Analysis of immersed tunnel settlement under complex load and multiple kinds of compound foundations

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Abstract. The silt type soil thickness is up to 3~18m laying at bottom of foundation trench of ramp section at west island of SZ-Link and the surface silt is of extremely low strength subject to the influence of sand disturbance. The island cut and cover section tunnel foundation is improved with PHC and the settlement is relatively small. To accommodate the coordinated deformation of the immersed tunnel the soft foundation of the tunnel needs special treatment. Through options comparison and selection, the solution for foundation of tunnel element E1 is advance gravel cushion and compacted blocky stone layer and DCM outside the island and high pressurized jet grouting pile in the island. To accurately analyze the settlement characteristics of the element E1, determine its settlement amount and to achieve smooth rigidity transition of immersion tunnel and cut and cover section the settlement analysis of element E1 under complex loads and multiple kinds of foundations were carried out in combination with testing researches of all kinds of foundations. The primary research work comprises: (1) conduction of laboratory model experiment of gravel cushion, blocky stone compaction experiment, DCM laboratory triaxial test and provision of deformation computation parameters for given different foundation improvement solutions; (2) conduction of 3D finite element numerical analysis for element E1, analysis of immersed tunnel settlement deformation characteristics and provision of the law of E1 settlement distribution.

1. Introduction
The ramp section at west island of SZ link is of poor ground conditions, most of which are fluid mud stratum and thick soft soils. The foundation of the cut and cover tunnel is treated with PHC and the settlement of tunnel elements are relatively smaller and element E1 of the immersed tunnel is butt connected to the cut and cover tunnel in the island. When the immersion tunnel is butt connected to the cut and cover tunnel, in order to achieve tunnel level basically in alignment with theory level after the element installation and occurrence of its settlement, additional elevation of installation level of the tunnel element is required (“pre-elevation amount). In SZ-Link advance gravel pavement is adopted and the pre-elevation amount has to consider not only the instant settlement amount of the immersed tunnel, but also the ground settlement to be developed slowly with elapse of time. Namely, the preset elevation amount equals sum of the preset elevation amount of the gravel cushion and overall preset elevation, where the former is intended to offset the instant settlement and control the vertical edge misalignment of adjacent elements while the latter is intended to offset post settlement after works completion. The principle drawing is illustrated in figure [1]. In order to assure the successful butt connection of element E1 to the cut and cover tunnel the ground improvement is required and the control goal of ground
improvement aims at controllability of total ground settlement and longitudinal differential settlement. The allowable longitudinal differential settlement (foundation stiffness) is to be determined by the internal force on the tunnel element and the allowable maximum opening amount of the joint. Ground improvement is not to be considered unless the untreated natural ground fails to meet the requirement of structural design by means of reasonable measures.

Figure 1. Conceptual drawing of preset elevation amount and preset higher dumping

Accurate calculation of immersed tunnel settlement is the precondition of determination of preset elevation amount and the settlement of immersed tunnel is mainly consisted of the three parts: compression of cushion layer, soil resilience and recompression settlement and secondary consolidation settlement. To immerse tunnels most solutions need foundation trench whose trench accuracy can hardly assured due to large excavations involved and highly difficult underwater works. Usually a bed layer is placed underneath the tunnel. Advance gravel cushion pavement is adopted in SZ link and the gravel cushion serves as a part of the foundation and its parameters are crucial to the estimation of the preset elevation amount of the immersed tunnel. In addition, the west island ramp section is of poor ground conditions and the soft ground requires improvement and the deep cement mixing, (DCM called in brief) has a successful case which was once applied in Busan tunnel, South Korea. With the development and application of domestic equipment such problems of limited use of DCM due to high cost and short of equipment have been eliminated. DCM solution through strengthening before excavation is recommended to be used in west island ramp foundation improvement for its advantages of good adaptability to soft foundation, no need of surcharge, shorter construction time, elimination of soil liquefaction, sound seismic resistance, good durability, good stiffness transition of immersed tunnel section. DCM cannot be penetrated into sea by construction vessel within range of element in island and area of the large cylinder. Since the high pressurized injection pile is of such advantages of simple equipment, convenient construction, high pile body strength and good and stabled strengthening effect, the scheme of high pressurized jet grouting pile is applied.

