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TAIWAN HIGH SPEED RAIL PROJECT – SEISMIC DESIGN OF BRIDGES ACROSS THE TUNTZUCHIAO ACTIVE FAULT

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SUMMARY

The US$15 billion Taiwan High Speed Rail Project, one of the largest and most challenging infrastructure projects in the world to date, is scheduled to commerce service by the end of 2005. The 345 km rail corridor is designed for trains travelling at maximum speed of 350 km/hr. A substantial 75% of the line is to run on elevated bridges, 15% in tunnels and 10% on grade or in cut/fill embankments. The civil works is being constructed under twelve separate designs and build contracts.

The location for Contract C250 (the contractor HBP is a joint venture of HOCHTIEF AG, Ballast Nedam and Pan Asia) is in central Taiwan and includes approximately 39 km of viaduct and bridge construction, short lengths of cut and cover tunnel and earthworks, plus a segment where the Tuntzuchiao active earthquake fault crosses the alignment.

Capacity design methods used for the seismic resistant design of the 185 m long 5-span bridge crossing the Tuntzuchiao active fault are described. The bridge was required to be designed for both amplified near-fault shaking effects plus fault rupture including 1.5 m horizontal and 0.5 m vertical ground offsets, without suffering a loss of span failure. Special detailing incorporated to allow the bridge to survive the fault movement without catastrophic damage included large monopile foundations, simply-supported and fully articulated deck spans, large movement pot bearings, plus a backup system of guide walls.

INTRODUCTION

The Taiwan High Speed Rail Project

Taiwan is formally known as the Republic of China (ROC) and located around 150 km off the southeast coast of China, separated from the mainland by the Taiwan Strait. With a total area of about 36,000 square kilometres, the island is 394 kilometres long and 144 kilometres at its widest point.

When in the mid of the 16th century the first European conquerors – the Portuguese – landed on the island of Taiwan, they called it "Ihla Formosa"; meaning "beautiful island". One reason for this is the 270-kilometre central mountain range, which has more than 200 peaks over 3,000 metres high. Foothills

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from the central mountain range lead to tablelands and coastal plains in the west and south, where the major part of the 22.5 million inhabitants, farming activities, and industries are concentrated resulting into population densities of over 1000 per km² in this area.

The lack of natural resources and a relatively small domestic market have made Taiwan dependent on foreign trade. Over the past two decades, Taiwan has progressively hi-teched its industries (e.g. computer hardware and software, telecommunications, precision machinery and biomedical industries) and is nowadays the world's 16th largest economy and the 14th largest exporter.

A well-developed transportation network is essential to Taiwan's export-oriented economy. Therefore, transportation has always been an important priority in national development programs. In May 1995, the government approved construction of the HSR using the Build-Operate-Transfer (BOT) model - the first major infrastructure project in Taiwan to be constructed by the private sector under this model. In 1998, the Ministry of Transportation and Communications (MOTC) and the Taiwan High Speed Rail Consortium (THSRC) signed the Construction and Operation Contract.

The planned 345-kilometre High Speed Rail (HSR) route will pass through the western corridor of the island from the capital Taipei in northern Taiwan to the second largest city Kaohsiung in southern Taiwan. The rail corridor runs through or close to Taiwan's main cities, manufacturing areas, business and administration centres. Along the HSR line ten stations, three depots and three maintenance workshops will be built.

The whole line crosses fourteen counties and cities, sixty-eight townships and thirty-two city zoning areas. The civil works are being constructed by 12 major design and build Joint Ventures and include in total 42 km mined tunnels, 8 km cut and cover tunnels, 247 km viaducts and bridges and 33 km cut and fill embankments.

