Geotechnical Issues in Design and Construction of Viaducts of the Taiwan High Speed Rail

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Abstract: With a total length of 345 km and an estimated construction cost of US$15 billions, the Taiwan High Speed Rail Project is considered to be one of the largest BOT (Build-Operate-Transfer) projects in the world. The route of the Rail runs through the populated west coast of Taiwan Island and links the City of Taipei at its northern end and City of Kaohsiung at its southern end. The geological conditions along the route vary tremendously, from rocky mountains in the north to thick sedimentary deposits in the south and, as a result, the northern section is constructed mostly by cut-and-fill, with bridges and tunnels, while the southern section is mostly elevated. Discussed herein are some major geotechnical issues in the design and construction of viaducts in the southern section with considerations primarily given to pile load tests, negative skin friction on piles, land subsidence, scouring and protection of riverbed, etc. Furthermore, as the route crosses three active faults, soil liquefaction potential and characteristics of ground motions have significant influences on the design of piles and are also discussed.

1. INTRODUCTION

With a total length of 345 km and an estimated cost of construction of US$15 billions, the Taiwan High Speed Rail Project (the THSR Project, hereinafter) is considered to be one of the largest BOT (Build-Operate-Transfer) projects in the world. The route of the High Speed Rail (the Rail, hereinafter) runs through the highly populated western coastal plains of the island and, as depicted in Fig. 1, links many major cities. With a design maximum speed of 350 km/hr, passengers will be able to make the trip from Taipei to Kaohsiung in 90 minutes instead of four and half hours by normal trains or cars. With an anticipated ridership of 300,000 per day, the Rail will certainly become a major transportation artery of the island. Once the Rail is open for revenue services, it is expected that traffic congestion will be a distant memory. The Rail is also expected to boost the national economy because of the increase in productivity as a result of better efficiency in transportation and redistribution of population and resources.

Taiwan is located at the Circum-Pacific earthquake belt. In fact, the island was created by the subduction of the Philippine Sea Plate underneath the Eurasian Plate. An annual movement of 7cm has been observed and, as a result, a complex geological structure is formed and geological conditions vary drastically from one place to another, as shown in Fig. 1. The foundations supporting the super-structures of the Rail also vary from place to place along the route. In the northern section, which accounts for one-fourth of the route, the terrain is mountainous and the Rail line is mostly constructed by cut-and-fill with bridges across rivers and tunnels through mountains. The rest of the route runs through the coastal plain and is mostly elevated with viaducts founded on piles.

Because of the differences in ground conditions and different methods of construction, geotechnical issues to be considered are also different from section to section. For the northern section, the major considerations are slope stability and prevention of geo-hazards, and for the rest of the route, economical design of piles is of primary importance. Discussed in this paper are the various geotechnical issues related to the design of the viaduct foundations, including pile loading tests, effects of liquefaction, ground subsidence, characteristics of ground motions and scouring of river bed, etc., on piles.

2. DESIGN REQUIREMENTS

Because of the high speed of trains, design criteria for the Rail are more stringent than those adopted for other structures. It is stipulated in design specifications that settlements of piles due to all the superimposed dead loads, including trackwork, shall be accounted for in design to ensure that the vertical alignment of the rails meets the criteria. To ensure that trains can run at their maximum speed of 350 km/hour, changes in the vertical alignments of rails as a result of differential settlements between neighboring piers after the completion of construction shall not exceed 1:1000 for simply supported spans and 1:1500 for continuous spans.

As mentioned above, the Taiwan Island is located on the Circum-Pacific Earthquake Belt, it experiences several major earthquakes annually. The devastating Chi Chi Earthquake of 1999 took more than 2,000 lives. Therefore seismic loads are major factors to be considered in designing infrastructures. For the THSR Project, two levels of ground shaking are considered:

(a). Type I Earthquake - damages are repairable
Type I Earthquake corresponds to earthquakes with a return period of 950 years, which has a 10% probability of exceedance in 100 years. In Type I earthquakes, structures are allowed to yield but damages to structures, if any, shall be repairable. The design peak ground accelerations (PGA) in the horizontal direction vary from 0.22g to 0.40g along the THSR route for Type I Earthquakes and the peak ground accelerations in the vertical direction are two-third of those in the horizontal direction.

(b). Type II Earthquake – trains can run at maximum speed and structures do not yield
Structures are not allowed to yield in earthquakes of this level and the operation of the system subsequent to earthquakes is expected to be unaffected. In order to make sure

