Design and Analysis Bridge

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Abstract

Bridge is a structure having a total length of above 6 metres between the inner faces of the dirt walls for carrying traffic on road or railway. The bridges shall be classified as minor bridge and major bridge.

Minor bridge – Bridge having a total length up to 60 meters. Clause 101.1 of IRC 5:1998

Major bridge – Bridge having a total length above 60 meters.

The bridges are designed and constructed adopting the following IRC specifications.

- IRC 5:1998 Standard specification and code of practice for road bridges- Section I general features of design
- IRC 6:1966 Standard specification and code of practice for road bridges – Section II load and stress
- IRC 21:1987 Standard specification and code of practice for road bridges- Section III cement concrete
- IRC 40 : 1995 Standard specification and code of practice for road bridges- Section IV (bricks, stones and masonry)
- IRC 22:1986 Standard specification and code of practice for road bridges- Section VI composite construction
- IRC 78:1983 Standard specification and code of practice for road bridges- Section VII formation and sub structure
- IRC 83:1987 Standard specification and code of practice for road bridges- Section IX bearings
- IRC SP:20 2002 Rural Road Manual
- IRC SP 13:2001 Guideline for the design of small bridges and culvert

- Component of Bridge

The component of the bridge is broadly grouped into

i) Foundation

ii) Substructure

iii) Superstructure

The foundations are different type viz., open foundation, well foundation, raft foundation and pile foundation. The substructure is the portion of the bridge structure such as pier and abutments above the foundation unit and supporting the superstructure. It shall also include returns and wing walls but exclude bearings. Superstructure is the portion of bridge structure above the substructure level viz., deck slab/beam, hand rail, foot path etc.

Introduction

Scope and Background

A bridge is a construction built to span physical obstacles such as a body of water, valley, or road, for the purpose of providing passage over the obstacle. Designs of bridges vary depending on the function of the bridge, the nature of the terrain where the bridge is constructed, the material used for construction and the funds available to build it.

A bridge has three main elements. First, the substructure (foundation) transfers the loaded weight of the bridge to the ground; it consists of components such as columns (also called piers) and abutments. An abutment is the connection between the end of the bridge and the road carried by the earth; it provides support for the end sections of the bridge. Second, the superstructure of the bridge is the horizontal platform that spans the space between columns. Finally, the deck of the bridge is

The guidelines for non linear analysis for bridge structure presents a collection of general recommendations for the modeling and analysis of highway bridges and overpasses subjected to earthquake ground motions, required for the design or evaluation of the capacity andductility of critical bridge components and systems.
The specifications and guidelines presented throughout the document are applicable for Ordinary Standard Bridges as defined according to the 2004 Caltrans Seismic Design Criteria (SDC), Section 1.1. Some general recommendations can be extended to Ordinary Nonstandard Bridges and Important Bridges, where more rigorous and advanced nonlinear analysis is required due to geometric irregularities of the bridge structure, including curves and skew, long spans or significant total length, multiple expansion joints, massive substructure components, or unstable soil conditions. For these special cases, the design engineer must exercise judgment in the application of these recommendations and refer to additional resources in situations beyond the intended scope of this document. The introductory chapter identifies the relevance and importance of nonlinear analysis procedures in bridge structures, including the advantages and drawbacks over simpler linear analysis. The different types of nonlinearities to be incorporated in the analytical bridge model are described briefly, with a list of the critical components of the structure that require detailed inelastic modeling to guarantee a desired level of accuracy. The appropriate model dimension (2D or 3D) recommended for the application of nonlinear analysis procedures is also justified in detail.

The second chapter titled load calculations includes considerations of traffic volume , general considerations and converting vehicle load into uni-axial load etc

The third chapter, titled *Bridge Modeling,* establishes a set of recommendations for the simplification of the geometry of the structure, definition of elements and materials, and the assignment of mass and boundary conditions, among others. A thorough explanation is presented that addresses the minimum requirements in the modeling in column bents. The nonlinear behavior of bridge abutments and foundations, as well as expansion joints integrated along the superstructure is discussed briefly.

The fourth chapter, titled *Bridge Analysis,* specifies the procedures and parameters used to simulate the seismic demand on the bridge structure in the form of imposed static and dynamic forces or displacements. The chapter provides an adequate and detailed methodology that allows the design engineer to conduct modal, gravity load, pushover, response spectra, and time history analysis, as well as to analyze the resulting response data of the bridge. References are provided to other resources for the use of response spectrum curves, selection and scaling of ground motions, and definition of additional parameters required for the different nonlinear analysis types.

