Site Characterization for the HZM Immersed Tunnel

Caractérisation du site pour le tunnel immergé HZM

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ABSTRACT: The 36 km long HZM Link, crossing the Pearl River estuary between Hong Kong in the east and Macao and Zhuhai in the west is rated one of the most important current infrastructure projects in China. It is slated for completion in 2016 and consists of a world record length of 6 km immersed tunnel, two artificial transition islands and some 30 km bridges with a dual three lane motorway. In order to provide the structural designers with the requisite input for proper soil structure interaction analysis a very extensive site characterisation was carried out comprising geotechnical boreholes, CPTUs and seismic testing with associated advanced laboratory testing. This paper describes the results and calibration of geotechnical boreholes, CPTUs and advanced laboratory tests to provide the requisite tool for inference of ground stratification and stiffness variation to be used in the structural modelling of the immersed tunnel, the design of piles and dredging slopes.

RÉSUMÉ : La liaison HZM de 36 km de long qui traverse l'estuaire du fleuve Pearl entre Hong-Kong à l'est, Macao et Zhuhai à l'ouest, est considéré comme étant l'un des plus importants projets d'infrastructure en Chine. Le projet qui doit être achevé en 2016 est composé d'un tunnel immergé d'une longueur record de 6 km, de deux îles artificielles de transition et d'environ 30 km de pont autoroutier à deux fois trois voies. Afin d'obtenir les éléments essentiels pour l'analyses de l'interaction entre les fondations et les structures, une campagne de sondages géotechniques très détaillée a été menée comprenant des forages, des tests de pénétration au cône (CPTU) et des sondages sismiques ainsi que les études en laboratoire correspondantes. Cet article décrit les résultats obtenus et méthodes de calibration des forages, CPTU et des essais en laboratoire mis en œuvre afin d'obtenir les éléments de base nécessaire pour la détermination des caractéristiques mécaniques des sols à utiliser pour la modélisation des éléments du tunnel immergé, la définition des pieux de fondation et l'étude des pentes de dragage.

KEYWORDS: Site characterization, immersed tunnel, CPTU, triaxial testing, undrained shear strength, settlements, spring stiffness.

1 INTRODUCTION

The Hong Kong-Zhuhai-Macao (HZM) Link crosses the Pearl River Estuary in south-eastern China in the Guangdong province connecting Hong Kong at Shek Wan, Lantau Island to the Pearl at Macau and to the district of Gongbei, Zhuhai in mainland China, see Figure 1.

Figure 1. Location of the HZM project in south-eastern China.

The link is 36 km in total length of which 6 km comprises the immersed tunnel. The remainder consists of two artificial transition islands and low bridges some 30 km in total length. The whole connection has the capacity of a dual three lane highway.

Provisions for two possible future 570 m wide navigation channels are planned along the immersed tunnel alignment with proposed design dredging levels some 15-20 m below existing seabed level.

The particular challenges for the design of the immersed tunnel are:

- the presence of very soft clays requiring extensive dredging profiles and soil improvement,
- very deep foundation level of the tunnel in order to allow for future navigation channels 570 m wide over the central part of the tunnel,
- up to 23 m sedimentation load over the central part of the tunnel,
- potential of differential settlements due to the highly varying loading and ground stiffness conditions,
- the need for mixed foundation solutions with end bearing or settlement reducing piles near the artificial islands and direct foundation for the central part.

In order to provide the structural designers with the requisite input for proper soil structure interaction analysis for Detailed Design, a very extensive site characterisation was required. The scope and findings of this site characterisation are described in this paper.

The Project Owner is the HZM Bridge Authority, and the design and construction is being undertaken by a Joint Venture headed by the contractor China Communications and Construction Company (CCCC) Ltd.

