 MPa as 95% lower fractile) was deemed conservative.

The strength of the Marlstone was σ_c = 13.5 whereas σ_c >> 18 MPa for the Limestone, i.e. above the strength of the concrete.

The toe bearing for the Marlstone was conservatively calculated as for a soil (11 MPa) as opposed to 45 MPa if rock parameters were used.

However, the drilling subcontractor TREVI experienced excessive time delays when drilling the 2 m diameter bored piles, 42 to 46 m long, into the Limestone, even after executing five Ø178 mm pre-drilling holes to the pile toe level within the foot print of each pile.

Moreover, the exploratory boreholes indicated the presence of cavities within the Limestone, at the base of the upper weathered zone and with heights up to 1 m.

The indicators were severe water loss and/or a sudden drop of the drill string.

Based on the observations the extent of the cavity feature was hypothesized as shown in Figure 3.

3 REMEDIATION AND MONITORING STRATEGY

The existence of the five Ø178 mm pre-drilling holes to the level of intended pile toe in Limestone turned out to be both a blessing and a curse. By re-opening the holes it would be possible to insert a "reinforcement" to transfer the load across the cavity feature but the holes at the same time prevented pressure grouting of the cavity as un-grouted holes would function as venting holes.

The solution adopted consisted in inserting a 12 m long steel reinforcing assembly composed of 6 Nos. of T40 mm reinforcement bars tack welded together around a central spacer ring into each pre-drilled hole. This solution was chosen in order not to cause delays by delivery to Africa of the preferred Ø90 mm GEWI piles and to utilize on site material. The reinforcement provided by the T-bars corresponds by and large to the reinforcement in the lower part of the Ø2 m bored pile.

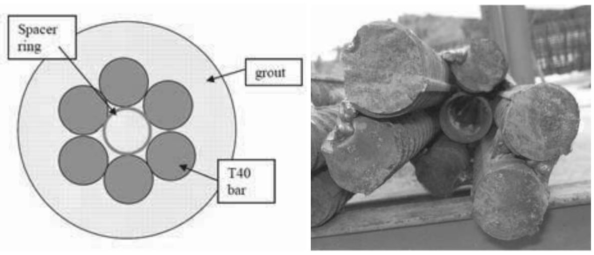

Figure 4. (a) Cross section through pre-drilling hole with assembly of T-bars; (b) Photo of T-bar assembly with spacer ring

The rationale of the solution is to transfer the axial load from the pile toe in Marlstone through the weathered Limestone and the cavity feature into the competent Limestone without risking excessive (differential) vertical displacement of the piles. Grout around the reinforcing assembly ensures stability of the reinforcement throughout the 12 m length. The reason for choosing this solution was that the pre-drilled holes were already available and it was not considered feasible to pressure grout the very localised weathered zones in the Limestone when grouting from just a few boreholes. The different degree of weathering across the foundation footprint could cause different settlement behaviour for individual piles, especially as the weathered zone would be directly below the pile toe where the loads are most concentrated.

The sequence of the reinforcement installation is: (i) re-open a pre-drilling hole, (ii) install 12 m long reinforcement assembly, with tremie pipe through the centre, so the top level is approximately 0.5 m below pile toe level, (iii) grout to the pile toe level allowing grout to permeate into potential cavities around the hole.

Grout of relatively high viscosity (Marsh viscosity 35- 50 sec.) was used. It consisted of Portland cement 42.5 with w/c ratio of 0.45 and thinner adjuvant type Rheobuilt with w/a ratio of 0.5 - 1.0%. Settling at 4 h < 5%. It would be expected that the grout could travel some 3-5 m away from the pre-drilling hole. However, as the cavity (cavities) may have a considerable horizontal extent also outside the foundation footprint as visualized in Figure 3a it was considered a must to also perform pressure grouting (of cavities) to ensure the design capacity of the pile group and to safeguard against excessive or differential overall displacements.

The grouting scheme devised consisted of primary and secondary grouting holes in staggered sequence as shown schematically in Figure 5. To maximize the effect the sequence of grouting is staggered.