The guidelines document presents ample recommendations for linear and nonlinear analysis of bridge structures appropriate for any structural analysis program, as well as specific details on the use of SAP2000 for such procedures. Additionally, a general review and definitions related to structural dynamics, applicable to both linear and nonlinear analysis, are presented throughout. The emphasis of the present document is the implementation of nonlinear analysis procedures used primarily for the estimation of the demand on a bridge structure, not the evaluation of its capacity for design purposes. The design engineer must determine the appropriate methods and level of refinement necessary to analyze each bridge structure on a case-by-case basis. This document is intended for use on bridges designed by and for the California Department of Transportation, reflecting the current state of practice at Caltrans. This document contains references specific and unique to Caltrans and may not be applicable to other parties, either institutional or private.

LOADS CONSIDERED

2.1 DEAD LOADS:

All permanent constructions of the structure form the dead loads. The dead load comprises of the weights of walls, partitions floor finishes, false ceilings, false floors and the other permanent constructions in the buildings. The dead load loads may be calculated from the dimensions of various members and their unit weights. the unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 kN/m" and 25 kN/m" respectively.

2.2 IMPOSED LOADS:

Imposed load is produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, load due to impact and vibration and dust loads. Imposed loads do not include loads due to wind, seismic activity, snow, and loads imposed due to temperature changes to which the structure will be subjected to, creep and shrinkage of the structure, the differential settlements to which the structure may undergo.

2.3 WIND LOAD:

Wind is air in motion relative to the surface of the earth. The primary cause of wind is traced to earth's rotation and differences in terrestrial radiation. The radiation effects are primarily responsible for convection either upwards or downwards. The wind generally blows horizontal to the ground at high wind speeds. Since vertical components of atmospheric motion are relatively small, the term 'wind' denotes almost exclusively the horizontal wind, vertical winds are always identified as such. The wind speeds are assessed with the aid of anemometers or anemographs which are installed at meteorological observatories at heights generally varying from 10 to 30 metres above ground.

Design Wind Speed (V,)

The basic wind speed (V,) for any site shall be obtained from and shall be modified to include the following effects to get design wind velocity at any height (V,) for the chosen structure:

a) Risk level;
b) Terrain roughness, height and size of structure; and
c) Local topography.

Risk Coefficient :(kI Factor) gives basic wind speeds for terrain Category 2 as applicable at10 m above ground level based on 50 years mean return period. In the design of all buildings and structures, a regional basic wind speed having a mean return period of 50 years shall be used

2.4 SEISMIC LOAD:

Design Lateral Force

The design lateral force shall first be computed for the building as a whole. This design lateral force shall then be distributed to the various floor levels. The overall design seismic force thus obtained at each floor level shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

Design Seismic Base Shear

The total design lateral force or design seismic base shear (Vb) along any principal direction shall be determined by the following expression:

n=Number of storeys in the building is the number of levels at which the masses are located. Distribution of Horizontal Design Lateral Force to Different Lateral Force Resisting Elements in case of buildings whose floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane shall be distributed to the various vertical elements of lateral force resisting system, assuming the floors to be infinitely rigid in the horizontal plane. In case of building whose floor diaphragms can not be treated as infinitely rigid in their own plane, the lateral shear at each floor shall be distributed to the vertical elements resisting the lateral forces, considering the in-plane flexibility of the diagram.

Dynamic Analysis-

Dynamic analysis shall be performed to obtain the design seismic force, and its distribution to different levels along the height of the .

Terrain, Height and Structure Size Factor (k, Factor)

Terrain - Selection of terrain categories shall be made with due regard to the effect of obstructions which constitute the ground surface roughness. The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Wherever sufficient meteorological information is available about the nature of wind direction, the orientation of *Pressure Coefficients* - The pressure coefficients are always given for a particular surface orpart of the surface of a building. The wind load acting normal to a surface is obtained by multiplying the area of that surface or its appropriate portion by the pressure coefficient (C,) and the design wind pressure at the height of the surface from the ground. The average values of these pressure coefficients for some building shapes Average values of pressure coefficients are given for critical wind directions in one or more

4. Bridge Modeling

BRIDGE GEOMETRY

Compilation of General Characteristics

The following information is required for the modeling of the basic bridge structural geometry:

• Total length of the bridge (L Total)=800m
• Number of spans and length of each superstructure span
• Total superstructure width (W superstructure)=16m
• Superstructure cross-sectional geometry
• Number and clear height of each column bent (H col)
• Column cross-sectional dimension in the direction of interest (Dc)

building and to the various lateral load resisting elements, for the following Buildings:

a) *Regular buildings* -Those greater than 40 m in height in Zones IV and V and those Greaterthan 90 m in height in Zones II and 111.

any building or structure may be suitably planned.

