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Seismic Analysis of Integral Bridges

Shilpa S[1], Thejashwini P T[2], Shruthi N P[3]

[1]Lecturer, Government Women's Polytechnic, Shiralakoppa
[2]SSE/DRG, South Westren Railway
[3]Lecturer, Government Polytechnic, Harihara

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Abstract - *A recent trend in bridge design has been toward the elimination of joints and bearings in the bridge superstructure. Joints and bearings are expensive in both initial and maintenance costs and gets filled with debris, freeze up and fail in their task to allow expansion and contraction of the superstructure. They are also a "weak link" that can allow de-icing chemicals to seep down and corrode bearings and support components.*

The primary concern in the design of integral bridges is that high stresses can develop in the superstructure and substructure as a result of secondary loads because of the continuity connection between the superstructure and the substructure. These stresses are the result of restrained thermal expansion and contraction, creep and shrinkage.

The aim of the Integral bridges has been found to outperform jointed bridges, decreasing maintenance costs, and enhancing the life expectancy of the superstructures and also has been a good choice for high speed railways.

However, a standard design method for integral bridges does not exist. Several factors must still be investigated to gain a better understanding of the behavior of integral bridges, and the factors that influence their analysis, design, detailing, and construction. The reasons for adopting integral bridges in India and elsewhere could be quite different. When earthquake forces are predominant or when considerations like increased resistance to blasts are to be reckoned with or there is a strong need of incorporating reduced inspection and maintenance features in the bridge structures, the integral bridge concept is an excellent option.

Hence this paper presents the seismic analysis of this integral bridge and their behavior for major earthquakes and to determine its suitability and safety in seismic regions especially for Bangalore region.

Key Words*: *Superstructure, Substructure, De-icing, Predominant, Seismic

1. INTRODUCTION

Integral bridges or joint fewer bridges are constructed without any movement joints between spans or between spans and abutments. These Bridges are widely used in countries like U.S.A, Germany, Europe, and china etc., seismically active areas and also in places where the weather effect is adverse and the failure of the bearings is predominant. The use of an integral bridge eliminates the need for deck joints and expansion bearings. The absence of joints and bearings significantly reduces costs during construction. More significantly, maintenance costs are also reduced since deck joints, which allow water to leak onto substructure elements and accelerate deterioration, are not needed. In addition, future widening or bridge replacement becomes easier. However, a standard design method for integral bridges does not exist.

Seismic Analysis of Bridge

The provisions for seismic are provided in IRC - 6(2000).The Bridges in Seismic Zones II and III need not be designed for seismic forces provided both the following conditions are met: a) Span is less than 15m b) Total length of bridge is less than 60m.

All other bridges shall be designed for seismic forces. The code defines response spectra analysis for bridges but whenever the exact analysis is to be performed a linear or non-linear time history analysis is performed.

Integral V/S Jointed Bridges

A review of some of the primary differences between integral bridges and their jointed counterparts should help to clarify why integral designs are gaining widespread interest and acceptance.

1. No Bearings and Joints: Integral bridges can be built without bearings and deck joints. Not only will this result in savings in initial costs, the absence of joints and bearings will reduce maintenance efforts.

2. Simplified Construction: Integral construction generally results in just four concrete placement days. After the embankments, piles, and pile caps have been placed and deck stringers erected, deck slabs, continuity connections, and approach slabs can follow in rapid succession.

3. Minimized Deterioration: The most obvious reason why integral bridges have become so popular, especially with transportation departments located in and above the Snow Belt, is their outstanding resistance to deicing chemical corrosion and deterioration. Since these bridges do not have movable deck joints at abutments deck drainage contaminated by deicing chemicals cannot penetrate bridge deck slabs and adversely affect the primary bridge members.

4. Simplified Bridge Replacement: When using multiple span integral bridges to replace single span structures with wall-type abutments, the great adaptability of integral bridges allows them to span across existing foundations, thus avoiding the need to remove them.

5. Secondary Effects: Like most of their jointed bridge counterparts, integral bridges are subjected to secondary effects due to shrinkage, creep, thermal gradients, differential settlement, and differential deflections.