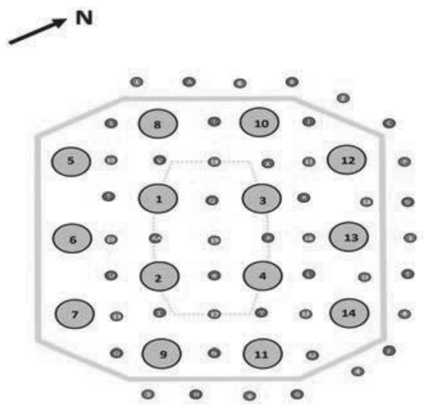

Figure 5. Schematic grouting scheme with primary grout holes in Arabic numbers and secondary grout holes in letters (pile numbers of the 14 Ø2 m piles shown for reference; piles 5, 6, 7 and 8 were drilled into the competent limestone prior to the remediation scheme).

The holes for pressure grouting (40 m deep) were drilled "destructively" from the ground surface using a Ø141 mm drill in the Marl (with casing) and a Ø105 to 115 mm drill to the full depth (without casing). The grout take was carefully measured using the same type of grout as before, but now with a settling <5% after 2 hours. The uncased part passing through the cavity feature was pressure grouted using packers with 12 bar pressure. The cased part was gravity grouted as the casing was withdrawn.

4 MONITORING OF REMEDIATION MEASURES

Based on the surface texture of the holes drilled for the piles (very uneven surface) the actual consumption would be expected to be higher than the theoretical consumption based on the bore and grout hole diameters. The consumption of concrete when casting the Ø2000 mm piles was:

- Piles P4/5 to P4/8 (46 to 50 m length): 113% ± 3.7%
- Remaining 10 piles (22 to 28 m length): 108% ± 1.8%

i.e. roughly 10% excess consumption.

As the Ø178 mm holes were re-drilled it seems likely that actual nominal consumption would be 115% of the theoretical consumption. Using this as baseline the excess grout take from the five reinforcement-holes below each of the ten Ø2000 mm piles are shown in Figure 6. In some cases re-grouting of the holes took place and hence a sequence number in excess of five occurs for some of the piles.

Although there is considerable scatter the excess grout take decreases by and large as a function of the sequence as would be expected as any cavity feature will be more readily filled during the initial grouting. Piles 3, 9, 11, 12 and 14 show grout take above average which is interpreted as a more persistent cavity feature at these locations.

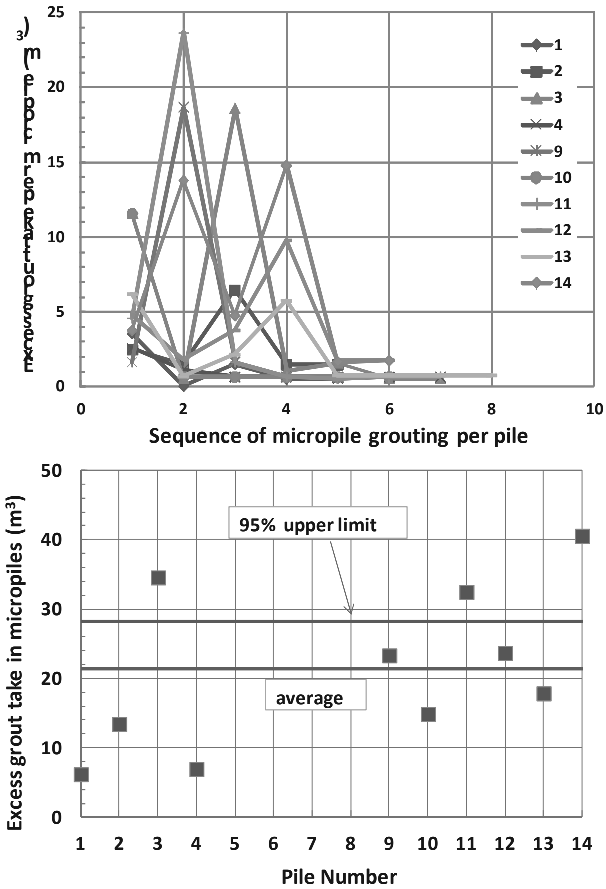

Figure 6. Excess grout take in "micropile" holes; (a) as function of sequence (2-6 weeks from grouting of fist to last pile); (b) accumulated values per pile location