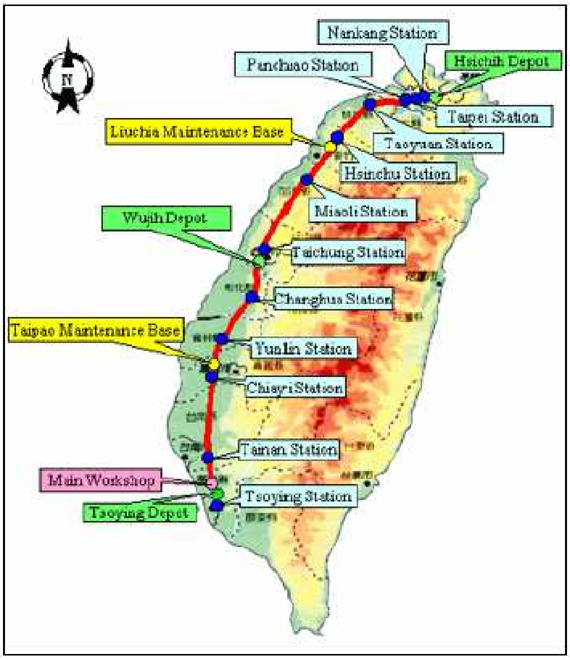


Figure 1: Taiwan High Speed Rail Project and Shinkansen Train

The overall construction cost of the HSR project is estimated to be US$15 billion, of which approximately 75 percent will come from private investment. Since the year 2000, all twelve lots have been under construction and together represent the largest infrastructure project presently being undertaken in the world. The construction work shall be finalized at the end of June 2004, with the track work and core system following and a start up of operations planned for October 2005, i.e. 68 months after commencement.

The Japanese Shinkansen train will operate on the tracks with cutting the journey between Taipei and Kaohsiung from 4.5 hours by existing train or highway vehicles to just 90 minutes.

Lot C250

The “HBP Joint Venture”, consisting of a joint venture of contractors, with the German HOCHTIEF Construction AG as the leader (55%), the Dutch Ballast Nedam International (30%) and the Taiwanese Pan Asia Corporation (15%), was awarded the contract for works of Lot C250. This Lot C250 is located approximately in the middle of the high speed rail route and runs close to the city of Taichung, which is the third largest city of Taiwan with around one million inhabitants.

Figure 2: Location of Lot C250 and Taichung City on a “clear day”

The contract Lot C250 is for the design and construction of the civil works of a 39.8 km long section of the HSR main line and the branch lines to the Wujih Depot, but excludes the construction of Taichung Station and the Wujih-Depot.

The following details provide some technical particulars of lot C250:

- Total length of Lot C250: 42.5 km (including branch lines to Wujih-Depot)
- Total length of bridges: 39.0 km (91.8%)
- Total length of tunnels: 0.7 km (1.60%)
- Earthworks: 2.8 km (6.60%)
- Construction period: 50 months
- Start / End : 01.05.2000 / 29.06.2004

The bridge section is split up into

- 26.9 km Full-span precast box girder bridges
- 1.9 km Cast in-situ 3-span bridges constructed on scaffolding or with free cantilever systems
- 4.0 km Cast in-situ bridges constructed with movable shoring systems
- 3.6 km T-beam bridges with precast concrete beams and in-situ deck and diaphragms
- 1.5 km Steel truss bridges
- 0.3 km Composite bridge
- 0.8 km Other in-situ concrete bridges

In comparison to other lots of the Taiwan High Speed Rail Project, Lot C250 is characterized by the largest quantity and variety of bridge structures, very dense urban areas, the crossing of four major rivers and the active Tuntzuchiao Fault traversing the rail alignment.

TUNTZUCHIAO FAULT

Seismic Setting of Taiwan

The island of Taiwan is located at the Circum-Pacific seismic zone at a complex juncture between the Eurasian continental and the Philippine sea plate. North and east of Taiwan, the Philippine sea plate subducts beneath the Eurasian plate to the north along the Ryuku Trench, while south of the island the Eurasian plate undertrusts the Philippine plate to the east along the Manila trench. Taiwan, therefore, occupies an unstable region between those two subduction systems of opposite polarity. Thus, seismicity is extremely active in this country.

Figure 3: Geotechnical Situation of Taiwan

Based on the distribution of recorded earthquakes, Taiwan can be roughly divided into three seismic zones: the north-eastern seismic zone, the eastern seismic zone and the western seismic zone.
Large magnitude earthquakes in the eastern and north-eastern zones dominate the island's high seismicity rates. The greatest recorded earthquake is the June 5, 1920 magnitude 8.0 at Hualien.
In the less seismically active western region generally shallow depth (<20 km) earthquakes occur. But, even though the occurrence rate in the western region is lower than that of the eastern region, they have a greater destructive impact as they strike the area of Hsinchu, Taichung, Chiayi and Tainan. Among the 125 destructive earthquakes, 30 events occurred in the western region. In the past 150 years 6 major earthquakes have occurred on the west coast of Taiwan: 1862, M=6.5; 1906, M=7.0; 1935, M=7.1; 1941, M=7.1; 1964, M=7.0; and 1999, M=7.6, killing in total more than 15000 people.
Based on the Central Geological Survey (CGS) of Taiwan the island has 51 active faults in total, which are classified into three groups: Holocene active faults, Late Pleistocene active faults, and Suspect active faults. In the first category, i.e. Holocene active faults, there are 9 faults recorded, with the Tuntzuchiao, Meishan, and Hsinhua faults running across or in the vicinity of the HSR route. In the second category (Late Pleistocene active faults) are 15 and the remaining 27 faults are the Suspect active faults.