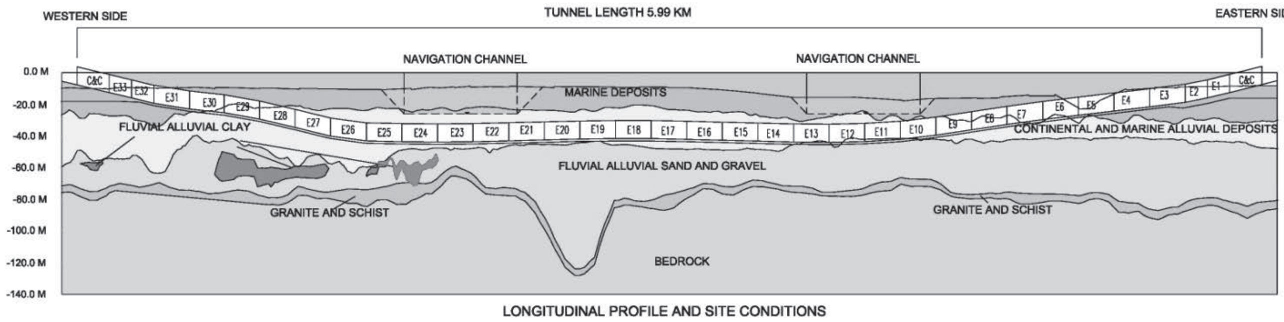

Figure 2. Simplified geological model along the immersed tunnel alignment.

2 GEOLOGICAL CONDITIONS

The project area is located in the Pearl River drainage basin, which historically has been shaped as a result of the uplift of the Tibetan Plateau during the Tertiary and Quaternary Periods, forming the present-day Pearl River Delta with its network system and estuarine bays (see Figure 1). The river delta is one of the most important and complex large-scale estuarine systems in China.

The Holocene development of the delta has been controlled and affected by the variations in the deposition of sediments, sea-levels and groundwater levels.

The soil deposits in the present-day Pearl River delta overlying weathered basement rock can be traced back to the Late Pleistocene and Holocene periods.

These deposits consist of three cycles of upward fining sequences of delta deposits, namely one Holocene and two Pleistocene delta cycle, which have been divided by two previously exposed and subsequently eroded surfaces.

Based on the described regional geology and the findings of the site investigations carried out for the project, the soil deposits and rock formations encountered along the alignment of the immersed tunnel, and in the locations of the artificial islands, can be grouped into five main units for soil deposits, and two main units for rock formations:

- Marine deposits of clays and sands formed during the Holocene period,
- Continental deposits of clays and sand from a once exposed surface formed during the late Pleistocene period,
- Marine alluvial deposits of clays and sands formed during the Mid to Late Pleistocene period,
- Fluvial alluvial deposits of clays and sands formed during the Early to Mid Pleistocene period,
- Residual soils formed during the Early Pleistocene period,
- Highly to completely migmatic schists formed during the Sinian period,
- Moderately to completely weathered migmatic granites formed during the Sinian period.

A simplified geological model is shown in Figure 2.

3 SCOPE OF INVESTIGATIONS

Three geotechnical investigation campaigns have been carried out for the project:

- Feasibility Study investigations carried out in 2004 and 2008: Only 16 Nos. boreholes were carried out in the vicinity of the immersed tunnel.
- Preliminary Design investigations carried out in 2009: 151 Nos. boreholes were carried out for the artificial islands and 115 Nos. boreholes, 29 Nos. CPTUs and seismic P-S suspension logging (in 10 Nos. boreholes) was carried out along the immersed tunnel alignment.
- Supplementary Soil investigations were carried out in 2010-2011: 80 Nos. boreholes, 364 Nos. CPTUs, 20 Nos. CPTUDs and seismic P-S suspension logging (in 6 Nos. boreholes) was carried out along the alignment of the immersed tunnel and at the locations of the artificial islands.

The Supplementary Soil investigations formed the main basis for Detailed Design, and the scope of and specifications for these investigations were defined by COWI as being a member of the design and construction Joint Venture. Site and laboratory works were followed closely by means of inspections carried out by COWI's geotechnical engineers, in order to ensure that all works were carried out in accordance with applicable standards.