6. High speed Railways: The structural continuity of integral ridges offers great advantages for high speed railways and also resulting in easy construction and cheap maintenance cost.

7. Other Considerations: Integral bridges should be restricted to sites where not less than 10 or preferably more than 15 feet of overburden is present (to ensure pile flexibility and effective pile end-bearing), to sites where appreciable settlement is remote (these bridges cannot easily be adjusted to compensate for large settlements), to sites where skews of 30 degrees or less are appropriate, and to uncrowded sites where embankments and extra spans can be added to avoid the use of wall-type abutments.

Limitations

Like any other type of design, the attributes of integral bridges are accompanied by some limitations

1. Approach Slabs: Integral bridges should be provided with approach slabs to prevent vehicular traffic from consolidating backfill adjacent to abutments, to eliminate live load surcharging of backfill, and to minimize the adverse effect of consolidating backfill and approach embankments on movement of vehicular traffic.

2. Joints off the Bridge: Cycle control joints, joints which facilitate longitudinal cycling of bridges and approach slabs, should be provided between approach slabs and approach pavement.

3. Pile Loading: One primary concern expressed about the construction of integral bridges with pile supported flexible abutments is the uncertainty about abutment pile flexural stresses.

4. Embankments: Since integral bridges receive significant support from embankments, such bridges should be built only in conjunction with stable, well consolidated embankments.

2. LOADS AND FACTORS INFLUENCING THE BEHAVIOR OF INTEGRAL BRIDGES

Temperature Loading

The Effective bridge temperature difference for Bangalore is given IRC-6 is Max temperature 37.5 degrees Celsius Min temperature 12.5 degrees Celsius

The bridge temperature when the bridge is effectively restrained is given by for places where thetemperature difference is >20^0c the Bridge temperature is given as

Mean of maximum andminimum air shade temperature +10^{0}c whichever is critical. (12.5-37.5)/2=12.5+10 = 22.5^{0}c.

Thermal Stress

High stresses can develop in the components of an integral bridge when the structure undergoes the thermal length changes of its bridge deck. Differences often exist in measured and theoretical temperature induced length changes and is one reason why integral bridges in some states have performed satisfactorily even though structural analysis indicated there should have been thermal stress problems.

These differences can be attributed to errors in the coefficient of thermal expansion, temperature gradients across the bridge cross sections, and resistance to movement provided by the abutment system and the soil pressure, which depend on the poorly understood soil- structure interaction.

Seismic Loading

As per provisions in IRC-6 the seismic analysis is to be performed for Zone II (Bangalore) because the span is greater than 15m and also the total length is greater than 60m.The horizontal seismic force shall be calculated as below
Where,

$$A_h = Z/2I/RS_a/g$$

Z = Zone factor given in Table 5(IRC -6-2000), is for the Maximum Considered Earthquake(MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so asto reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE). I = Importance factor, depending upon the functional use of the structures. Important bridges = 1.5, Other bridges = 1, R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0. The values of R given as 2.5

Sa/g = Average response acceleration coefficient for rock or soil sites (IRC-6-2000) and based on appropriate natural periods and damping of the structure. Thesecurves represent free field ground motion.

Soil Stiffening and Settlement

As the bridge superstructure goes through its seasonal length changes, it causes the structurally connected abutments to move away from the soil they retain in the winter and into the soil during summer which is non- linear with respect to the preceding one i.e.in each winter the abutment moves slightly inward than it did in the preceding winter and the same with the summer. As a result of this net soil displacement towards the abutment, the summer lateral earth pressure over time as the soil immediately adjacent to each abutment becomes increasingly wedged, called “Ratcheting”.

