

An Efficient Vertical Seismic Response of a Precast Segmental Bridge Superstructure: A Study

Bhukya Ramu

ABSTRACT: *This paper investigates numerically the vertical non-linear dynamic reaction of an easy span segmental bridge superstructure that consists of ABC layout standards. The superstructure segments are confused collectively via continuous internal unbonded tendons to make certain the system's more desirable self-centering capability. A collection of Incremental Dynamic Analysis (IDA) is carried out with the aid of scaling a set of vertical historic earthquake floor motions to distinctive intensity ranges. The response of these segmental superstructure bridge device is evaluated via assessing the variability of feature response portions and assigning performance limit states.*

KEYWORDS: unbonded tendons; incremental dynamic analysis; non-linear dynamic response; limit states

I. INTRODUCTION

Steel bridges are commonly considered to perform nicely in earthquakes, and the implication is regularly made that they have to be used extra regularly in seismically active areas. It seems that this argument is based totally on the truth that few if any, steel bridges have collapsed in North American earthquakes, in comparison to the overall performance of structural concrete bridges. If a metal bridge is described as one with a metal superstructure and a metal substructure, there are only a few of these in western North America, or even fewer had been subjected to strong floor motions within the remaining decade or so. However, if a steel bridge includes those with concrete substructures ~piers, and columns! The populace will increase significantly, but continues to be a long way less than that of structural concrete bridges ~in western North America!. Even so, overall performance facts for those bridges are tough to locate, and especially for bridges subjected to sturdy shaking.

Nevertheless, it is able to be inferred that metallic bridge superstructures are liable to damage even for the duration of low-to-moderate shaking, and appear to be greater fragile than structural concrete superstructures in this regard if not designed well. Typical harm consists of unseated girders and screw-ups in connections, bearings, pass-frames, and growth joints. In some instances ~appreciably in the course of the Kobe earthquake! Predominant gravity load-sporting participants have failed, brought on in a few times by way of the failure of additives somewhere else inside the superstructure ~a bearing, as an example!. It may also, therefore, be argued that the recognition loved by using steel bridges is because of the reality that very few steel bridges were subjected to robust floor movement, and the absence of disintegrating may be due to a lack of exposure as opposed to the inherent potential of metallic bridges. Supporting this view is the statement that harm at some stage in low-to-moderate shaking shows a degree of fragility in metal bridges not visible in structural concrete superstructures.

It is crucial to word on this argument that seismic layout specs for bridges within the United States do no longer require the specific design of bridge superstructures ~concrete or metallic! For earthquake masses. The assumption is made that a superstructure that is designed for out-of-aircraft gravity hundreds has enough strength, by default, to withstand in-plane earthquake loads. This assumption appears to be justified for structural concrete superstructures, that are heavier and stiffer than their steel opposite numbers but can be unfounded for positive styles of steel superstructures, including trusses or slab-and-girder superstructures, both of which may be flexible in-aircraft.

Improvement in the seismic overall performance of steel bridges is warranted, in conjunction with layout hints for each steel sub- and extremely good-structures. Better perception is required concerning

the weight routes as well as the capacities of person components and assembled systems. Applications of modern technologies, which includes ductile end cross frames or diaphragms! And other embedded energy dissipators, deserve further take a look at.

II. RELATED WORKS

The prototype bridge structure used for this study is the one considered by Megally et al. (2002). It is a single-cell box girder bridge that consists of five spans with three interior spans of 30.5 m (100 ft) and exterior spans of 22.9 m (75 ft) for a total length of 137.2 m (450 ft). Each span of the prototype structure is post-tensioned with a harped-shape tendon. The segmental superstructure model considered in this study represents the centre span of the prototype bridge with two overhangs. The interior span length is 30.5 m (100 ft) and the length of each overhang is 7.6 m (25 ft) for a total length of 45.7 m (150 ft).

In order to comply with the ABC techniques for precast segmental bridges, the superstructure is divided into segments. A segment-to-segment joint is provided at the mid-span section, where high bending moments and low shear forces are induced. Mid-span is also the location where maximum relative displacement of the segments is expected when the superstructure is subjected to vertical seismic loading. The segmental superstructure consists of six interior segments 6.1 m (20 ft) long and two exterior segments 4.6 m (15 ft) long. The precast segments are match cast, which means that each segment is cast against the previous one so that the end face of one segment is an imprint of the neighbour segment. No shear keys or epoxy adhesives are considered at the joints. The uniform behaviour of the system is achieved through post-tensioning of the segments with internal unbonded tendons (DYWIDAG, 2009)

