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SEISMIC DESIGN AND CONSTRUCTION OF THE CUT-AND-COVER TUNNEL UNDER THE JORGE CHAVEZ AIRPORT IN CALLAO, PERU

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Abstract

The construction of the third phase of the Nestor Gambetta Freeway is underway in Callao, Lima-Peru. This stage completes the Nestor Gambetta Freeway connecting the Callao Port Terminal with the Panamericana Norte Freeway.

This phase comprises the construction of a tunnel, called “Túnel Callao”, that runs beneath the future second runway at the Jorge Chávez International Airport. It includes a 1 km cut-and-cover box structure and two approach ramps, which make a total length of 2.4 km. This facility resolves the inconvenient overlap between the new freeway and the airport expansion.

It is located in a highly seismic area with a high water table. The paper presents an overview of the design and construction, with emphasis in seismic design. The design earthquake was determined based on the latest AASHTO criteria. The seismic design takes into account pressures and racking displacement demands. The racking displacements were based on site specific soil characteristics and state-of-the-art research on soil interaction analysis. In a final stage, performance-based earthquake engineering (PBEE) approach was applied in order to investigate and demonstrate that the tunnel seismic performance is adequate for big earthquakes. The state-of-the art-practice in tunnel seismic design was used in order to get a safe and economic structure and with a predictable seismic behavior.

Keywords: Cut-and-Cover Tunnel; Seismic Design; Racking Displacement; Performance-based earthquake engineering

1. Introduction

The Nestor Gambetta Freeway is a major route in Callao Region. In the last ten years, it has had several stages of expansion and modernization promoted by the urban sprawl, the country's economic growth and an unprecedented expansion of the Callao Port. In line with this, the need for the expansion of Peru's main airport, Jorge Chavez International Airport, was evident. As the airport expansion required a second runway and its alignment overlaps with the Gambetta Freeway, the logical solution approved by the government, was to depress the Gambetta Freeway in the conflicting length, giving birth to the project called “Tramo III-B de la Av. Néstor Gambetta – Callao”, which is the 3th phase of the expansion of the Nestor Gambetta Freeway and is currently under construction.

The third phase comprises the construction of a 1 km tunnel that runs beneath the future second runway at the Jorge Chávez International Airport and two approach ramps that make a total length of 2.4 km (see Fig. 1). The tunnel is a cut and cover box structure with three cells: two of them for highway transit with three lanes each and the third one for a two-way railway transit (see Fig. 4a). The approach structures have an open U section as shown in Fig. 4b. All structures, but the tunnel´s top slab, are cast-in-place reinforced concrete. The top slab (roof) consists of precast inverted tee-beams, topped with cast-in-place concrete. The project is located in the highest seismic zone in Peru, with a high water table.

The construction was awarded to "CTC" consortium —comprising Andrade Gutierrez, Queiroz Galvao, both from Brazil and ICCGSA from Peru— through a rigorous public selection process. The total project cost is approximately US$ 0.4 billion. The structural design engineering was developed by GCAQ Civil Engineers from Lima, Peru, based on the tunnel’s basic engineering developed by CJC from Brazil.

The paper presents an overview of the design and construction of the project, with emphasis in the seismic design of the cut-and-cover tunnel.

Fig. 1 – Project Overview.

2. Site Conditions

The geological profile and project vertical alignment are shown in Fig. 2, according to the Geological and Geotechnical Study [1]. The soil profile along the project alignment shows two main stratums: a first soft stratum conformed by silt-sands, sands, silts and clays, with a variable depth between 2.0 m-4.0 m; beneath this there is a second stratum conformed by an alluvial conglomerate gravel layer (GM, GP-GM y GP), which extends beyond the maximum depth of investigation (40 m). The second stratum is part of the subsoil of Lima called "Conglomerate of Lima" and, according to past studies, has at least a depth of 100 m in this area. The measured shear-wave velocities vary from 100 m/s to 220 m/s for the first stratum and from 300 m/s to 600 m/s for the second stratum, with densities increasing with depth.

There is a high water table, with measured levels around 1-2 m below the ground surface. The hydrological study estimated the probable maximum water level to be 1 m below the ground surface. For design purposes, we considered the maximum water table level to be 0.50 m below the ground surface.