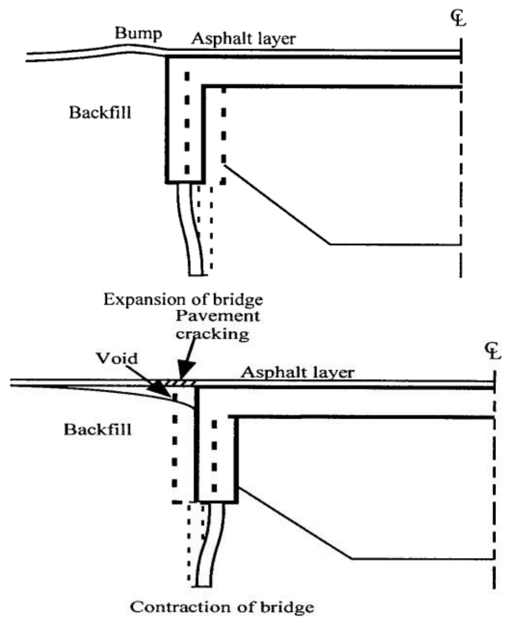


Fig – 1: Seasonal variation of integral bridges

Abutment Wall - Soil Interaction

The soil-structure interaction has the largest single influence on the behavior of integral bridges. Unfortunately, it is also the most difficult to accurately predict because the reactive soil pressures are a non-linear function of the magnitude of the displacement and deflected shape of the wall, and the deflected shape of the wall is a function of soil pressures. Variables that affect the soil-abutment interaction include abutment wall, pile, wing wall, approach slab, and pile configurations; soil characteristics (primarily soil stiffness); total movement; and superstructure stiffness among others.

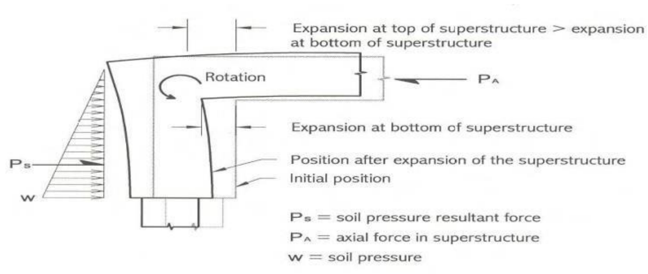


Fig- 2: Abutment rotation as a result of thermal gradient and eccentricity between soil pressure and axial force in the superstructure.

Shrinkage and Creep Effect

Creep and shrinkage effects are assumed to be opposite in nature and hence tend to cancel each other out, they are generally ignored in bridge design. Those maximum shrinkage moments occur within 30 days of form striping with negligible creep effects. Creep effects balance shrinkage effects after 7 to 8 months. For integral bridges, especially those constructed entirely of concrete, shrinkage results in a permanent shortening of the bridge, which will be principally resisted by the bending stiffness of the substructure. Creep, especially for bridges constructed entirely of concrete, will result in a gradual elongation of the bridge, which will be principally resisted by the stiffness of the soil and structure acting together.

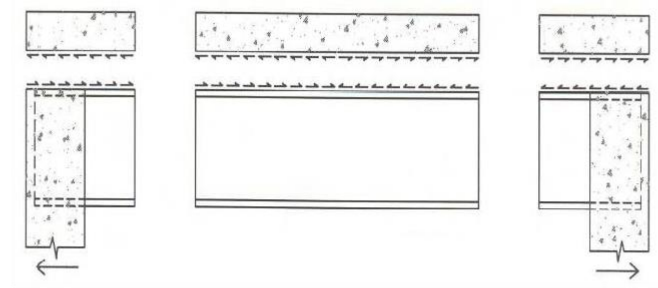


Fig – 3: Shrinkage –induced stresses in a steel girder built integrally with the concrete deck and abutments

3. OBJECTIVES AND SCOPE OF PRESENT STUDY

- To develop the complete three dimensional, finite element model of integral bridge and to validate model.
- To conduct parametric studyby seismic analysis on three dimensional finite element model and to study its behavior under temperature and seismic loading.
- A comprehensive literature review has been under taken and based on the study following conclusions are drawn

i. Lawver *et al*. (2000) and Thippeswamy*et al.* (1995) suggested that thermal-induced movement of an integral bridge caused greater stresses in integral bridge components and hence they should be considered carefully.