Topography(ks Factor) - The basic wind speed Vb takes account of the general level of siteabove sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliff, steep escarpments, or ridges quadrants. In order to determine the maximum wind load on the building, the total load should be calculated for each of the critical directions shown from all quadrants. Where considerable variation of pressure occurs over a surface, it has been subdivided and mean pressure coefficients given for each of its several parts.

Then the wind load, F, acting in a direction normal to the individual structural element or Cladding unit is:

• Distance from column top to center of gravity of superstructure (Dc.g.)
• Length of cab beam to centroid of column (L cap)
• Cap beam width (Bcap)
• Location of expansion joints
• Support details for boundary conditions

The definition of the individual behavior of major bridge components entails the following data:

• Concrete material properties for concrete of superstructure (f'c, Ec)
• Concrete and reinforcing steel material properties ($\sigma-\varepsilon$) of column bents
• Reinforcement details of column bent cross section
• Foundation soil geotechnical properties
• Abutment general geometry

• Number and properties of abutment bearing pads
• Size of expansion joints

- **MODEL DIMENSION**

A three-dimensional (3D) model of the structural system is required to capture the response of the entire bridge system and individual components under specific seismic demand characteristics. The interaction between the response in the orthogonal bridge directions and the variation of axial loads in column bents throughout the analysis are captured more accurately in a 3D model. This enables correct evaluation of the capacity and ductility of the system under seismic loads or displacements applied along any given direction, not necessary aligned with the principal axis of the bridge.

If the primary modes of the structure are highly correlated due to special mass distribution or geometry characteristics, they will significantly affect the dynamic response of the bridge, which must then be represented adequately through a three-dimensional model. Since the modal contribution is a key aspect in bridge analysis, and since the ground motions applied in a time history analysis are decomposed into three orthogonal directions and applied at an angle with respect to the principal axes of the bridge, a global analysis of the system is required. A two-dimensional (2D) model consisting of plane frames or cantilevers will fail to capture the particular geometric characteristics of the entire bridge and the interaction between structural subsystems. The actual distribution of forces among critical components of the bridge is determined according to their relative stiffness. The flexibility of the superstructure in the transverse direction, the relative stiffness of the column bents according to their heights and cross-sectional properties, and the abutment characteristics are imperative aspects to consider in the analysis that cannot be modeled correctly using a two-dimensional model.

The use of combinations rules for the interaction of responses in orthogonal directions to estimate the maximum demand on critical bridge components are applicable only for linear elastic structures, and could result in significant errors when extrapolated to the inelastic range. Particularly in the case of special bridge systems with irregular geometry, curved or skewed, with multiple transverse expansion joints, massive substructure components, and foundations supported by soft soil, the dynamic response characteristics exhibited are not necessarily obvious beforehand and may not be captured in a separate subsystem analysis. According to Section 5.2 of SDC 2004, for structures supported on highly non-uniform soils, a separate analysis of each individual frame is recommended in addition to the conventional three-dimensional multi-frame analysis.

Local analysis of an individual component or subsystem may be used to assess the critical values of their strength and ductility capacity and provide a general approximation of the expected range of response of the entire bridge system. If desired, local analysis is performed in the transverse and longitudinal directions for bridge column cross sections with biaxial symmetry, following the recommendations of Sections 5.3–5.5 of SDC 2004. Local analysis fails to capture the interaction between different components or subsystems of the bridge, and could therefore result in significant errors in the estimation of the demand on the analyzed component.