To sum up above through comparison of multiple kinds of solutions the E1 foundation scheme in SZ link is: advance gravel cushion and compacted blocky stone layer and DCM outside the island and high pressurized jet grouting pile in the island. In this paper the mechanical parameters for the gravel cushion, compacted blocky stone layer, DCM are determined by means of laboratory model test, laboratory triaxial test, field test in place and element finite numerical calculation model built and in combination with design document and site construction state the E1 settlement characteristics are hereby analyzed.

2. E1 foundation and tunnel roof backfill treatment

The soils at base of west island ramp section is dominantly silt type and sand natured soil. The original place of silt stratum ②1 is a kind of fluid silt and deep excavation will be involved by means of replacement by extensive excavation and siltation will be more and more serious and thus this excavation scheme is not feasible. Since DCM scheme achieved success in Busan tunnel, South Korea
and by DCM method surcharge loading is not needed in addition to such advantages of shorter construction time, avoidable liquefaction, sound seismic performance and better stiffness transition of immersed tunnel than DCM. Thus DCM can be the recommended solution to west island ramp foundation.

Within the area of K11+941.2~K11+993.04 within range of E1, 1 meter thick gravel cushion and 1.1 meter thick blocky stone compacted layer is placed on top of DCM (figure 2, figure 3, figure 4). DCM marine construction vessel cannot enter the range of island of E1 and cylinder and the area to be strengthened is deep and high strength is required. Thus high pressurized jet grouting pile scheme is applied and at the location outside the cylinder and close to island inside 35 centimeter thick plain concrete C30 is placed on top of the high pressurized jet grouting pile at bottom of gravel cushion to achieve smooth and gentle transition from the immersion section to cut and cover section.

To sum up above the ramp tunnel foundation at west island in SZ Link comprise: the composite foundation design solution applied consists of advance gravel cushion and compacted blocky stone layer and outside island DCM and island high pressurized jet grouting pile.

Figure 2. The longitudinal section of the immersed tunnel E1 ground improvement (unit: m)

Figure 3. Plane layout of DCM (unit: cm)
The thickness of the backfill on the tunnel roof impacts greatly the settlement amount and its backfill thickness on tunnel roof shall be determined subject to geological conditions and vessel impact protection requirement. The longitudinal of backfill on E1 is illustrated in figure 5.

The element E1 at head of the west island penetrates through the island revetment structure and the places where backfill loads of island revetment are acting and the maximum vertical loads at base of element under revetment wall at island head reach up to 260kPa and the loads increase sharply. The range of 40 meters of E1 within island is founded on the ground of surcharge loaded and still 3~5m fluid silt stratum lie underneath the element outside the island and most of the stratum lie within range of 28 diameter steel cylinder. Hence it is very difficult to coordinate the E1 foundation design and the longitudinal settlement of the cut and cover longitudinally and stiffness smooth transition. Therefore, load reducing structure is required at island head location. The loading reducing structure is concrete hollow box as shown in figure 2 and after loading reduction the maximum load underneath the element at normal water level condition is 138kPa and the maximum load underneath the element at the lowest water level in 100 years is 151kPa.
3. Computation model and the parameters

3.1. 3D numerical calculation model for E1

In setting up the model such stratum information from section K11+941.2 (E1 end outside the island), section K11+982.2, section K12+025.1 and section K12+065 (E1 end inside the island) to set up model boreholes.

