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Taiwan High Speed Railway – Contracts 260 & 270 Geotechnical aspects and pile testing

Les aspects de geotechnical et entasse l'essai

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ABSTRACT
The paper describes the ground investigations carried out during the design period to supplement the tender information provided by the employer, the Taiwan Highspeed Railway Corporation (THSRC). The Grund Investigation was carried out in 2 or 3 stages consisting of boreholes with SPT's and disturbed samples. At some locations in Contract 270 CPT tests were also undertaken, where soil conditions allowed the probe to penetrate the strata. The majority of the foundations to the viaducts are piled. The design of the 2.0m diameter piles was based on local practice verified by 16 No full size pile tests. The test piles varied in length being 30m to 44m in Contract 260 and 55m to 72.5m in Contract 270, to replicate the pile lengths required to support the viaduct structure. The paper presents the results of the load tests, the verification & development of the design equations

RÉSUMÉ
L'article décrit les investigations de sols effectuées durant la période de préparation afin de compléter les informations du dossier de consultation fourni par le Client, la Société Taïwanaise du Chemin de Fer à Grande Vitesse (THSRC). Les investigations ont été menées en 2 ou 3 campagnes de forages destructifs avec SPT et prélèvements de sols. Dans les zones du Contrat 270 où la géologie permettait à la sonde de traverser les couches de terrain, des tests CPT ont également été réalisés. Les fondations des viaducs sont pour les majorité constituées des pieux. Le dimensionnement des pieux de 2,0 m de diamètre a été basé sur la pratique locale et vérifié par 16 essais de chargement en vraie grandeur. Les pieux d'essai ont des longueurs variant de 30 à 44 m pour le Contrat 260 et de 55 à 72,5 m pour le Contrat 270, ces longueurs étant représentatives de celles nécessaires aux fondations des structures des viaducs. L'article présente les résultats des essais de chargement, la vérification et le développement des équations de dimensionnement

1 INTRODUCTION

To enable the rapid development of urban areas in the western part of Taiwan, a high-speed rail link is being constructed from Panchiao, Taipei County, in the north to Tzoying Kaohsiung City in the south. Total route length of the Taiwan High Speed Rail Link (THSRL) is 345 km. It will have six stations including Panchiao, Taoyuan, Hsinchu, Taichung, Chiayi, Tainan and Tzoying. The railway runs on viaducts for much of the route south of Paguashan tunnel. The southern section of the route runs roughly north south crossing an alluvial plain, with a number of rivers which generally flow east to west. The soils along this section of the route are alluvial to depths that exceed 100m. The route lies within an area of high seismicity and on potentially liquefiable ground This paper describes the geotechnical works carried out for the designs of the piled foundations, for the two contracts C260 and C270 awarded to the Joint Venture Bilfinger Berger Continental Engineering Corporation Joint Venture (JVBBCEC). Figure 1 is a plan showing the position of the contracts C260 and C270 in the context of the overall project.

The ground conditions along the route were investigated by the Taiwan High Speed Railway Corporation (THSRC) before tenders were invited, further investigations were carried out by the contractor, JVBBCEC. The additional investigations included boreholes or cone penetration tests at every viaduct pier position. Data from the investigations was either digitized or supplied in a digital format so that the information could be used in the design process without further processing.

During the tender design JVBBCEC had developed preliminary relationships between standard penetration test results (SPT N) and pile load capacity. Post award of the construction contracts C260 and C270, JVBBCEC had to validate the design relationships prior to detailed design of the large diameter piles supporting the THSR Viaducts.

Full scale pile tests were carried out on non-working piles using the Osterberg Cell (Ocell) system with full instrumentation of the pile shafts. The data from the pile tests was used to optimize the pile design equations that had been used for the tender designs.

