Challenges in seismic design of incrementally launched bridges of Northern Marmara Motorway

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Abstract

This paper presents the challenges in the seismic design of the incrementally launched bridges of the Northern Marmara Motorway project in Istanbul Turkey. First it describes the initial design for these viaducts, then it presents the alternative design based on incremental launching method (ILM) and finally does a comparison of their performances. Regarding the earthquake resistance studies, traditional multimodal response spectrum analysis was carried out. This was then completed by a nonlinear time history analysis considering the behaviour the fluid viscous dampers, the cracking of concrete and steel rebar yielding at the plastic hinges. The results of these two analyses are compared in terms of forces and displacements, indicating the conservatism of the modal spectral analysis.

Keywords: bridge, launching, damper, plastic hinge, nonlinear time history analysis

1 Introduction

The Northern Marmara Motorway is a 115 km long ring road around Istanbul. It includes the iconic 3rd Bosphorus Bridge designed to be the symbol of modern Turkey, but also not less than 37 bridges spread over both the European and the Asian sides. Most of these bridges are made of precast I-girders, however three of these bridges are constructed using the incremental launching method (ILM). The three ILM bridges of Northern Marmara Motorway project, also designated as the 3rd Istanbul Ring Road project, are viaducts V6 Left and Right which are 445 m long, viaducts V14 Left and Right which are 427 m and 280 m long respectively, then viaducts V17 Left and Right which are 640 m long. At the time of writing this paper, the launching of viaducts V6 and V17 is completed, while for viaduct V14 the work is in progress. We will focus on viaduct V6 in the following, all three viaducts being similar.

2 Presentation of the original design based on precast I girder

Conforming design was based on precast I girder, length 41 m maximum for span length of 40 m from bearing centreline to bearing centreline and 43,5 m maximum from pier centreline to pier centreline. The precast I girders are placed on laminated elastomeric bearings (300 x 400 x 85. mm). The simply supported spans are connected through link slab, to form 3 to 4 spans modules, an expansion joint being placed in between each module. The total cross section of each deck is 17,52 m² that is 0,8 m equivalent thickness. Each individual girder has a depth of 1,8 m without the top slab, and 2,05 m with the top slab. The slenderness ratio is 1/19,5 considering the total depth and the span length from bearing centreline to bearing centreline. Each beam weighs 68,4 tons and is placed with beam launching gantry. Beams are precast on site in several precast yards. Each beam has 30 strands of 15,2 mm diameter, providing a ratio of 56,9 kg/m^3 if we consider the PSC beam alone, or 36,9 kg/m^3 if we consider the beam and the top slab.

Piercap or headstock are large reinforced concrete element with inverted T shape cross section. The piers have constant cross section of 9 m x 4,5 m with 0,7 m wall thickness. Piers are designed for earthquake with plastic hinges for both directions, with a response modification factor of R=3 which is consistent with ref [1] requirements.

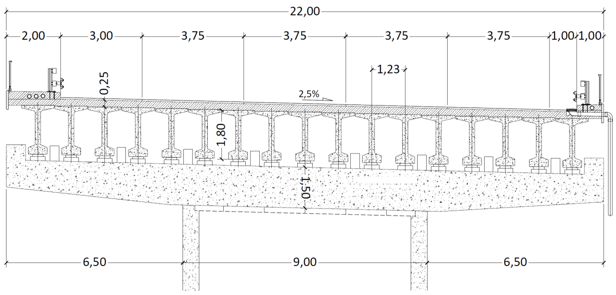


Figure 1. Cross section of the original design

3 Presentation of the alternative design with incrementally launched bridge

3.1 Overview

The alternative design offered by Freyssinet is based on conventional incrementally launched bridge. While this construction method is mastered in many countries, it is rarely used in Turkey, making this project unconventional in many ways. The deck is continuous from one abutment to the other. Each deck is a single cell box girder 22 m wide, for a section of 13,72 m² at midspan and 14,24 m² on support, which correspond to equivalent thickness of 0,624 m and 0,647 m respectively.

Deck is launched uphill from abutment 1 to abutment 10 using strand jacks attached to the abutment. The deck geometry has a constant radius in plane of 1 500 m. In elevation, the profile is made of a sag, a straight alignment with 1.5% slope and then a hog. The bottom launching surface of the deck had to be modified into a circular arc laying on an inclined plane, while the top running surface of the deck remained unchanged. Difference of profile between top and bottom was achieved by varying the depth of the box girder by not more than 255 mm. The deck is broken down into segments whose length correspond to span length with concrete joint located closed to contra flexure point. It is interesting to notice the fact that it is unusual to have launching segments up to 55 m length. It is beneficial for cycle time but increases the investment on the casting bed.