The boreholes for the Supplementary Soil investigations were split into two types of boreholes: the GITB-series where geotechnical in-situ testing was carried out and disturbed samples were retrieved, and the TCB-series that were used entirely to retrieve undisturbed samples of fine grained soils. Most of the boreholes were carried out in pairs, each pair consisting of one GITB borehole and one TCB borehole, and as a general rule the GITB and TCB boreholes were drilled within five meters of each other, in order to produce mirror boreholes displaying similar geological and geotechnical properties. The drilling depths varied from 29 to 107 m below existing seabed level. The general distance between boreholes (and borehole pairs) was on average approx. 200 m in the longitudinal direction.

In general the CPTUs were carried out along three lines parallel to the tunnel alignment at distances of 0 m, +25 m and -25 m from the tunnel axis. The probing positions were staggered (cf. Figure 3), in order to effectively allow for one CPTU carried out at 25 m spacing along the projected centreline of the entire immersed tunnel alignment. Furthermore, additional CPTUs were carried out near the artificial islands. The CPTUs were carried out to penetration depths varying from 28 to 43 m below existing seabed level (basically to refusal in the fluvial alluvial sands and clays underlying soft deposits of marine clays).

A typical arrangement of investigations along the immersed tunnel alignment is shown in Figure 3.

The complete results of the Supplementary Soil investigations were provided by the geotechnical sub-contractors, Fourth Harbour Design Institute (FHDI) and Fugro, in native AGS 3.1 format.

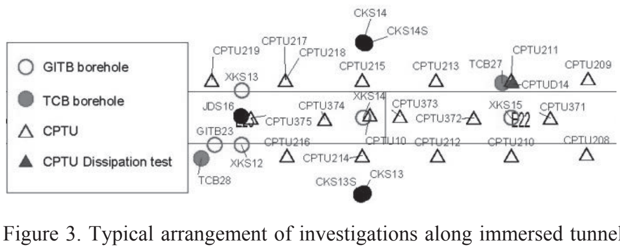

Figure 3. Typical arrangement of investigations along immersed tunnel alignment.

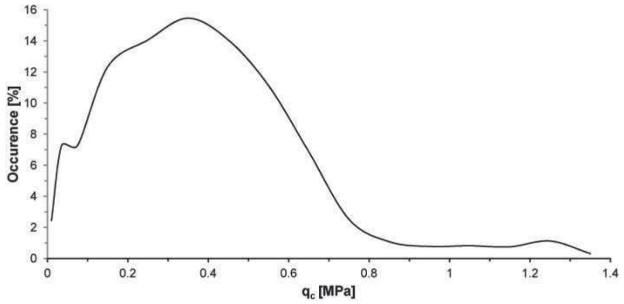

Figure 5. Example of q_c frequency distribution "foot print" for Marine Clay.

Based on the frequency distributions, representative ranges were established for the three principal CPTU properties, which in turn were used as filter criteria for a template predicting the geological unit.

The pore pressures varied greatly within each geological unit and were not used as a criterion for the geological interpretation, but merely as a guide when visually cross checking the results.

The interpretation template set up in this way worked on the premise that if a data set fell within the established "foot print" criteria, the template would subsequently yield the associated geological unit. The criteria were regarded as a key to a typical geological interpretation, not an unambiguous analysis. The final geological interpretation was therefore based on additional factors such as the combined appearance of the qc, Rf and u2 distributions combined with cross referencing to nearby boreholes.

Approximately 400 Nos. CPTUs (including those carried out during the Preliminary Design investigations) were interpreted using this method. This allowed for a 3D stratigraphical model to be set up for the geotechnical interpretation of the subsurface conditions surrounding the tunnel alignment, see e.g. Figure 6.