ii. Dicleli and Suhail (2003) recommended the maximum length of concrete integral bridges to be 190m in cold climates and 240m in moderate climates and steel integral bridges are limited to 100m in cold climates and 160m in moderate climates. In clay soil, they recommended the maximum length of concrete integral bridges to be 210m in cold climates and 260m in moderate climates and steel integral are limited to 120m in cold climates and 180m in moderate climates. The FHWA technical advisory recommended the following length limits for integral abutment bridges to 91.4m for steel, 152.4m for poured- in-place concrete and 182.9 for prestressed concrete.

iii. The study accounting the effect of predrilled holes with loose granular fill and varying backfill for thermal, gravity and seismic loads is recommended by Farajiet *al.* (2001) to streamline the design process for integral abutment bridge.

iv. The investigation is in process by bridge researchers to develop concepts and simplified procedures for performance-based seismic evaluation of bridges. As far as our knowledge goes, no technical papers are found on the performance-based seismic analysis of integral bridge.

4. NUMERICAL EXAMPLE OF AN INTEGRAL BRIDGE

Superstructure

The 150-m long viaduct portion of the flyover consists of five continuous spans of reinforced concrete (RC) voided deck slab having individual spans of 22.5m + 30m + 40m + 30m + 22.5 m with 2.5m overhang on either side. The total deck width is 9.9m having 9-m wide clear carriageway and 450-mm wide crash barriers (at base level) on either side. The superstructure depth is generally kept uniform (total depth of 1.7m) expect at the location of central piers where it gradually increases to 2.2m.The grade of concrete used for structure is M45. In order to avoid the reinforcement congestion, Fe 500 grade steel has been used in all the structural elements of the flyovers.Reducing the pier thickness from 1.5m (central piers) to 1.15m (outer piers) gradually increasing the pier heights by lowering the top level of footings below ground from 0.5m for central piers to 2m for outer piers.This way the flexibility of the end and next to end piers is increased 6 to 4 times respectively as compared to the central piers.

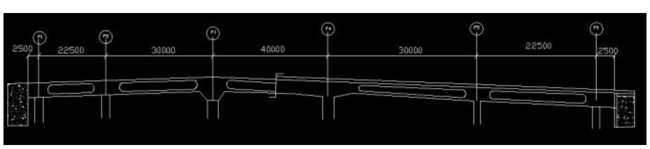


Fig – **4**: Typical sectional elevation

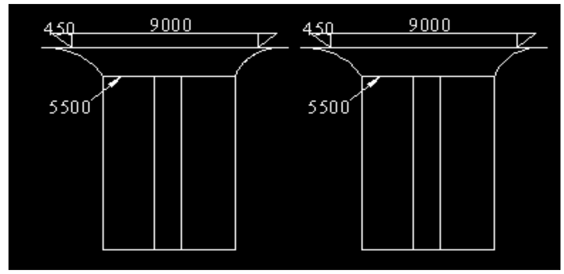


Fig – 5: Typical cross-section of flyover at mid-span and support

Design requirements

The live load on the bridge is considered as per IRC-6 specifications. As per provisions in IRC-6 the seismic analysis is to be performed for Zone II (Bangalore)because the span is greater than 15m and also the total length is greater than 60m and the he load combination considered were as per IRC: 78 and IRC: 6 respectively.

5. FINITE ELEMENT MODELLING

The entire bridge is modelled as single monolithic bridge using bridge wizard tools in SAP(2000). Each element is modelled as node. Nodes are connected using frame element byestablishing connection between the deck and the pier. Substructure pier and abutment are modelled as monolithic