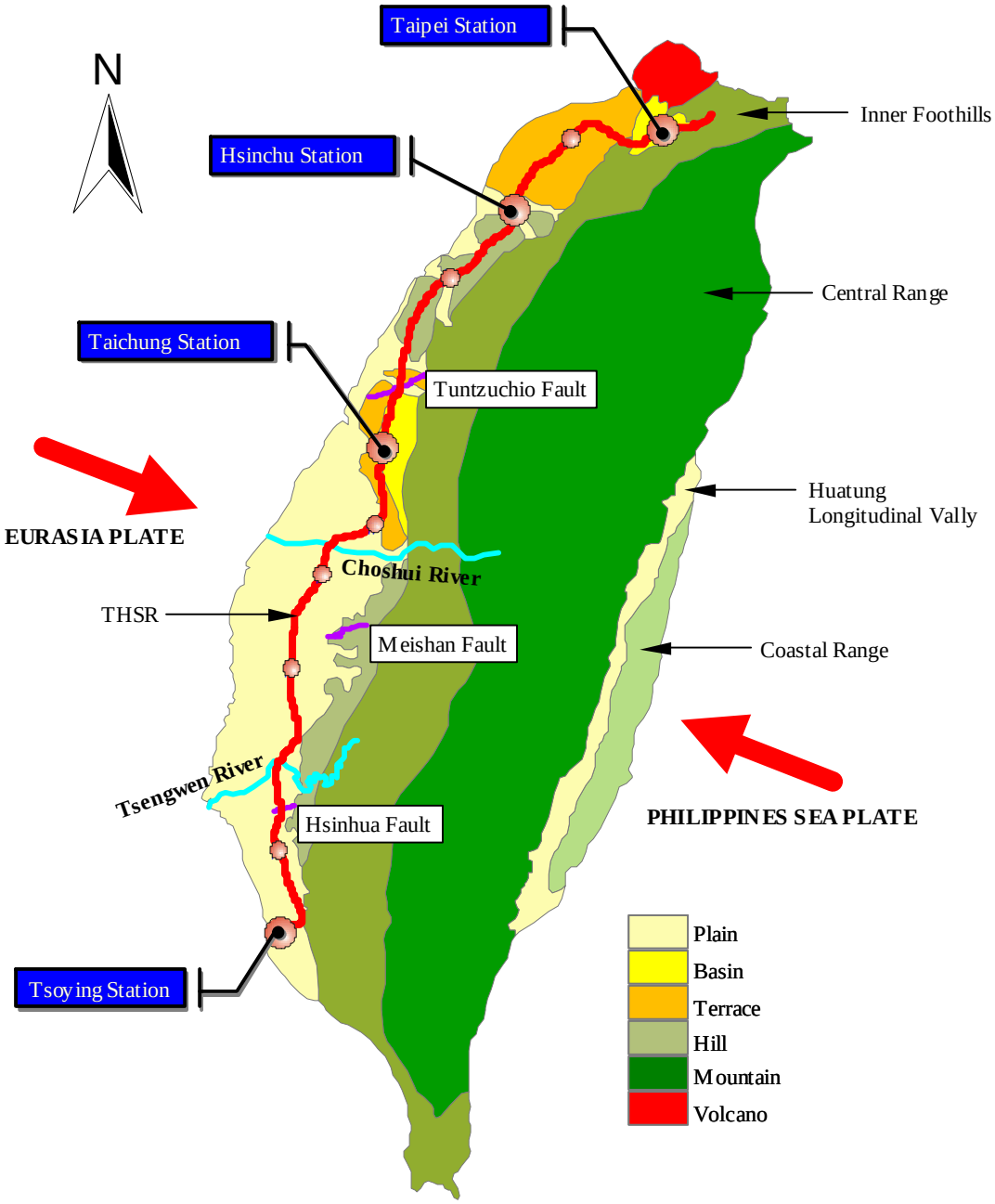


Fig. 1. General geology of Taiwan Island and route of the High Speed Rail.

that trains still can run at their full speed of 350 km/hr, it is mandatory to inspect the deformations of tracks after earthquakes and make sure they are within allowable ranges. The design PGA in the horizontal direction for Type II earthquakes are one-third of those specified for the Type I earthquakes and the design ground acceleration in the vertical direction are two-thirds of those in the horizontal direction. Under Type II earthquakes, maximum lateral displacement shall not exceed: 50mm tilt from vertical axis at top of caissons, 50mm relative displacement between pile head and pile toe, and 50mm relative displacement between barrette head and barrette toe.

In the Chi Chi Earthquake of 1999, many bridges collapsed with their decks falling to the ground. Learning from this lesson, design codes have been revised with considerable improvements on structural details. Shear keys, longitudinal restrainers and transverse girder stoppers are provided to prevent bridge decks from falling. Furthermore, emphases have been given to ground motions near faults.

3. PILE FOUNDATIONS

Shallow foundations are usually more economical to use and are used where there is a suitable bearing stratum at shallow depths and settlements are expected to be within tolerance. Deep foundations are used when shallow foundations are unsuitable or uneconomical. However, for the viaduct section of the THSR, with a total length of 244km, piles are exclusively used to support the superstructures because of (i) presence of thick alluvium soil deposit, (ii), large earthquake forces, and (iii) limited right of way (generally only 18m).

3.1 Design of Pile Foundations

Bearing capacities of piles are doubtlessly the primary consideration for pile design. According to the design specifications, to ensure piles to have sufficient end bearing, piles are required to enter the bearing soil formations for at least 2.5 m. In rock formations, the minimum penetrations of piles required vary from 0.5 m for rocks with compressive strengths of 5 MN/sq m to 2.5 m for rocks with compressive strengths of 0.5 MN/sq m. If there is a compressive stratum underlying the bearing soil formation, the thickness of this bearing soil formation below the toe of the pile shall be adequate to resist punching and settlement of the pile is required to be within the specified limits. Notwithstanding, in no case shall this thickness be less than three times of the diameter of the pile.

Since both physical and engineering properties of the subsoils have profound effects on the behavior of pile foundation and the THSR route passes through variable subterrain conditions, it is important to understand the effects of soil parameter on pile foundation in order to arrive at economic designs within relatively short period of time. Parametric studies of sensitivity of pile design to soil parameters were carried out. Figure 2 illustrates one case of such study. A group of 4 piles with a same diameter of 2 m and same length of 59 m was subjected to a lateral load of 1,200 tons which corresponds to the ultimate load case incorporated with the Type I earthquake load. Piles are spaced at a distance of 8.5m center to center and a factor of 0.78 is adopted for group effects. The connections between the piles and the pile cap are assumed to be rigid and rotation is not allowed.

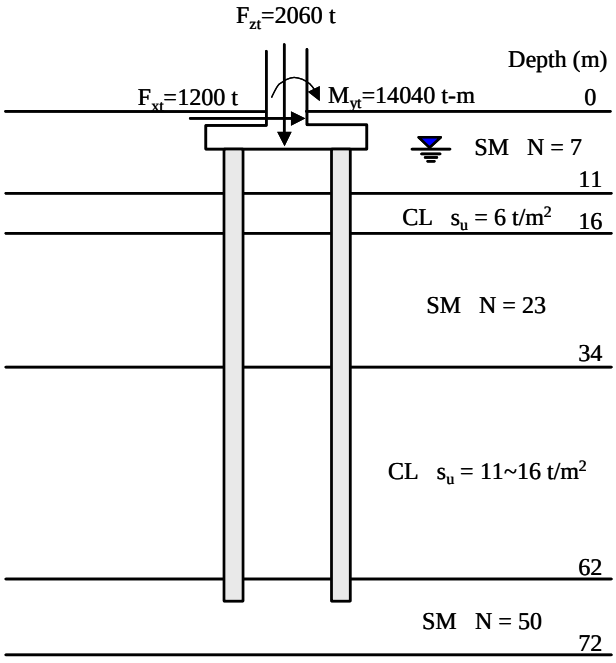


Fig. 2. Case for a sensitivity study of soil parameters on design of pile foundation.

The ground consists of alternating layers of sand and clay as shown in the figure. If the soil strength and stiffness, represented by N values of Standard Penetration Tests, for the top 10 m below ground surface decrease to one-third of their original values, then it is necessary to increase the pile length by about 4m and to increase the amount of steel reinforcement in the pile head by 50 per cent in order to satisfy the design requirements. On the other hand, if the soil strength remains unchanged but the horizontal soil stiffnesses decreases to one-third of their original values along the full length of piles, the lengths of piles have to be increased approximately by about 2m and the steel reinforcement in the upper portion of the piles has to be increased by 40 per cent. The results are summarized in Table 1.

Table 1. Pile design as affected by changes in soil parameters.