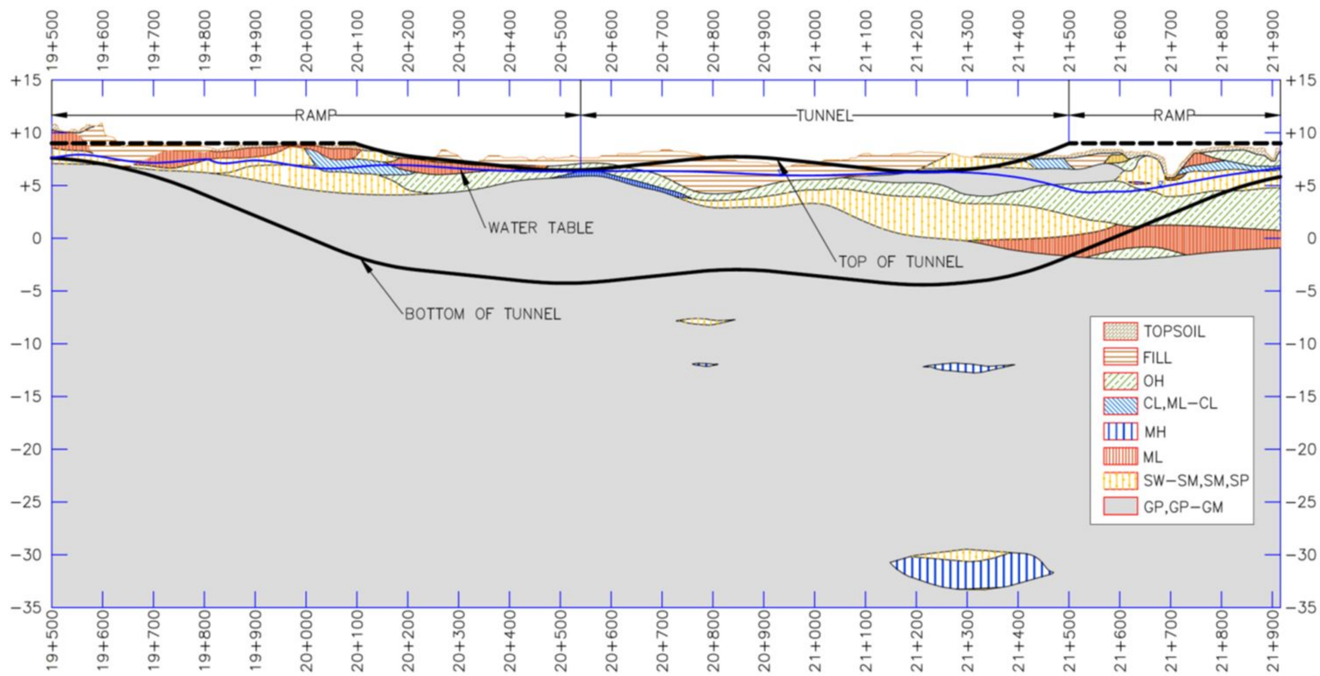

Fig. 2 – Geological profile and project alignment.

3. Seismicity and Tsunami Hazard

3.1. Seismicity

Over the past 500 years, the Peruvian coast has been hit by numerous destructive earthquakes and tsunamis historically documented. The main source of the seismic events affecting this region is the subduction of the Nazca plate beneath the South American plate. This movement generates large-scale seismic events that can overcome a magnitude of 8 on the moment scale Mw.

From the Seismic and Tsunami Hazard Study [2], peak ground accelerations (PGA) and uniform hazard spectrums were determined for different return periods for rock and soil site conditions, as shown in Table 1 and Fig. 3a, respectively. Fig. 3b shows the design spectrums for Site Class B (rock) according to AASHTO LRFD 2012 [3].

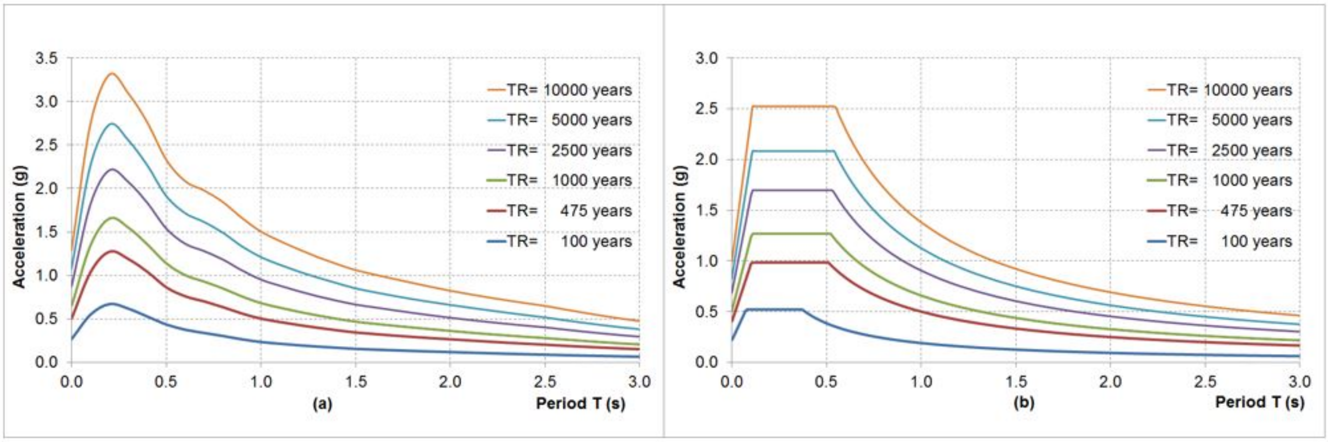

Fig. 3– (a) Uniform hazard spectrums. (b) Design spectrums.