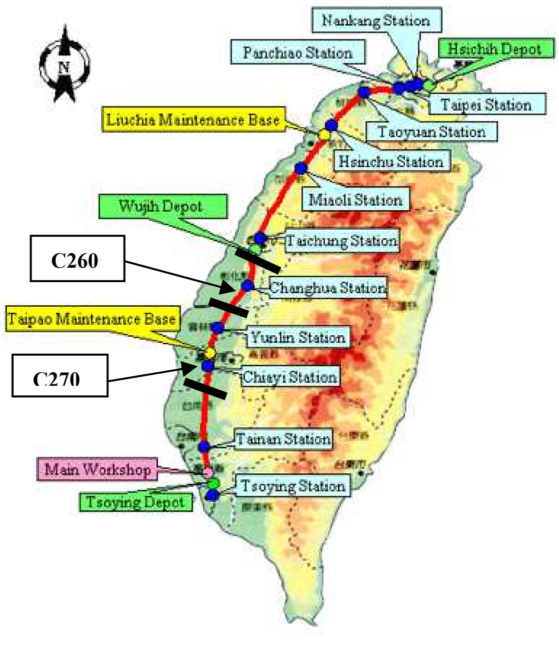

Figure 1 Map showing project and contracts

During construction the methods of forming the piles were reviewed and where necessary changes were agreed. In some areas of difficult ground a small number of piles were formed with defects that required remedial works. A complex 3D finite element analysis of the pile groups where these remedial works were undertaken was necessary to confirm that the dynamic response of these asymetric groups would satisfy the design specification.

2 THE GROUND INVESTIGATIONS

Prior to starting the pile testing programme, the JVBBCEC started a phased ground investigation contract. The ground investigation data provided with the tender documents was based on boreholes, formed by 240 No wash boring, and 19 No CPT tests in sections designated C260 and C270. All of the tender issue information was issued in hard copy; this data was transcribed into an Excel spreadsheet, which allowed the data to be set into the Association of Geotechnical Specialists (AGS 1998) data transfer format for uploading into Holebase (Key Systems 1998) a geotechnical database system.

During the first phase investigation, intended to ensure that at least one pier in every four was investigated to below the anticipated pile toe depth, 158No. boreholes were formed in C260 and 215No. in C270. All boreholes were formed using the wash-boring process, SPT tests were performed at 1.5m intervals in all strata and disturbed samples were taken for grading and plasticity (Atterberg Limits) tests in the laboratory. In cohesive soils undisturbed core samples were taken. The core samples were used to determine the bulk density of the soils in addition to the other classification tests.

In the second phase investigation boreholes were formed at piers where no previous GI had been carried out or where the design process was indicating that previous boreholes did not penetrate to at least 3 pile diameters below the design toe level. The requirement for boreholes to penetrate to the depth was a fundamental principle of the design specification. . In C260 these investigations were performed by drilling boreholes, while in C270, where the soil conditions were suitable, Cone Penetration Tests (CPT) were carried out. The interpretation of the CPT data was a subject of much discussion between the designers, the checkers and the client. To resolve the differences of opinion between the parties the JVBBCEC carried out a third phase of investigation, boreholes were formed at locations in C270 where the results of the CPTs could not be rationalized with the SPT N results from the previous adjacent boreholes.

All of the data from the post-tender investigations carried out by JVBBCEC was provided to the designers in hard copy and in an Access database. A routine was written by Maunsell Ltd, to extract data from the Access database in a format that was compatible with the inputs required by Holebase. The data transfer process was developed to eliminate errors introduced by repeated data entry.

A total of 429 boreholes were formed in C260 and, in C270 496No boreholes and 865 CPTs were carried out. All of this data was added to the Holebase database for use in the liquefaction potential assessment and the design of the piles.

3 THE PILE TESTS

At the beginning of the project a location in C260 was chosen as a site where trials of the different methods of pile construction could be undertaken. The first test pile was constructed using a 2.0m diameter casing oscillator and grab. The pile was constructed with the bore full of water to balance the ground water pressure, at about 3.0m below GL. The 30m long pile was tested using a single Osterberg cell (Ocell) set at about 20m below the pile head. The results of the load test showed that the mobilized shaft friction was lower than the values used in the tender design – further research was planned to see if other methods of construction would give a higher value of shaft friction.

A pile was bored using a polymer mud using a rotary auger, however as the rate of boring was very slow the JVBBCEC considered other methods. The JVBBCEC next constructed a test pile using a reverse circulation drilling rig (RCD), this method was adopted for all production piles in C270, in C260 some pile were constructed under polymer using a grab.

The schedule of all the design validation test piles is shown in Table 1. All of the piles were constructed using a polymer drilling mud using the RCD process. Some of the shorter piles were tested using a single Ocell, while the longer piles were tested using a double Ocell arrangement. Figure 2 shows a typical double Ocell configuration and the instrumentation used to monitor the behavior of the piles during the tests.

