ANALYSIS AND DESIGN OF SUPERSTRUCTURE - EARTHQUAKE RESISTANT BRIDGE

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ABSTRACT

Our project deals with the RCC design of an earthquake resistant bridge. The location is near railway station in Thanjavur which is facing major traffic problem due to the train moving & the Public felt inconvenient to cross the busy Track. We have done a traffic survey and designed all the structural partsfor the project. The bridge is having 10m span Length .The slab is designed by Limit stress method as per the recommendation of IRC:-21-2000 and IS 456-2000.Dead & live load for the Pire Column, slab, beam. All the elements are designed by using M25 Concrete Grade, & fe415 Grade steel.

Keywords: Column, beam, slab

I INTRODUCTION

Our Nation being primarily an agricultural country. 90% of Population is depending upon industrial activities. The Roads, Railways and bridges are very important for growth of economy of the country. With the occurrence of every major earthquake, there has been in the past, almost aworld-wide tendency to past, almost a world-wide tendency to increase the capacity demand of the structure to counteract such events. The structural engineer"s having energy-dissipating and energy-sharing devices and those that can control the response of the system. These ideas can be used in economical earthquake resistant design of bridges.

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1.1 BRIDGES

Bridge is a structure having a total length above 6m between the inner face of the dirt walls for carrying traffic or other moving loads over a depression or obstruction such as railway. They are classified as minor or major bridges as per the criteria given below:

Minor Bridges – Span greater than

6m up to 60m

1.2 PLASTIC HINGING AND DURABILITY

There is a difference in seismic design aspects of bridges and buildings. The reduced degree of indeterminacy of bridge structures to reduced potential of dissipating energy and load re-distribution. In bridges, the superstructures are the main structural elements for resistance to earthquake. For energy dissipation, ductile behavior is necessary during flexure of these structural elements under lateral earthquake loads. The formation of plastic hinges or flexural yielding is permitted to occur in elements during severe shaking to bring down the lateral design forces to acceptable levels. The yielding would lead to damage, plastic hinging are restrict design at points accessible for inspection and repair, That is parts of the substructure that lie from foundation upwards. There is No plastic hinges are allowed to occur in the foundations or in the bridge deck.

1.3 SUPERSTRUCTURE DISLODGEMENT PREVENTION & INTEGRAL BRIDGES

Bearings, in general, are comparatively fragile and brittle elements (see Figure 3). Usual bearings of various types (metallic, elastomeric, pot, etc.) can be designed to have the capacity of sustaining lateral forces of about 25-30% of their vertical load carrying capacity (IRC, 1999). For larger lateral forces, as in the cases of Zones IV and V (IRC, 2000; BIS, 2002), it is more suitable and economical to provide resistance to these forces separately by some other structural element.

Integral bridges need wide exposure because bearings and expansion joints are elements that are of serious concern in earthquake-prone areas. As already mentioned, they also happen to be the weak points in bridge structures from the point of view of strength, durability and maintenance.

1.4 BASE ISOLATION AND ELASTOMERIC BEARINGS

The use of bearing that reduce the seismic forces can be effectively used in the structure. By decoupling the structure from seismic ground motions it is possible to reduce the earthquake-induced forces in it. This can be done in two ways:

- Increase natural timing of the structure by base isolation,
- Increase damping of the system by energy-dissipating devices.

By incorporating a layer of discontinuity which has a low lateral stiffness as compared to the structural elements.

1.5 ENERGY SHARING

Sometimes it is advantageous that the seismic energy entering from the ground into the structure does not get localized. Special devices exist which can avoid significant energy accumulation and ensure its distribution to various structural elements. Here, the idea is not to reduce the total seismic energy entering into the structure but to judiciously distribute.

1.6 ESTIMATE:

The cost of the project includes expenditure on various items from planning to the yearly maintenance up to the end of useful life of the project. Generally they are,

1) Cost of preliminary survey

2) Cost of acquisition of land

3) Cost of various structure

4) Maintenance cost

5) Operation cost

6) Cost of tools and plants

7) Cost of establishment of construction.