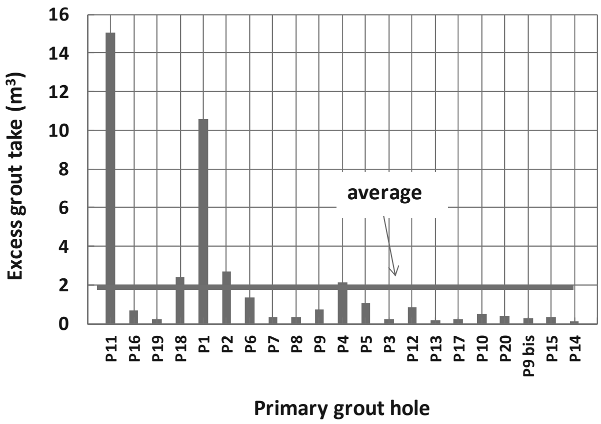

Figure 7. Excess grout take versus sequence for primary grout holes

The actual sequence of the subsequent pressure grouting in primary (Figure 7) and secondary (Figure 8) deviated slightly from the sequence in Figure 5 but followed the principal intent. The theoretical grout take was based on the nominal drilling diameters but with reference to the considerations for the piles and pre-drilling holes this may entail some 15% underestimation of take.

As seen in Figure 7 and Figure 8 the excess grout take was very limited in the majority of holes.

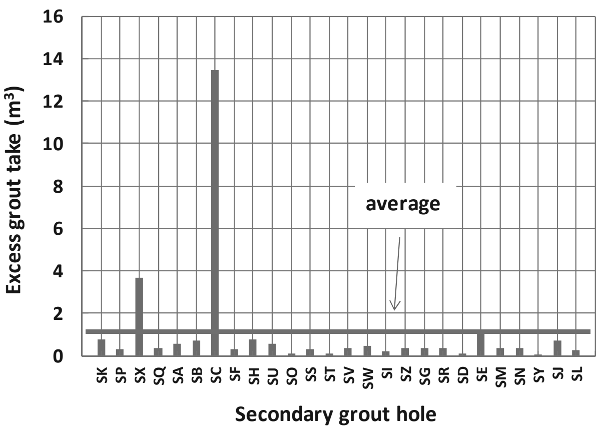

Figure 8. Excess grout take versus sequence for secondary grout holes

The excessive grout takes over the average value are by and large concentrated in areas where a cavity feature had already been indicated by the "micropile" grouting. Furthermore, it is clear that the grout take during primary grouting (1.9 $m^3 \pm 3.8$ m^3) compared to the secondary grouting (1.1 $m^3 \pm 2.6$ m^3) is an indirect confirmation of the success of the grouting scheme.

Excluding the seven values exceeding the average value the excess grout take is 0.6 $m^3 \pm 0.5$ m^3 for primary and 0.4 $m^3 \pm$ 0.3 m^3 for secondary grout holes compared to the minimum theoretical grout take of 0.5 m^3.

The tentative evaluation in terms of cavity feature extent, based on the combined result in terms of excess grout take, is shown in Figure 9. The Figure lends credibility to the conclusion that the horizontal extent of the cavity feature was relatively modest and close to the hypothetical extent envisaged (Figure 3).

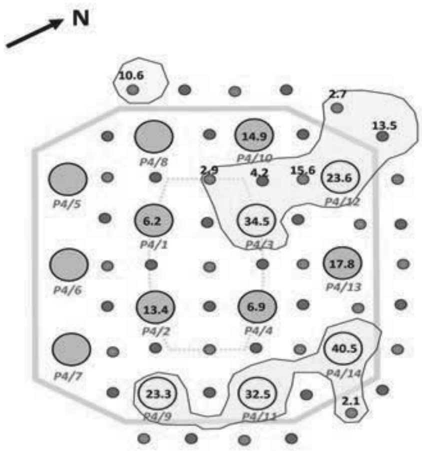

Figure 9. Tentative evaluation of the extent (shading) of the cavity feature below Pylon P4.

5 VERIFICATION BY PILE LOAD TESTING

To verify the capacity of the piles with pile toe in Marlstone the most onerous pile in relation to the cavity, P4/3, was tested by means of an O-cell load test carried out by Fugro LOADTEST.

The principle of the test is shown in Figure 10. In this case two Ø870 mm O-cells were placed 1 m above the intended toe level of the pile.

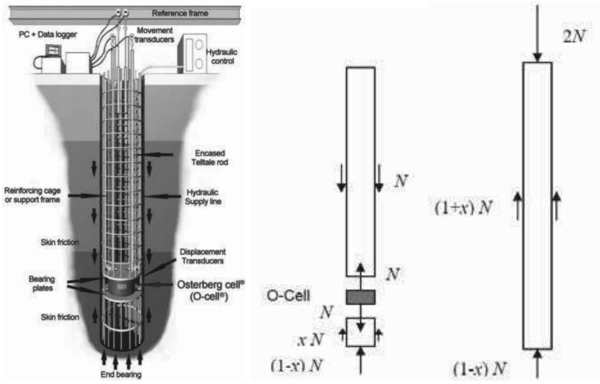

Figure 10. O-cell load test for a bored pile. (a) schematic set-up; (b) load distribution in O-cell test; (c) load distribution in top down loading

The 1 m pile length below the O-cells safeguards against tilting of the cells, due to uneven strength/stiffness distribution at the toe of the pile. The pile length above the O-cell is 26.1 m.