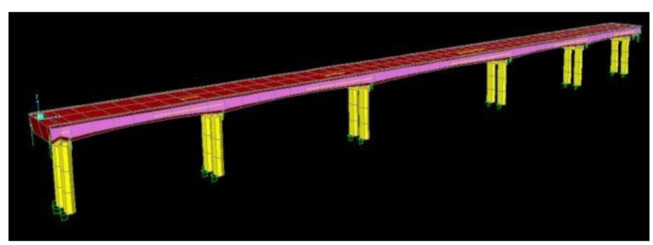


Fig – 6: Modelling of integral bridge

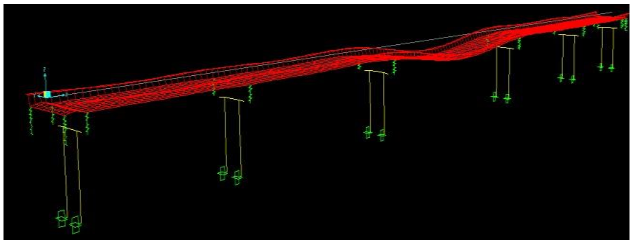
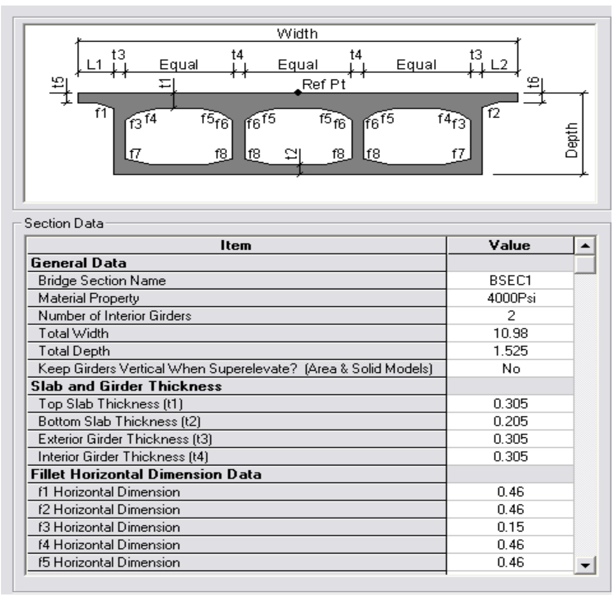
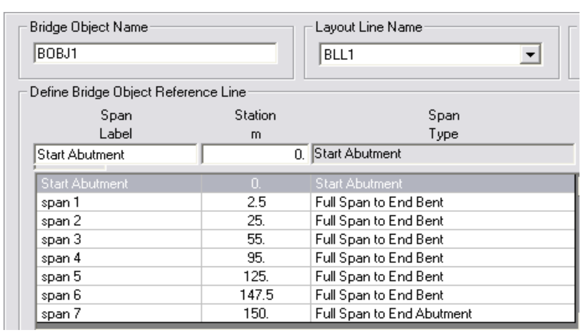


Fig – 7: Deflected shape of an Integral bridge



Item	Value
General Data	
Bridge Section Name	BSEC1
Material Property	4000Psi
Number of Interior Girders	2
Total Width	10.98
Total Depth	1.525
Keep Girders Vertical When Superelevate? (Area & Solid Models)	No
Slab and Girder Thickness	
Top Slab Thickness (t1)	0.305
Bottom Slab Thickness (t2)	0.205
Exterior Girder Thickness (t3)	0.305
Interior Girder Thickness (t4)	0.305
Fillet Horizontal Dimension Data	
f1 Horizontal Dimension	0.46
f2 Horizontal Dimension	0.46
f3 Horizontal Dimension	0.15
f4 Horizontal Dimension	0.46
f5 Horizontal Dimension	0.46

Fig – 8: Deck section details of the considered bridge



Span Label	Station m	Span Type
Start Abutment	0.	Start Abutment
Start Abutment	0.	Start Abutment
span 1	2.5	Full Span to End Bent
span 2	25.	Full Span to End Bent
span 3	55.	Full Span to End Bent
span 4	95.	Full Span to End Bent
span 5	125.	Full Span to End Bent
span 6	147.5	Full Span to End Bent
span 7	150.	Full Span to End Abutment

Fig – 9: Different spans of the considered bridge

1) The entire bridge is modelled in SAP 2000 and the loads are considered as per IRCspecification. The vehicle considered is IRC class 70R loaded on single lane at a time and classA loaded on two lanes at a time. The temperature stresses are applied both as temperature gradient and thermal variation. The live load applied on the superstructure is both as Bridge liveand moving loads.