- **CAP BEAM MODELING**

The cap beam is a concrete element connecting the superstructure and the column bents, helping a multi-column bent bridge resist, through frame action, lateral loads or displacements applied primarily in the transverse direction of the bridge. For single-column bent bridges, the cap beam is built to facilitate the connection of the bent to the superstructure and reinforce the joint. The
Bridge Modeler feature available in latest versions of SAP2000 resolves many of the issues regarding cap beam modeling discussed in the present section.
In the case of multi-column bent bridges, an elastic element representing the cap beam should be modeled as a frame element with a solid rectangular cross section with dimensions according to plans. The material properties used for this element include the Modulus of Elasticity Ec, Weight wc, and Mass Rcc of reinforced concrete, as defined by SDC 2004, Section3.2.6. The definition of a stress-strain relationship of the concrete material, as well as other properties, is Not required for this elastic element.
The cap beam is connected through rigid or moment connections to the superstructure since both elements are usually constructed monolithically without joints. The use of joint constraints between column top nodes, representing node slaving or a rigid diaphragm perpendicular to the Global Z direction, will produce an overestimation of the bridge's stiffness, primarily in the transverse direction. The flexibility of the cap beam should be accounted for in the model, instead of joint constraints, if sufficient design details are specified for such an element. Since the concrete superstructure and cap beam are cast simultaneously into a single element, the superstructure's flexural stiffness enhances the torsional stiffness of the cap beam. The actual dimensions of the cap beam-superstructure system resisting torsion are greater than the cross-sectional dimensions of the cap beam element exclusively. The torsional constant of the cap beam J should therefore be modified by an amplification factor C by applying Property Modifiers to that value.

In order to model the correct torsional stiffness of the cap beam-superstructure system, it is necessary to verify that the cap beam twist, which is the difference between the column top rotation and the superstructure rotation, has reduced to 5% of its original value obtained without amplification factors (see Fig. 2.5). The value of the C factor should be adjusted accordingly by multiples of 10 until reaching the desired value of the cap beam twist and approximating rigid

- **MODELING OF PIER COLUMNS**

General Considerations

According to the bridge geometry described in specific plans and Section 3.1 of SDC 2004, the foundation of the bridge column will be defined at the level of base fixity. The clear height of the column (H column) is to be taken according to Figure 3.3 of SDC 2004 Guidelines. The top of the column will be defined at a distance of Dc.g. (difference between the bottom flange or slab and the vertical centroid of the superstructure cross section) above the clear height of the bridge Column.
Inelastic three-dimensional beam-column elements are used to model the column and shaft for each of the piers in the bridge. A beam-column element connects each of the nodes at the geometric centroid of the column cross section, using a minimum of five elements to model the column, according to Section 2.1.3.

It is recommended to define a separate segment at the column top with the length Dc.g. defined above, representing the portion of the column embedded in the bent cap. An end (rigid)

- **Observations**

1. Uncoupled behavior in each orthogonal direction results in a significant overestimation of column strength for 3D analysis (40% in the case of pushover at 45o), even in the case of circular symmetry of the cross section. A bias factor between 0.7 and 1.0 can be taken in order to reduce the strength values resulting in a 3D analysis of the bridge.

2. Convergence problems occur in SAP2000 after yielding during nonlinear time history analysis, possible solutions for which are:

- Divide the plastic hinge zone into smaller discrete elements, with an additional rotational mass assigned to the nodes. The arbitrary value of this mass should be relatively small, not greater than 1e-3, to avoid overestimating the existing mass of the structure, but rather provide an artificial tool for numerical stability during the analysis algorithm.
- The recommended degrading slope defined for the moment-rotation or moment curvature relationship of the hinge should be in the order of the elastic stiffness. Since the elastic properties of the hinge are not defined, but rather calculated automatically in SAP2000 through the elastic section, the definition of the degrading stiffness is determined iteratively by the user. The value of the degrading slope is increased progressively by the user until convergence or stable response of the bridge is achieved, which can be monitored, e.g., through displacement time history plots.

3. The uncoupled plastic hinge fails to adjust the capacity and ductility of the column

According to the fluctuation in column axial load, expected during a static pushover or dynamic analysis. The use of such plastic hinge model is not recommended when large variations of column axial load occur.

- **ABUTMENT MODELING Importance**

Abutments are earth-retaining systems designed to provide unimpeded traffic access to and from the bridge. Abutments also provide an economical means of resisting bridge inertial loads developed during ground excitations. Abutment walls are traditionally designed following principles for free-standing retaining walls based on active and passive earth pressure theories.

However, such pressure theories are invalid for abutment walls during seismic events when inertial loading from the massive bridge structure induces higher than anticipated passive earth pressure conditions (Lam and Martin 1986).

Abutment behavior, soil-structure interaction, and embankment flexibility have been found by post-earthquake reconnaissance reports to significantly influence the response of an entire bridge system under moderate to strong intensity ground motions. Specifically for Ordinary Standard bridge structures with short spans and relatively high superstructure stiffness, the embankment mobilization and the inelastic behavior of the soil material under high shear

- **Abutment Geometry and Behavior**

The different components of a typical seat-type abutment system are presented in Figure 2.19. Some of the typical abutment types used for highway bridges are classified by ATC 32 and include pile cap, stub, stub "L", cantilever, cantilever "L", spill-trough, and rigid frame abutments (see Fig. 5.1 of ATC 32). These abutments are alternatively categorized as seat and diaphragm abutment types, according to Chapter 7 of SDC 2004. Munfakh (1990) and Schnore (1990) discuss the advantages and disadvantages of various types of walls and abutments.