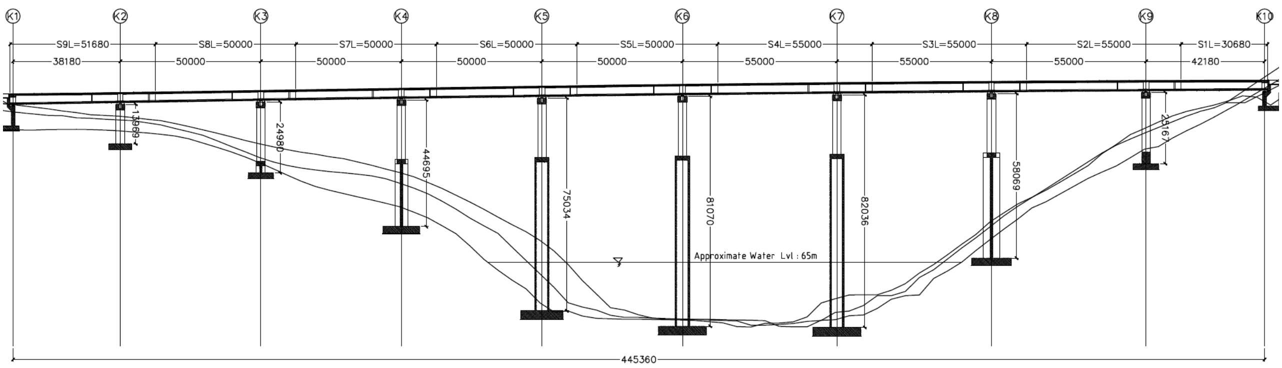


Figure 2. Elevation of viaduct V6

Figure 3. View of viaduct V6

3.2 Deck design

The cross section is 3,4 m deep for a maximum span of 55 m thus a slenderness ratio is 1/16 which is typical for incrementally launched bridges. Live loads are H30 S24 as per KGM Technical Specification for highway bridges in Turkey. Roadway width measured between curbs is 19 m which corresponds to 5 design lanes of 12 feet as per AASHTO. Live load shall be the highest between lane loading and truck loading. H30 S24 truck loading corresponds to 3 axles of 60 kN-240 kN-240 kN with 4,25 m spacing between first two axles, and 4,25-9 m between last two axles. H30 S24 lane loading is made of 15 kN/m combined with a concentrated load of Pm= 135 kN for moment.

The launching post tensioning tendons are concentric and made of 19C15 and 13C15 tendons, using 15,7 mm diameter strands with 1 860 MPa ultimate tensile strength. In typical segments, we have 12 x 19C15 tendons at the top and 4 x 13 C15 plus 4 x 19C15 tendons at the bottom. In the first two segments, to take the additional bending at the back of the launching nose, 4 additional 19C15 and 2 x 19C15 tendons were provided respectively at the top and the bottom. Total tonnage for launching PT is 208,43 tons. Launching tendons are overlapped on 4 meters. At the stressing end, post tensioning anchorages are located on the concrete face in the casting bed, and at the non-stressing end, pockets are provided to fit the stressing block and the wedges. With pockets, couplers and blisters are avoided; this simplifies the internal formwork of the box girder and increases efficiency for post tensioning work. Launching tendons are two segments long, with fifty percent of the launching tendons being overlapped at each construction joint.