Table 1- PGA accelerations

Return Period (years)	Site Class B (Rock)			Site Class C (Soil)		
	Period (s)			Periodo (s)		
	PGA	0.2	1.0	**PGA**	0.2	1.0
100	**0.23g**	0.52g	0.19g	**0.27g**	0.67g	0.24g
475	**0.41g**	0.98g	0.39g	**0.51g**	1.27g	0.50g
1000	**0.52g**	1.27g	0.51g	**0.66g**	1.66g	0.68g
2500	**0.68g**	1.69g	0.69g	**0.87g**	2.21g	0.95g
5000	**0.83g**	2.01g	0.87g	**1.08g**	2.73g	1.21g
10000	**0.99g**	2.52g	1.06g	**1.29g**	3.31g	1.50g

3.2. Tsunami Hazard

Due to the proximity of the sea, the tsunami hazard was evaluated. The tsunami study concludes that, because of the project elevation and the fact that it is 1.2 km away far from the sea, the project will not be affected by a tsunami of great magnitude.

4. Seismic Design Criteria

The design performance criteria asks that the structure must withstand without damage a 1 in 100 years seismic event; with minimal damage, a 1 in 1000 years seismic event; and with reparable damage, a 1 in 2500 years seismic event.

5. Design Solution

The project comprises a 1 km tunnel section and two approach ramps, with a total length of 2.4 km. The tunnel section is a cut-and-cover box structure and the ramps are U-shaped sections. All structures, but the top slab of the tunnel, are cast-in-place reinforced concrete. The top slab consists of precast inverted tee-beams, topped with cast-in-place concrete. The ground cover over the tunnel varies along the alignment and above it there is a 2.00 m of structural fill as part of the new airport runaway. Fig. 4 shows the typical cross sections of tunnel and ramps with three components. The central component is a 10 m wide train transport corridor with two railway lines, while the lateral ones are prepared for 3 road traffic lanes. Additionally, there are two small services compartments on each edge.

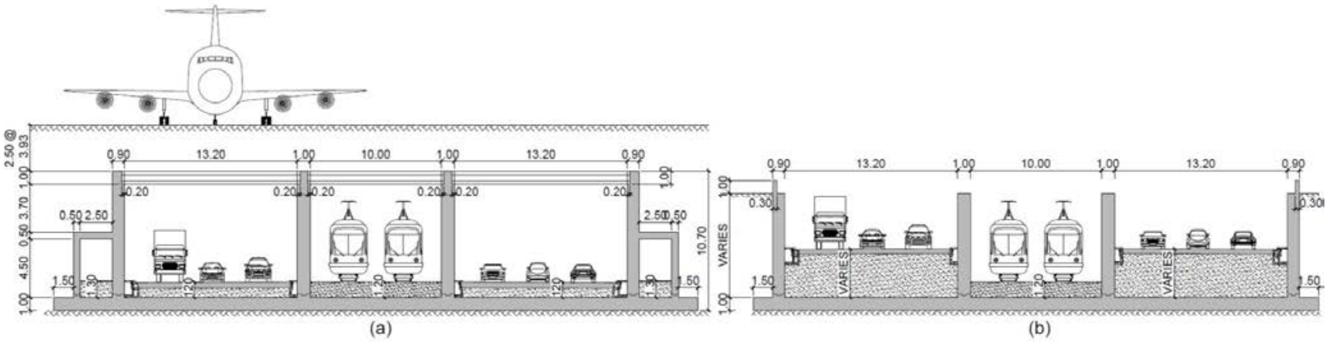

Fig. 4 – (a) Typical Cross Section of Tunnel. (b) Typical Cross Section of Ramp (unit: m).

6. Seismic Analysis

Various authors like Wang [4] and Hashash [5] exposed that the ground deformations and soil-structure interaction controls the loads exerted by the earthquake on underground structures. For cut-and cover box structures, the dominant seismic effect is the relative horizontal deformation between the top and bottom slabs (racking deformation).