A realistic abutment model should represent all major resistance mechanisms and components, including an accurate estimation of their mass, stiffness, and nonlinear hysteretic behavior. Values of embankment critical length and participating mass were suggested by many research studies in order to quantify the embankment mobilization. Among them are Kotsoglou and Pantazopouloi (2006), Zhang and Makris (2002), and Werner (1994). The consideration of the abutment system participating mass has a critical effect on the mode shapes and consequently the dynamic response of the bridge, captured primarily through time history analysis. The load Wing wall
Bearing pads Back wall
Exterior
Expanded
Polystyrene
Stem
Footing
Vertical piles
Battered piles
Superstructure
Shear keys

Coordinate System

- The coordinate system used for the modeling and analysis of the bridge is shown in Figure 2.1. The global X-axis is in the directi

Effect of superstructure end restraints in single- and multi-column bent bridges.

Multi-column bent bridge
Preliminary model
Abutment model
Roller Complete model
Single-column bent bridge
Complete model
Deformation levels dominate the response of the bridge and the intermediate column bents
The proper evaluation of the dynamic characteristics and response of abutment systems under transverse and longitudinal excitations is the main focus of many ongoing research studies.
The findings of these studies will also play an important role in predicting the functionality of the bridge following an earthquake.

One of the chord connecting the abutments, denoted as the longitudinal direction; the global Y-axis is orthogonal to the chord in the horizontal plane, representing the transverse direction; while the global Z-axis defines the vertical direction of the bridge. For the analysis and design of elements of the bridge using two-noded elements, a local coordinate system is used. It is recommended that the orientation of all frame elements in a bridge structure without a skew coincides with the positive direction of the global axis; namely, the coordinate of node *I* of the frame will be smaller than node *j*. In the case of bridge structures with skew supports, the orientation of the superstructure elements should coincide with the skew coordinates, not the global axis. The nomenclature for twist or torsion, as well as axial force or deformation of an element will be denoted as the direction *1-1* or axial direction. Shear forces and deformations, as well as moments and rotations will be specified as directions

- **Node and Element Definition**

For the seismic analysis of highway bridges it is customary to use three-dimensional beam column elements (line or frame elements) with corresponding cross-sectional properties, to represent the superstructure and the components of the bents (columns and cap beams). The geometry, nodes, and connectivity of the elements in the model will be determined according to plans, following the recommendations of this chapter. The present guidelines document focuses on the three-dimensional spine model of the bridge structure with line elements located at the centroid of the cross section, following the alignment of the bridge; however, some of the**XyZ**

6. Traffic study

The traffic in terms of the cumulative number of Standard axles (8160 Kg) to be carried by the pavement during the design life. The following information is needed:
i) Initial traffic after construction in terms of number of commercial vehicles per day (CVPD)
ii) Traffic growth rate during the design life in percentage
iii) Design life in number of years
iv) Vehicle damage factor (VDF)
v) Distribution of Commercial traffic over the carriageway.

- Initial Traffic: Estimate of initial daily average traffic flow for any road should normally be based on atleast 7 days, 24 hour classified traffic counts. In case of new roads, traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area.

- Traffic growth rate: Traffic growth rates should be estimated by study. If adequate data is not available, average annual growth rate of 7.5% may be adopted. The factor is reduced to 6% for roads desiged adopting IRC:SP 20-2002
- Design life: The Design life is defined in terms of cumulative number of Standard axles that can be carried before strengthening of the pavement. Normally the pavement for NH & SH is the designed for life of 15 years, Expressways and Urban roads for 20 years and other roads for 10 to 15 years. When it is not possible to provide the full thickness of pavement at the time of initial construction, stage construction technique should be resorted to. Roads in Rural areas should be designed for a design life of 10 years.
- Vehicle damage factor (VDF): VDF is arrived at from axle load surveys. The indicative value of VDF factor is given below:

Initial traffic in terms of commercial vehicle per day Terrain (Rolling/Plain Hilly)