III. MODELLING APPROACH

A two-dimensional numerical model of the segmental superstructure, which incorporates material and geometric nonlinearities, is developed and

analyzed using the inelastic dynamic analysis software RUAUMOKO (Carr, 2007). The segments of the superstructure are modeled using linear elastic body kind contributors besides for a vicinity at the ends of every segment that's discretized into several axial non-linear springs. The springs are connected to the ends of the superstructure beam factor through inflexible body links. Prior to loading, a joint between two adjacent segments is closed and the complete section is in compression. When a vertical seismic load is applied, the joint opens and a compressive contact sector paperwork. In order to correctly simulate the contact region among adjoining segments, the wide variety and distribution of touch springs will be decided on such that the area and 2d second of inertia of the contact springs do not significantly deviate from the place and 2d moment of inertia of the superstructure's go-segment. The hysteresis rule used to version the touch springs is the bi-linear with slackness hysteresis rule (Carr, 2007). A huge initial slackness displacement cost is chosen to make certain that no tensile forces are evolved within the springs.

Based on the geometry of the superstructure's go-section, the touch quarter between two adjoining segments is modeled with 11 nonlinear touch springs, eight of that are located on the pinnacle and bottom plate where most neighborhood deformations are expected. In order to compute the axial stiffness of the touch springs, their length will be predicted primarily based on the geometric traits of the superstructure's pass-segment. For this look at, the portion of the superstructure's segments modeled by using contact springs is assumed to be same to h/eight, in which his the intensity of the move-phase, consequently 23.0 cm (9.0 in) at every end.

The post-tensioning system consists of 80 unbonded mono-strand tendons with a strand diameter equal to 15.2 mm (0.6 in) and an ultimate strength equal to 1860 MPa (270 ksi). In order to minimize the number of numerical elements, the 80 tendons are modelled as a single tendon of equivalent cross-sectional area and parabolic profile. The unbonded tendons are modelled using several bi-linear elastic spring elements in series. More information on the adopted

modelling approach can be found in Anagnostopoulou (2009).

Figure 2.1 shows the geometry of the developed segmental superstructure and the corresponding numerical model. The location of the segment-to-segment joints is indicated together with the various structural elements (e.g., concrete segments/frame members, joints/contact spring elements, tendons/spring elements). By performing a modal analysis of the numerical model, the first natural period of the system is found to be 0.305 sec (3.28 Hz) and the second and third natural periods are found to be 0.106 sec (9.42 Hz) and 0.007 sec (14.14 Hz), respectively.

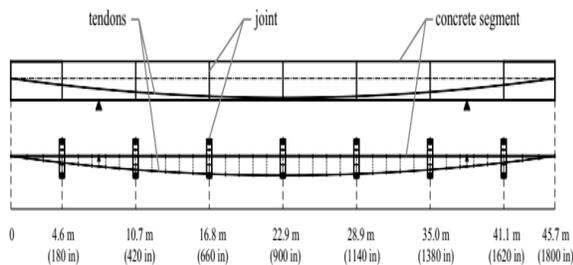


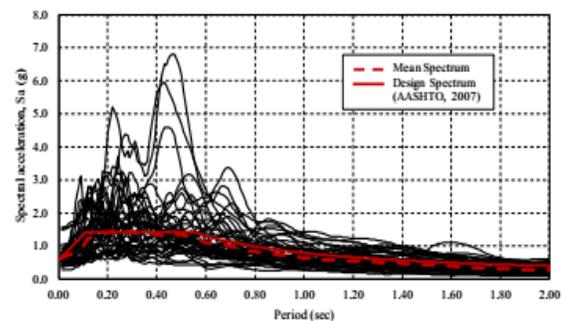
Figure 2.1. Geometry of the one-span segmental bridge superstructure and the developed numerical model

IV. INCREMENTAL DYNAMIC ANALYSIS

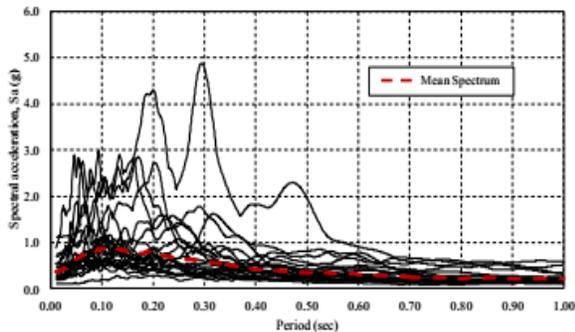
The site location for the bridge model is assumed to be in the Western United States, close to the City of Los Angeles. The design spectrum is selected from the AASHTO LRFD Bridge Design Specifications (2007) and represents a seismic event with a 10% probability of exceedance in 50 years (475-year return period). The acceleration coefficient, A , is assumed to be equal to 0.60 (Seismic Zone 4) and the site coefficient, S , equal to 1.20 (Soil profile type II). Moreover, the Design Earthquake (DE) and Maximum Considered Earthquake (MCE) response spectra are defined according to ASCE/SEI 7-05 (2005). For the specific site location and 5% of critical damping, ASCE specifies the following design and maximum earthquake spectral response