2) A equivalent earthquake signature of Coalinga, Imperial and El centro earthquake with PGA of 0.151, 0.157, 0.36g is used for time history analysis which is recorded at rock level.

6. RESULTS AND DISCUSSIONS

Longitudinal Variation of Stresses for different loads on bridge

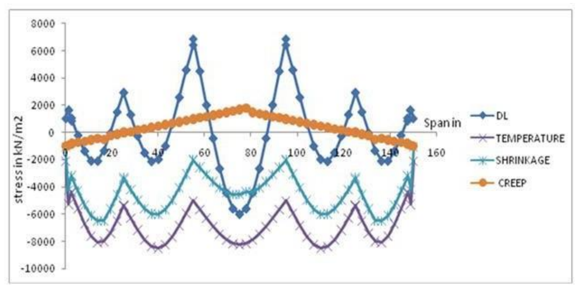


Chart – 1: Variation of longitudinal stresses at top for various loads

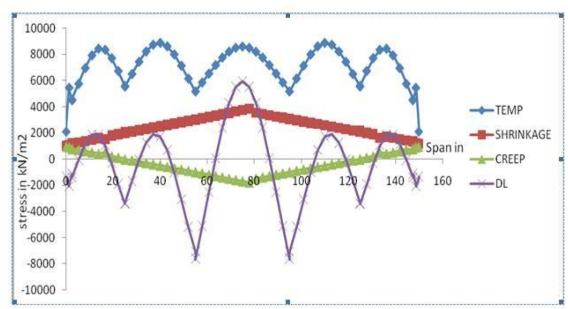


Chart – 2: Variation of longitudinal stresses at bottom for various loads

Discussions

1. From the Graph 5.1 and Graph 5.2 variation of stresses resulting in both compressive and tensile forces along the deck top and bottom can be observed. At same time braking stresses resulting almost negligible stresses.

2. Temperature gradient produces absolute compressive stresses at the top of the deck. This is a significant issue with jointless bridge systems because it adds to the compressive stresses caused by other loads and increases the potential for cracking.

3. Shrinkage produces compressive stresses at the top and tensile stresses at the bottom in all systems. This creates the worst scenario because the primary loads also produce compressive stresses at the top fibre and tensile stresses at the bottom over a pier.

4. There is a decrease in compressive stresses caused by creep; in other words, creep relives shrinkage. Therefore, the common design assumption that creep and shrinkage have opposite effects is reasonable.

5. Further from Graph 5.1 and 5.2 it can be observed that in jointless bridges fully restrained against elongation and end rotation, researchers believe that shrinkage and creep stresses are opposite in nature, negate each other. In present study it can be concluded that shrinkage and creep effects do not negate completely, and there is always a residual stress, which may cause the concrete to eventually crack.

6. Cracking of concrete partially relives shrinkage stresses. Therefore, the effects of shrinkage on the superstructure should be properly accounted for in the analysis and design of joint less bridge.

Longitudinal Variation of Stresses for various combination of earthquake loads

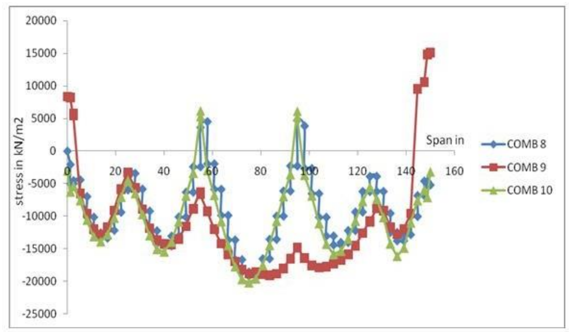


Chart – 3: Variation of longitudinal top stresses for various combinations of earthquake loads

