Incrementally Launched Viaducts of the Northern Marmara Highway: Design and Construction in High Seismic Areas

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Abstract

Launching bridges is an efficient and fast way of constructing viaducts. The method involves setting up a casting yard, forming each span in segments, and launching each successive span over piers creating a continuous structure. This technique is used for post-tensioned deck bridges as well as steel girder bridges. Depending on the size and slope of the deck, launching of a concrete deck may take approximately 5meters/hour. Apart from the savings in the construction time and material quantity of the continuous post-tensioned deck, pier and foundation optimization under seismic load effects generates additional savings. As part of the Northern Marmara Highway, three viaducts; V06 (448m), V14 (429m/282m) and V17 (642m) are designed and constructed using this method. Pier heights range from 8m to 81m. Given the difference in pier heights, under seismic load effects, the shortest piers have to be disconnected from the deck, otherwise they will increase the longitudinal seismic force significantly and attract most of this demand. Therefore, the flexibility and higher effective period of the tall piers are taken advantage of to create economy in design. In longitudinal direction only a few tall piers are fixed to the deck, leaving the rest free to slide: to increase the effective period. In transverse direction, all piers are fixed to the deck with an innovative portal frame shape in order to create similar stiffness and distribute the transverse loads more equally between the different piers. Freysaş is also involved in the design and execution of railway bridges using incremental launching. The BTZ viaducts in Algeria (post-tensioned concrete deck) and AKHG viaducts in Ethiopia (steel girder) are launched railway viaducts. The piers are designed to stay elastic longitudinally. Material optimization, bearing

arrangements, and the pier cross-section layout of the Northern Marmara Highway viaducts will be presented in detail.

1 Introduction

Incremental launching method is one of the bridge construction methods which gives advantages for long span and multi span continuous bridges. The method is suitable for concrete, steel or composite deck bridges. 40m-60m spans with up to 1200m total continuous deck lengths are built with this method. Casting yard is needed to construct the deck before launching. The temporary sliding bearings are used to decrease launching effects on the substructure with low friction coefficient of the sliding material during launching. Also, it is possible to use the temporary support/pier during launching to increase the span length. This method allows to shorten the construction time, reduces the material quantity by using hyperstatic continuous deck, and decreases the number of bearings and expansion joints.

Three viaducts of the Northern Marmara Highway are built with ILM method. Maximum span length is 55m and total continuous deck lengths vary between 282m to 642m. The deck is 22m wide single cell box girder section. The pier sections are made of two parts: wall section in upper part and rigid shaft section in lower part. This configuration of the sections is important to get equally horizontal force distribution under seismic actions on the piers. In the seismic design of the viaducts, the tallest piers (3 or 4 of the piers) are fixed in longitudinal direction in order to increase the natural period of the system and reduce the seismic acceleration. In transverse direction, all piers are restrained to deck, and pier sections are formed to get similar stiffness for each piers.

In this paper, the seismic design details of the Viaduct V06, which has tallest pier with 81m height will be presented.

2 Design of Precast System

Precast system consists of 17 precast I girders with 40m length from bearing centerline to bearing centerline. Elastomeric bearings (300x400x85) are used to connect the girder to pier cap. The girders work as simply supported beam, connected with link slab to form 3 to 4 span modules. Each span module is separated from the adjoining module by expansion joint. The girder depth is 1.8m and total deck depth is 2.05m including top slab. The equivalent thickness of the deck is 0.8m with 17.52m2 cross sectional area.

30 strands with 15.2mm diameter are placed in each girder. Strands used per girder concrete volume is 56.9kg/m^3. This ratio is equal to 36.9kg/m^3 when the top slab volume is added to the concrete volume of the girder.

Constant and one type of cross section (9.0m x 4.5m with 0.7m thickness) is used for all piers. For seismic design of the piers, response modification factor is taken as R=3 per AASHTO [1] to allow the plastic hinge in both directions.

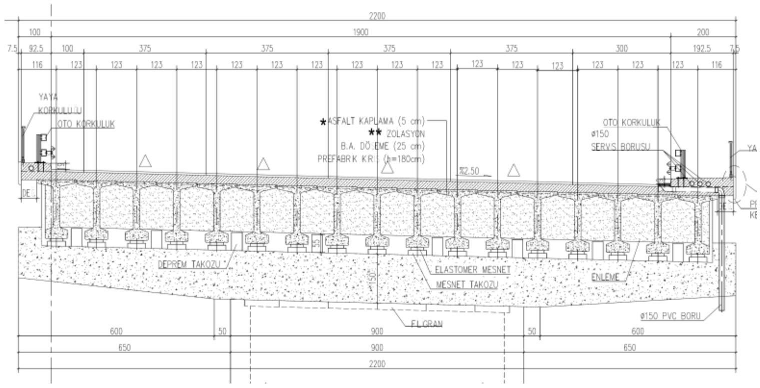


Figure 1. Cross section of precast system.