Wang [4] outlined a simplified method for seismic analysis of box cut-and-cover tunnels which many engineers call "Racking Deformation Method". In this method the free-field racking deformation ($\Delta_{\text{free-field}}$) is determined and then converted to structure racking deformation ($\Delta_{\text{structure}}$) using the racking coefficient (R), see Fig. 5, $\Delta_{\text{structure}} = R \cdot \Delta_{\text{free-field}}$; then the racking deformation is applied to the structure considering the static equivalent load method reproduced in Fig. 6, so the tunnel is designed to resist the imposed ground deformation under seismic event. This method is indicated in the AASHTO Tunnel Manual [6] and, in fact, is one of the most used methods in seismic design of cut-and-cover tunnels.

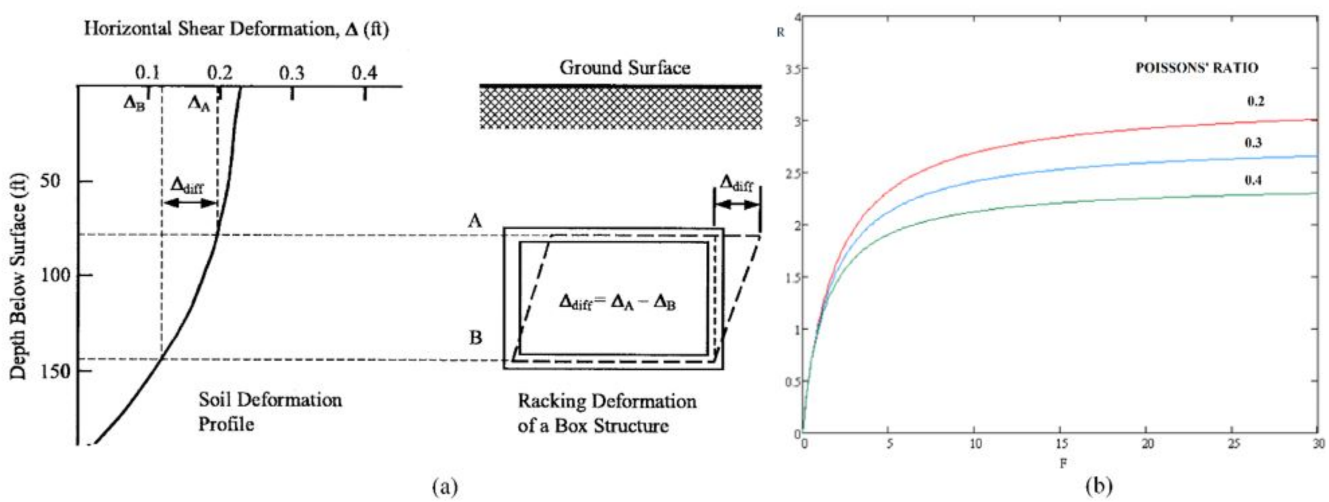

Fig. 5 – (a) Racking Deformation of a Box Structure; (b) racking coefficient. (Wang 1993 [4]).

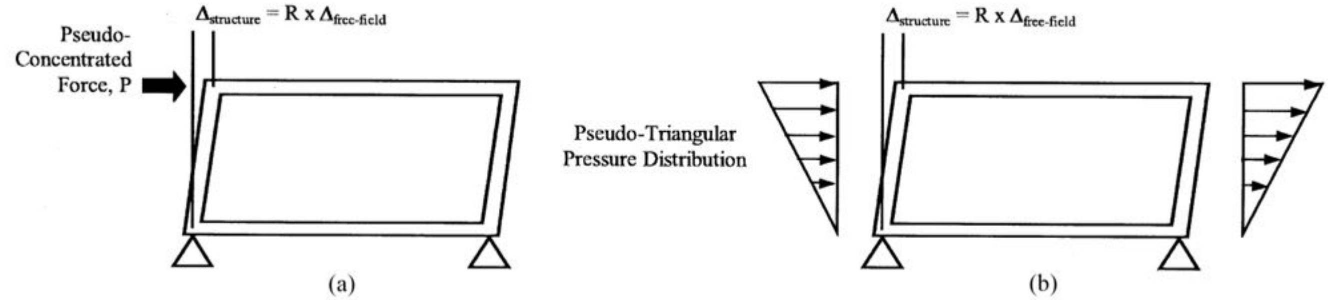

Fig. 6 – Simplified Frame Analysis Model (Wang [4]): (a) pseudo-concentrated force at the corner; (b) pseudo-triangular pressure distribution on the side walls.

For this project the free-field racking deformation ($\Delta_{free\text{-}field}$) is calculated from one-dimensional site response analysis using equivalent linear (SHAKE91 [7], EERA [8]) and non-linear (NERA [9]) approaches. These analyses considered detailed soil profiles and 20 spectral matched ground motions (see Fig. 7). From these analyses the free-field racking deformation is around 0.6 cm for the 1 in 1000 years design earthquake. For the typical section of the tunnel the flexibility ratio (F) is around 17, and from Fig. 5b, the racking coefficient (R) is about 2.50, so the resultant design structure deformation ($\Delta_{structure}$) is 1.5 cm.