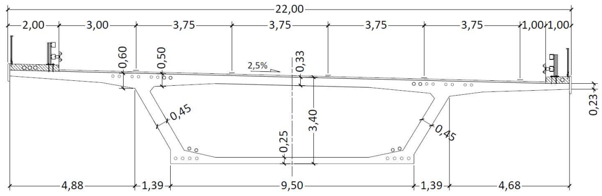


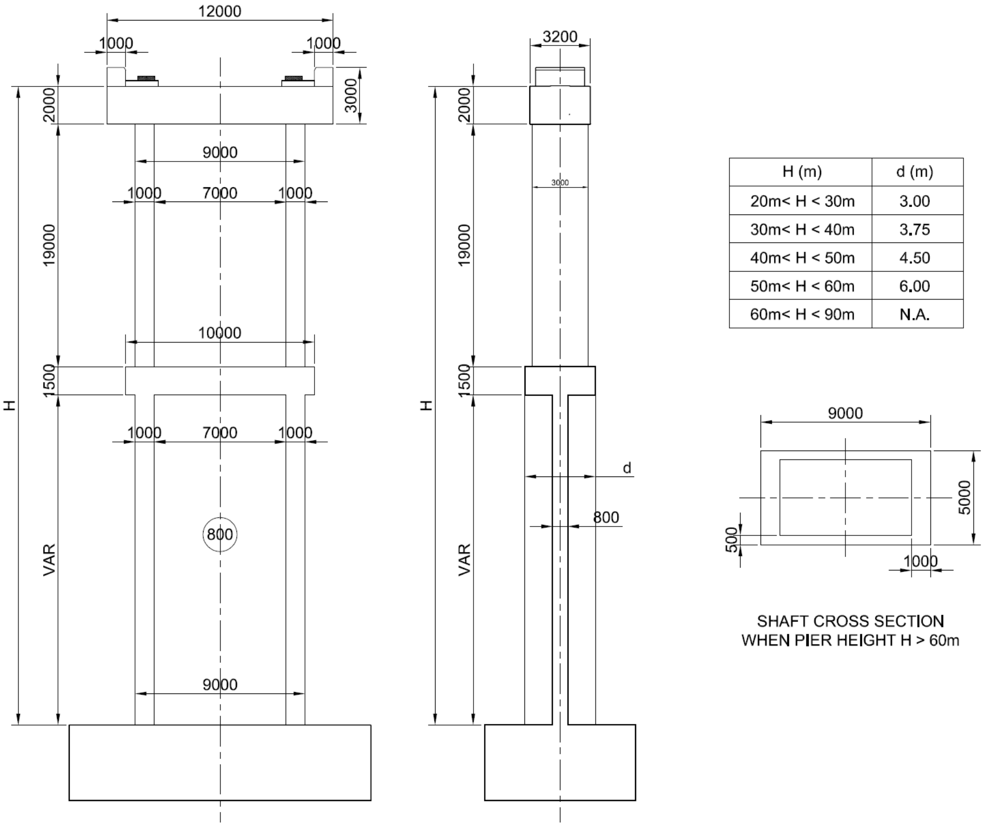
Figure 4. Typical cross section

Continuity prestressing is made of 4 external tendons 19C15 or 25C15 in typical spans, and 2 x 19C15 tendons in end spans. External tendons are made of bare strands injected with cement grout in HDPE duct. Tendons are two spans long and fifty percent of the external tendons are overlapped across pier diaphragm. External tendons are deviated in two lower deviators located consistently at 14,5 m from pier centreline. Tonnage of external tendons is 41,32 tons. Total longitudinal prestressing tonnage is then 250 tons which corresponds to a ratio of 38,6 kg/m^3 which is typical for incrementally launched road bridges made of prestressed concrete box.

Transverse PT tendons are also provided with 3B15 flat anchorages. Transverse tendons run from tip of the top slab to the top of the opposite web, by alternating between left and right side, in this way the spacing between transverse tendons is 0,7 m in the cantilever top slab and 0,35 m is the inner top slab. In other words, transverse post tensioning tendons density is twice as large in the inner part as in the cantilever part. This arrangement is structurally efficient and cost effective since at each and, one anchorage is replaced by a dead end. Tonnage of transverse tendons is 72,6 tons which corresponds to 7,4 kg/m^2, which again is a typical ratio. As for the passive reinforcement ratio, this is about 165 kg/m^3 for the deck.

3.3 Pier design

Longitudinally, the deck is fixed on 4 central piers (5 to 8) and sliding free on the other piers and abutments, while transversally the deck is restrained on all piers and abutment. The fixed piers being tall and flexible, the forces arising from creep, shrinkage and temperature are small in comparison with the seismic forces. The number of fixed piers was chosen to reduce the period of the fundamental longitudinal vibration mode. With 4 fixed piers, the period of this longitudinal mode is equal to 5 seconds, which was considered as an upper bound. The piers are quite slender in the longitudinal direction, with slenderness ratio approaching the limit of 100 above which second order analysis are necessary to encounter for P-Delta effect (see clause 8.16.5.1 of ref [1]). To reduce the longitudinal seismic demand on the fixed piers, two fluid viscous dampers with maximum force of 2 200 kN each were placed at each abutment. The bridge being launched, a built-in longitudinal capacity exists at least in one abutment that exceeds the forces imposed by the fluid viscous damper. Hence incrementally launched bridges are well suitable for seismic devices to be attached between the deck and the abutment, because we reuse the launching capacity at the abutment to withstand seismic forces.