Note that when loading top down (the design load situation) it takes the double axial load to mobilise the same shaft and toe bearing as with the O-cell test and hence a greater elastic pile compression will occur.

For the test the pile was fitted with strain gauges, at six levels in the Marlstone, and tell-tales at four levels including the toe in order to be able to get a detailed understanding of the stress distribution along the shaft. Unfortunately, these measurements were inconsistent and deemed unreliable except for tell-tales 1 (level +560.5), 2 (level +565.3) and 3 (level +571.0). These measurements were combined with reliable values of the upwards and downwards displacement of the O-cell and the bi-directional O-cell load with a maximum load of 40.1 MN in each direction.

The capacity of the O-cell was higher, but in order to safeguard against any detrimental effect from the testing on the working pile the Owner decided to limit the load on the pile to 1.5 SLS.

The load test included three step-wise loading unloading cycles:

(i) to a level corresponding to the maximum SLS load in the pile group i.e. 26.5 MN

(ii) to 1.5 SLS ~ 40.1 MN (which exceeds the highest ultimate state load for earth quake of ULS-EQ = 32.4 MN for Pile P4/3) and

(iii) to SLS = 26.5 MN.

Creep was observed for up to 120 minutes for the designated loading steps and during shorter periods (typically 10 minutes) for the unloading steps.

The downward displacement of the bottom plate of the O-cells as a function of the gross downward O-cell load is shown in Figure 11.

A load of some 3 to 4 MN is required before displacements are initiated. This corresponds reasonably well to the tension capacity of the concrete section less the O-cell area (1.95 m^2) assuming $\sigma_t \sim 0.06\ \sigma_c = 0.06 \times 35.5 = 2.1$ MPa and thus a breaking load of $1.95 \times 2.1 = 4.1$ MN.

This is slightly higher than the load required for breaking the tack welds initially holding the O-cells closed (reported as 2.94 MN).

From Figure 11 it is apparent that the load-displacement curve is almost linear until the maximum load of 40.1 MN applied.

The creep rate increases with load level with a maximum of 2.0 mm/log cycle of time for the maximum load.

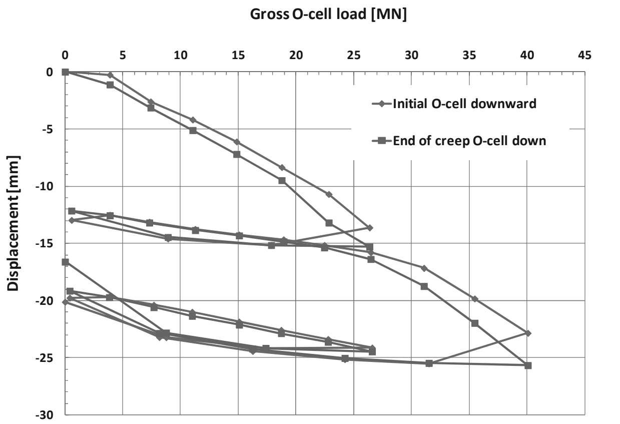

Figure 11. Displacement of lower O-cell plate versus applied downward gross O-cell load

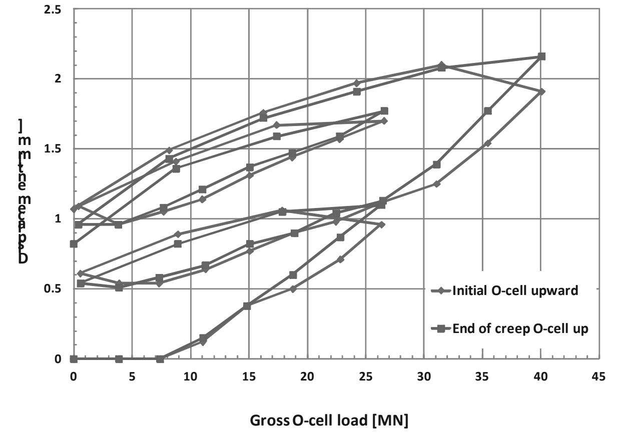

Figure 12. Displacement of upper O-cell plate versus applied upward gross O-cell load

Considering, that the major part of the load below the O-cells is taken by toe bearing the creep rate is acceptably low. The corresponding creep rates for the top plate of the O-cells are an order of magnitude smaller and hence insignificant.