3 Design of ILM System

The Viaduct V06 has been chosen to present the design detail and to compare with the precast system.

3.1 Viaduct Details

The viaduct V06 consists of two bridges side by side and total bridge length is (38m+4X50m+3X55m+42m) 445m. It has a horizontal curve with 1,500m radius in plan.

The single cell box girder with 22m width and 3.4m depth is designed as post-tensioned in longitudinal and transverse direction. In the deck design, live loads are H30 S24 truck loading and 15kN/m lane loading with concentrated load of Pm=135kN as per KGM Technical Specification [2]. According to motorway width (19m) 5 design lanes of 3.0m as per AASHTO [1] are considered.

The launching post tensioning tendons are placed at the top and bottom part of the cross section. 16 tendons (8 no 19C15 and 4 no 13C15) are placed at the top and 4 no 19C15 tendons are placed at the bottom. In the first two segments, 4 additional tendons 19C15 and 2 additional tendons 19C15 are placed at the top and bottom to take additional bending because of the launching nose. Total quantity of the launching PT is 208.43tonnes. 4 tendons 19 or 25C15 in typical spans and 2 tendons 19C15 in end spans are placed as an external continuity prestressing. Total longitudinal PT quantity is 250tonnes. In the section, transverse PT tendons 3B15 are located with 0.7m spacing for cantilever slab and 0.35m spacing for inner slab. Transverse PT quantity is 72.6tonnes. The total PT quantity for the whole bridge is 322.6tonnes.

Figure 2. Viaduct V06

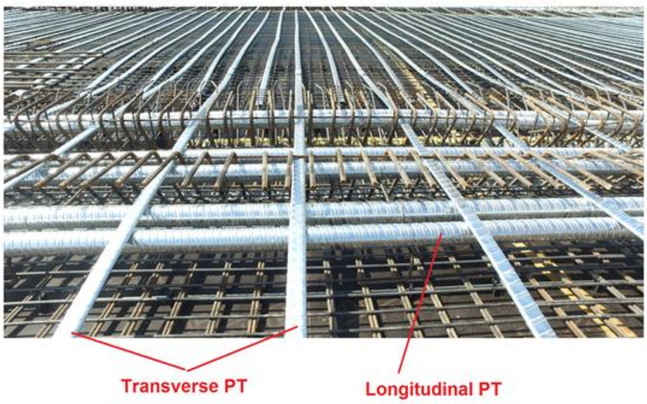


Figure 3. Transverse and longitudinal PT layout.

Pier heights vary between 10m to 81m. The wall section height is 19m and it is constant for all piers. If the pier height is greater than 19m, the shaft section is used.

Figure 4. Viaduct V06 – Elevation view

The purpose of using a flexible wall section is to decrease the pier stiffness and provide to get similar stiffness in transverse direction.

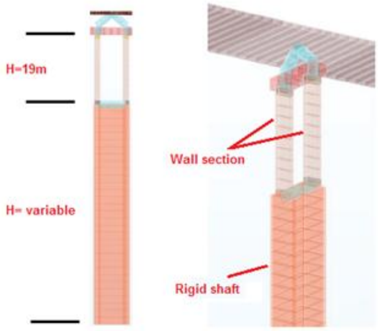


Figure 5. Section configuration of piers.

The deck connection on the piers has importance with regards to distribution of the seismic action. In longitudinal direction, the deck is fixed on 4 tall piers, and free to slide on the others. Transversally, the deck is fixed on all piers. In longitudinal direction, the fundamental period of the system is about 5.0 sec, and corresponding displacement of the fundamental period is 506mm. Hence, longitudinally, two fluid viscous dampers (FVDs) are placed at each abutment to reduce the seismic effects on the fixed piers and limit the pier displacements. The maximum force of the FVDs is 2,200kN with 180mm displacement capacity.

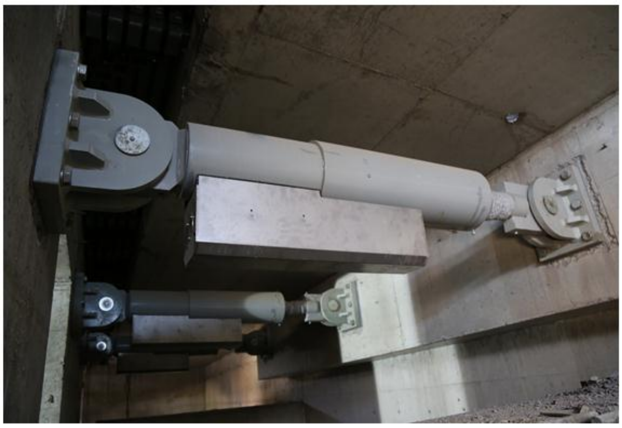


Figure 6. FVD application for bridge.