H (m)	d (m)
20m< H < 30m	3.00
30m< H < 40m	3.75
40m< H < 50m	4.50
50m< H < 60m	6.00
60m< H < 90m	N.A.

Figure 5. Pier concrete outlines

Transversally, the pier shape was dictated by seismic design. Each pier has an identical portal frame at the top. The portal frame allows significant reduction of stiffness of the pier in the transverse direction, and also smooth out the differences of stiffness between piers of variable heights. Accordingly, the transversal stiffness is more governed by the flexibly of the portal frame rather than by the pier height. The portal frame was designed with plastic hinges to form both at top and bottom of each leg. As in annex C of ref [2], reduction of stiffness for ductile member was considered following recommendation of clause 5.6 of ref [3]. Taking into account this stiffness reduction, the fundamental transversal mode has a period of 2,4 seconds.

Overall, the reinforcing steel ratio was about 200 kg/m^3 in the piers, but it went up to 300 kg/m^3 in the legs of portal frame, due to high concentration of steel in the plastic hinges areas.

3.4 Comparison between the original and final design

The ILM alternative design brought many advantages to the project. The main rationale for comparison lies in the quantity savings as it can be seen in table 1. Reducing overall quantities means of course reduction of the cost, and also reduction on the footprint on the environment.

Other advantages were overcoming the difficulty of precast beam erection in deep valley, enhancing the safety during construction and introducing innovative seismic design approach, with seismic devices like viscous dampers and isolation through flexible portal frames at pier top.

Item	Quantity	Conforming Design (Precast I girder)	Alternative Design (ILM box girder)	Variation
Deck	Concrete (m^3)	110 033	101 476	-8%
	Rebar (tons)	22 375	15 221	-32%
Pier	Concrete (m3)	113 672	48 162	-58%
	Rebar (tons)	23 669	12 041	-49%
Total	Concrete (m^3)	223 706	149 639	-33%
	Rebar (tons)	46 044	27 262	-41%

Table 1. Quantity comparison between conforming and alternative design

4 Seismic design

4.1 Seismic design with response spectrum analysis

Seismic design was performed as conventional response spectrum analysis. It corresponds indeed to common practice for earthquake engineering and is defined as the reference analysis procedure in clause 4.1.6 of ref [2]. We adopted a response modification factor, also designated as behaviour factor, of 4 in the transverse direction and 1,5 in the longitudinal direction for the ductile members, which are limited to the legs of the portal frame. We note that table 3.7 of division 1A of ref [1] allows to adopt up to 5 for multiple column bent which is the case transversally, and up to 3 for single column bent which is our case longitudinally. In the longitudinal direction however, since we have an isolation system through the fluid viscous dampers, the response modification shall be half of the factor without isolation but not less than 1,5 according to clause 6 of ref [4], that is why we have adopted 1,5 longitudinally.

The spectrum used was a site specific spectrum. Structure had to remain elastic (no yielding of the reinforcement) under small earthquake corresponding to 72 years return period, while under medium earthquake corresponding to 475 years return period, limited damages were accepted with formation of plastic hinges. Non-linear behaviour of the fluid viscous damper was turned into equivalent linear behaviour through equivalent stiffness and equivalent damping, as per the design procedure set out in clause 7.5.4 of ref [2]. In the longitudinal direction this is a one degree of freedom problem that can be solved manually by iterating on the design displacement. Fluid viscous dampers were sized such as to limit the equivalent damping to 30%, which is the upper bond for use of simplified method based on equivalent linear damper properties as defined in clause 7.5.3 of ref [2]. In the global finite element model, the fluid viscous damper was modelled as a spring more or less coincident with deck centroid, whose stiffness was equal to the effective stiffness calculated manually and a modal damping of 30% was applied to the fundamental longitudinal mode only, leaving the other modal damping as 5%. The results of the multimodal response spectrum analysis were almost identical to the results of the fundamental mode spectrum analysis, at least for deck and fixed pier longitudinal displacement, showing the adequacy of this simplified method. However multimodal response spectrum analysis was necessary to capture the longitudinal vibration and corresponding earthquake forces in the free sliding piers.