Change in soil parameters	Pile Length	Reinforcement
Decrease N and kh in top 10m to 1/3	+ 4m	+ 50%
N unchanged, kh for the full length reduces to 1/3	+ 2m	+ 40%

Response of a full structure-foundation-soil interaction system is usually analyzed by replacing soils with equivalent springs, including horizontal spring (k_h), vertical spring (k_v), rotational spring (k_r) and torsional spring (k_t). The same pile group is used as an example to demonstrate the sensitivity of design of superstructure to soil parameters. For bridge piers with 8m, 12m and 16m in heights, the amount of steel reinforcements required is 2.1, 2.4 and 2.7 per cent, respectively. Decreasing the stiffness of horizontal springs by 4 times will reduce the amount of steel reinforcement required in the bridge pier by 21, 3 and 0 per cent respectively. Decreasing both the stiffness of vertical and rotational springs by 4 times will decrease the amount of steel reinforcement required by 17, 14 and 3%. The results are summarized in Table 2.

Table 2. Reinforcement required in bridge piers as affected by changes in equivalent soil springs.

Change in soil parameters	Reinforcement in piers		
	Pier ht. 8m	Pier ht. 12m	Pier ht. 16m
kh decrease by 4 times	– 21%	– 3%	– 0%
kv and kr decrease by 4 times	– 17%	– 14%	– 3%

An analysis is also performed for the case of 8m high bridge pier and it is found that the results are not sensitive to changes in torsional spring (k_t).

The above analysis indicates that decreasing soil stiffness will increase the reinforcement required in piles but will decrease the reinforcement required in piers. The use of conservative soil properties is not necessarily always conservative. Therefore, the soil parameters adopted in the design should be obtained from representative soil strata and reliable in situ and laboratory test results. Local experience and back analyses from full scale loading tests and monitoring results often play important roles in the foundation design.

3.2 Construction of Piles

For a 155 km section in the mid to south section of the route, more than 20,000 piles have been installed for supporting the viaducts, mostly 1.5m to 2.5m in diameter and 35m to 72m in length. Piles with 1.8m and 2.0m in diameter and 50m to 60m in length are most common. Because the variations in ground conditions along the THSR route, different construction methods are adopted for the installation of piles. Due to restriction on noise

and vibration, pile driving is banned and all the piles are cast-in-place piles. In the north segment of the viaduct section, the route mostly runs along foothills and the ground consists of mostly terrace deposits with a layer of gravel at surface underlain by layers of sandstone or mudstone. Bored holes were protected by casing and dug by hammer grab (within gravel layer) or drilling bucket (within rock). In some cases, the holes were fully cased and in others holes were cased only to the top of bearing stratum.

Toward the south, the major soil compositions are interbedded sand and clay. Piles are generally installed by using the reverse-circulation piling method for the reason that the method is popular in Taiwan and a large number of piling machines are available. In the Tainan and Kaohsiung areas, mudstone layer and sandstone layers exist at the toe levels of piles. Although all the machines were capable of penetrating into mudstone and sandstone, the progress of drilling was rather slow.

In some local areas, limestone was encountered and escape of drilling mud was a problem. Grouting had to be carried out to seal off water paths and sealant (e.g. wood scraps) was added in the drilling mud to reduce leakage. For limestone layer over 5m in thickness, steel casing with shoe was used for the full thickness of the limestone layer to contain drilling mud and improve the efficiency of construction.

At the peak of piling, there were more than 100 sets of piling machines working along the route at the same time. In addition, because the schedule is extremely tight, in most of cases, design had to go parallel with construction and confusion might occur as a result. This led to the concern on the quality of works. To ensure the quality of works, first of all, as a rule, a check boring had to be made at the location of each pier to reveal the local ground conditions and to determine the required length of piles to be installed. This is particularly important for piles of which significant portion of their capacities come from end bearing. Furthermore, it is required by the specifications that Designers and Contractor's Independent Checking Engineer (CICE) must certify that the geotechnical conditions actually encountered are consistent with those assumed in the designs and the Designers must modify their designs if not. Any modifications in design should be certified in writing by the CICE in accordance with the Quality Plan.

3.3 Integrity Tests of Piles

Honeycombing, segregation and necking are common problems with bored piles and presence of soil and cracks in pile shafts may also occur occasionally. For the THSR project, as a rule, over 20 per cent of the total number of piles were subjected to non-destructive integrity tests. In addition, piles suspected to be defective as indicated by piling records were also subjected to tests. If test results indicated that the pile might be defective, the test results should be provided to the Designer to evaluate the influence of these defects and take necessary remedial measures. Sometimes, continuous coring was performed to confirm the quality of the piles in doubt.

3.4 Pile Load Tests

3.4.1 Tests for bearing capacities

It is stipulated in the specifications that an appropriate number of advance piles have to be tested to failure to verify the design assumptions and to confirm the ultimate bearing capacities of piles. The locations and lengths of test piles were determined in considerations of pile type, pile capacity, and soil conditions. Generally, for each geotechnical unit with similar ground conditions, one out of every 1,000 piles was tested. Conventional loading tests were normally carried out to loads of, up to, 4,400 tons. Test piles were instrumented with rebar stress transducers mounted at different depths along their shafts for measuring the strains induced together with displacement transducers at top for measuring total settlements. With sufficient instrumentation, it was possible to compute distribution of skin friction and end bearing of the piles from the test results.

Osterberg Cells (O-Cell) were used in a few occasions as an alternative to kentledges as loading devices. Other methods, such as Dynamic Pile Load Test Method and Statnamic Load Method were acceptable, however, their validity had to be verified by performing conventional loading tests on piles with similar ground conditions. In tests using Osterberg cells, as shown in Fig. 3, piles were cast in three sections with two cells installed between neighboring sections for jacking against each other. The bottom sections were very short so the frictional resistance could be to