| Table 1 The soil parameters |
|-----------------------------|
| stratum                      | Constitutive model | Unit weight kN/m³ | E50/E0 MPa | Eood MPa | Eur MPa | Power m | Cohesion kPa | Friction angle ° | Poisson’s ratio ν | Rinter |
| 21sludge                     | HS                | 14.93             | 1.57        | 2.53     | 4.71    | 1       | 5.64       | 10.7               | 0.2               | 0.67   |
| 22sludge                     | HS                | 14.93             | 1.57        | 2.53     | 4.71    | 1       | 5.64       | 10.7               | 0.2               | 0.67   |
| 224silt                      | HS                | 19.25             | 7.94        | 7.94     | 23.82   | 0.5     | 9.8        | 22                  | 0.67              |
| 23sliltey clay               | HS                | 14.33             | 2.33        | 3.65     | 6.99    | 1       | 6.63       | 11.17              | 0.2               | 0.67   |
| 224silt                      | HS                | 19.25             | 7.94        | 7.94     | 23.82   | 0.5     | 9.8        | 22                  | 0.67              |
| 226medium sand               | HS                | 20.19             | 13          | 13       | 39      | 0.5     | 9          | 24.3                | 0.67              |
| 31clay                       | HS                | 19.44             | 5.79        | 9.7      | 17.37   | 1       | 26.4       | 10.1               | 0.2               | 0.67   |
| 6121sandy soil like highly weathered granite | HS  | 19.89 | 50 | 50 | 150 | 0 | 30.5 | 30.6 | 0.2 | 0.67 |
| 6122fragmentary weathered granite | HS  | 19.2 | 33.33 | 33.33 | 1000 | 0 | 14.5 | 29.6 | 0.2 | 0.67 |
| 613medium weathered granite | HS | 26.2 | 3000 | 3000 | 0 | 8 | 28 | 0.2 | 0.67 |
| gravel backfill              | MC                | 22                | 12          | -        | -       | -       | 1          | 36                  | 0.67              |
| sand backfill                | HS                | 21                | 8           | 8        | 24      | 0.5     | 1          | 30                  | 0.2               | 0.67   |
| Ceramsite concrete           | MC                | 12                | 17.5        | -        | -       | -       | -          | 38                  | 0.2               | 0.67   |
| Cylinder + backfill sand     | Linear elasticity | 18                | 1765        | -        | -       | -       | -          | -                   | 0.17              | 1      |
| Concrete bed/PHC             | Linear elasticity | 21                | 30000       | -        | -       | -       | -          | -                   | 0.2               | 1      |
| Tunnel structure             | Linear elasticity | 26.203           | 345000      | -        | -       | -       | -          | -                   | 0.167             | 1      |

The water level in the model considers the mean water level of seabed surface and elevation of the mean water level is +0.52m.

Based on the external conditions 3D numerical calculation model for E1 was set up and plastic calculation mode was adopted to make analysis. The displacement in the element immersion stage was adjusted to zero and the calculation model is illustrated in figure 6.

In order to accurately calculate the settlement of E1 relevant testing researches were conducted. The design parameters of the gravel cushion, compacted blocky stone layer, DCM and high pressurized jet grouting piles were determined and provided.
3.2. Laboratory model experiment for gravel cushion
Through carrying out laboratory model test for gravel cushion (figure 7), the deformation modulus of the gravel cushion was obtained. The test used 1:1 geometrical similarity ratio and based on plane stress assumption single ridge roof and bilateral 1/2 grooves were used and outside of the model was in complete confining state. Full-size physical modeling experiment was conducted, in which the gravel cushion section top is 1.8m wide, bilateral V shaped 1/2 groove 0.6m wide and 0.40m high.

The gravel cushion is of grading 20~40mm and thickness of 1m. In addition, in the process of leveling of the gravel leveling vessel the gravel in the drop pile exerts a pre-load on the gravel bed placed and this pre-load effect influences deformation characteristics of the gravel bed to some extent. In the experiment the pre-load on the gravel cushion is 30kPa. Through model experiment on two groups of gravel cushions in which the gravel cushion is 1m thick respective and preload is 30kPa (figure 8), the deformation modulus of the gravel cushion is respectively 25.4MPa in section 0~30kPa and 8.9MPa in section 30~110kPa.
3.3. Blocky stone cushion compaction experiment

The blocky stone cushion under E1 is of thickness 1.1m–3m and the construction method of compacted blocky stone uses excitation compacting force of 150kPa and compaction duration 45s were verified by model tests (figure 9).