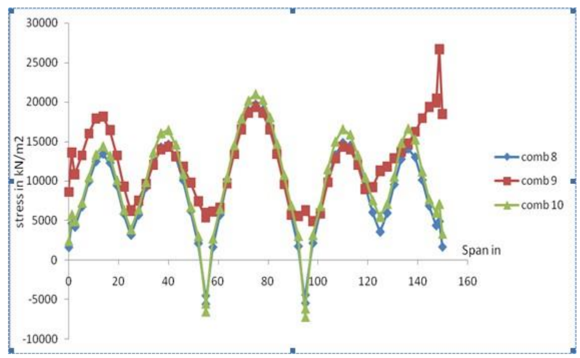


Chart – 4: Variation of longitudinal bottom stresses for various combination of earthquake loads

comb 8 - DL+0.5ML+TEMP+0.5BR+EQ1 comb 9 - DL+0.5ML+TEMP+0.5BR+EQ2 comb 10 - DL+0.5ML+TEMP+0.5BR+EQ3

Discussions

1. It can be seen that except at the end spans tensile stresses are produced at the top section and compressive stresses at bottom which again add up to the primary and secondary loads resulting more critical section.

2. Imperial earthquake produces major stresses at mid span and El Centro at the end spans.

Amplification of Responses

Acceleration

An acceleration data of three earth quake EL Centro, Coalinga and Imperial Valley of PGA0.338g, 0.151g, 0.157g were scaled down to 0.15g for Bangalore region using software seismo signal and then response of the structure was calculated in both horizontal and vertical direction. Since the acceleration in vertical direction will be less than horizontal the, response of the structure in vertical direction was calculated for 0.67 (two-third) times the PGA in horizontal direction i.e.0.1g.A critical joint no 1515 was selected from the model as it showed large deflection along the deck region and joint 121 in deck and pier joint.

Table -1: Comparison of different earthquake acceleration in horizontal and vertical direction

Earthquake Record	Deck section (1515)		Pier and Deck section (121)	
	Acceleration in horizontal direction m/sec²	Acceleration in vertical direction m/sec²	Acceleration in horizontal direction m/sec²	Acceleration in vertical direction m/sec²
COALINGA	0.26	2.2	0.29	0.161
EL CENTRO	0.0608	2.99	0.298	0.181
IMPERIAL	1.23	5.18	0.711	0.22

Discussions

1. The deck section showed 22% amplification in vertical direction where as 3% amplification in horizontal direction since the stiffness of deck in vertical direction is less than horizontal direction for Coalinga earthquake.

2. In pier and deck section amplification of 3% was seen in horizontal direction and 1.6% in the vertical direction this is also because of stiffness in pier and deck section in horizontal direction is less than vertical direction.

3. Of the entire three earthquake considered imperial earthquake showed more amplification than other two in both deck section and pier and deck section.

4. Imperial earthquake proves to be more critical than coalinga and El centro.

5. The vertical stiffness of the deck section and horizontal stiffness of the pier section proves to be more critical.

7. SCOPE FOR FURTHER STUDY

Through an extensive literature search on the subject of integral bridges and their connection details to the deck slab and approach slab is done, it is apparent that there is still a great deal of research to be performed on these structures.

1) A parametric study can be performed with different length of bridge and different pier and abutment height with soil structure interaction.

2) Integral bridge with different skewness can be studied to determine the maximum length to which the bridge can be restricted.

3) Since the field of integral bridge is an emerging field, more research can be carried out to bring out a common design code

4) A parametric study can be performed with different type of soils and different water content and its effect on the length of integral bridges.

5) More studies on integral bridges for high speed railways have to be carried out.

8. CONCLUSIONS

1) In the present study, of load combination shows that DL+temp is a critical combination compared to others.

2) In the present study it is observed that shrinkage induces compressive stresses and creep induces tensile stresses. The combination of shrinkage and creep stresses cancel out each other resulting in small residual stresses. Hence the study on these types of stresses is not necessary

3) The deck joint no 1515 showed amplification of acceleration by 22% in vertical direction and 3% amplification of acceleration in horizontal direction. So it can be concluded that the stiffness in vertical direction is smaller compared to horizontal direction.

4) The pier and deck joint no 121 showed 3% amplification of acceleration in horizontal direction and 1.6% amplification of acceleration in vertical direction. Hence it is inferred that deflection in deck and pier joint is more in horizontal direction than vertical direction.

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