Bearing definition on the piers are shown in Figure 7.

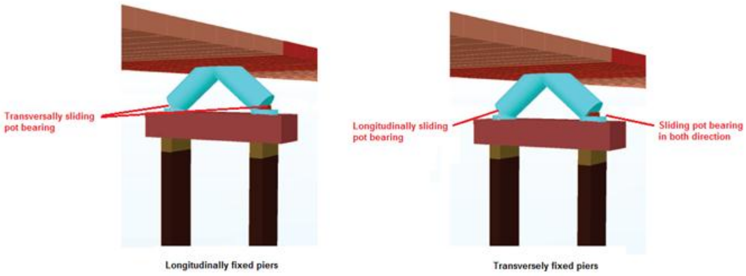


Figure 7. Bearing definition on longitudinally and transversally fixed piers.

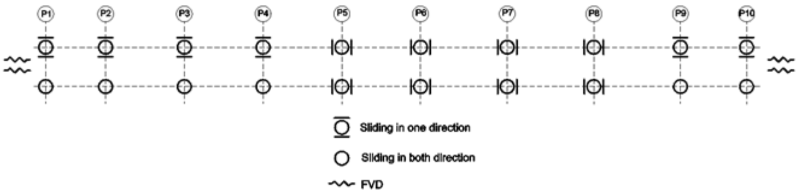


Figure 8. Bearing arrangement.

3.2 Comparison of the Precast System and ILM System

ILM system brings great advantages to this project. With the ILM system, the material quantity is decreased significantly, creating cost savings. The details are shown in Table 1. Moreover, using expansion joints at only the abutments by the help of the continuous deck, decreased the initial and long term costs of the bridge. Two sets of bearings under the deck section at each pier and abutment are used. The precast system requires two bearings under each I-girder.

Table 1. Comparison of the Precast and ILM System.

Item	Quantity	Precast system	ILM system	Variation
Deck	Concrete (m^3)	110,033	101,476	-8%
	Rebar (tonnes)	22,375	15,221	-32%
Pier	Concrete (m^3)	113,672	48,162	-58%
	Rebar (tonnes)	23,669	12,041	-49%
Total	Concrete (m^3)	223,706	149,639	-33%
	Rebar (tonnes)	46,044	27,262	-41%
Number of bearings		374	20	-95%
Number of expansion joints		4	2	-50%

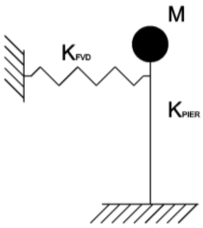
3.3 Seismic Design of the ILM System

3.3.1 Response Spectrum Analysis

In the seismic design, site specific spectrum was considered per DLH [3]. Two levels of earthquake were considered in the seismic check. For small earthquake (72 years return period earthquake), it was assumed that there was no plastic deformation and the structure has an elastic behavior with no yielding of the reinforcement. Under moderate earthquake (475 years return period earthquake), structures are allowed to respond into the inelastic range, so that the damages are repairable.

Design under moderate earthquake is based on the capacity design with ductile behavior. The plastic hinges will be form at top and bottom of the wall system. The response modification factor is taken R=4 in transverse direction and R=1.5 in longitudinal direction for ductile members. In longitudinal direction, the structure has an isolation system through the fluid viscous dampers (FVD). Response modification factor is equal to 5 for multiple column system without the isolation in transverse direction. According to clause 6 AASHTO Isolation Guide [4], the response modification factor shall be taken half of the factor for non-isolated bridges, and it shall not be less than 1.5. Thus, the response factor has been adopted as R=2.0 instead of R=1.5.

In the longitudinal direction, 2 FVDs on each abutment is used to reduce the longitudinal seismic displacements. The isolation system is simplified as a single degree of freedom system with equivalent stiffness and equivalent damping of the FVDs as per in clause 7.5.4 of EN 1998-2 [5] and Setra [6]. Iterative calculation is done to find the design displacement with limited equivalent damping of 30% for the simplified method in clause 7.5.3 of EN 1998-2 [5].



Effective period:

$$T_{eff} = 2\pi\sqrt{\frac{M}{K_{eff}}}$$

Spectral acceleration: $S_a(g)$

Damping correction factor:

$$\eta = \sqrt{\frac{0.10}{0.05 + \zeta_{eff}}}$$

Spectral displacement:

$$S_d = S_a(g)(\frac{T_{eff}}{2\pi})^2$$

Damped displacement:

$$D = S_d.\eta$$

Figure 9. Single degree of freedom system.