4.2 Seismic design with non-linear time history analysis

Upon request from KGM, the Turkish administration, Freyssinet was requested to complete the response spectrum analysis by a nonlinear time history analysis in which the fluid viscous dampers are modelled with their intrinsic nonlinear force displacement constitutive model and the plastic hinges are represented using a multi-fibre beam element based on mechanical properties of concrete and steel reinforcement. While it is common practice to use non-linear time history analysis to evaluate with accuracy the response of a bridge isolated with non-linear seismic devices, it is not common to model the material non-linearity of reinforced concrete in the plastic hinges. In clause 4.2.4.1 of ref [1], it is stated that non-linear time history analysis shall only be used in combination with standard response spectrum analysis, to provide an insight into the post elastic response of the bridge, and shall not be used to relax the requirement of the response spectrum analysis.

10 times histories were provided by Bogazici University, Kandilli Observatory and Earthquake Research Institute. The time histories were derived from real seismic events, but they were scaled according ref [5] in order to average the SRSS spectra from all horizontal component pairs does not fall below the corresponding ordinate of the response spectrum used in the design. It is interesting to note that in previous edition of ref [5] or in clause 3.2.3 of ref [2], the average SRSS spectra of all horizontal components had to be above 1,3 and not 1,0 of the response spectrum used in the design. The coefficient 1,3 is consistent with usual seismic combination rule in which final response spectrum analysis obtained after combining 1,0 times the response spectrum in one direction with 0,3 times the response spectrum in the other direction. Reducing this coefficient from 1,3 to 1,0 may cause the non-linear time history analysis to be less severe than the response spectrum analysis, since the overall seismic action is reduced by the scale factor on the time histories.

Clause 4.6.5.2 of ref [6] explains quite precisely the procedure and the types of model to be used when dealing with non-linear time history analysis with plastic hinges. Instead of modelling considering the entire reinforced concrete structure with non-linear constitutive material laws, we concentrated the non-linearity in the plastic hinge regions only. The length of the plastic hinge is computed according to analytical formula in ref [2] and the finite element model integrates the plastic hinge as well as the dampers non linearly, the rest of the model staying as linear elastic model. For our analysis, we used two finite element programs used by two different teams of engineers: SAP 2000 (Computer and Structure, Inc, operated by Freyssinet) and Code_Aster (EDF, operated by NECS). The reason why two FEM softwares were used is to ensure for quality in results.

In SAP 2000, the software can incorporate a plastic hinge through a pre-defined moment curvature diagram calculated automatically by the program based on reinforcement and concrete layout. Or alternatively, it can make use of fibber hinges. This consists in meshing the cross section into steel and concrete fibbers. Each fibber is associated to a stress-strain constitutive law, whether it is unconfined concrete, confined concrete or reinforcing steel.

The software allows distinguishing core (confined) and cover concrete by using analytical formula such as the one elaborated by Mander. The following graph displays the stress strain relationship for concrete, showing that the model of Mander for confined concrete is nearly identical to the model detailed in annex E of ref [2] and also that confined concrete exhibits compressive strength up to 1,85 times the strength of unconfined concrete and an ultimate strain up to 7 times the strain for unconfined concrete.

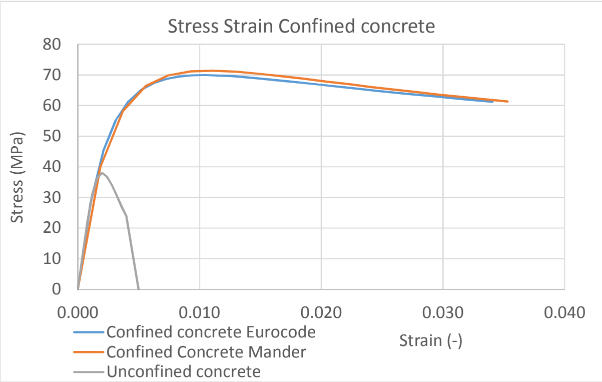


Fig. 6: stress strain relationship for concrete

In the longitudinal direction, the deck displacement found by the finite element analysis is very close to the one obtained through direct integration of the motion equation through manual Newmark integration scheme (one degree of freedom problem). One can conclude that the formation of the plastic hinges in the transverse direction does not affect the longitudinal response of the bridge, hence longitudinal and transverse responses are uncoupled.

