Non Linear Seismic Performance of Two Span Prestressed Girder Bridge

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Abstract: - The linear method of analysis of bridges deals with elastic range of structure where the displacement and forces are not exact but accurate and may be high thus overestimating the design criteria. This results in a heavy design which attracts more seismic force. To design bridge for less force needs to employ nonlinear analysis which considers the inelastic range which gives exact account of parameters like displacement, forces and hinge locations. This enables the structural designer to design sections of bridge accordingly without overestimating the design criteria. This paper aims at studying performance of two span prestressed girder bridge with geometric nonlinearity. Demand and capacity spectrum curves are evaluated accordingly using pushover analysis.

Keywords: Demand spectrum, capacity spectrum, nonlinear analysis.

I. INTRODUCTION

Bridges are life line facility that must remain functional following disaster. Most places in India are connected by bridges. Bridges are lifeline structures in disaster prone regions. In major cities and also in rural areas and in strategic locations bridges are used for daily transportation. The Bridges must remain functional even after the earthquakes as they play great role in rescue operations. So, it is very important to select proper structural system and also need to study the response of bridges to dynamic forces by both equivalent Static method as well as Dynamic method and to find out the design parameters for seismic analysis. The effect of aerodynamic pressure must be considered in the analysis of bridges.

During uttarakhand disaster, rescue work was slowed because of collapsed bridges and damaged roads. Though the disaster was not seismic in nature it showed the vulnerability of human life against natural disasters for lack of immediate medical response, which could have been much faster otherwise. The bridge network in northern and north eastern India is less compared to china. It slows our response in highly mountainous region, which is seismic zone of high risk. Bridges of considerable seismic resistant type will ensure our National safety by providing access to remotest strategic locations for our defence forces for longer duration considering the seismic nature of ground.

In present study, the bridge is located in Zone V, with two spans and prestressing arrangement for girders. It has a column bent considered to be a non linearity. We analyse this bridge for geometric nonlinearity while the material nonlinearity is not considered in present study. Codes like IRC-18, IRC-21, and AASHTO-LFRD Codes are referred wherever necessary. The principle adopted while studying this bridge is First design then analyse (linear and non-linear analysis) and then redesign is adopted to employ a perfect structural system.

The prestressing part of girders and related values of prestressing force, cable profile are calculated manually, and then the bridge sub-structure and superstructure is modelled on CSI BRIDGE software. Analysis is carried out by both linear and nonlinear methods. The values obtained after seismic analysis are base shear, displacement which are used to plot demand and capacity spectrum curves. The fundamental time period of bridge is calculated using imperial formula and compared with software calculated time period for first mode shape. The performance point is considered where capacity spectrum curve and capacity demand curves meet. For pushover analysis ATC-40 method is employed.

II. GEOMETRY OF BRIDGE

Primary dimensions:
Clear span=20m
Effective span=18.8m
Total length of girder=19.9m
Clear width of road-way=7.5m

With above primary dimension we provide girder and cross beam arrangement supporting deck slab above as shown in *figure 1*. There are two interior girders and two exterior longitudnal girders. The spacing is calculated by primary design calculations which are to be made before commencing with software modelling. The primary design is carried out as per bridge design specifications of IRC(Indian Road Congress) guidelines. Thus the philosophy adopted here is Primary design followed by software modelling and analysis and then studying the results to redesign as required and as per judgement of the designer. To modify the primary design requires slill combined with knowledge which develops with sufficient experience and practice in bridge design. Thus while modelling the dimensions of girders and deck slab are obtained from the primary calculations done manully reffered from good bridge design book. From primary calculation we get primary dimensions of girders and deck slab which are then used in software modelling. These dimensions are modified after analysis of software results. However we restrict our study in obtaining performance point of bridge which is done after running software analysis. Redign is not done as it is a different topic and not necessary for this paper.

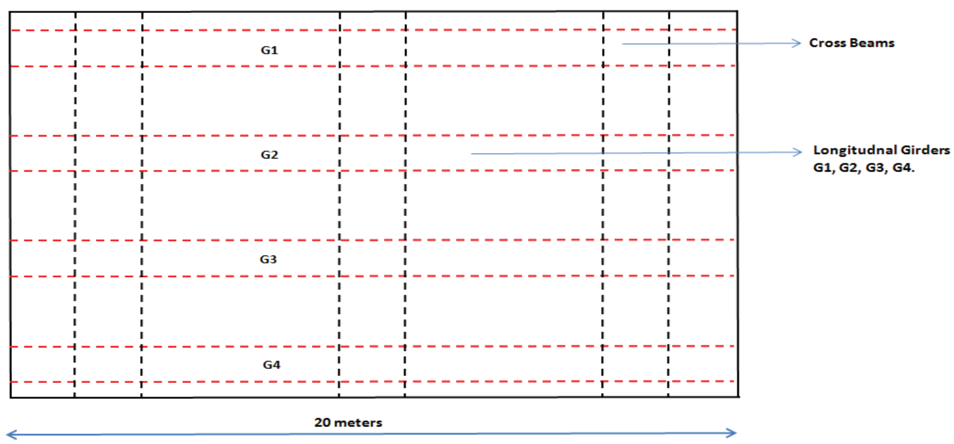

Figure 1. Plan view showing arrangement of cross beam and longitudnal girders

Consider a bridge 20m in clear span having longitudnal girder and cross beam arrangements as shown. The longitudnal girders are prestressed and high tensile strength steel cables are used. Using IRC-18 specifications
Overall depth required=75X18.8=1400mm
Thickness of slab=150mm
Thickness of web=150+50=200mm
Spacing of precast girders=2.2m
Spacing of cross beams=4.7m
Details of girders G1, G2, G3 and G4 are shown in *figure.* The girder dimensions are obtained from primary design calculations. They are prestressed and modelled in software with all primary design values. The software accommodates provision for prestressed girders. We need to specify which profile is to be adopted like parabolic, linear etc. This input for software can be given manually based on our primary design of prestressed girder of simply specify the profile and eccentricity and software will calculate the prestressing force and its compatibility in accordance with loading requirement. We can change the profile of cable or other parameters and see the response of structure. All the girders are symmetrical and identical, however in later stage can be modified as per requirement.