The calculated structure deformation is used to determinate the static equivalent load as explained before. These loads are applied to the tunnel sections modeled in the SAP 2000 computer program, considering frame elements with effective flexural stiffness (35% gross stiffness for slabs, 70% gross stiffness for walls and 50% gross stiffness for short walls). The soil was represented by ground non-tension springs using subgrade modulus specified in the Geological and Geotechnical Study [1]. Moments and shears are obtained from the structural model, and then combined with other loads as will be explained in the next section.

In addition, some two-dimensional models were made in PLAXIS [10]. As the calculated deformations were somewhat lower than those obtained from the 1-D analysis, the design was based on the 1-D results.

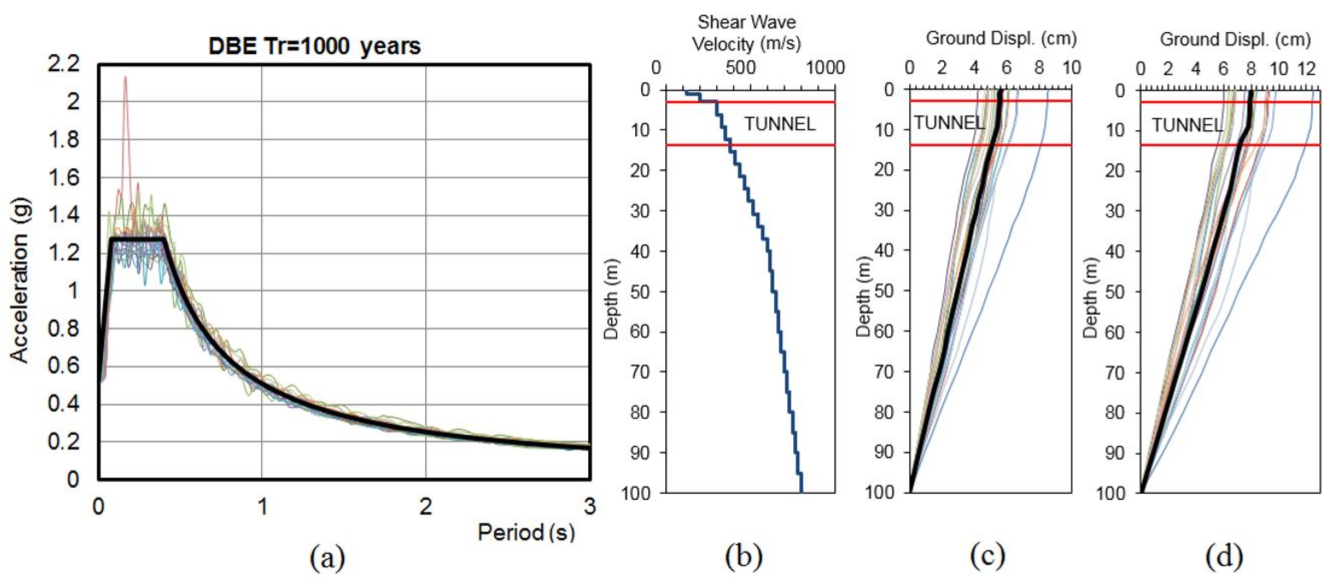

Fig. 7 – (a) Target spectrum and spectrum of 20 spectrum-matched ground motions for 1000-year return period hazard level in site class B. (b) Shear wave velocity profile. (c) Mean relative displacement of the ground for 1000-year return period earthquake. (d) Mean relative displacement of the ground for 2500-year return period earthquake.

7. Design

The structural design follows mainly AASHTO Tunnel Manual [6], FHWA Tunnel Manual [11], AASHTO LRFD 2012 [3] and ACI 318 [12].

Actions on the tunnel include static loads and design seismic effects. The loads combinations are according to chapter 5 in AASHTO Tunnel Manual [6].

Some of the relevant static loads are soil pressures, fill loads on the roof, water pressures, and live loads from a Boeing 747-400 ER FREIGHTER airplane, the latter is according to Jorge Chavez Airport Authority. The Boeing 747-400 ER FREIGHTER has a maximum design taxi weight of 414.15 t or 96.9 t per main strut. This load is distributed, for variable depths, according to AASHTO LRFD [3] sec. 3.6.1.2.6. Conservative values for maximum pressures over the roof of the tunnel were taken as: 5.45 t/m^2 and 3.50 t/m^2 for a 2.50 m and 3.93 m fill cover, respectively.