0-150
1.5 0.5
150-1500
3.5 1.5
More than 1500
4.5 2.5

- Distribution of Commercial traffic over the carriage way:

i) Single lane : Design should be based on total number of commercial vehicle in both directions multiplied by two
ii) Two lane (single Carriageway) : 75% of the total number of commercial vehicle in both the direction.
iii) Four lane (single Carriage way) : 40% of the –do iv) Dual Carriageway: 75% of the number of commercial vehicle in each direction. For dual 3 lane and dual 4 lane carriageway, the distribution factor will be 60% and 45% respectively.
Computation of design traffic under IRC 37: 2002
The design traffic is considered in terms of Cumulative number of standard axles to be carried during the design life of the road.
Computed by the equation
N= 365x [(1+r)n-1] x A x D x Fr
<u>Where</u>

- N: The cumulative number of standard axles to be catered for in the design in terms of MSA
- A: Initial traffic in the year of completion of construction in terms of number of commercial vehicles per day
- D: Lane distribution factor
- F: VDF
- n: Design life in years
- r : Annual growth rate of commercial vehicles (for 7.5% annual growth rate r=0.075)

<u>The traffic in the year of completion is estimated using the following formula:</u>

- **A= P (1+r) x**

Where
P = Number of Commercial vehicle as per last count
x = Number of years between the last count and the year of completion of construction

Computation of design traffic under SP 20:2002

The traffic for design life is computed as –

Number of commercial vehicles per day for design **A = P(1+r)n+x**

Where

r= Annual growth rate of commercial vehicle (i.e 6%)

P, x & n = as above

DESIGN OF GIRDERS:-

Structural properties

Span=2a

No of main beams=n=4

Girder span p=2.2m

Effective Width= np =8.8m

Cross beam spacing= q = 5m

SECTION PROPERTIES:

Cross beam – Mid span = 905 sq.m

905*1000 Sq mm

DISTRIBUTION COEFFICIENTS:-

Second moment of area of composite girder I=1632*108mm

Longitudinal stiffness I = 1038*10000000000/2200

=0.742*108mm4

Second moment of area cross beams j=695*108

Distribution transverse stiffness(j)=J/q

= 695*108 /4700

=0.148*108 mm4

Binder stiffness parameter

ϴ=(b/2a)(i/j)0.25

ϴ=4.4/19.6(742/148)0.25

=0.34

Distribution translational stiffness $i_0=I_0/P$

I_O=(67*108 /2200)

=3.05* 106 mm4

Distribution transverse torsional stiffness $j_0 = j_0$ /q

J_0=79*108 /4700

=168*106 mm4

Torrsional stiffness parameter =α=G(i_0 + j_0)/2E(iJ)1/2

=1/4.6

α =0.03

DEAD LOAD:

The dead load due to kerbs and hand rails = 7.32 KN/m (acting 284mm from edge of kerb)

Bending moment due to dead load:

B.M at center for intermediate girder G_2& G_3 =6874 KN-m

B.M at center for end girders G_1 &G_2=677 KN-m

Moments due to cast in situ concrete &wearing course:

B.M at center for girder G_2 & G_3=338 KN-m

B.M at center for end girder G_1& G_4 =202 kN-m

LIVE LOAD:

Moments due to kerb& hand rails

Total moment due to kerb and hand rail=650KN-m

Moment due to live load

-live load class A

Impact factor = 1.10

B.M at mid span including impact

=1.10*350(30/2 – 36/4)

=5428.5KN-m

Live load class A:

Impact factor fraction= 5/6+L

=5/6+30

=0.138

The max B.M occur under 114 KN load

Max B.M One lane class A=1376 KN

Max B.M two lane with impact=2*1.18*1376

STRESS DUE TO BENDING MOMENT:

The girders G_1& G_2

Cables of 12-18ф hig

Ultimate stress of 8mm wires=1500Mpa

Allowable stress install creep & shrinkage

=1500*0.6=900N/m2

Allowable force at transfer for one 12-8ф

=12*π/4(8)2*0.9=543KN

Allowable force at jacking end for one cable =0.7*1.5*543/0.9

=633KN

Therefore 6 cables are provided

Permissible stress in concrete at 4 days of prestressing

Permissible compressive stress=0.5*f_{cj}

=0.5*0.6*40

=12Mpa

Permissible tensile stress = 1.20 Mpa

At 21 days
Permissible compressive stress=18Mpa
Permissible tensile stress=1.8Mpa

FIRST STAGE PRESTRESSING:-
Stress at bottom fibre= 0.82(1.82+4.80)
Stress at top fibre= 0.82(1.82+3.08)
Concrete gain 60% of 28 day stress within 4 days