When the blocky stone cushion thickness takes 1.15m, vibratory compaction was made in three operations with each compaction duration of 15s. The compacted amount of 1.15m thick blocky stone cushion is shown in table 2.

| experiment | Cushion thickness/m | Compacting duration | Base plate settlement amount/cm | Cushion compacted amount/cm | Compaction rate | Top surface height variation of compacted blocky stone cushion/cm | Compacted specimen state |
|------------|---------------------|---------------------|---------------------------------|----------------------------|-----------------|---------------------------------------------------------------|--------------------------|
| Group 1 experiment | 1.15                | 30s (first time, 2nd Layer compaction15s+1s) | 0.46                            | 11.53                      | 10.03%          | -11.4cm ~ 9.8cm                                               | Complete                 |
|               |                     | 15s (third compaction) |                                | 2.28                        | 1.98%           | -4.2cm ~ -4.7cm                                               |                          |
|               |                     | 45s (accumulated calculation) |                                | 13.81                      | 12.01%          | -4.2cm ~ -4.7cm                                               |                          |
In situ static load experiment was conducted after compaction and the settlement curve is illustrated in figure 10.

![Figure 10. Load settlement curve](image)

The experiment results show that when compaction duration is 45s ideal compacted blocky stone cushion can be formed and the compacted cushion can achieve deformation modulus up to 50MPa, and compaction amount can be controlled to around 14cm with compaction ratio around 12%.

When the blocky stone thickness is 1.5m the compaction duration is controlled to be 45s. The compaction amount is shown in table 3.

| experiment       | Cushion thickness/m | Design thickness/m | Compaction duration | Bottom slab settlement amount/cm | Cushion compaction settlement/cm | Compaction rate | Blocky stone layer top height variation after compaction/cm | State of compacted block specimen |
|------------------|---------------------|--------------------|---------------------|----------------------------------|----------------------------------|-----------------|----------------------------------------------------------|----------------------------------|
| Group two        | 1.54                | 1.5                | 45s                 | 1.44                             | 17.04                            | 11.07%          | -6.0cm – 4.4cm                                          | intact                           |

In situ static loading experiment was carried out after compaction and the settlement curve diagram is shown in figure 11.

![Figure 11. p-s curve of group static loading experiment](image)
Static loading experiment shows that the deformation modulus of the blocky stone cushion may reach around 34MPa. To guarantee the compaction results 3 meter thick cushion was divided into two layers to be compacted.

To sum up above experiment results: for the 1.15m thick blocky stone the deformation modulus of the cushion is set to be 50MPa while for the 1.5m thick blocky stone, the deformation modulus of the cushion is set to be 34MPa.

3.4. Laboratory triaxial experiment for DCM

The blocky stone cushion under E1 is of thickness 1.1m~3m and the construction method of compacted blocky stone uses excitation compacting force of 150kPa and compaction duration 45s were verified by model tests (figure 9).

After through DCM improvement the foundation is calculated with compound foundation parameters. According to “Technical specification of building foundation treatment” (GB 50007-2011) after DCM improvement the compound foundation was calculated with the calculation parameter formula (1).

$$E_{sp} = mE_p + (1 - m)E_s$$  \hfill (1)

Where: $E_{sp}$ is the compression modulus (kPa) of compound soil stratum; $E_p$ is the compression modulus (kPa) of cement soil reinforcing body; $E_s$ is the compression modulus (kPa) of soils between piles; $m$ is replacement percentage in areas.

The DCM improvement for foundation under E1 adopts the solution: piles are arranged at longitudinal spacing of 3m between pile shafts and varied transverse spacing of 3m, 4m and 5m and pile replacement rate in areas are respectively 30.94%, 38.68% and 51.57%. The relevant parameters of the compound foundation are calculated by formula (1).

After DCM pile works was completed core was taken and the core sample taken received laboratory triaxial experiment and the stress-strain curves of different core samples taken from different depth were obtained (figure 12). The ratio of 1/2 stress of the peak value stress to the strain was taken to calculate its deformation modulus. The mean deformation modulus the specimen was 156.3MPa and the specific testing results are shown in table 4 and the stress-strain curve diagram is shown in figure 13.