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1.7 TRAFFIC SURVEY

Traffic survey was made on 25.09.2016, in the project site from 6.00 pm to7.00 pm. This time was selected on the basic of the past traffic study as an average of peak hour. The site was observed and the number of vehicles passed was converted to PCU‟s (Passenger car unit).

1.8 TRAFFIC PROJECTION:

The passenger car unit of a vehicle type has been found to be depends up on the size, and speed of the vehicle type and environment. They are not dependent on the flow and road width 10000PCUs/hour. The capacity of junction was estimated at 6547 PCS‟s / hour. The design period is taken as 30 years. One year would be taken for the construction. So traffic is projection.

II. LITERATURE REVIEW

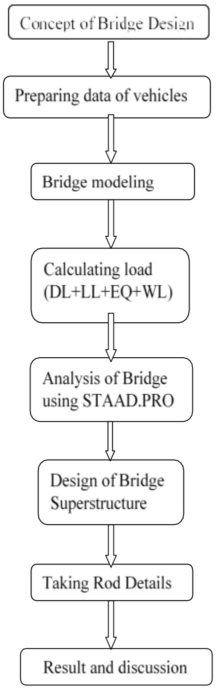
Kaar et al., (1960) investigated the development of continuity in precast, prestressed concrete bridge girders used in conventional designs for extending span lengths. The conventional design used deformed reinforcement in the CIP deck slab over the girders to provide continuity designed for resisting the live loads. The width of the diaphragms was greater than the spacing between the ends of the girders and helped to provide lateral restraint to strengthen the concrete in compression. The results from this study found that this continuity connection detail was desirableas it permits sufficient redistribution of moment and is simple to construct and relatively economical.

Mattock et al., (1960) carried out extra tests on the continuous connection for precast slab, prestressed concrete bridge concrete girders with introduction of details for resisting the positive moments resulting from creep and shrinkage. They conducted static and dynamic load tests on half-scale component specimens of a two-span continuous connection between girders with CIP deck and diaphragm. They tested two connection details for positive moment resistance: (i) fillet welding the projecting ends of the reinforcement bars to a structural steel angle, and (ii) bending the projecting ends of the reinforcement to form right angle hooks and lapping them with the longitudinal diaphragm reinforcement. It was suggested to use an inside radius of the hook larger than the bar diameter and a minimum distance of 12 times bar diameter from the edge of the precast member to the inside face of the hook to develop the yield strength of the reinforcement bars.

Ficenec et al., (1993) described the project phases and implementation of new girder flow technology for two bridge structures in Nebraska. In this new girder continuity system, the girder segments were made continuous by splicing, coupling, and tensioning the pre-tensioning strand extensions at the adjacent ends of the girder segments. The bridge consisted of 5 spans with 90‟ exterior spans and 125‟ interior spans using 4 „ 6 “. deep Nebraska Type 4-A girders. The main viaduct bridge consisted of six spans with 86 „ and 114‟ exterior spans employing 4‟ in. deep Nebraska Type 4-A girders and 172‟ interior spans employing 6‟ 3”. deep Nebraska Type BT-1A girders.

III.METHODOLOGY

This chapter briefly explains the methodology adopted for this Bridge Design Project. In the first phase, Concept of Bridge design



IV.ANALYSIS AND DESIGN OF SUPERSTRUCTURE

The superstructure consists of the girder, deck slab and crash barriers. The girders rest on the bearings structure through forces and moments are transferred to the sub-structure.

4.1 SECTION PROPERTIES

The section properties of the girders are initially assumed as per standards or from previous experience and later checks are done in the design stages to ensure safety. From calculations the Moment of inertia (MI), cross-section area and center of gravity of the section is calculated.

4.2 LOAD CALCULATIONS

4.2.1 DEAD LOAD

The dead loads are calculated by considering 1m of the box section. The dead load for top and bottom slab calculating as a udl load and side walls are calculated separately as a Uniformly Distributed Load applied on the respective members. The concrete density is generally taken as $25kN/m^3$.