Piles which were subjected to a horizontal load test, were fitted with four inclinometer tubes, to monitor the horizontal movements of the pile shafts.

Two piles were subjected to a vertical tension test to verify the design assumption that the results of the Ocell tests are valid for both compression and tension loading cases. Previously published data from Ocell tests had only compared the results of the Ocell tests with conventional top loaded static pile tests.

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Table 1: Details of Piles Constructed for Pile Tests

Test Pile	Type of Test	Pile Length	O-Cell Depths (m) Upper	Lower	Date Cast
PT270-05	O-cell	57.1	40.1	53.1	30/09/00
PT270-06	O-cell	60.4	46.2	56.8	12/10/00
PT270-07	O-cell	63.35	48.0	61.0	15/12/00
PT270-08	O-cell	55.0	34.3	53.3	16/03/01
PT270-09	O-cell	60.3	43.9	55.9	08/12/00
PT270-10	O-cell	72.4	37.9	66.4	24/03/01
PT270-11	O-cell	62.93	47.4	62.9	04/04/01
PT270-12	O-cell	60.63	47.29	57.9	10/03/01
TT270-01	Static Tension	40.7	NA		27/04/01
HT270-01	Horizontal	30.0	NA		16/12/00
HT270-02	Horizontal	30.0	NA		12/04/01
HT270-03	Horizontal	30.0	NA		17/04/01
PT260/01	Vertical	30.3	25.15		09/09/00
PT260/02	Vertical	38.35	34.15		03/11/00
PT260/03	Vertical	39.8	34.		14/12/00
PT260/04	Vertical	44	38.0		01/12/01
PT260/05	Vertical	43.6	25.1	43.7	02/03/01
PT260-06	Vertical	45.7	41.11	25.05	16/03/01
PT260-08	Vertical	41.06	22.5	39.03	24/02/01
TT260-02 (Tensile)	Vertical	28.00	NA		26/04/01
HT260/02 (Horizontal)	Horizontal	25.0	NA		27/11/00
HT260/03 (Horizontal)	Horizontal	30.0	NA		

The tests were carried out not less than 28 days after piles had been cast, to allow time for curing and for the friction between the soil and the concrete in the pile shaft to develop

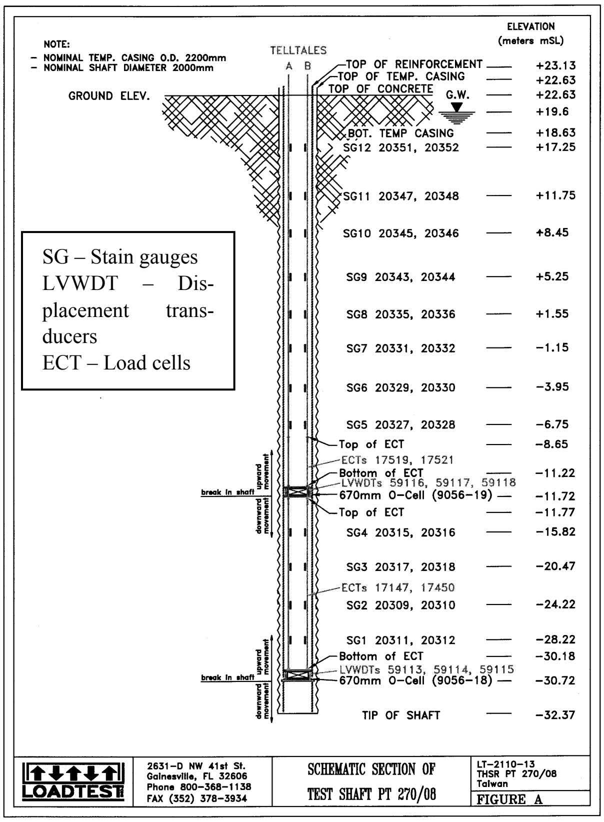

Figure 2 Typical test pile arrangement

4 ANALYSIS OF THE PILE TESTS

Boreholes and CPT investigations were carried out at the site of each test pile. Pile design was to be carried out considering each SPT N value to be representative of the soil for the length of pile mid way to the adjacent SPT tests. No attempt was made to 'average' the values or to produce 'design' lines for soil strata. The SPT N profile was specific to each pier location The process is roughly equivalent to taking the average value over any depth range. The initial design equations for shaft friction were:

For Non - Cohesive Soils

$q_s = 3.3 x N (kN/m^2)$ (1)

for SPT N < 50 (limiting q_s to 165 kPa)

For Cohesive Soils

$q_s = (100/16) x N (kN/m^2)$ for $N \leq 4$ (2)

$q_s = (21/16) x N + 26 (kN/m^2)$ for $N \geq 5$ (3)

Intermediate values are linearly interpolated, with the limits:

$q_s \leq 100$ *kPa for piles constructed under bentonite* (4)

$q_s \leq 150$ kPa for piles under other supporting fluids (5)

The values of shaft friction calculated using these equations are the ultimate values , to which the specified factors of safety were applied.

The initial design equations used for end bearing or base resistance were:

For sands with up to 40% fines:

Base stress $= 80$ *N kPa for* $20 \leq N \leq 50$ (6)

For gravels with more than 40% fines:

Base stress = 75 N kPa for $20 \leq N \leq 50$ (7)

For gravels with up to 40% fines:

Base stress $= 80$ *N kPa for* $20 \leq N \leq 60$ (8)

For sands with more than 40% fines:

Base stress $= 75$ *N kPa for* $20 \leq N \leq 60$ (9)

For cohesive soils i.e. silts and clays:

*Base Stress = (900/16) * N kPa* (10)

The results of the pile tests in C260 validated the relationships used in the tender designs as shown in equations 1 to 10.

The results from the pile tests in C270 validated the relationship for shaft friction in granular soils shown in equation (1), for sections of pile shaft in contact with cohesive soils the relationships for shaft friction were modified as shown in equations (2a) and (3a)

$q_s = 7.9*N$ (for $N<5$). (2a)

$q_s = 2.7 N + 26$ (to a maximum of 150kPa for $N>= 5$) (3a))

The results from all of the pile tests in C270, are shown in Figure 3. Some data points were excluded as they were considered to be 'invalid'. The invalid data generally came from gauges which were either very close to the O-Cells or from points near the top of the pile shaft, where there was insufficient movement of the pile shaft to mobilize the peak shaft resistance. The design lines are shown in Figure 3, together with the best fit lines calculated from linear regression methods.

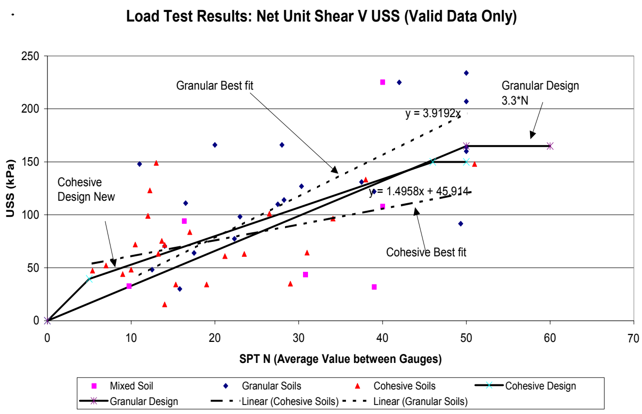

Figure 3. Graphical presentation of processed test pile results from C270

The end bearing or base resistance relationships were also modified based on the interpretation of the test results to be:

$Q_B = 53 * N$ *with a max limit of 2500kPa with SPT N values.*
or
$Q_B = 72 * N_{cpt(R\&C)}$ *with a max upper limit of 2500 kPa.* (11)

The low end bearing capacity of the piles gave some concern and the construction processes for cleaning out the base of the excavation were carefully applied during construction of the production piles to ensure the best possible contact between concrete and the soil at the pile toes.

A different relationship was developed for pile end bearing resistance in terms of an inferred SPT N value derived from CPT investigation results (Eqn 11) The inferred N values from the Robertson & Campenella (1983) method is justified by the fact that the CPT inferred N values were normally lower than the measured SPT N values from boreholes. Where possible the design team avoided founding piles in silts, sandy silts or clayey silts so that the base resistance available was maximized..

The static tension tests carried out, one in each contract, showed that the O-Cell tests are equally valid for predicting the compressive shaft friction and the tension shaft friction.