Because of the high water table on site, buoyancy was a major concern in the design. To counteract buoyancy, we have considered the weights of: the structure, the interior plane concrete fill —which was provided where necessary— and the earth fill required by the airport. Frictional forces were omitted in all cases. Buoyancy was checked for the Limit State IVA, as indicated in Table 5.5.2-1 from AASHTO Tunnel Manual [6]. This is equivalent to a minimum buoyancy safety factor equal to 1.1 (SF_{min} = 1.1) for the initial condition. In the final service condition, the minimum safety factor for the tunnel sections and ramps sections are 1.30 and 1.25, respectively.

The structural design for strength load combinations follows the common procedures in codes. For example, a total of twenty-four strength load combinations were used for the typical transverse section of the tunnel in order to take into account all loads scenarios.

After that, extreme event load combinations including earthquake loads are checked and additional reinforced is provided when necessary.

In the initial design stage of the tunnel section, the design for the extreme event combination considered the seismic load according to Mononobe-Okabe “M-O” theory, modified for box structures in, for example, Kaul [13]. For the final design the seismic effect took into account the Racking Deformation Method calculated as described in the preceding section. In line with what is reported in the literature, the design forces (moments and shears) obtained using “M-O” theory were found to be greater than those obtained using “Raking Deformation Method”, see Fig. 8. For typical reinforcement details of the tunnel section (see Fig. 9).

For the ramp sections, the seismic load is applied on the cantilever walls according to “M-O” theory, using K_h = A/2, also hydrodynamic pressures are considered.

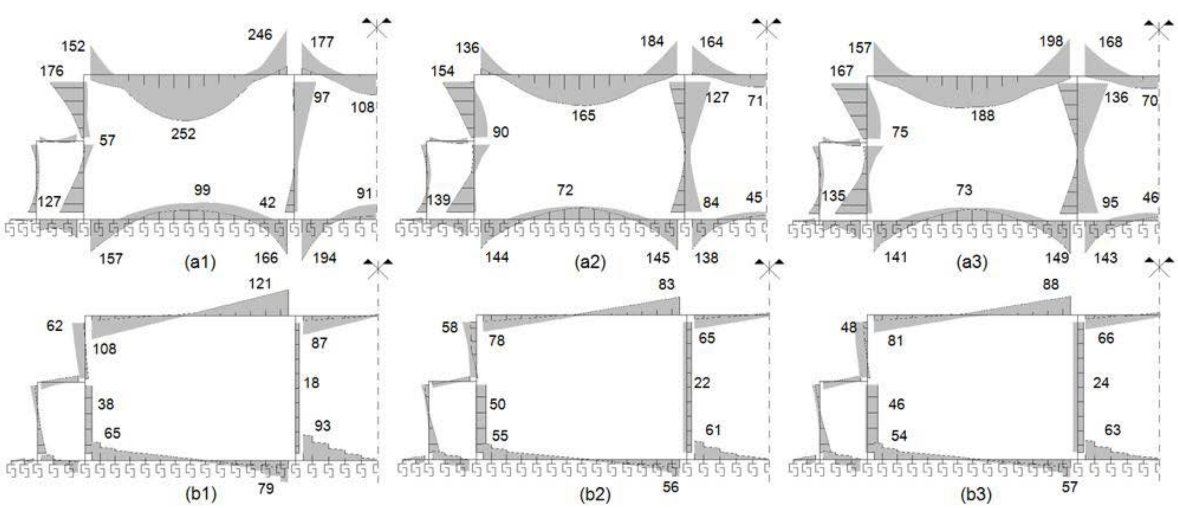

Fig. 8 – (a) Bending moment envelopes (unit: t-m/m); (b) Shear force envelopes (unit: t/m). (1) Strength I; (2) Extreme Event I (M-O); (3) Extreme Event I (Raking).