Fault description of Tuntzuchiao Fault

The Tuntzuchiao Fault is a Holocene fault located 15 km north of the city of Taichung with an East-North-East (ENE) striking direction. The Fault's first (and so far last) movement was registered during a strong motion earthquake in 1935. The epicentre of this earthquake was located some kilometres east of the HSR track at the north-south striking Chelungpu Fault with the hypocentre located at a depth of about 10 to 15 km. It is likely that the Tuntzuchiao Fault was generated during this 1935 event as the result of a simple cross-shearing in the block between the Chelungpu Fault in the East and the Changhua thrust Fault in the West. Not previously registered as an active tectonic fault, the Tuntzuchiao Fault is assumed to be of secondary character, a so-called tear fault.

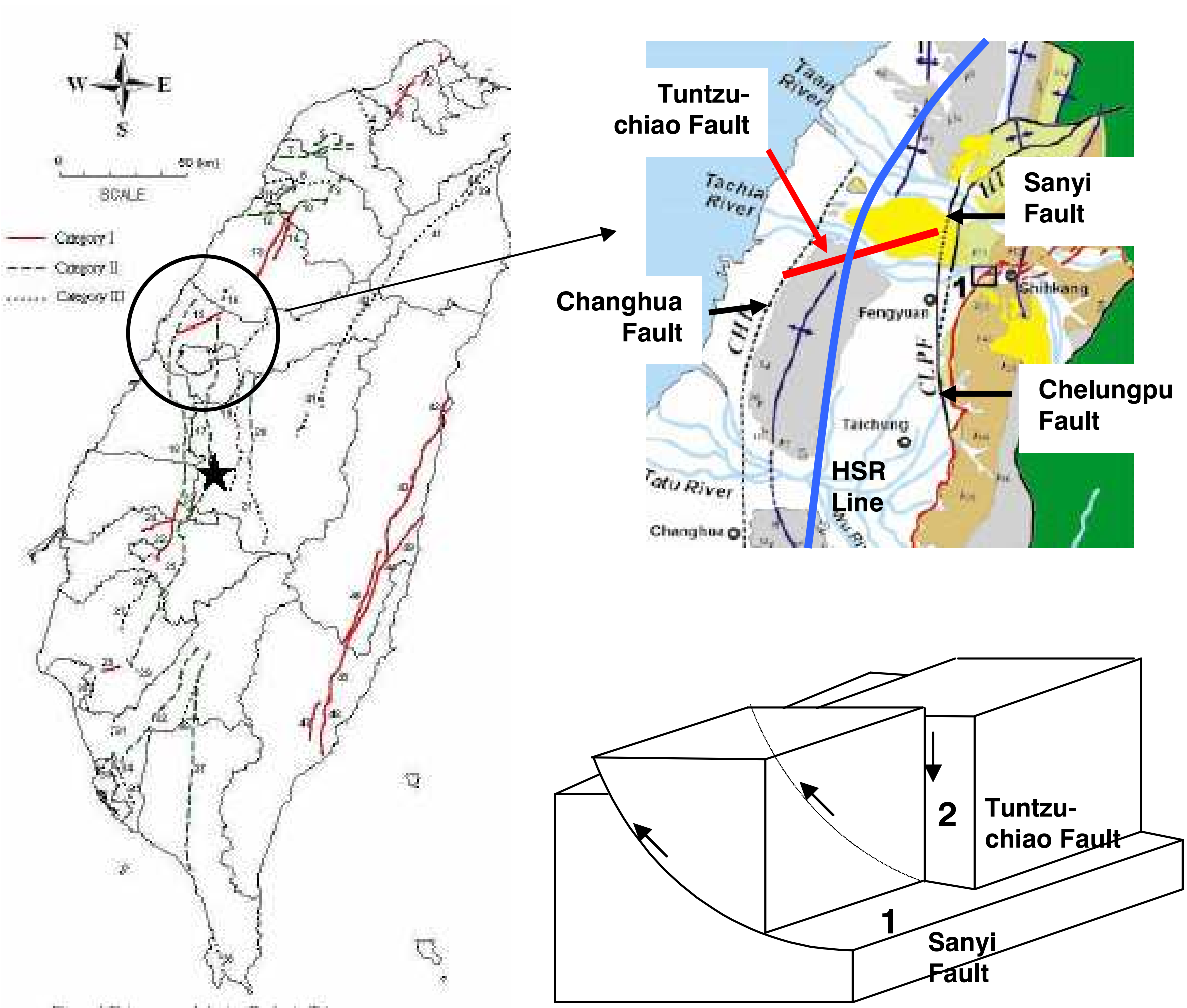


Figure 4: Active Faults in Taiwan and Tuntzuchiao Fault Characteristics

STRUCTURES CROSSING THE TUNTZUCHIAO FAULT

The Tuntzuchiao Fault crosses the HSR alignment within a zone of approximately 750 m long at an angle of close to 45 degrees to the longitudinal axis of the HSR line. The expression of the fault at the surface is likely to be well defined due to the cemented nature of the conglomerates underlying the site. So, the width of the fault zone is likely to be influenced only by the size of the boulders in the conglomerate, and the fault movement is expected over a relatively narrow width of approximately 1.0 m.