The load-displacement curve for the upward O-cell displacement is shown in Figure 12. The upwards displacements are significantly less than the downward displacements (Figure 11) and they are only initiated at an O-cell load of 7 MN after which the load-displacement curve is essentially linear. It is apparent that shaft stresses are locked into the system at unloading resulting in a non-recoverable displacement of approximately 1 mm after unloading from the gross O-cell load of 40.1 MN (net load 38.8 MN after subtraction of pile weight).

From the readings of tell tales 2 and 3 it is apparent that the displacements decrease very significantly with the distance from the O-cells and that initiation of displacements requires significantly greater O-cell loads than needed to initiate downward displacement.

Tell tales 2 are at the transition from Marlstone to Marl 2 and hence the difference between tell tales 1 (top of O-cells) and 2 indicate the "rock socket" shortening in the Marlstone as shown in Figure 13.

This deformation corresponds to the accumulated displacement between pile and rock for developing the shaft resistance. As seen from the Figure the displacement is very small and almost mirrors the upward displacement of the O-cell upper plate (Figure 11). It means that the displacement between pile and rock at the top of the Marlstone is only 0.6 mm and thus the pile capacity is very significantly higher.

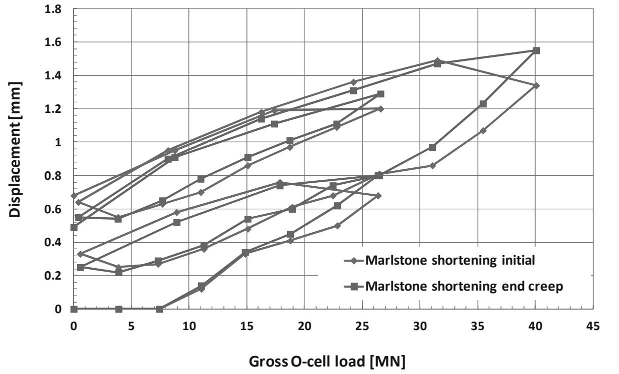

Figure 13. Shortening of the pile ("rock socket") in Marlstone between tell tales 1 and 2 versus applied O-cell gross load.

The recorded working curves for the upper pile shaft and the lower pile segment may be fitted by "linear fractional" (hyperbolic) functions, i.e. $y = (ax + b)/(cx + d)$. This facilitates extrapolation of the working curves and production of synthesized top load settlements curves.

The methodology by Fleming (1992) was used to produce the predicted top load settlement curve shown in Figure 14.

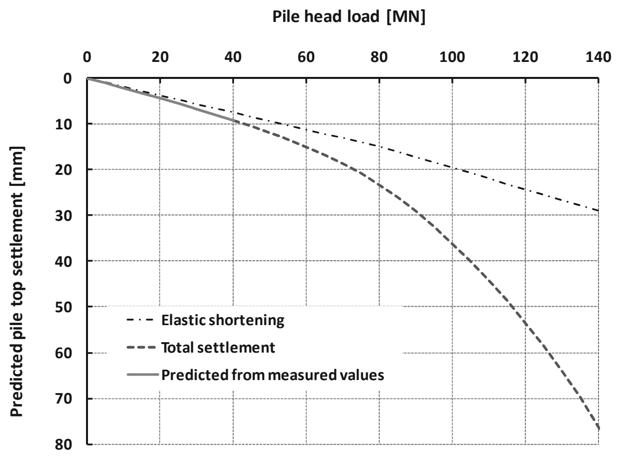

Figure 14. Predicted top load settlement curve for Pile P4/3

It is apparent from Figure 14 that a top loading of 26.5 MN (corresponding to the maximum SLS load in the P4 Pylon pile group) will cause a pile head settlement in the order of 6 mm. Twice the load, 53 MN (twice the SLS load and larger than the maximum ULS load of 45.5 MN in the pile group) will result in only 13 mm settlement.

Thus, even for the ultimate limit state (ULS-EQ) loading of the actual pile of 29.6 MN the predicted settlements are only slightly larger than the elastic part of the working curve.