Time history	Max Longitudinal displacement (mm)
1	141
2	143
3	203
4	50
5	44
6	44
7	203
8	101
9	91
10	59
Average	104

Table 2. Maximum longitudinal from non-linear analysis

The maximum displacement averaged over the 10 times history analysis is 104 mm as compared to 179 mm in the response spectrum analysis. Therefore in average the response spectrum analysis is providing higher displacements by 40%. However, individual results from the non-linear time history analysis show a significant dispersion around the mean value (see table 2). It is therefore recommended to use a large number of time histories rather than the minimum allowed in order to be able to use the average of the results rather than the maximum as pointed in ref [2] clause 4.2.4.3 and avoid singularity in the results.

In the transversal direction, the yielding of the plastic hinge increases the transverse displacement as well as the pseudo period of the oscillation. In terms of forces, the bending moment transferred to the bottom of the pier by the plastic hinge is capped at the moment capacity of the plastic hinge section, so the forces in the piers resulting from time history analysis don't exceed those from response spectrum analysis. Actually, non-linear time history analysis is more about checking the safety margin with respect to ultimate deformation. The program enables to plot the hysteresis behaviour of the plastic hinge through a moment-rotation diagram (see following graph). The ultimate curvature capacity of the plastic hinge is Φ_u=0,15 m^{-1}, the analytical plastic hinge length is L_p=1,16 m so the design rotation capacity is $\gamma_{p,d}$ = L_p F_u / $\gamma_{R,p}$ = 0,124 rad. In the below plot we see that rotation demand does not exceed 0,02 rad, indicating a comfortable safety factor on plastic hinge failure by excessive deformation.

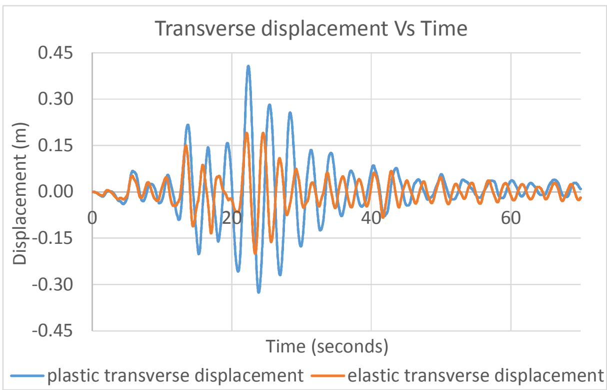


Figure 7. Transverse displacement of the deck above pier 6 Vs time

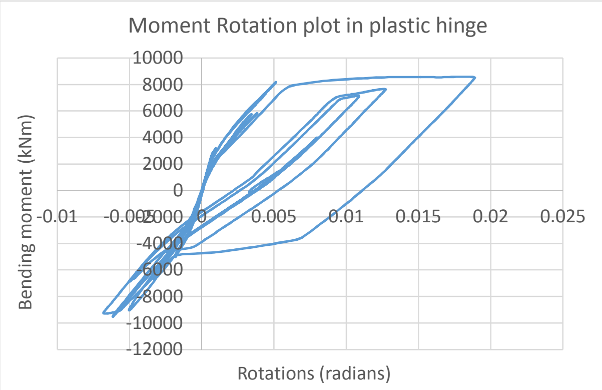


Fig. 8. Moment rotation diagram in plastic hinge from non-linear time history analysis

5 Conclusion

In this paper we have presented the alternative design done by Freyssinet on three of the approach viaducts of the 3rd Bosphorus Bridge in Istanbul Turkey. Alternative design based on incremental launching method optimised the quantity of material very significantly in comparison with the original design based on precast I girder design. As for the seismic design, sophisticated non-linear time history analysis was performed to complete the results of the response spectrum analysis. Longitudinally, the average deck displacement was found to be 40% lower than the corresponding response spectrum displacement. Transversally, the model exhibited clearly the formation of plastic hinge with energy dissipation through steel reinforcement yielding. The rotation demand was less than 14% of the rotation capacity showing a very comfortable safety margin on the plastic hinge failure. Hence the key learning is that plastic hinges designed with response spectrum analysis don't seem to provide structural deficiency due to insufficient rotation capacity. Static push over analysis could also have been performed in order to compute numerically the safety margin on failure by excessive deformation. Finally, complete non-linear time history analysis from both SAP 2000 and Code_Aster provide more accurate and realistic results of the bridge response to earthquake. However such analysis is less computationally efficient and provides less severe results than conventional response spectrum analysis, which we believe is sufficient in most cases and shall remain the reference design procedure for regular bridge earthquake design.

6 References

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