Before starting the iteration, initial values are taken as 30% for damping and 178mm for displacement. Total mass of the system is 26,750tons (262,417kN). Total stiffness of fixed piers is 34,776kN/m. The calculated period for 178mm displacement with initial damping is 3.62sec. Then, the system stiffness is found as Keff=80,500kN/m considering total mass and system period. As a results, the stiffness of FVDs is calculated as KFVD=Keff–Kpiers= 45,724kN/m.

For the response spectrum analysis, global system is modelled in Sap2000 Software [7], and FVDs are defined as spring elements with their equivalent stiffness calculated iteratively. In addition, 30% equivalent damping of the system is defined as modal damping in the model. According to AASHTO Isolation Guide [4], this damping shall be considered for only the modal periods greater than the 0.8 of the effective period. In other cases, 5% damping shall be used for all other modes. The corresponding response spectrum is given in Figure 10.

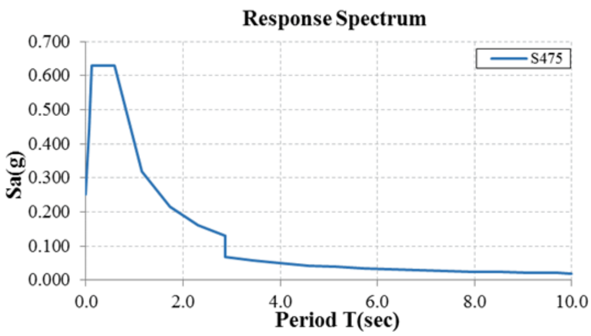


Figure 10. Damped spectrum with %30 damping.

3.3.3 Non-linear Time History Analysis

Time history analysis is done as per EN 1998-2 [5]. The analysis should be performed when the equivalent damping exceeds 30%. Per this method, minimum 7 pairs of earthquake accelerogram records shall be considered. The computer model shall be analyzed under these records, and the design checks should consider the average response of these 7 records. If there are less than 7 accelerogram records, the structure shall be designed under the record which gives maximum effects.

Time history analysis of the 3 viaducts are performed considering the fluid viscous damper properties. FVDs are modelled with their non-linear force displacement behavior. In addition, the plastic hinges of the wall system are defined as multi-fiber beam element using the non-linear material model of concrete and reinforcing steel.

Ten earthquake records were provided by Boğaziçi University, Kandilli Observatory and Earthquake Research Institute. In the response spectrum analysis, the seismic load combination in two orthogonal directions are considered (1.0Ex+0.3Ey; 1.0Ey+0.3Ex) per AASHTO Isolation Guide [4]. However, in time history analysis two orthogonal earthquakes are applied at once with 1.0 factors, and the corresponding displacement is calculated as the square root of sum of squares.

Time history analysis is performed in SAP2000 Software [7] and Code Aster Software [8]. In the analysis, all concrete members are assumed as ductile element, and the non-linearity is considered only at the plastic hinge regions. The plastic hinge length is calculated per Caltrans [8]. Non-linear properties of the FVDs are defined in the model as link element properties.

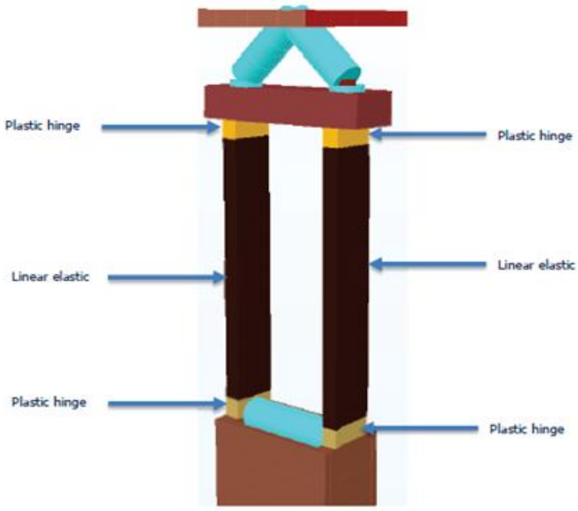


Figure 11. Plastic hinge locations.

When the deck displacement is calculated as 179mm from response spectrum analysis, the average result of the ten times histories is calculated as 104mm. It can be argued that the time history analysis gives more realistic results as compared to the response spectrum analysis.

4 Conclusions

In this paper, alternative bridge systems for viaduct V06 of the Northern Marmara Highway is presented. The comparison of the precast system and ILM system is summarized with details. It is shown that the seismic analysis of the ILM system yields significant reduction in material quantity. Moreover, comparison of seismic analysis methods have been presented. The seismic displacement calculated from the time history analysis is 40% less than the results obtained from the response spectrum analysis. However, the time history analysis is less computationally efficient than the response spectrum analysis, which in our cases provided more severe results for the seismic design of the bridge.

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