The potential fault rupture displacements of the Tuntzuchiao Fault are in such a range that in case of a fault rupture structural damages on the HSR alignment can not totally be excluded. For that reason, HSR infrastructure crossing the Tuntzuchiao Fault has to meet special requirements and has conceptually to consist of structures with limited damage and distinctive repair characteristics.

In particular, the infrastructure crossing Tuntzuchiao fault is required to be designed using embankments with containment earthworks to accommodate the potential rupture offset and subsequent re-alignment. The containment earthworks are required to have a minimum height 2m and a 2m wide flat top. Where embankments are not possible and bridges have to be constructed to cross the Tuntzuchiao Fault, simply supported span and appropriate measures are required to prevent loss-of-span failures.

From these requirements the following layout resulted for the infrastructure:

- Two embankments, 300 m and 250 m long and up to 22 m high, which form the major part of the infrastructure. Due to the different right of way routes around the embankments the slopes of the two embankments are not equal, i.e. Embankment I has a slope of 1:2.15 and Embankment II a slope of 1:1.6.
- A 55 m long single span steel truss bridge, which is needed to cross an existing Highway, supported on pier caps.
- Two simply supported bridge spans (in situ concrete) at each side of the steel bridge (2x30 m and 2x35 m) supported on 15 m tall piers and on abutments at the connection to the embankment.
- A 5 cell water/road culvert, which crosses Embankment II.

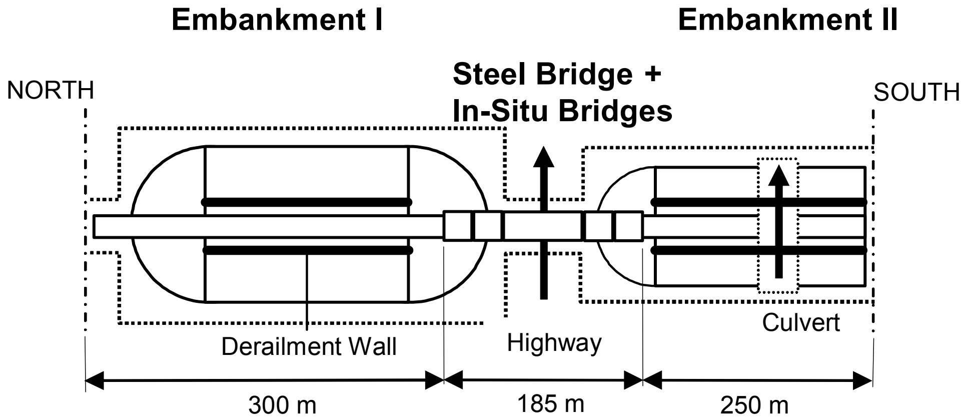


Figure 5: Layout of the Infrastructure crossing the Tuntzuchiao Fault

Special near fault design criteria were specified for the design of the bridge structures in the Tuntzuchiao Fault zone. For more information regarding the design of the embankments (pseudo-static analysis and seismic durability design using time history simulations) it is referred to [1].

BRIDGE DESIGN

General

The bridge structures along the HSR line are required to be designed to safeguard against major failures and loss of life under Type I earthquakes (severe, return period 950 years with 10% probability of exceedance) and to ensure adequate service performance under Type II (moderate) earthquakes.
When subjected to a Type I earthquake it is acceptable for the structure to respond in the inelastic range, provided the ductility demand does not exceed the available ductility. All damage is to be repairable and capacity design procedures are required to be used. In general it is expected that inelastic behaviour will be restricted to the piers and that the deck will remain within the elastic range.
When subjected to a Type II earthquake (1/3 of Type I) no yielding of reinforcement or structural steel is permitted, and the displacement of the deck shall be such that trains can brake safely to a stop from their full design speed of 350 km/hr.
The viaducts and the abutments are within Seismic Zone 1B, with design ground acceleration of 0.34 g. The site consists of more than 30 m of conglomerates over mudstone.