4.2.2 LIVE LOAD

For calculation of load dispersion along the traffic direction, as per Cl 305.16.3, IRC 21-200-, the effect of contact of wheel or track load in the direction of span length shall be taken as equal to the dimension of the tyre contact area over the wearing surface of the slab in the direction of the span plus twice the overall depth of the slab inclusive of the thickness of the wearing surface.

4.3 LOAD COMBINATIONS

All the live load cases are each combined with the Super-imposed Dead Load (SIDL) as these come under the service condition and the critical value is taken as the maximum results from inner and outer girder results, which is used for carrying out the design. The critical load combinations considered for design are as follows.

1. DL+SIDL+EP
2. DL+SIDL+EP+LLS-L
3. DL+SIDL+EP+LLS-R
4. DL+SIDL+EP+LLS-BOTH
5. DL+SIDL+EP+LLS-L+LL
6. DL+SIDL+EP+LLS-R+LL
7. DL+SIDL+EP+LLS-BOTH+LL
8. DL+SIDL+EP+LLS-L+LL+BR-L

9. DL+SIDL+EP+LLS-R+LL+BR-R

4.4 SHEAR CONNECTOR DESIGN

The deck slab and girder being cast and placed separately are not monolithic and hence requires a connection for load transfer. This is facilitated with the help of shear connectors which are cast in the girders in the casting yard with the shear connectors projecting above the top of the girder. The deck slab is cast around this supporting the superstructure withstand shear.

4.5 DIFFERENTIAL SHRINKAGE STRESS

As the various components of the superstructure are cast at different times, the concrete is of different age and shrinkage occurs in the various components non-uniformly. This generates a stress known as differential shrinkage stress.

V.DESIGN AND CALCLATION OF SUPERSTRUCTURES

DETAILS OF SUPERSTRUCTURE

Grade of Concrete = 35.00 MPa

Grade of Steel = 500.00 MPa

Density of concrete = 25.00 kN/m^3

Overall span of RCC Girder= 18.00 m

Effective Span = 17.50 m

Spacing between bearings = 3.00 m

Thickness of wearing coat= 0.06 m

Depth of RCC Girder = 1.50 m

% camber = 2.50 %

Total width of deck slab = 12.00 m

Moment of inertia = 0.18 m^4

Ztop = 0.24 m

Zbottom = 0.23 m

Composite Section at support

Total Area = 1.61 m^2

Moment of Inertia= 0.47 m^4

Ztop slab = 0.78 m

Ztop girder = 1.26 m

INTERMEDIATE SECTION

Total Area = 1.47 m^3

Moment of Inertia = 0.44 m^4

DIAPHRAGM

Moment of Inertia = 0.11 m^4

Ztop = 0.15 m

Zbottom = 0.15 m

LOAD CALCULATION

Thickness of deck slab = 0.23 m

Distance between bearings= 3.00 m

Thickness of wearing coat= 0.10 m

DEAD LOAD

Dead load of girder at the support =23.11 kN/m

Dead load of girder at midspan= 15.80 kN/m

Dead load due to deck slab=17.25 kN/m

SIDL due to wearing coat on girder= 6.60

GIRDER DESIGN

CONSTRUCTION STAGE

Design of girder for construction stage (Dead Load alone)

Clear Cover = 40.00 mm

Effective depth = 1368.00 mm

196.00 mm from bottom of girder

Permissible compressive stress in bending = 13.33 MPa

Permissible tensile stress in reinforcement = 240.00 MPa

Stresses for given moment

Bending Moment ,

Actual stress in concrete, σc =6.94 Mpa

Actual stress in bottom most steel layer,

= 93.84 Mpa

SERVICE STAGE

Clear Cover= 40.00 mm

Diameter of main bar = 32.00 mm

196.00 mm from bottom of girder

Effective cover provided= 132.00 mm

Area of steel provided= 12063mm^2

Stresses for given moment

Actual stress in concrete, σc= 3.07 Mpa

Actual stress in bottom most steel layer,

= 122.06 Mpa

Final Stresses

Actual stess in concrete at top of girder flange, σc = 7.89 Mpa

Actual Stress in bottom most steel layer,

σst = 215.90 MPa

Curtailment Design of Composite girder for Service stage

Diameter of main bar = 32.00 mm at No. of bars = 5.00 Nos.