6 EVALUATION OF ROCK SOCKET CAPACITY

The maximum toe and shaft design loads for pile P4/3 are 4.5 to 4.9 MN and 21.0 to 26.5 MN, respectively. The range reflects upper and lower bound soil stratification and soil spring assumptions. As seen from Figure 11 and Figure 12 the load test indicates much higher values even within near elastic load-displacement behaviour.

For the toe bearing of the rock socket the mobilisation of the resistance may be assumed to vary roughly proportional to the square root of the mobilised displacement, where $\delta_{ult,t}$ (toe bearing) ~ $0.1 \cdot D = 200$ mm:

$$\frac{\sigma_{mob}}{\sigma_{ult,t}} = \sqrt{\frac{\delta_{mob}}{\delta_{ult,t}}} \tag{1}$$

For the development of shaft resistance the following expression may be tentatively used (Steenfelt & Abild, 2011):

$$\frac{\tau_{mob}}{\tau_{ult,s}} = \sqrt{\frac{4\,\delta_{mob}}{3\,\delta_{mob} + \delta_{ult,s}}} \tag{2}$$

where $\delta_{ult,s}$ (shaft resistance) is of the order 3-10 mm.

Based on the lower bound characteristic ultimate shaft friction resistances indicated in Table 1 and δ_{ult} = 5.5 mm a total shaft resistance above the O-cell for the 26.1 m pile of 38.8 MN is found based on the mobilisation ratios inferred from Eq. (2). Average displacements in Marlstone, Marl 2, Marl 1 and fill, based on the tell tale measurements, have been used in this assessment. The corresponding ultimate shaft resistance at 100% mobilisation is 54.2 MN.

For the toe resistance and δ_{mob} =25.7 mm full development of 9.5 MN shaft resistance is assumed on the lower 1 m pile ($\delta_{mob} > \delta_{ult}$). This means that the mobilised toe resistance is 30.6 MN and by application of Eq. (1) an ultimate toe resistance of 85.5 MN may be deduced.

Thus, based on conservative estimates for the characteristic shear strength of the layers involved an ultimate capacity of the P4/3 pile of almost 140 MN is inferred. This would be close to the characteristic structural capacity of the pile as it corresponds to a maximum characteristic stress of 44.6 MPa at the pile toe.

Considering that conservative parameters have been applied it is concluded that the pile has more than sufficient capacity and that there is no reduction of capacity from the presence of the cavity feature or the weathered Limestone below the toe of the pile

7 SUMMARY & CONCLUSION

The recorded cavity feature below the foundation for Pylon P4 necessitated remedial measures. At the same time the Contractor preferred pile toes at a higher level, above the cavity feature, in order to reduce construction time. These issues were addressed by:

- the installation and grouting of five 12 m long reinforcing elements ("micro piles") from 0.14 to 0.78 m distance below the pile toe level and into the intact Limestone
- pressure grouting of 20 primary and 26 secondary grout holes over and slightly beyond the foot print of the P4 Pylon
- load testing of Pile P4/3 situated at the most onerous position over the recognised cavity feature

It was concluded that the cavity feature was not a consistent feature but was concentrated around piles 3, 9, 11, 12 and 14. Even if some additional cavity feature should exist to the west of pile 8 or to the north of pile 12 it is entirely unlikely that this would have any detrimental effect on the bearing capacity of the pile group.

The remediation measures were therefore deemed successful and required no further tertiary grouting to be carried out.

The main conclusions following the load test were as follows:

- There is no evidence of any detrimental effects from the cavity feature on the pile capacity.
- Both the shaft and the toe capacity are far from being exhausted at the maximum bi-directional load of 40.1 MN.
- A maximum characteristic toe bearing stress of 11 MPa was conservatively assumed in the design. This stress was almost reached in the test but at a toe displacement of only some 25 mm, corresponding to a low degree of mobilisation (of the order 24%). Thus, the cavity remediation works, including the reinforcement of the weathered Limestone, has been successful and allows for a high toe bearing capacity in ULS (cf. Figure 11).
- Extrapolation of shaft and toe resistances to 200 mm toe displacement and >5 mm shaft/soil displacement show capacity at or above the structural capacity of the concrete pile of approximately 110 MN (at the 7 days compressive concrete strength of 33.5 MPa).

Based on the thorough investigation of the cavity feature and the closely monitored remediation work it was possible to successfully conclude the foundation works for the Pylon P4 and start the casting of the Pylon as seen in Figure 15.

Figure 15. Status of P4 construction December 2012

8 ACKNOWLEDGEMENTS

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