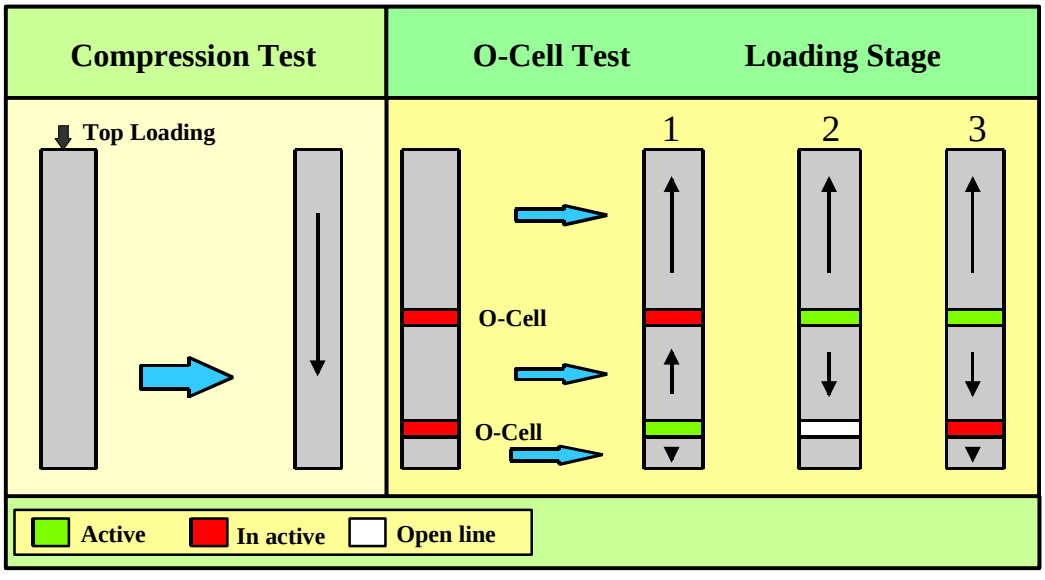


Fig. 3. Pile load tests using Osterberg Cells.

tally ignored. In the first stage of test, the upper cell were locked so the upper and the middle sections moved together. The lower cell was pressurized and the end bearing of pile could be determined with the friction on the upper and the middle section of pile as reaction. In the second stage of test, the lower cell was released so there would be no reaction from the bottom section of pile simulating soft-toe. The upper cell was pressurized and the upper and the middle sections of piles moved in opposite directions and the frictional resistance on the two sections could be determined separately. In the third stage of test, the lower cell was locked so end bearing came into play and allowed the frictional resistance on the upper section of pile to fully develop.

The use of Osterberg cells has the great advantage that the ultimate end bearing capacity of full-scale piles can be measured. Cells installed at the bottom of test piles were able to mobilize end bearing of a pile with the frictional resistance on the full length of piles as reaction. The position of the cells were determined in consideration of ground conditions so that cells would be capable of mobilizing the full skin frictions for the upper and lower sections of the test piles. Skin frictions of piles were grossly underestimated when test programs were prepared and in many cases the cells were incapable of mobilizing the skin friction on piles to the full extent. The frictional resistance on piles was re-estimated and the positions of cells were adjusted in later stages based on the results obtained previously.

It has been a well known fact that workmanship has profound influences on bearing capacities of piles. Relaxation of geostress tends to reduce the strength of soil with time, it is thus desirable to cast piles as soon as possible. Furthermore, water tends to soften soils and usually bentonite is used for stabilizing the side wall. It has been found that better skin friction can be obtained if polymer is used to stabilize the holes. The use of polymer is getting more and more popular. Test results indicated that a 50 per cent increase in skin friction of sandy soils could be achieved if polymer is used and concrete is promptly cast through reduction in caking effect.

The end bearing capacities of piles are greatly affected by the construction method and quality of construction as well. Soft toe is a common problem for cast-in-situ piles. The problem of soft toe caused by sedimentation of soil at the bottom of excavated holes can be solved via injecting cement grout at high pressure in stages to strengthen the weak zone surrounding the pile toe. With pressure grouting at toe, it has been found that the end bearing capacities of piles are much improved and settlements of piles are much reduced as shown in Fig. 4. The effects of pressure grouting for piles embedded in clays are, however, less certain.

For ensuring the quality of construction and for confirming that the settlements of working piles are within required design limits, an average of 0.5 per cent of working piles were tested. Conventional static tests were carried out to loads of, up to, 2,200 tons. However, some of the contractors opted dynamic load tests or Statnamic load tests instead of conventional loading tests and performed comparative studies beforehand to establish correlations between the results obtained by using different methods.

3.4.2 Pile resistance in tension

To determine the tension resistance of piles, conventional tension tests were carried out individually on piles located neighboring those for compression tests. The results indicate that the ratio of ultimate tension capacities to compression capacities without end bearing ranging from 0.62 to 1.05 (62 to 105 per cent), with a mean of 81 per cent and a standard deviation of 16 per cent.

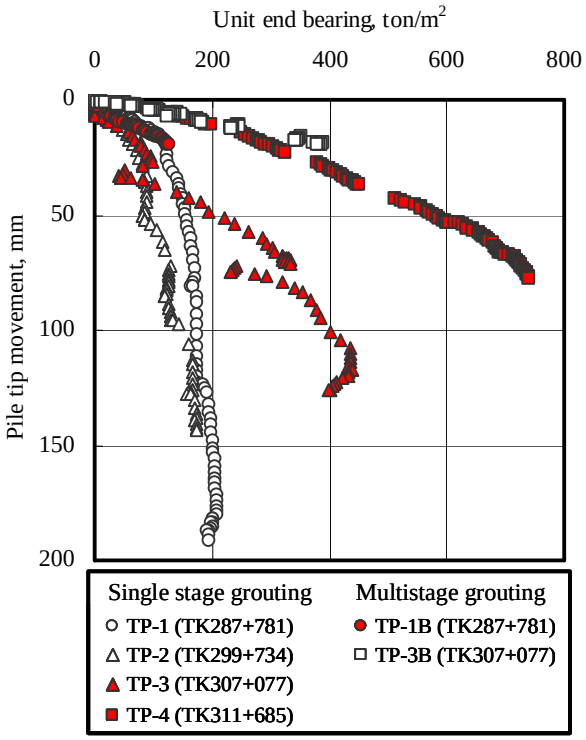


Fig. 4. Effects of the grouting on end bearing of capacity of piles.

These ratios are much greater than the limit of 40 per cent stipulated in the design specifications. It may be worth the effort to reconsider this limit if tension capacity is a governing factor in design of piles in the case due to seismic loads. If this limit of 40 per cent were raised to 50 or 60 per cent, considerable savings could be achieved. In such cases, the governing factor will then be compression, instead of tension, capacity of the piles. For river crossing bridges, the lengths of piles are generally dictated by the compression capacity due to existence of relatively thick overburden and consideration for scouring.

As mentioned in above paragraphs, Osterberg Cells were used in several cases for pile loading tests. In those tests, the upper sections of the test piles were subjected to upward loads at their bottom and practically behaved as tension piles. Therefore the skin friction mobilized tend to be lower than those obtained in the conventional loading tests. The ratios of ultimate tension capacities of piles from conventional tension tests to the compression capacities in tests using Osterberg Cells range from 1.0 to 1.14 (100 to 114 per cent).