![Figure 12. Laboratory triaxial test of DCM core sample](image)
Figure 13. Stress-Strain curve of core samples taken from different depth

Table 4 Deformation modulus of core samples

| Rock core No. | Specimen description | Sample size (mm) | Sampling depth (m) | Deformation modulus E50 (MPa) |
|---------------|----------------------|------------------|--------------------|-------------------------------|
| 3-22.4~22.6   | Moderate cement content, containing small amount of coarse sand | 100 200 | -22.4 -22.6 | 260.1 |
| 9-24.4~24.6   | Moderate cement amount, containing small amount of coarse sand | 100 200 | -24.4 -24.6 | 229.2 |
| 15-26.5~26.7  | Better cement content, containing small amount of clay speckle | 100 200 | -26.5 -26.7 | 140.3 |
| 27-30.3~30.5  | Better cement content, containing small amount of coarse sand | 100 200 | -30.3 -30.5 | 123.9 |
| 30-31.2~31.4  | Better cement content, containing small amount of coarse sand | 100 200 | -31.2 -31.4 | 82.4 |
| 33-32.3~32.5  | Better cement content, containing small amount of coarse sand | 100 200 | 32.3 32.5 | 101.8 |

Mean value 156.3

3.5. Reference value taken for high pressurized jet grouting pile
SZ Link is not far from the Hong Kong-Zhuhai-Macao bridge and their geological conditions are not very different. By learning from the high pressure jet grouting piled foundation settlement experiment at the island – tunnel interface section of Hong Kong-Zhuhai-Macao Bridge (Wang, 2017), the mean SPT blow number was no less than 20 blows in the compound foundation of the high pressure jet grouting piled in the Hong Kong-Zhuhai-Macao Bridge Project the mean deformation modulus of the corresponding compound foundation of high pressure jet grouting piles was 81.6MPa. In SZ link the SPT blowing amount no less than 20 blows is required for the compound foundation of high pressure jet grouting piles which is consistent with that of Hong Kong-Zhuhai-Macao Bridge Project and the deformation modulus of the compound foundation of high pressure jet grouting piles is set to be 81.6MPa.

4. Force deformation characteristics of E1 in complex geological conditions
According to the site works state after immersed tunnel is immersed and installed backfill bilateral and on tunnel roof are to be carried out followed by such works procedures of wave wall works at island head, ballast concrete works and opening to traffic and operation. The deformation cloud map of the immersed tunnel subjected to loading cases are selected as case 1 tunnel bilateral and roof backfill,
loading case 2: wave wall works at island head; loading case 3: ballast concrete works and opening to traffic and operation to analyze its characteristics of deformation by forces.

4.1. Stress characteristics analysis of IMT base stress
The external loads of loading case 1 are: The backfill on tunnel roof outside island is 4.754m high and the unit weight underwater of gravel cushion is 22kN/m³ and unit weight above water is 18 kN/m³. The hollow box at location of cylinder at island head is 6.4m high and the hollow box is equivalent to be solid body and its unit weight is set to be 6.078kN/m³. The backfill material in lower hollow box is crushed stone and unit weight of the crushed stone layer is 22kN/m³ and unit weight above water is 18 kN/m³. The backfill material in the island is partially ceramsite concrete and the backfill ceramsite concrete is 4.473m high with its unit weight of 12 kN/m³. 1.6m high blocky stone backfill layer lie on top of the ceramsite concrete. 1.05 negative buoyancy in the tunnel is considered and vertical load of 6.66kPa is imposed inside the tunnel element.

The external loads in loading case 2 are: in addition to the loads imposed in loading case 1, the breakwater at island head is equivalent rectangular with height of 7m, length of 15.972, unit weight of 10kN/m³. The blocky stone backfill on top of deloading hollow box is simulated with loads and the area loads imposed on top of the hollow box is 48.4kPa.

The external loads in loading case 3: In additional to above loads ballast concrete and vehicular loads are considered and 31.03kPa vertical loads are imposed inside the immersed tunnel.

According to above external loading conditions and foundation improvement forms the stress cloud map of tunnel base under all various loading conditions are shown in figure 14.

(a) bilateral and top backfill  (b) island head breakwater  (c) ballast concrete works and opening operation

Figure 14. Tunnel base stress cloud map

The longitudinal effective stress curve at element center is shown in figure 15

Figure 15. Element center stress comparison curve

Figures 14 and 15 show that with the constant increased external loads outside tunnel the tunnel base stress increases progressively and the tunnel base stress distribution pattern is in positive correlation
with tunnel roof loads. The tunnel roof loads are utmost at island head location where the tunnel base stress reaches its peak value which is 202.74kPa.