7 GEOTECHNICAL INTERPRETATION

The interpretation of the results of the oedometer tests carried out yielded the modulus number, m, recompression modulus number, m_r, secondary compression index, C_α, secondary recompression index, $C_{\alpha r}$, coefficient of consolidation, c_v and excess preconsolidation pressure, Δ 1_{pc} (= 1_{pc} - 1_{v0}).

The use of CPTUs was a key element in the evaluation of the settlement/stiffness variation along the alignment of the Having established the modulus number, m, for a range of soil deposits through laboratory oedometer testing, the modulus modifier, a, can be determined based on the formula:

I L §ä ß ¾ Ý (1)

where q_{tM} is the stress-adjusted cone resistance and 1 is a reference stress (=100 kPa).

Based on the modulus number from the oedometer tests and the stress adjusted cone resistance from CPTU testing, the modulus modifier, a, was derived or each soil deposit from (1).

The modulus modifier is plotted in Figure 7 assessing all oedometer results for fine grained samples. The results shown in this figure indicate relatively little data scatter and a general grouping of fine grained soils around 2 to 5 and 60 to 90 for the coarse grained soils (the latter values are not shown in Figure 7).

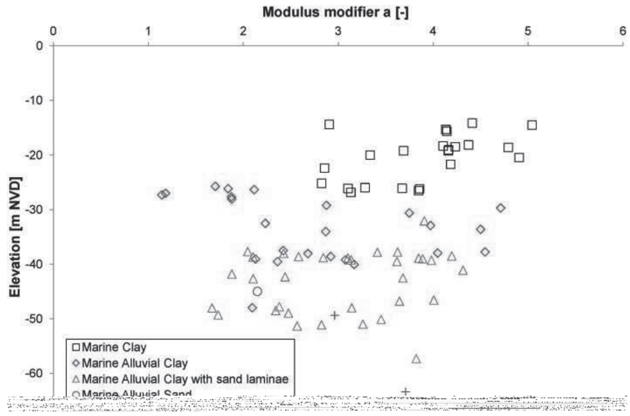

Figure 7. Modulus modifier, a, for selected geological units as derived from oedometer and CPTU testing results.

The recompression branch of the oedometer tests on fine grained soils indicated a linear correlation rather than a log-linear correlation. Further, the recompression modulus number, m_r, resulting from the reloading branches was found to vary with load for the fine grained soils. A reasonable approximation was achieved by applying different m_r values above and below an in situ stress of 100 kPa.

The resulting recompression modulus modifier, a_r, was therefore defined for in situ stress below and above 100 kPa.

Relatively little data scatter was observed in the a_r values, with a general grouping of a_r values for fine grained soils around 14 to 25 and 14 to 33 for in situ stress above and below 100 kPa, respectively.

Figure 6. Example of contour plot generated based on the compiled 3D stratigraphical model showing top of Continental/Marine Alluvial deposits in the location of the East Artificial Island.

The SHANSEP concept derives from the empirical observation that the ratio of the undrained shear strength, s_u, to the effective confining stress, σ'_v, is approximately constant for a given Over Consolidation Ratio (OCR) and varies linearly with OCR^{Λ}:

$$\frac{s_u}{\sigma'_v} = S * OCR^{\Lambda} \tag{2}$$

where S is the proportionality constant (also referred to as the c/p ratio) and Λ is the memory exponent. These values were estimated from the CAU triaxial testing carried out on undisturbed samples.

The S (or c/p-ratio) value was determined based on CAU tests loaded anisotropically to >150% of the assumed preconsolidation stress (as determined from the Batch I IL oedometer tests) and then sheared. The S-value thus determined was used for the determination of the Λ value for tests loaded anisotropically to below the assumed preconsolidation stress. Due to relatively high uncertainty with regards to the determination of the preconsolidation pressure, the memory exponent was found difficult to determine with accuracy.

For the clay deposits found along the alignment of the immersed tunnel average S and Λ values shown in Table 1 were found.

Table 1. Average values of S and Λ for clay deposits found along the immersed tunnel alignment.