3.4.3 Lateral pile load tests

Lateral pile load tests were conducted in ground with different soil conditions for determining horizontal modulus of subsoil reaction and to verify the design parameters. Horizontal displacements and lateral loads were measured by using displacement transducers and load cells, respectively. An inclinometer was also installed in each test pile to determine the profile of horizontal displacements along the pile during the test. Test results revealed that the responses of test piles are quite similar to those predicted by the computer programs LPILE as shown in Fig. 5. It is therefore concluded that the input soil parameters are reasonable and the program LPILE is a useful tool for predicting response of piles.

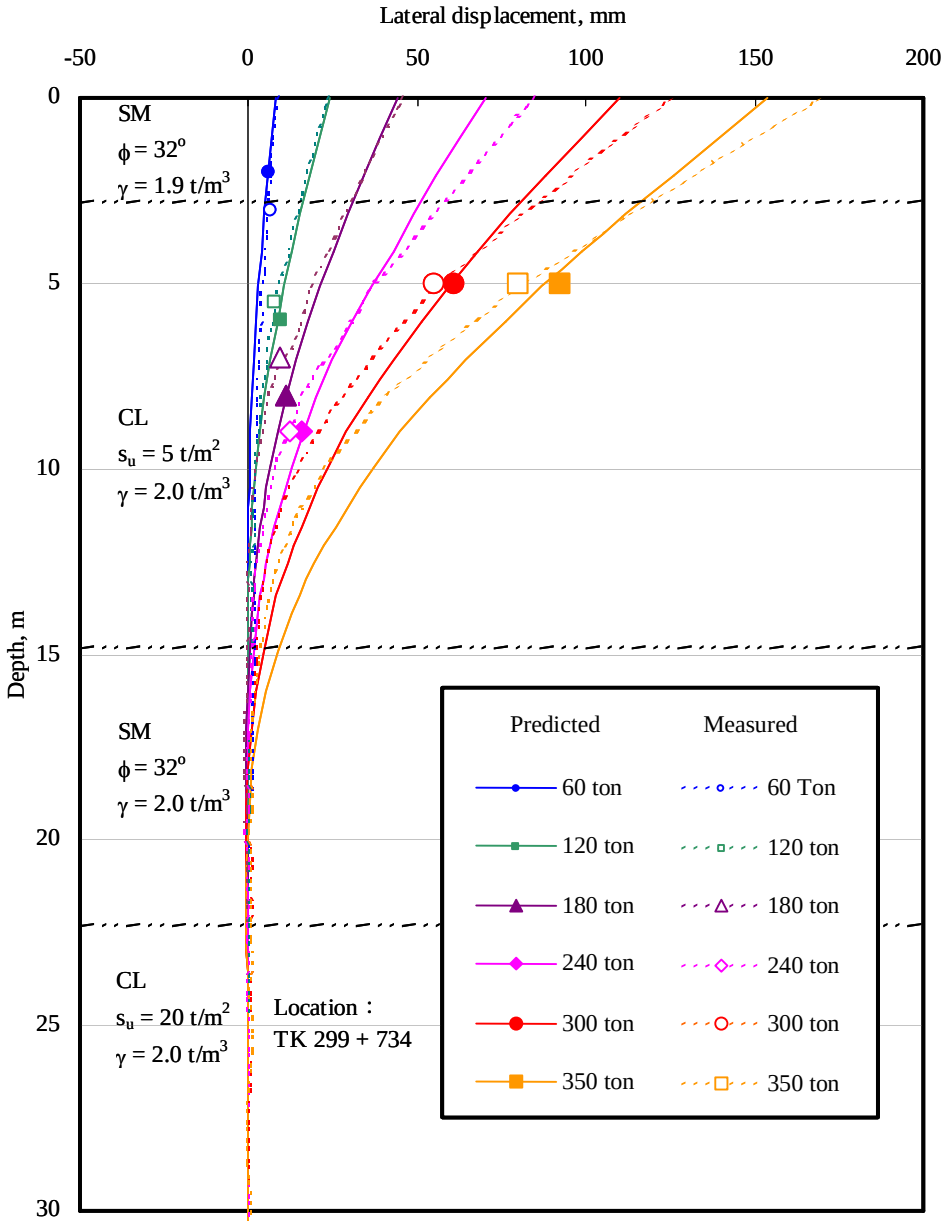


Fig. 5. Comparison of lateral pile load test results with predicted values from LPILE program.

4. SOIL LIQUEFACTION

During strong earthquakes, pore water pressure in saturated soils may increase due to the application of cyclic shear stresses induced by ground motions and cause reduction of effective stresses leading to partial or total loss of shear strength of soils. The soil deformation accompanied may be limited (cyclic mobility) or unlimited (liquefaction). According to the records of the Central Weather Bureau in Taiwan, there were at least 5 earthquakes in which soil liquefaction was observed in the southwestern part of Taiwan in the last century, therefore, liquefaction is a real threat. As the consequences of liquefaction, loss of foundation support, flow slides, slumping, lateral spreading, ground subsidence, etc., may occur. For the THSR project, contractors are specifically required to pay attention to liquefaction potential of soils and take it into consideration in foundation design. If liquefaction is indeed a potential threat, either the liquefiable soils be removed or piles be used to transfer loads from structures to a stable stratum.

4.1 Evaluation of Liquefaction Potential

Basically, liquefaction potential is expressed in terms of the ratio of soil resistance to the shearing stress induced. The average earthquake-induced shearing stress is usually estimated from the expected peak ground acceleration at the site using empirical correlations with earthquake magnitudes. Regarding soil resistance to cyclic loadings, it is normally determined either by laboratory

tests, such as cyclic triaxial or simple shear tests (the so called analytical method of evaluation) or by in-situ tests such as standard penetration tests and cone penetration tests (the empirical method of evaluation).

The commonly used simplified methods for liquefaction evaluation in Taiwan are those proposed by Seed et al (1985), Tokimatsu & Yoshimi (1983), Japan Road Association (1996), and National Center for Earthquake Engineering Research, USA (Youd & et al, 2001). According to a study conducted by the National Center for Research on Earthquake Engineering, Taiwan for Chi-Chi Earthquake in 1999, the results obtained by using the method proposed by the National Center for Earthquake Engineering Research, USA in 1998 (Youd et al, 2001) gives better correlation with the field observations in comparison with others. However, for the THSR project, the method proposed by the Japan Road Association (1996) is adopted. This method has incorporated the experience from the Hyogoken-Nanbu Earthquake in 1995 and has the merits that reduction in soil strength as liquefaction develops is accounted for.