acceleration parameters: SDS equals 1.415 g, $SD1$ equals 0.784 g, SMS equals 2.123 g and $SM1$ equals 1.176 g. The segmental superstructure model is analyzed using the far-field earthquake ground motion ensemble defined in FEMA P-695 (2009). The ensemble consists of twenty-two historical strong ground motions. Initially, a scaling procedure is applied to the horizontal components of the P-695

ground motions: for the DE and MCE intensity levels, the acceleration response spectrum of each record ($2 \times 22 = 44$ records) is multiplied by a factor defined as the ratio of the DE and MCE spectral accelerations, respectively, at a target period equal to the fundamental period of vibration of the superstructure model, over the geometric mean of the forty-four horizontal spectral acceleration values at the same target period. Given that the current bridge design codes provide little guidance on the development of vertical design spectra, the scale factors obtained from scaling the horizontal components of the ground motions are also used for scaling the vertical components of the ground motions to the DE and MCE intensity levels (the vertical component of one record is not available, so 21 records are considered in total). Figure 3.1 shows the results of the scaling method for the horizontal and vertical components of the P-695 ground motions at the DE intensity level.



a) Horizontal response spectra



b) Vertical response spectra

Figure 3.1. Horizontal and vertical Design acceleration response spectra

IV. PERFORMANCE LIMIT STATES

The overall performance limit states of the segmental superstructure can be defined whilst the numerical version is analyzed below a vertical cyclic sinusoidal displacement-managed load, which fits the profile of the version's first mode of vibration. Four overall performance restrict states are identified: the onset of joint commencing; the cracking of the phase; the onset of spalling of the segment's excessive concrete fibers; and either the crushing of the segment's constrained core or the yielding of the tendons. The first overall performance restriction nation, specified via PS1, is related to the onset of joint commencing. According to AASHTO (2007), a segmental superstructure should behave as a monolithic gadget for the serviceability restrict states and allow for joint opening underneath the closing limit states. The allowable compressive concrete pressure for pre-burdened additives with unbonded tendons at carrier restrict kingdom is same to $0.60f_c$ while; no tensile stresses shall develop. Considering a concrete compressive electricity, FC' , equal to 34.5MPa (5.0 ksi), the allowable compressive stress is 20.7 MPa (three.Zeroksi) and the corresponding concrete stain, ϵ_c , is 0.07%. In terms of maximum vertical displacement, the AASHTO (2007) specifies a deflection restrict identical to $L/800$ under the effect of vehicular masses, where L is the span duration of the bridge. For the case of the segmental superstructure version with an indoor span period

same to 30.5 m (one hundred ft), the deflection restriction is about equal to three.8 cm (1.5 in). The 2nd overall performance restriction kingdom, unique by means of PS2, is associated with the initiation of cracking in the ends of each section and adjoining to the phase-to-phase joints. The pressure degree at which cracking of the unconfined concrete occurs is believed to be identical to 0.12% (AASHTO, 2007). The third performance restrict country, detailed by means of PS3, is related to the onset of spalling of the extreme concrete fibers adjacent to the phase-to-segment joints. The pressure stage at which the cover concrete ceases to hold any stresses is assumed to be same to 0.30% (AASHTO, 2007).

The fourth overall performance restriction state, specified via PS4, is associated either with the crushing of the restricted concrete middle or the yielding of the tendons. The strain level at which crashing of the limited concrete takes place is thought to be same to at least 1.0%.

V. CONCLUSION

This paper offers the outcomes of a numerical study that investigates the response of a phase concrete bridge superstructure while subjected to vertical earthquake motions. The recommended device which makes use of inner unbonded tendons as the handiest continuous reinforcement along the bridge's length is designed to show off high ductility and more desirable self-centering competencies. The number one device used in this research is a numerical version, which includes material and geometric nonlinearities, and is analysed below a fixed of multi-data Incremental Dynamic Analysis (IDA).

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BIODATA



Bhukya Ramu Completed B.E-Civil Engg. from Bhaskar Engineering College, M.Tech Geo-Technical Engg. Specilization from vidya vikas institute of engineering and technology, Currently working as site engineer in Automotive Manufacture pvt. Ltd. since 2015.