4.2. Immersed tunnel base settlement characteristics analysis

The tunnel base stress cloud map under all various loading conditions are shown in figures 16.

![Figure 16. Tunnel base settlement cloud map](image)

(a) bilateral and roof backfill  (b) island head breakwater works  (c) ballast concrete works and opening to traffic operation

Under these given three loading cases the tunnel center longitudinal settlement curves are shown in figure 17.

![Figure 17. Tunnel center longitudinal settlement comparison curves](image)

The comparison and contrast results show that the greater the external load is the larger is the tunnel structure settlement. By means of deloading inside island and tunnel base foundation improvement the island settlement inclines to stable with maximum settlement of 59.33mm. The tunnel center utmost settlement values are shown in table 5.

| Works stage                  | loads                                                                 | settlement/mm |
|------------------------------|-----------------------------------------------------------------------|---------------|
| Stage 1 Bilateral and roof backfill | Structural selfweight+ roof backfill                                 | 37.27         |
| Stage 2 Island head breakwater works | Structural selfweight+roof backfill +breakwater+partial siltation | 53.25         |
| Stage 3 Ballast concrete works and opening to traffic and operation | Structural selfweight+roof backfill+ waterbreak+ permanent siltation+ ballast load+ opening to traffic load | 59.33         |

5. Conclusion

In order to analyze the settlement characteristics of E1 element of immersed tunnel of SZ Link to determine its settlement in this paper gravel cushion laboratory model experiment, blocky stone compaction experiment, unconfined compressive strength experiment for DCM were carried out and the
parameters of high pressure jet grouting piling experiment of Hong Kong-Zhuhai-Macao Bridge were learnt from to provide basis for accurate value selection as the foundation parameters. Based on above parameters the characteristics of deformation by settlement of E1 at island head was analyzed and the following primary conclusions are drawn:

1) The gravel cushion deformation modulus parameters are obtained by carrying out laboratory gravel cushion model experiment and adopting 1:1 geometric similarity ration and based on plane stress assumption. Namely the 1.0 thick gravel cushion deformation modules value with 30kPa preloading considered to be: 25.4MPa for section 0~30kPa and 8.9MPa for section 30~110kPa.

2) Under the conditions that the excitation vibration force is 150kPa and vibrating duration of 45s, 1.15m thick blocky stone cushion after compacted may form ideal compacted blocky stone cushion with the deformation modulus up to 50MPa of the compacted cushion and settlement amount by compaction is controlled to be around 14cm with compaction rate around12%: 1.5m thick blocky stone can achieve deformation modulus up to around 34MPa after compaction. To assure the compaction effect during works the 3m thick cushion can be divided into two layers to be compacted.

3) Core sample was taken after completion of DCM piling works and the core specimen drawn received laboratory triaxial test, through which the stress-strain curves of core sample from different depth were obtained and the ratio of 1/2 peak stress value to strain was used to derive its deformation modulus. The mean average deformation modulus of the specimen is 156.3MPa.

4) The SPT blow number was 21 blows in the high pressure jet grouting piled compound foundation of the Hong Kong-Zhuhai-Macao Bridge project and the corresponding mean deformation modulus is 81.6MPa of the high pressure jet grouting piled compound foundation. The geological conditions of the SZ Link and the Hong Kong-Zhuhai-Macao Bridge are not of much difference and it is required that the mean standard penetration blow number shall not be less than 20 blows for the high pressure jet grouting piled compound foundation and the deformation modulus of the high pressure jet grouting piles consults the modulus of high pressure jet grouting piles of Hong Kong-Zhuhai-Macao Bridge.

5) Based on above researches through finite element numerical analysis, with the constant increased external loads of immersed tunnel the tunnel base stress and settlement amount also increase progressively and the tunnel base stress and settlement distribution pattern is in positive correlation with the tunnel roof loads. The tunnel base stress and settlement at the island head location is utmost and through island deloading and tunnel base foundation improvement the island settlement inclined to stable and the utmost settlement amount was 59.33mm.

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