It is a well known fact that characteristics of ground motions at a site are affected by the proximity of the site to faults. Furthermore, the characteristics of faults are important to be considered in estimating ground motions. In the early days, studies were mostly based on data obtained in California where San Andreas Fault was responsible for several major jolts. Therefore, expressions for attenuation of ground motions established previously are more applicable to motions due to rupture of strike slip faults. It is only in recent years that ground motions generated by rupture of thrust faults drew attention from researchers. In earthquakes due to rupture of thrust fault, such as the Chi Chi Earthquake of 1999, ground motions on the two sides of the faults are drastically different. The THSR route crosses 3 major faults and ground motions near these faults could be significantly different from those far away from the faults.

During the design, the National Center for Research on Earthquake Engineering was engaged by the Designer to study ground motions for the section of the Rail route near the Hsinhua Fault. (Moh and Associates, 2001). Figure 6 shows the Boore-Joyner-Funel near field attenuation of ground motion with distance from the fault, as well as the Campbell's relationship using Taiwan's local strong motion data. Based on the results of this study, it is recommended that when the distance is less than 2.5km from the fault line, the near source effect should be considered. In other words, over a distance of 5km, the viaduct design shall be based on the near-field seismic hazard. However, the THSR specifications also refer to the use of the method proposed by the Japan Road Association (1996). The JRA method takes into account of the effect of in-situ stress which has unique properties of higher seismic intensity and lower number of cycles due to near source. Therefore, by using the JRA method, although the PGA is increased from 0.34 to 0.50g, liquefaction potential at the site in fact becomes even slightly lower.

It should be noted that notwithstanding the fact that soil liquefaction potential does not increase at locations near faults, the seismic loads on structures do become larger because of increased PGA. Taking the case of the Hsinhua Fault for example, for a bridge pier foundation consisting of 4 numbers of piles of 2.0m diameter, the designed pile length increases from 59m to 61m and the amount of reinforcing steel increases by 8 per cent when the near fault effect is considered.

4.2 Soil Properties Affected by Soil Liquefaction

Where potential for liquefaction exists under the excitation associated with either the Type I or Type II earthquakes, the liquefiable soils shall be removed, or soil improvement techniques be used, or deep foundations such as piles or caissons be used. The design should consider reduced soil strengths depending on the value of liquefaction resistance factor F_L and the depth of soil susceptible to liquefaction.

For piles, reduction in soil strengths will lead to decrease in bearing capacities. In the mean time, the reduced soil stiffness will induce larger lateral movements and larger bending moments when piles are subjected to lateral force due to earthquakes. Consequently, the pile length, reinforcement, and even the pile diameter have to be increased. This has significant effect on the pile design. The example of soil parametric study discussed in section 3.1 clearly illustrates such effects.

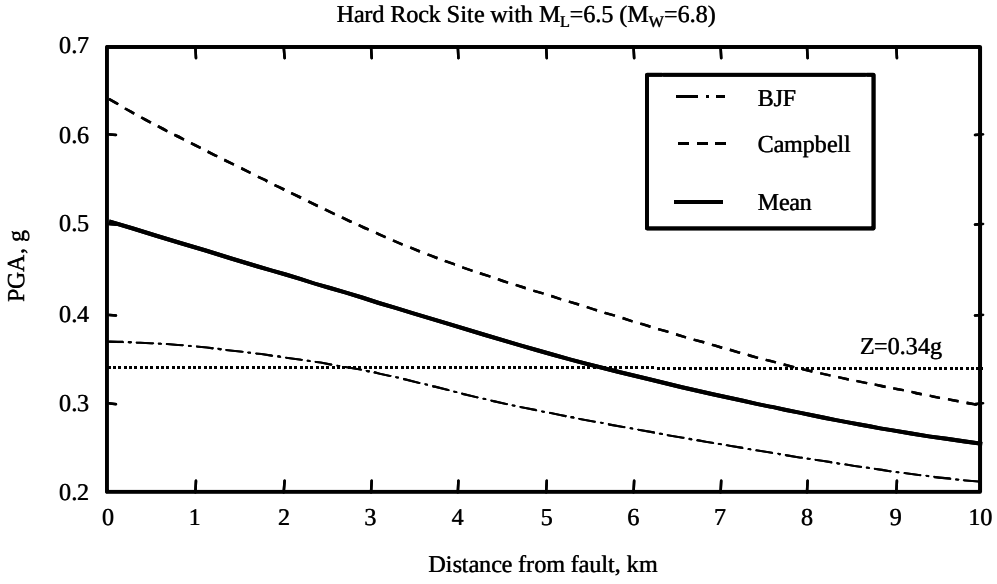


Fig. 6. Near-field ground motion attenuation with distance from fault.

4.3 Negative Skin Friction Caused by Soil Liquefaction

The design of piles needs to take into consideration the effects of negative skin friction resulting from dewatering or liquefaction of surrounding soils. Analysis should be carried out to compute ground subsidence caused by dissipation of excess pore pressure within liquefied soil layers after an earthquake and the resultant negative skin friction on piles. If negative skin friction were developed, it is considered as an addition to the working load as stipulated in the THSR design specifications. However, the Chinese Building Codes issued by the Ministry of Interior(1998a, b) stipulate that negative skin friction due to liquefaction is considered as short-term forces and is not necessarily to be combined together with other short term forces, such as wind loads, impacts, and traffic loads, at the same time. Therefore, checks are undertaken separately. In general, it is found that for the THSR Project, negative skin friction does not govern the pile design.

4.4 Flow Slide

According to the THSR design specifications, it is necessary to study the potential influence of flow slide caused by soil liquefaction. Based on Seismic Design Specification for Railway Bridges issued by the Ministry of Transportation and Communications (1999), and codes by Japan Road Association (1996), flow slide should be considered if one of the following situations is encountered:

(a). The differential height between the bottom of the sea (river in the case for the Rail) and seashore (or riverbank) revetment exceeds 5m.

(b). The thickness of liquefiable soil within 100 m from seashore (riverbank in the case for the Rail) exceeds 5 m and the liquefaction soil is continuous.

Because most rivers along the route have more than 5m of height difference between the river bottom and adjacent revetment, effects of flow slide due to soil liquefaction have to be considered in design of the bridge foundations.

The dynamic behavior of pile foundations in liquefiable zone is very complicated because liquefaction advances with time during earthquakes. In the beginning of the earthquake, both reactions and displacements of the pile foundation are mainly controlled or forced by the inertia force transferred from the superstructure. Once after the peak of earthquake, the soil may have already been liquefied and even the phenomenon of flow slide of stratification may start to occur. During this phase, the inertial force transferred from the superstructure is possibly decreased due to the shock absorption effect of soil liquefaction. Displacement of the foundation caused by cyclic seismic loading will obviously increase due to weakening soil condition. After the earthquake stops, the loads on the pile foundations are mainly those due to permanent displacement caused by the flow slide.