Potential Rupture Offset of Tuntzuchiao Fault

Based upon the 1935 earthquake data, design movements for the fault were defined as

- a right lateral strike slip displacement of 1.5 m connected with
- a vertical slip displacement of 0.5 m.

Considering the angle between fault and HSR line the design movements are 1.06 m slip along the longitudinal bridge axis and 1.06 m slip transverse to the bridge axis. The northern side of the fault moves east relative to the southern side, thus tending to spread the abutments apart if the fault rupture is located between them. And, the north side of the fault is expected to move downwards relative to the southern side.

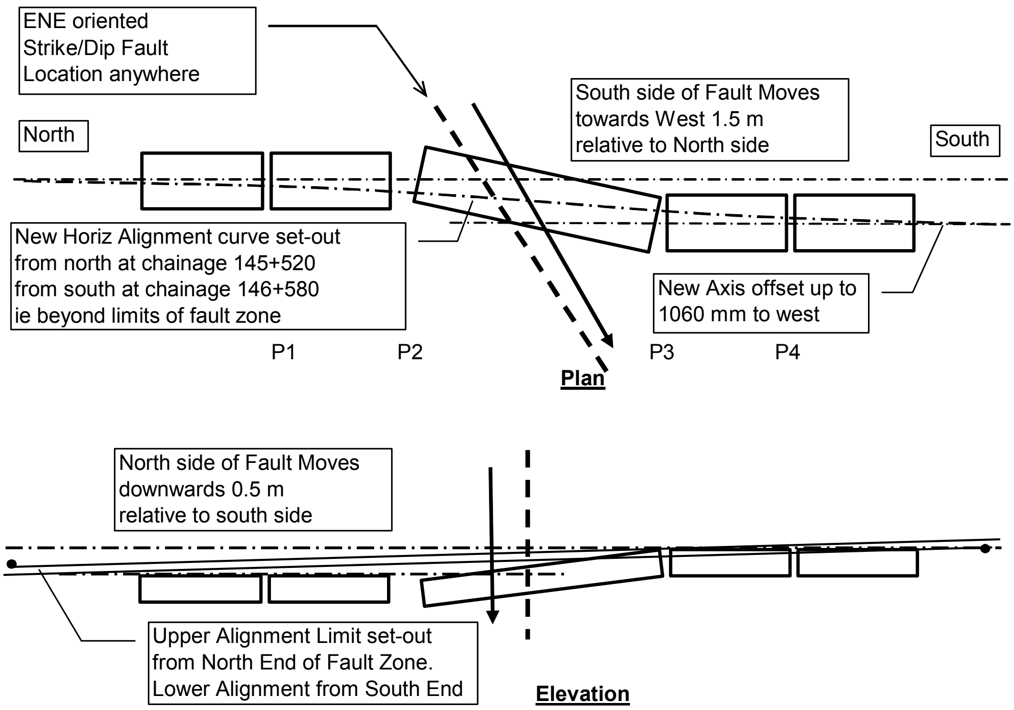


Figure 6: Provisions for Fault Movements and Re-Alignment for a potential fault rupture within the span of the steel bridge

Near Fault Effect of Tuntzuchiao Fault

Depending on the maximum moment magnitude potential of a fault the ground motions are amplified at close distance to the fault. Accordingly, ground shaking and then the corresponding structural response are attenuated with distance from the source. The attenuation relationships are based on the earthquake magnitude and other parameters.

The Taiwanese National Centre for Research on Earthquake Engineering (NCREE) developed two procedures regarding the design of the structures in active fault zones [2]:

- “Development of Near-Fault Design Response Spectra” and
- “Ductility Capacity Check”.

Near-Fault Design Response Spectra

For the specific site of the THSR route NCREE proposed attenuations function of Boore, Joyner and Fumal (BJF) [3] in order to define the 5% damped demand spectrum for a strike-slip fault near the source. The BJF function is defined as:

$$\log_{10}(PGA) = -0.136 + 0.229 \cdot (M_W - 6) - 0.778 \cdot \left(\log_{10} \sqrt{(r^2 + 5.57^2)} \right) - 0.371 \cdot (2.881 - \log_{10} 1400) \,. \qquad (1)$$