196.00 mm from bottom of girder

Area of steel provided = 8042.48 mm^2

Permissible compressive stress in bending = 13.33 MPa

Permissible tensile stress in reinforcement = 240.00 MPa

Check for neutral axis

If neutral axis lies within the range, for kd<Df

For kd >Df

Revised Neutral axis depth, kd 272.79 mm

Neutral Axis depth factor, k= 0.17

Stresses for given moment

Bending Moment,

Actual stress in concrete, σc =2.48 Mpa

Actual stress in bottom most steel layer, = 126.35 Mpa

σst = (D-d1'-kd) x σc x m/kd

Final Stresses (Curtailment Design)

Flange, σc = 5.99 Mpa

σst = 224.43 Mpa

Final stress in steel = 226.79 Mpa

SHEAR DESIGN

Nominal shear stress, τv = 1.35 MPa

Shear in concrete, Vuc = 287.64 KN

Provide 2 legged 12 mm dia stirrups @125 mm spacing

Minimum shear reinforcement, Asv,min = 41.55 mm^2

Shear at 4.25m distance away from support = 274.25 kN

Section is adequate

Percentage Reinforcement at the section = 1.64 %

Shear in concrete, Vuc = 249.39 kN

Provide 2 legged 12 mm dia stirrups @ 250 mm spacing

SHEAR CONNECTOR DESIGN

Provide 2 legged 12 mm dia shear connectors @150 mm spacing

Area of connector, As = 226.19 mm^2

Provided = 150.00 mm

DIFFERENTIALSHRINKAGE STRESSES

Grade of concrete= 40.00 Mpa

Total Area = 1.32 m^2

Moment of inertia= 0.41 m^4

Ztop = 0.77 m^3

Zbottom= 0.34 m^3

Stress in compression flange =1.36 Mpa

DIAPHRAGM DESIGN

Gross Area, Ag = 592000mm^2

Design sagging moment = 294.70 kNm

Design Hogging Moment = 568.15 kNm

Effective depth provided= 1380.50 mm

Ast,reqd = 1891.33 mm^2

No. of bars required = 4

Ast provided = 1963.50 mm^2

Design of bottom reinforcement

Ast,reqd = 981.03 mm^2

Minimum steel to be provided= 1184.00 mm^2

Area of steel to be provided =1184.00 mm^2

Ast provided = 1206.37 mm^2

Design of shear reinforcement for Diaphragm

Maximum shear force at support =910.05 kN

Nominal Shear Stress =1.65 Mpa

Maximum shear stress =2.50 MPa

OK

Spacing of stirrups = 180.00 mm

Shear stirrups required = 410.41 mm^2

Provide 12 mm, 4 LVS @180.00 mm spacing

Area of provided stirrups= 452.39 mm^2

SAFE

Design of Side reinforcement

Provide side face reinforcement = 592.00 mm2

Diameter of bar to be provided = 10.00mm

Provide 4.00 bars of10.00 mm at 250 mm

VI. CONCLUSION

This Project work is an attempt in understanding the design of Balanced Cantilever Bridge and this paper has looked at aspects of the detailed design of a multi-span balanced cantilever bridge which were investigated using a global model of the structure: the analysis of construction stages from cantilevering to determination of the long-term moment distribution as well as the design of the bridge to resist earthquakes. The regularity of the structure inherent to the balanced cantilever technique let the user take advantage of Span more than 15m to 20m, BCB are economical.The software also provides the required tools to carry out the analysis and design of a complex bridge structure according to the methodology proposed in Indian standard code practices. In return the Balanced Cantilév̀er Bridge depth of girder small & small quantities of reinforcement is Need and the Project stands with good scope.

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