From the above recounted interactions between soil and structure under liquefaction of stratification caused by seismic action, it is understandable that different time frame exists between the development of flow slide and occurrence of the peak of earthquake. Therefore, the effect of flow slide does not have to be included or considered as a case in the earthquake forces in order to prevent being over-conservative in design.

According to both seismic design specifications mentioned above, the forces acting on piles and the resultant displacements should be checked for the following three conditions:

(a). no liquefaction or flow slide

(b). only liquefaction occurs

(c). only flow slide occurs

After investigating designs in several cases in the Rail Project, it is found that the forces produced by flow slide are relatively small as compared to the seismic forces. However, when the range and depth of liquefaction are great and the liquefaction is continuous from shallow stratification, the amount of steel reinforcement in the pile head may need to be increased.

5. GROUND SUBSIDENCE

5.1 Ground Subsidence Caused by Dewatering

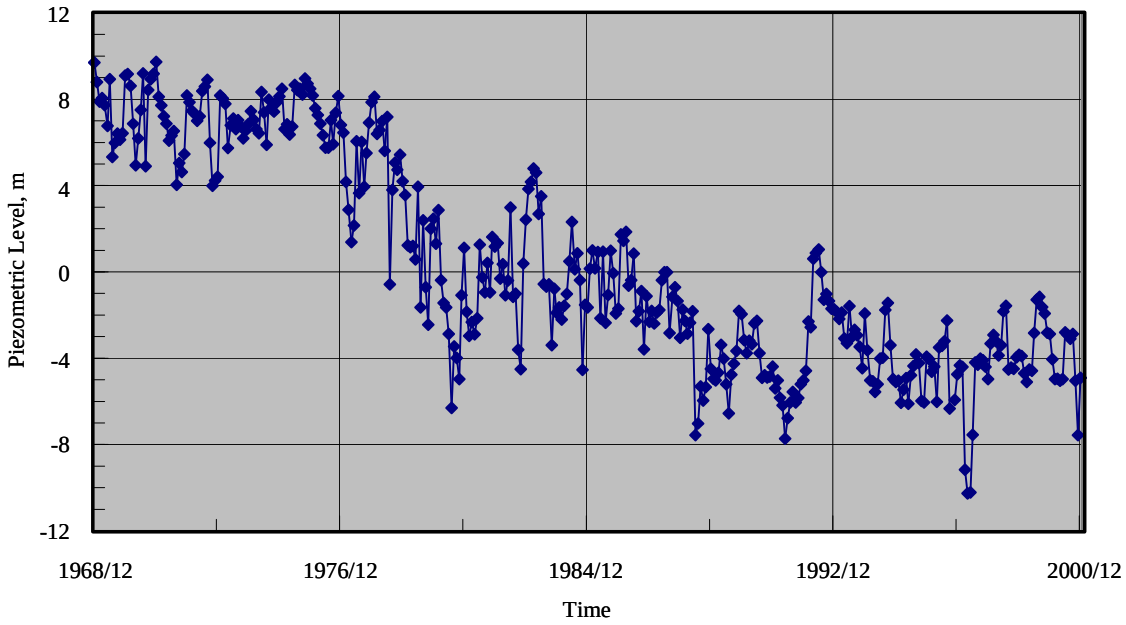
According to a report published by the Water Resources Planning Commission (WRPC) of Taiwan, there were once 200,000 pumping wells in operation on the island, drawing 7.3 billion tons of water from the ground annually, of which 44 per cent was utilized for irrigation, 33 per cent for fish farming, and the remaining 23 per cent for industrial and domestic use. A total of 1,170 sq. km of land has experienced ground subsidences varying from 0.5m to 2.5m. Although dewatering of groundwater by means of pumping wells obviously leads to serious land subsidence along the southwestern shoreline of Taiwan, the influence of land subsidence on the THSR project may be small since the route is located at least 10 km away from the seashore. Notwithstanding, further investigation and exploration are required due to the great significance of the THSR project and the importance of the structures.

Along the THSR route, ground subsidence is the most serious in Yunlin area which is located to the south of the Choshui Creek because of excessive pumping for irrigation and for fish farming. There are no reservoirs in the area, groundwater is the sole source of water supply. The area is generally underlain by thick layers of clay, consolidation settlements become eminent. Figure 7 (Water Conservancy Agency, 2001) shows groundwater levels along the THSR route at Kecuo in the Yunlin area. It can be obviously noted that the underground water level in the area has dropped by 15 m since 1968. Figure 8 (Water Resources Bureau, 2000) shows the accumulative ground subsidence in the two years period of October 1990 to September 2001 in the Yunlin area along the THSR route and the drawdowns of groundwater level in the period from 1968 to 2000. It can be noted that the total land subsidence in the 11- year period vary from 20 cm to 100 cm.

From the data mentioned above, the average subsidence rate is more than 2cm/yr in the Yunlin area in the past ten years. Based on this fact, the influence of land subsidence on pile foundation was studied for estimating the influence of negative skin friction on piles. Furthermore, differential settlements of the bridge piers were checked in order to make sure that design specifications are met. A special study was carried out to evaluate the long term effect of land subsidence on structures along the THSR in the section south of the Chosui River (Moh and Associates, 2002).

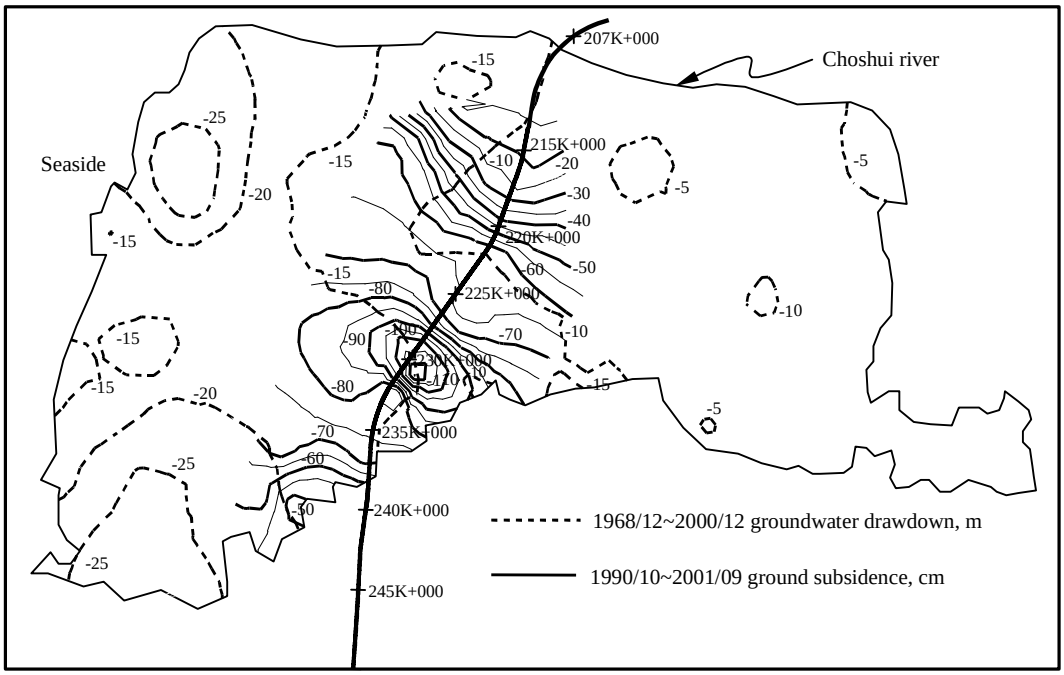
5.2 Effect of Ground Subsidence on THSR Structures