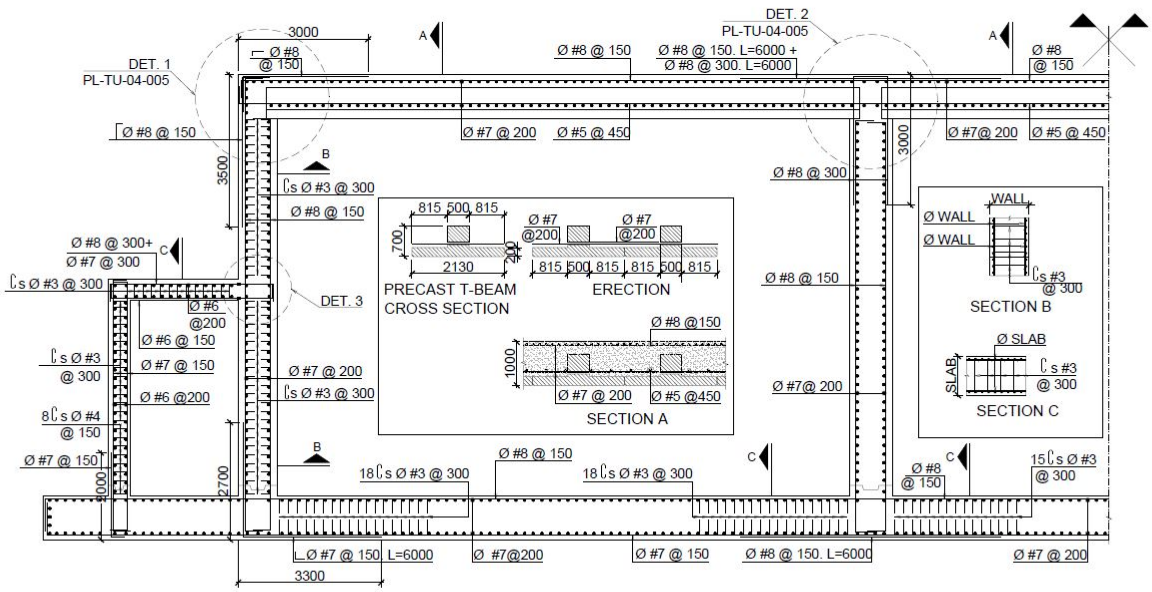

Fig. 9 – Typical Reinforcement Details of the tunnel section. Precast T-Beam reinforcement not shown for clarity.

In the longitudinal direction, the tunnel and ramps are designed as 30 m long structural modules, in order to reduce the induced seismic axial strain and to control the temperature and shrinkage deformations. We have provided reinforced concrete shear keys at the joints between the modules as shown in Fig. 10, Shear keys are designed for vertical and transversal seismic shears and to prevent differential settlements along the tunnel.

All joints between modules are provided with a water stop. This was designed to provide the required flexibility and strengths to withstand the stress and strain, produced by the imposed seismic longitudinal deformation between the structural models.

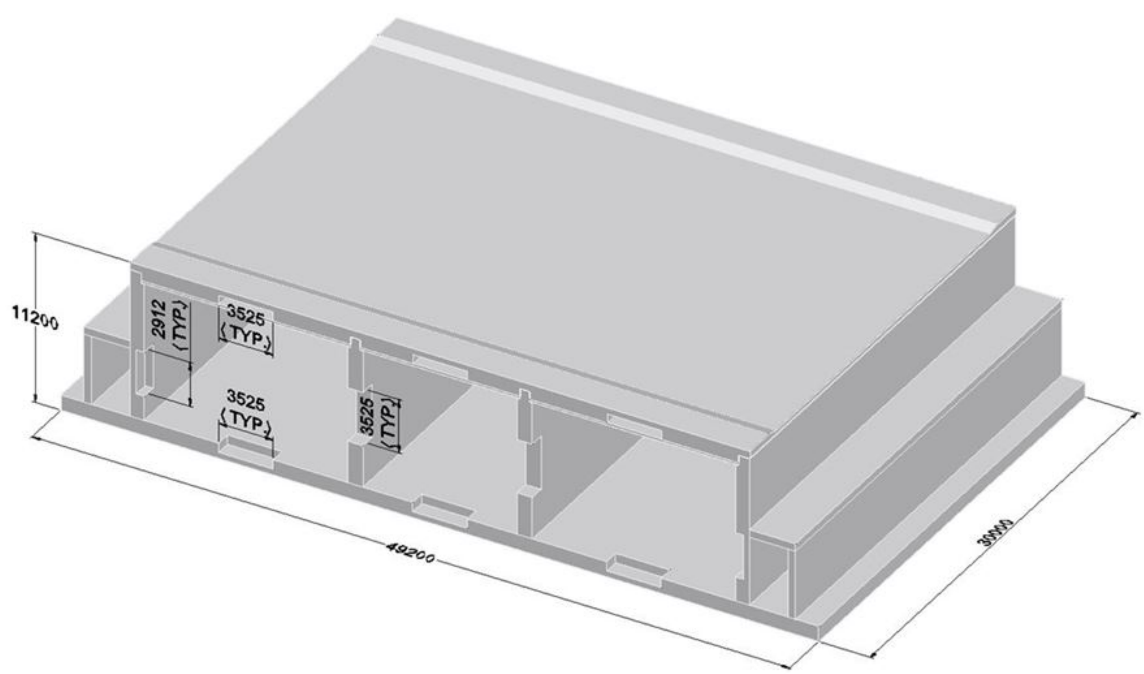

Fig. 10 – Shear Key concept for the tunnel section. Units in millimeters.

8. Seismic Performance Assessment

The tunnel structure was analyzed non-linearly using pushover analysis that considers effective stiffness, expected material properties and hinges according to FEMA 356 [14] for bending concrete members. Racking load patterns (triangular pressure and concentrate force) are added to the static loads and applied monotonically in order to obtain the lateral capacity of the structure or “pushover curve”.