With the Campbell law

$$PGA = 0.00577 \cdot \exp(1.77089 \cdot M_L) \cdot (r + 0.13481 \cdot \exp(0.78846 \cdot M_L)^{-2.13137} \qquad (2)$$

the arithmetic mean of (1) and (2) was determined, and considered as an appropriate attenuation relationship for the normalised Near Fault Design Response Spectrum ("Mean Curve approach"):

$$PGA^{Att}(r) = \frac{1}{2} \cdot \left[PGA^{BJF}(r) + PGA^{Campbell}(r) \right], \qquad (3)$$

with PGA as peak ground acceleration, M_w as moment magnitude, r as rupture distance and M_L as local magnitude. Accordingly, the required seismic demand for Tuntzuchiao Fault was given by

$$PGA(r) = \max\left[PGA^{Att}(r);\; Z \right]. \qquad (4)$$

With Z = 0.34 as basic design seismic ground acceleration coefficient in the Tuntzuchiao Fault zone (corresponding to Type I earthquake with 950 years return period and 10 % probability of exceedance) the design response spectra for the Tuntzuchiao Fault zone shown in figure 7 are resulting.

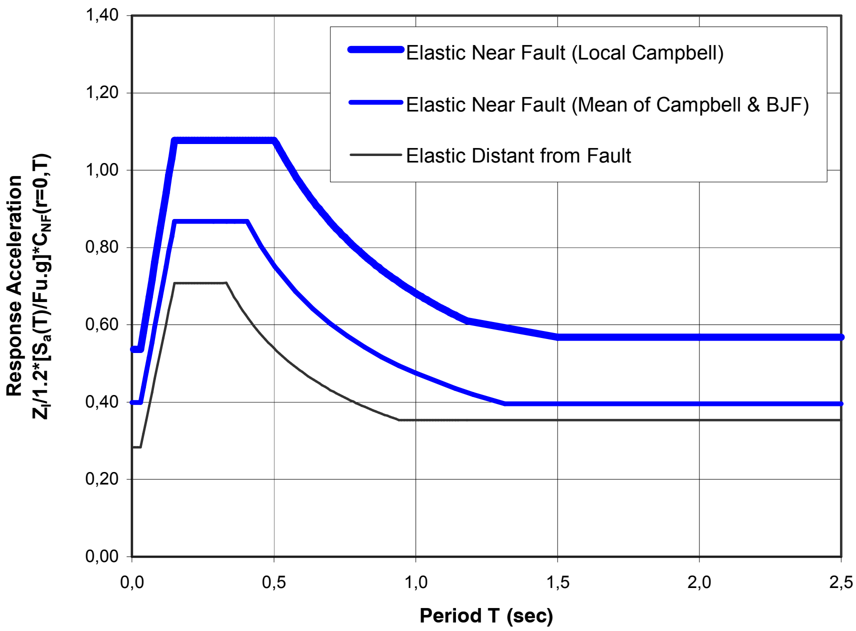


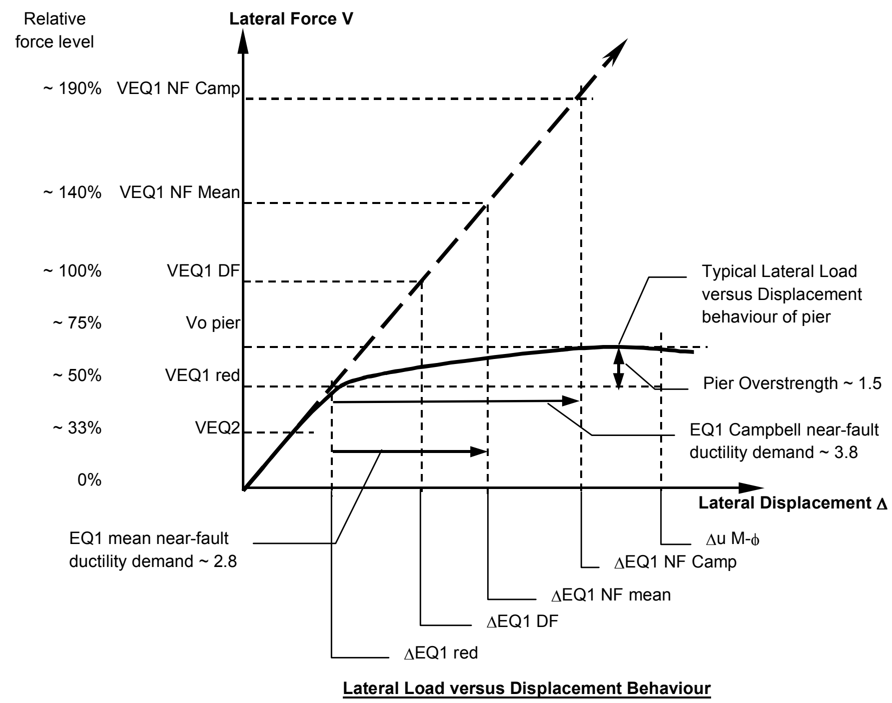
Figure 7: Tuntzuchiao Active Fault Design Response Spectra

Ductility Capacity Check

As an alternative approach NCREE permitted to design the structures in the active fault zone firstly in accordance with the design specifications (distant from fault), and then, secondly, to undertake a ductility check to confirm compliance with the seismic demands based on the Campbell attenuation curve.