The analysis of the lateral stiffness of the horizontal pile test showed that the comparison between the actual pile behaviour and that of a numerical model, with stiffness parameters derived from the horizontal modulus of subsoil reaction formulae showed reasonable agreement at maximum anticipated load levels. It was concluded that the horizontal pile tests validated the assumptions made in the Design Manual for horizontal or lateral loadings

5 PILE DESIGN

The design of the piles followed a complex procedure which was set up to ensure compliance with the THSRC Design Specification. The Design Specification set the factors of safety to be applied to the pile shaft friction and base resistance for each of three loading cases as shown in Table 2.

Table 2 Factors of Safety from Design Manual

	Safety Factor		
	Normal Load Case	Exceptional Load Case	Ultimate Load Case
End Bearing Capacity	3	2	1.25
Skin Friction	2	1.5	1.25
Tension	0	2.5	1.5

The pile design specification also detailed that the design of the pile was to include reduction factors for liquefaction based on the Japanese Road Association (JRA, 1996) procedure. To be considered as soil with the potential to liquefy three conditions need to be satisfied:

- Saturated soil layers in the 20m below ground surface where the ground water table is less than 10m below the ground surface,
- The fines content of the sample, F_{MC}, is less than 35% (F_c < 35%) or if the fines content is greater than 35% (F_c > 35%) and the Plastic Index is less than 15% (PI < 15%),
- The mean grain size D_{50} does not exceed 10mm ($D_{50} \leq$ 10mm) and the percentage of soil finer than 10% (D_{10}) is less than 1.0mm ($D_{10} \leq$ 1.00mm).

The JRA procedure makes use of the energy corrected SPT N value to determine the liquefaction resistance factor (F_L). If the calculated value of F_L is less than l.0 the soil is consider to be vulnerable to liquefaction under the specified ground accelerations. A reduction factor (D_E) which is applied to soil strength and stiffness in design calculations can be taken from Table 3

Table 3 Reduction Factors for Coefficient of Subgrade Reaction

F_L	**Depth from ground surface (m)**	**Reduction Factor D_E**	
		$R \leq 0.3$	**$0.3 < R$**
$F_L \leq 0.33$	$0 \leq z \leq 10$	0	0.167
	$10 < z \leq 20$	0.33	0.33
$0.33 < F_L \leq 0.67$	$0 \leq z \leq 10$	0.33	0.67
	$10 < z \leq 20$	0.67	0.67
$0.67 < F_L \leq 1.0$	$0 \leq z \leq 10$	0.67	1
	$10 < z \leq 20$	1	1

In the Project Design Specification (THSRC 1999) reference is made to a parameter T_g, the first mode period, which is used to determine if the area around any particular borehole is to be categorised as a 'particular site' and the design characteristics to be applied if the area is a 'particular site'. The values of T_g were included in a report prepared by the National Centre for Research on Earthquake Engineering (NCREE, 2000). The extent of the site related to each T_g value and hence designated as a particular site was defined in the NCREE report.

The definition of a particular site was given in the Design Specification as Tg > 1.00 seconds, the precise cut off point is based upon the analysis of the factual soils data. A sensitivity check of some piers where the Tg value was close to 1.0 was carried out by the designers to check if the sudden change from a normal to a particular site gives a marked difference in the structural form.

The design process also included considerations for 'near fault effects' where the route either crossed or was close to known active faults. In the near fault areas the seismic coefficient was increased as defined in the Design Specification.

The potential effects of scour at river crossings was included for piers located between river flood banks. Scour depths of up to 10m were included in rivers where the most sever scour can occur.

In some areas of the route the ground profiles can lead to 'flow slides' occurring during earthquakes due to soil liquefaction. The geometry of sites with the potential to develop flow slides was checked and where appropriate the lateral effects of flowing liquefied soils was included in the pile designs.

6 CONCLUSIONS

The designs of the piled foundations in contracts C260 and C270 have been carefully prepared and checked in detail by the Contractor's Independent Checker, Moh & Associates, overseen by the client and their independent checkers the International Rail Engineering Group (IREG).

The ground investigations and the testing of the non-working piles has allowed the deign process to go forward with confidence that the piles will be safe, will meet the requirements of the Design Specification and will represent an economic solution for the contractor.

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