Fig. 11 shows the pushover curve obtained for a typical section and the demanded lateral displacements for 1000-year and 2500-year return period earthquake levels. This analysis shows that the structure remains essentially elastic even for 2500-year return period earthquake level, and fully complies with the seismic design criteria. In addition, the structure can resist lateral displacements around twice the design earthquake lateral displacements with minimum or no damage.

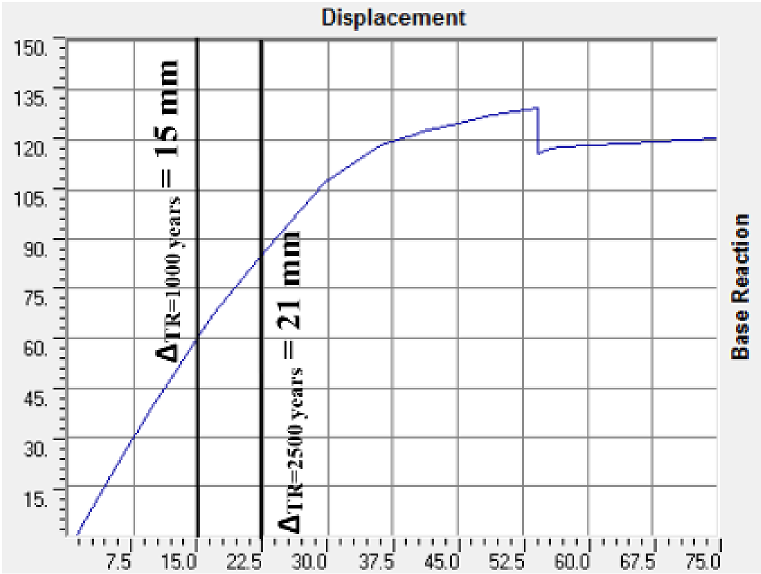

Fig. 11 – Pushover curve of typical tunnel section.

9. Construction

Construction takes place in an open dry trench. Prior to the excavation, the water level was lowered to a level below the planned excavation in order to get to “in the dry” condition, for this purpose a series of pumps were installed on site during all construction time.

The tunnel walls and bottom slab were cast-in-place (see Fig. 12 and Fig. 13); the top slab (roof) consists of precast inverted tee-beams (see Fig. 14a and Fig. 14b), and topped with cast-in-place concrete (see Fig. 14c), rough interface was prepared accordingly.

Fig. 12 – (a) Excavated trench. (b) Bottom slab reinforcing. (c) Bottom slab and wall dowels.

Fig. 13 – (a) Bottom slab reinforcing and wall dowels. (b) Wall reinforcing. (c) Walls.

Fig. 14 – (a) Erection of precast inverted T-beams. (b) Precast inverted T-beams set on walls. (c) Roof reinforcing.

The typical pour sequence is of 30 m length, as is the typical tunnel module, so there are no vertical construction joints; an exception is in the 2.00 m thick bottom slabs of two modules of ramp sections.

Due to the importance of this facility, it was decided to follow a design for complete water-tightness in order to prevent any water infiltration. For this purpose, the tunnel section was enveloped with a continuous external water proofing membrane and complemented by sealing all exterior joints between modules with waterstop elements (see Fig. 15a). To ensure water-tightness, waterstop elements were specified to take any seismic induced deformation while resisting water pressure.

The ground cover over the roof of Gambetta tunnel (see Fig. 15b) has a variable depth from 2.50 m to 3.93 m, which includes a 2.00 m structural fill as part of the new airport runaway. Fig. 16 shows Callao tunnel views from the outside and inside.

Fig. 15 – (a) External waterproofing system. (b) Fill cover over roof. (c) Open trench construction.

Fig. 16 – Gambetta tunnel: (a) aerial view from tunnel section; (b) aerial view from ramp section; (c) inside view.

10. Conclusions

This paper presented the key points in the seismic design and construction of Callao Tunnel, which is one the first projects of this type built in Peru. The “Racking Displacement Method” is used for the final seismic design. This method has proven to be very useful and easy to apply, so it is recommended for use in the design of underground box type structures in areas of high seismicity. In addition, this design is not only safe but also economical compared to the application of the traditional Mononobe-Okabe theory.

Finally, the seismic assessment for 1000-year and 2500-year return period earthquakes was done using current Performance-based earthquake engineering procedures. It allows to prove that the tunnel response is essentially elastic even for the 2500-year return period earthquake.

In order to avoid the interruptions in the service and very difficult inspection and repairs works in the underground facility, it is recommended to design transportation facilities like this one for the following seismic performance: (a) to withstand a 2500-years return period earthquake with a reparable damage; (b) to withstand a 1000-year return period return with minor damage; and (c) to respond without damage for frequent earthquakes.

11. Acknowledgements

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