Soil deposit	*Nos. of tests*	*S (avg.)*	*Λ (avg.)*
Marine clay	2	0.31	0.7
Continental clay	2	0.40	NA
Marine alluvial clay	7	0.31	1.0
Marine alluvial clay with sand laminae	4	0.36	0.7

Notes: NA = Not Applicable

The results of the CAU triaxial tests were also used to provide a correlation to results of CPTU testing, and thereby for providing an estimate of the N_{kt} cone bearing factor as used in the following equation (e.g. Lunne et al 1997):

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{3}$$

where σ_{v0} is the overburden pressure at the cone tip and q_t is the cone resistance corrected for pore pressure.

For the clay deposits found along the alignment of the immersed tunnel, the N_{kt} values were found to be 17 on average for the four deposits referenced in Table 1.

8 SETTLEMENT/SPRING STIFFNESS CALCULATION

Based on the geotechnical interpretation of the geology and settlement characteristics of soil deposits, the settlement and spring stiffness was calculated for each individual CPTU location.

The settlement analysis was carried out using the Janbu (1963) tangent modulus method, which accounts for the general non-linear load deformation relationship of soils. The settlement equations differ between coarse grained (sandy) and fine grained (clayey and silty) soils, and whether or not the preconsolidation stress is exceeded.

All in all four different equations were established.

Eq (4) for coarse grained soils below and above the preconsolidation stress:

$$\varepsilon = \frac{\Delta\sigma'_v}{m_r\sigma_r} \qquad \varepsilon = \frac{\sigma'_p - \sigma'_0}{m_r\sigma_r} + \frac{\sigma'_1 - \sigma'_p}{m\sigma_r} \tag{4}$$

and Eq (5) for fine grained soils below and above the preconsolidation stress:

$$\varepsilon = \frac{\Delta\sigma'_v}{m_r\sigma_r} \qquad \varepsilon = \frac{\sigma'_p - \sigma'_0}{m_r\sigma_r} + \frac{1}{m} ln\frac{\sigma'_1}{\sigma'_p} \tag{5}$$

Here ε is the vertical strain, $\Delta\sigma'_v$ is the increase in effective vertical stress from the tunnel (σ'_1-σ'_0), σ'_p is the preconsolidation pressure, σ'_0 is the in-situ vertical stress prior to loading, σ'_1 is the final vertical effective stress and σ'_r is a reference stress of 100 kPa.

The secondary settlement was calculated from (Terzaghi et al. 1996):

$$\varepsilon_s = \frac{C_\alpha}{1+e_0} log\frac{t}{t_p} \tag{6}$$

where C_α is the secondary compression index, and t/t_p is the ratio between the lifespan of the structure and the time for primary consolidation (t/t_p = 100 was conservatively assumed).

When the final load was lower than the preconsolidation stress, the secondary recompression index, $C_{\alpha r}$, was used instead of C_α.

The calculation of settlement was terminated at the top of rock, and due to the limited penetration of the CPTUs into the fluvial alluvial deposits of sand and gravel, the settlement calculations were based on SPT N data between the bottom of the CPTUs and the top of rock. An empirical q_c/N correlation dependent on the grain size distribution was used (Kulhawy & Mayne 1990):

$$\frac{q_c/p_a}{N} = 5.44(d_{50})^{0.26} \tag{7}$$

where p_a is a reference stress of 100 kPa, d_{50} is the mean grain size in mm and q_c is given in kPa.

The spring stiffness was then calculated as:

$$Spring\ stiffness = \frac{load\ from\ tunnel + siltation}{total\ settlement} \tag{8}$$

The settlement/spring stiffness calculations were carried out in purposefully set up Excel spreadsheets.

The settlement/spring stiffness calculations were carried out for some 400 Nos. CPTUs, and considering that each CPTU could contain up to 6,000 measurement points, running the entire series of calculations could take up to 2 hours.