This check used moment-curvature method calculating the moment capacity and curvature of the column cross-section based on the actual reinforcement details and stress-strain curves of both confined concrete and reinforcement.
It then determines the ultimate curvature and bending moment in the plastic hinge zone, and the ultimate displacement at the top of the piers, which is required for the calculation of the ultimate allowable ductility factor $R_{a,N.F.}$. Thereby, $R_{a,\ N.F.}$ is limited to 4, which is twice the allowable ductility factor R_a=2.0 (distant to fault).
Based on the ultimate allowable ductility factor the ultimate seismic force reduction factor $F_{u,\ N.F.}$ is calculated using equal energy principle.
With the above described normalised Near Fault design response spectrum including Campbell factor and the increased seismic force reduction factor $F_{u,\ N.F.}$ the member force at the pier base is computed. This member force is then compared with the allowable horizontal member force (distant from fault) to determine if the pier column is seismically adequate and the plastic hinge ductility is sufficient.
In addition a shear check has to be performed to ensure that no shear failure occurs prior to the flexural failure.

The Ductility Capacity Check was chosen for the seismic design of the piers in the Tuntzuchiao Active Fault in order to fulfil the design requirements of NCREE.



Parameters	VEQ1 NF Camp	elastic EQ1 response with Campbell near fault amplification
	VEQ1 NF mean	elastic EQ1 response with mean near fault amplification
	VEQ1 DF	elastic EQ1 response (distant from Fault)
	Vo pier	overstrength capacity of pier based on moment-curvature
	VEQ1red	reduced EQ1 forces (distant from Fault)
	VEQ2	EQ2 forces (distant from Fault)
	Δu M-ϕ	available ultimate displacement from M-ϕ ductility capacity check
	ΔEQ1 NF Camp	EQ1 displacement requirement for near-fault Campbell curve.

Figure 8: Schematic representation of the lateral force versus displacement demands on the bridge system and the behaviour of the piers as part of the Ductility Capacity Check

STRUCTURAL CONCEPT AND CONSTRCUTION

General

The structural design concept of the bridges across the Tuntzuchiao Fault is based on standard C250 bridge components, which are modified accordingly, to meet the special active fault design requirements. The bridge structure comprises a simply supported and articulated deck system supported on pile supported single piers and abutments.

The dynamic calculation is performed with a multi-modal response spectrum analysis using a lumped mass model to reflect the structural behavior during earthquake. The 3-dimensional model represents the main structural components of all the bridge spans and the abutment as boundaries and all masses of the bridge including all superimposed dead loads such as slab track system, cable troughs, derailment walls and parapets. The supporting foundations are included in the model with its rotational and displacement stiffness by equivalent springs.

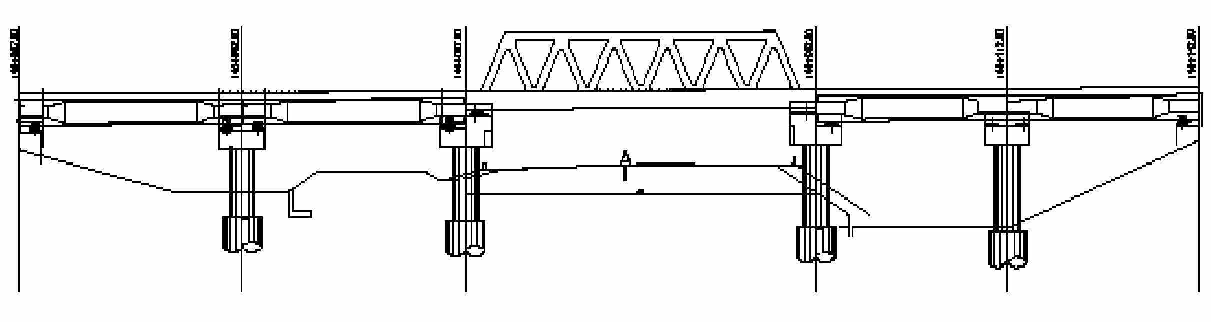


Figure 9: Layout of bridges across the Tuntzuchiao Fault

Pile Foundations

Concept

Due to the cemented conglomerates and the expected sharp fault expression on the surface, the piers are founded on a single solid pile (monopile) with 6 m diameter and 20 m length. The monopile system was chosen instead of a normal multi-pile foundation to minimize the risk of foundation damage caused by a fault rupture passing directly under the pier. That arrangement avoids or reduces the possibility of sharing of footing support across the fault, and reduces the risk of the foundation being subjected to significant stress in the event that the fault moves under it – particularly for vertical movements.