SECOND STAGE PRESTRESSING:-
At 21 days – concrete gain 90% of 28 day stress
Initial force=543KN
Eccentricity=854-125mm
=729mm

STRESS DUE TO FIRST STAGE CABLE:-
Loss due to relaxation in steel=1.38%
Loss due to shrinkage =
9.57-[2*10-4 (2.1*105)/900]*100 = 4.90

Modular ratio=Es/Ec = 2.1*105/31620
=6.64

Average stress due to 2nd stage = 0.5(10.11+3.11)
= 6.61Mpa

Loss of prestress due to elastic shortening = 0.5*6.64*6.61
=21.95Mpa
%loss due to elastic shortening= (21.95/900)*100
=2.44% **Design of deck slab**:
The slab consisting of central gap slab and the hunches due to pre cast girder at the end is considered as behaving like an arch. Two possible alignments for the line of thruster shown.
Dead load per m2 =(0.15*24)+(0.08*22)
=5.36KN
B.M due to dead load parameter = 5.36*1542 /8
= 159KN-m

LIVE LOAD:
Consider IRC class AA tracked vehicle
Depression along the span=850+(150+80)2
=1310mm
Intensity of loading including impact of 25% = (350*125)/(4.8*131)
= 69
B.M due to live load including impact of 25% perimeter width
=17.KN-m

Total Bending Moment = 1.6+17.6
=19.2KN-m

Check for stability of slab:

Max raise possible for end panel c= 2/3(d)+1/3(dl)
= (2*75)/3 + 75/3

Max raise possible for central panel=100m
Max thrust needed per m = M/C = 19.2*1000/75
=250KN

Assume 12-5 ϕ cables at 800mm c/c
Allowable initial pre-stress for 5ϕwire=0.6*1600
=960Mpa
Force required for cable= 0.8*256 = 205 KN
Final pre-stress required in cable s =(205*1000)/(12*19.6)
=872Mpa
Loss due to slip off 2.5mm= 2.5*2.1*105 /8000
= 66Mpa
Average stress in concrete= (12*19.6*974)/(650*150)
=2.3Mpa
Loss due to creep= 4*10-4 *2.3*2.1*105 /10
=21Mpa
Final pre-stress in steel= 974-(35+40+21)
= 878Mpa
The value is less than 960Mpa allowable and is greater than 872Mpa required

DESIGN OF INTERMEDIATRE CROSS BEAMS:
Live load B.M
Using morice-lottle method

M_y=b(M_0-r_1-M_{30}-r_3+M_{50}-r_5)

Assuming the track load
Υ_n=μp_0 sin(nπ/2)/nπ - sin(nπc/2a) n=1,3.5
ϴ=0.34 a=9.84 c-1.84
b=4.4m
The returns can be evaluated and are determined as
r_1=0.362p_0
r_3=0.323p_0
r_5=0.252p_0

p_0=intensity of loading with impact due to one track

=350*125/3.6

P_0=121.5 KN/m

Spacing cross beams=5m

B.M to be resisted by each cross beam=5*69.5

=347KN-m

Stress at bottom fiber=347*106/0.73*108

=4.076Mpa

Stress at top fiber =347*106/3.44*108

=0.94Mpa

B.M Due to dead load:

Stress at bottom fiber=6.5*106/0.73*108

=0.09Mpa

Stress at top fiber=6.5*105/3.44*108

=0.02Mpa

Net stress due to dead load and live load:-

Stress at bottom fiber = -4.48-0.09

= -4.57Mpa

Stress at top fiber =0.95+0.02

=0.97Mpa

PRESTRESS:

The cross section of a cross beam

2-cables of 12 to 7ɸ are provided in the web and the flange has cables of 12 to 5ɸ at 800mm c/c