Assessment of potential effects of land subsidence on the safety of THSR structures includes:(a) bridge foundations, (b) vertical alignment and (c) negative skin friction on foundation piles. According to the design specifications, allowable angular distortion between any two points along the THSR should not exceed 1/1,000. Based on measurements carried out by the THSR Corp. in 1998-2000, the estimated angular distortions, except at locations near St. TK245+000, were all below the allowable value.



(Data from Water Converancy Agency, MOEA (2001))

Fig. 7. Monitoring results of observation wells along the THSR route at Kecuo.



(Data form Wtaer Resources Bureau, MOEA (2000) and THSR Corporation (2001))

Fig. 8. Observed ground subsidence and drawdown of groundwater level along the THSR route in Yunlin area.

The THSR design specifications also stipulate that "…Vertical Curves need not be provided in the structure or embankment if the difference in gradient is less than 1.0 per cent in full-speed sections, or if the calculated vertical curves mid ordinate is less than 10mm in lower-speed sections…". Since the calculated angular distortions along the THSR route at the present are all less than the specified limit, no adjustment of the vertical alignment appears to be needed.

According to the design standards for Japanese National Railways, design of foundation piles should consider effect of negative skin friction when the rate of land subsidence exceeds 2cm/year. When the subsidence rate is more than 4cm/year, 100 per cent negative skin friction should be considered.

For the section of the THSR considered, between TK218+000 and TK+237+000, the average rate of land subsidence is between 0.5 and 16.0 cm/year, development of negative skin friction is a

potential problem. However, it is important to identify the source of land subsidence caused by withdrawal of groundwater. If the land subsidence as measured by the settlement of ground surface, is caused by consolidation of deep soil strata, negative skin friction may not need to be considered. Results of analysis indicate that about 30 per cent of the consolidation settlement occurred within the depth of most of the piles and it is less than 2cm/year. It is therefore concluded that influence of negative skin friction due to ground subsidence on bearing capacity of pile foundation need not be considered.

Analysis has also been conducted to determine whether the differential settlement are within tolerable allowances. Based on the observational data and results of analyses, it has been found that if excessive pumping is continuing for a long period to come, the safety of THSR operation may be endangered. Pumping thus has to be strictly regulated and monitoring of groundwater table is crucial for ensuring that the situation will not deteriorate beyond control.

6. FOUNDATIONS IN RIVERS

The Rail crosses several major rivers along its route and design specifications require that the safety of foundation shall be checked against scoring of riverbed. These rivers usually are dry but are quickly flooded in heavy rains. The annual rainfall on the island is more than 2000mm and a significant portion of this amount is caused by a number of typhoons in summers. Downpours frequently resulted in landslides and debris flow in mountains and some of them were fatal with heavy casualties. These rivers are very short with large gradients. As the water rushes toward the sea through plains, it scours riverbed and bridge foundations and quite a few bridges collapsed in the past as a result.

Scouring is a dynamic phenomenon and may occur as a gradual event with predictable amount annually or as a rapid event with sudden formation of large cavities under bridge piers within hours. Furthermore, how the materials will shift from place to place will depend on the configuration of bridge foundations and the terrain of the ground and is rather difficult to predict. Take the Tsengwen Creek as an example, a long-term scouring depth of 0.6m is expected and depth of local scouring is expected to be 5.41m. Furthermore, a depth of 2m is expected for temporary shifting of the river bed.

According to specifications issued by the Water Conservancy Agency of the Ministry of Economic Affairs (2001a) and the design specifications for the THSR project, structures should be founded at depth below the lowest point along the cross-section of the river and below the depth of long-term scouring at the location of the foundation, whichever is lower. This requirement is reasonable in normal cases but appears to be much too conservative if it is combined with requirements for other temporary conditions, such as earthquakes. It is also stipulated in the design specification for the project that the depth of scouring to be considered is only half of its value during earthquakes. Similarly, during earthquakes, a return period of 1 year of flow is adopted in determining the hydrodynamic loads on foundations of bridge piers, instead of a return period of 200 years for major rivers and 100 years for minor rivers for cases without earthquakes.

The structural design of substructure systems shall take both the existing ground elevation and the planned riverbed elevation after scouring into consideration. When pier foundations are constructed in water, it is only necessary to bury the foundations below the riverbed by 0.6m if protective measures are taken against scouring. However, such protective measures must not raise the water to a level which would cause concerns on safety of flood control works or the ecological environment of the river.

7. CONCLUDING REMARKS

The Taiwan High Speed Rail Project is a mega size infrastructure development in terms of investment, construction size and complexity. In order to meet the operation time target, the Project is being carried out on fast track turn-key basis. In other words, design and construction are proceeding in parallel. This requires extremely close coordination between the designers and constructors. Due to the high operation speed, safety becomes the top concern. Design Specifications for the project are therefore much more stringent than conventional highway and railway development.

In this project, just like many other infrastructure projects, foundations of structures are the first to be constructed which often dictates the entire time schedule of the construction. Under the extremely tight schedule of the THSR, and the variable complex geological conditions along the route, the tasks for producing an economical, safe and timely design are great challenges to the geotechnical engineers. In this paper, some of the major geotechnical issues in the design and construction of viaducts of the project are described and discussed, including soil characterization, foundation type, soil liquefaction, ground subsidence, and negative skin friction, near fault effect and scouring of river crossing bridge foundation.

For ensuring safety and economy of the project, pile load tests are essential for verification of the design assumptions and selection of parameters in the design. In the THSR Project, the specifications require check borings to be carried out at every pier location. Furthermore, it requires the verification of soil bearing stratum in comparing with the design condition by the Designer as well as Independent Checking Engineer. All these procedures ensure the construction quality of this BOT project.

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