The variation of calculated settlement and spring stiffness along the immersed tunnel alignment is shown in Figures 8 and 9, respectively.

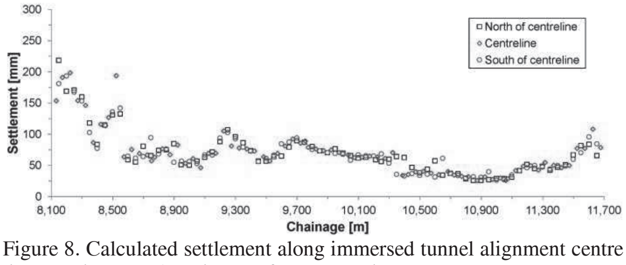

Figure 8. Calculated settlement along immersed tunnel alignment centre line and lines at 25 m distance from centreline.

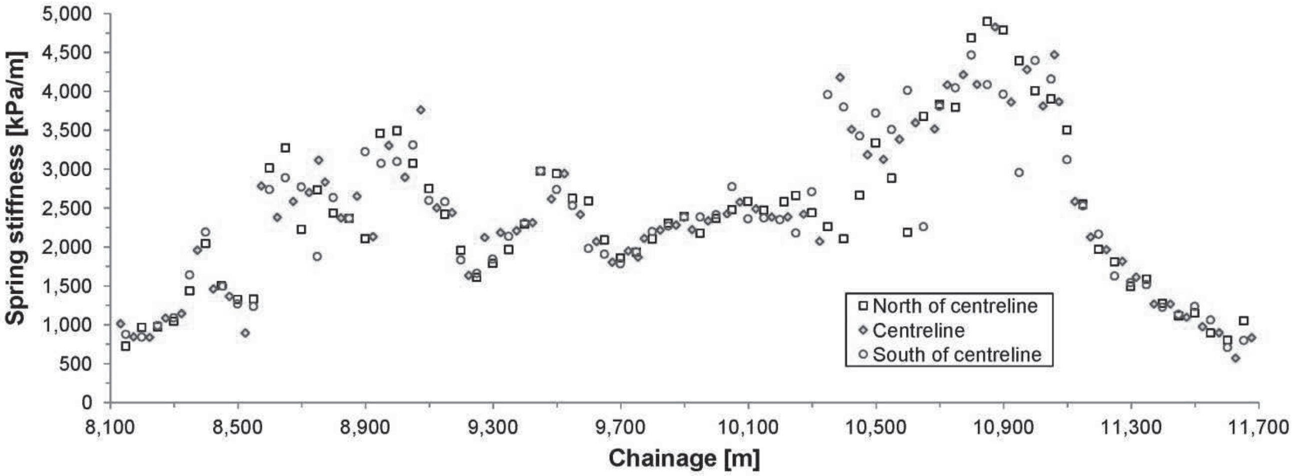

Figure 9. Calculated spring stiffness along immersed tunnel alignment centre line and lines at 25 m distance from centreline.

9 CONCLUSION

The design of the 6 km world record long immersed tunnel with highly variable soil and loading conditions poses significant challenges to both the geotechnical site characterization and the soil-tunnel interaction.

The structural tunnel design is very sensitive to differential settlements and rotations of individual tunnel elements and segments and thus to variation is soil stiffness along and across the tunnel alignment. Rather than resolving to empirical rules for handling the soil stiffness variation (Monte Carlo simulation or additional sinusoidal variation around the mean stiffness) the variation was handled directly by the tight mesh of CPTU probing points along and across the alignment.

Thus, the CPTUs provided a strong tool for clear geological unit delineation and allowed for very detailed settlement and soil stiffness assessment along the entire tunnel. The CPTU data were correlated with results from oedometer and CAU triaxial test results to provide site specific correlations regarding stiffness and undrained shear strength.

The geotechnical site characterization thus facilitated the tool for interaction between geotechnical and structural design of the tunnel elements and allowed for a robust and safe design.

10 ACKNOWLEDGEMENTS

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