Shear due to dead load=10KN

Total shear force=164KN

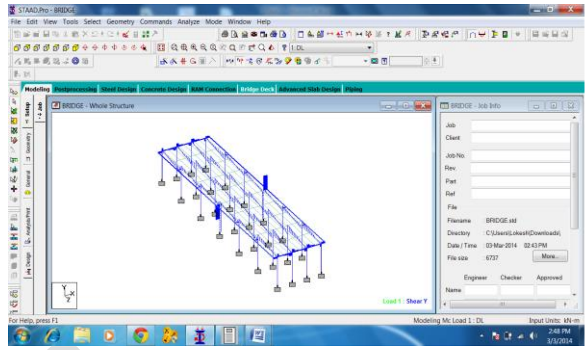
Shear stress at level of NA=1.1Mpa

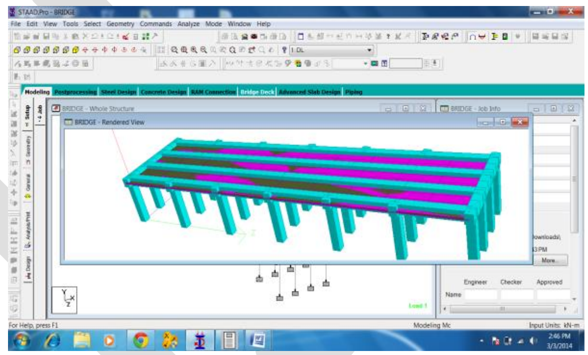
Compression due to pre-stress=2.0Mpa

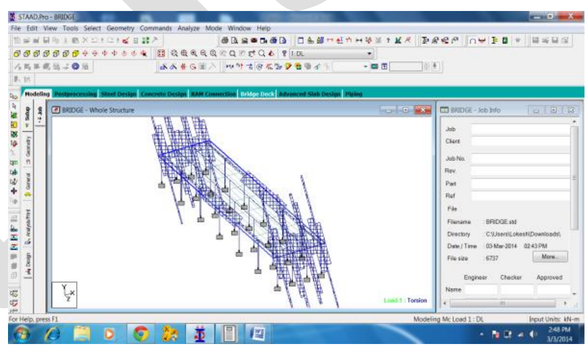
Provide nominal stirrups with 2-legged 10ɸ stirrups at 300mm.

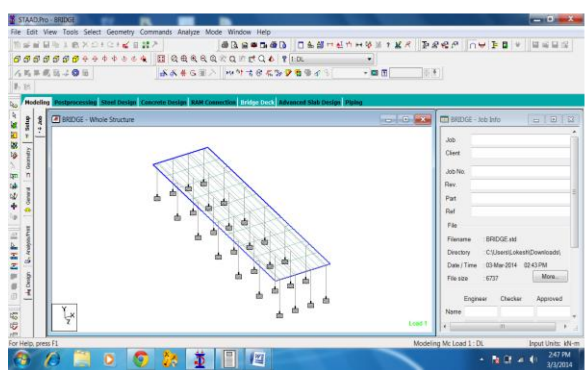
Provide nominal longitudinal mild steel bars with 6-10ɸ bars spaced two (1)top two (2)bottom two

(3)mid point









CONCLUSION:

In our project, we are going to design minor Bridge. We plan on covering every aspect of the redesign. This is going to include the design of the actual replacement bridge, the affect this bridge will have on the surrounding area through an environmental impact, and the logistics associated with the construction phase. In completing this project, we are going to have to use a number of tools. We will have to get bridge history reports in order to see the deficiencies of the current bridge, including height issues and pier quality. We are also going to have to determine what the ASHTO design standards are and apply them to this bridge. Through these events, along with others, we expect to get a good understanding of the construction phase and end up with a product similar to what was designed and approved by Mass Highway for this bridge.

STAAD PRO has the capability to calculate the reinforcement needed for any concrete section. The program contains a number of parameters which are designed as per IS: 456(2000). Beams are designed for flexure, shear and torsion.

Design for Flexure:

Maximum sagging (creating tensile stress at the bottom face of the beam) and hogging (creating tensile stress at the top face) moments are calculated for all active load cases at each of the above mentioned sections. Each of these sections are designed to resist both of these critical sagging and hogging moments. Where ever the rectangular section is inadequate as singly reinforced section, doubly reinforced section is tried.

Design for Shear:

Shear reinforcement is calculated to resist both shear forces and torsional moments. Shear capacity calculation at different sections without the shear reinforcement is based on the actual tensile reinforcement provided by STAAD program. Two-legged stirrups are provided to take care of the balance shear forces acting on these sections.

Beam Design Output:

The default design output of the beam contains flexural and shear reinforcement provided along the length of the beam.

Desk slab Design:

Desk slab are designed for axial forces and biaxial moments at the ends. All active load cases are tested to calculate reinforcement. The loading which yield maximum reinforcement is called the critical load. Desk slab is done for square section. Square columns are designed with reinforcement distributed on each side equally for the sections under biaxial moments and with reinforcement distributed equally in two faces for sections under uni-axial moment. All major criteria for selecting longitudinal and transverse reinforcement as stipulated by IS: 456 have been taken care of in the Desk slab design of STAAD.

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