Construction

Before the excavation of the pile shaft started the ground water was drawn down by a gallery of wells. Around each of the four mono-piles four 600 mm wells with 40 m depth were installed. Excavation itself was done with a backhoe in 5 m lifts. The excavated walls are protected against collapse with a 25 cm thick shotcrete lining, which is reinforced with wiremesh.

Once the pile is fully excavated the reinforcement cage is installed, which contains a heavy confinement of welded stirrups (about 220 cm²/m at the top of the pile). The longitudinal reinforcement of the mono-pile (around 3200 cm²/m) was designed for the overstrength plastic moment and also checked for cracking due to hydrationtemperature of the massive concrete body. The casting of the shaft was done with concrete class 245 kg/m² (24.0 MPa) in one go, i.e. approx. 500 m³ per mono-pile.

Additionally, a grouting system was installed, which allowed high pressure grouting between the pile shaft and the shotcrete lining in order to ensure that the pile shaft is always in full contact with the surrounding soil without any reduction of the skin friction and lateral bedding.

Figure 10: Mono-pile Construction

Abutments

The abutments are embedded into the embankments and are supported on multi-pile foundations with 1.50m pile diameter. Under earthquake shaking the abutments will resist transverse forces from the end spans, but do not restrain the deck longitudinally. Special attention had to be spent on the slope stability check in connection with the embankment.


Figure 11: Abutment with extended seating length and special bearings

Piers

Concept

The bridge is supported on circular solid piers with a diameter of 4 m. These piers also resist the lateral earthquake loads on the bridge. All piers are of approximately equal length to provide uniform distribution of earthquake effects.

The plastic hinge zones at the base of the piers are designed for the normal reduced Type I earthquake forces (distant from the fault). Capacity design methods are used to ensure that yielding will only occur at the pier bases and that damage due to earthquake shaking is expected to be limited to the pier bases and can be easily inspected and repaired.

Because the near fault Type I earthquake shaking is amplified compared with the distant from fault shaking, the ductility demands at these piers are greater than for bridges located distant from the fault to

sustain the Type I near-fault shaking effects. Ductility capacity checks were carried out to demonstrate that adequate ductility is available.
The pier caps are considerably larger than the pier caps of the other C250 bridges as they allow the deck to be re-aligned after a fault movement event and to allow for large bearing movements in case of a fault rupture event.

Guide Walls
Longitudinal guide walls are provided at each abutment and each pier cap to retain the deck safely on the abutments and pier caps after fusing of the pot bearings. Approximately 1 m of horizontal clearance is provided between the girders and the longitudinal guide-walls. This clearance is not required to accommodate fault displacements. It is provided only to facilitate realignment of the deck and track after a fault rupture.

Construction
The piers are cast in 5 m lifts with the use of steel formwork. The concrete class is 280kg/m² (27.5 MPa). Due to the high loading the piers are heavily reinforced with around 2400 cm² longitudinal reinforcement and 200 cm²/m transversal confinement reinforcement at the base.



Figure 12: Piers with extended seating length and special bearings

Pot Bearings
Each girder is supported and connected to its supporting piercaps and abutments via a standard configuration of pot bearings, similar to other C250 bridges. Each span has a fixed shear key together with two free bearings at one end, and two bearings which resist transverse forces and slide longitudinally at the far end. The pot bearing system provides the normal primary means of transferring lateral load between the deck and piercaps/abutments.
The sliding longitudinal bearings at one end of each span are designed to provide the necessary additional longitudinal movement capacity required to accommodate the chosen fault displacement allowances. This is done with sliding surfaces on the pier caps.
In addition the bearings are designed with a fuse system, which will allow the anchor bars to shear off at a certain horizontal load.
At some bearing locations additional elastic and ductile Holding Down Devices are provided on spans where large uplift forces occurred, and where it was not practical to design the pot bearings to resist those forces. Due to larger elasticity of the Hold Down Bars in comparison to the normal bearing clamps the vertical bearing forces were reduced significantly.

Figure 13: Pot bearings in open mould and on pier caps with sliding surfaces

Bridge Spans

The deck is able to articulate horizontally, vertically and longitudinally at the piers and abutments to accommodate expected earthquake shaking-induced displacements and fault rupture movements between adjacent piers, and between adjacent piers and abutments.

Figure 14: Steel bridge and In-situ concrete bridges

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