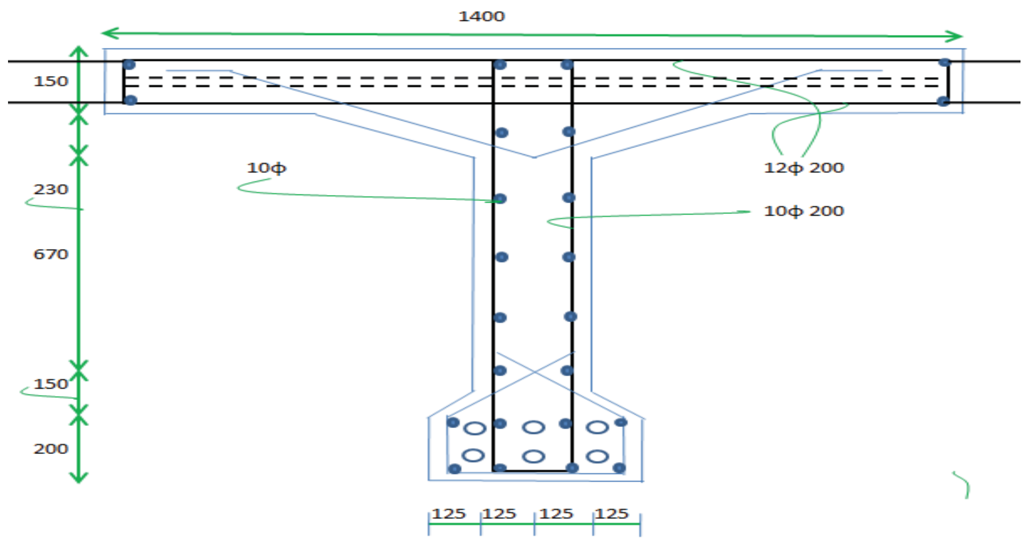

Figure 2. Cross section showing details of longitudinal girders. Dimensions in mm.

III. MODELLING OF BRIDGE

Sub structure and super structure are modelled on CSI Bridge software. The girders are prestressed hence the arrangement of cables is to be specified. The prestressing force can be calculated manually however here we use software calculated values. The software CSI Bridge uses programs that calculate section properties based on user input of cross sectional dimensions. All one must input is material properties, the dimensions of section defined, loading cases to be considered, for example in this paper loads considered are dead load, live load, prestress force, impact loading and seismic effect. The analysis methods used by software are static pushover analysis, response spectrum analysis as specified by user. Some modelling steps are shown below however all steps cannot be described to be precise as we are supposed to see the results and obtain performance point for our bridge. The results can be thus obtained after running the analysis.

Modelling of girder:

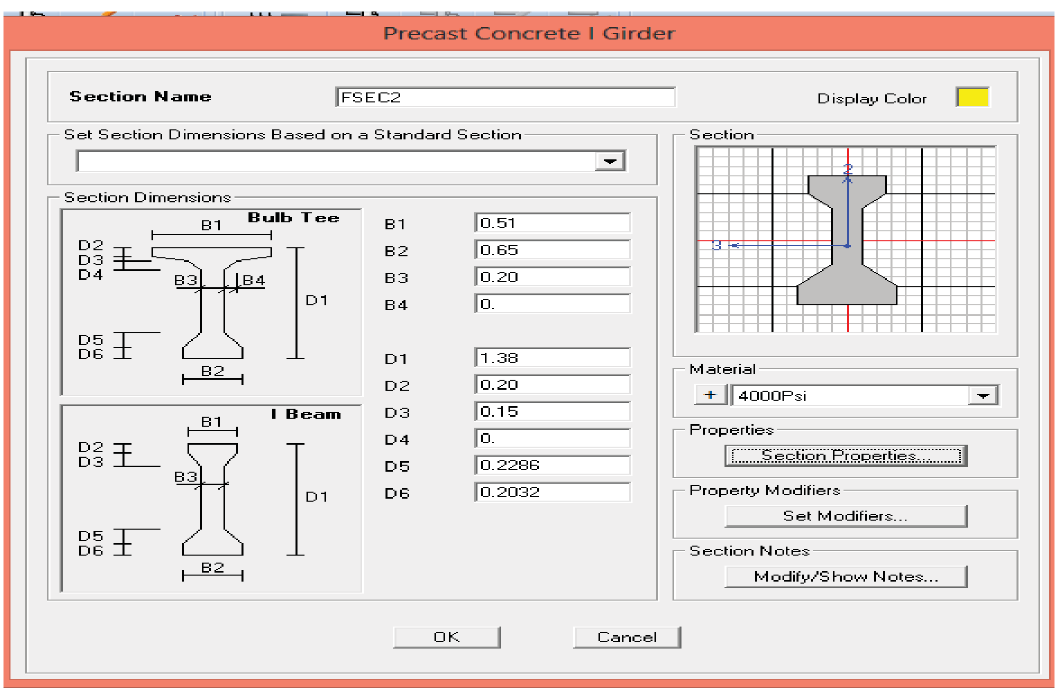

Figure 3. Snap from CSI Bridge software for girder modelling

Modelling of deck section along with girders

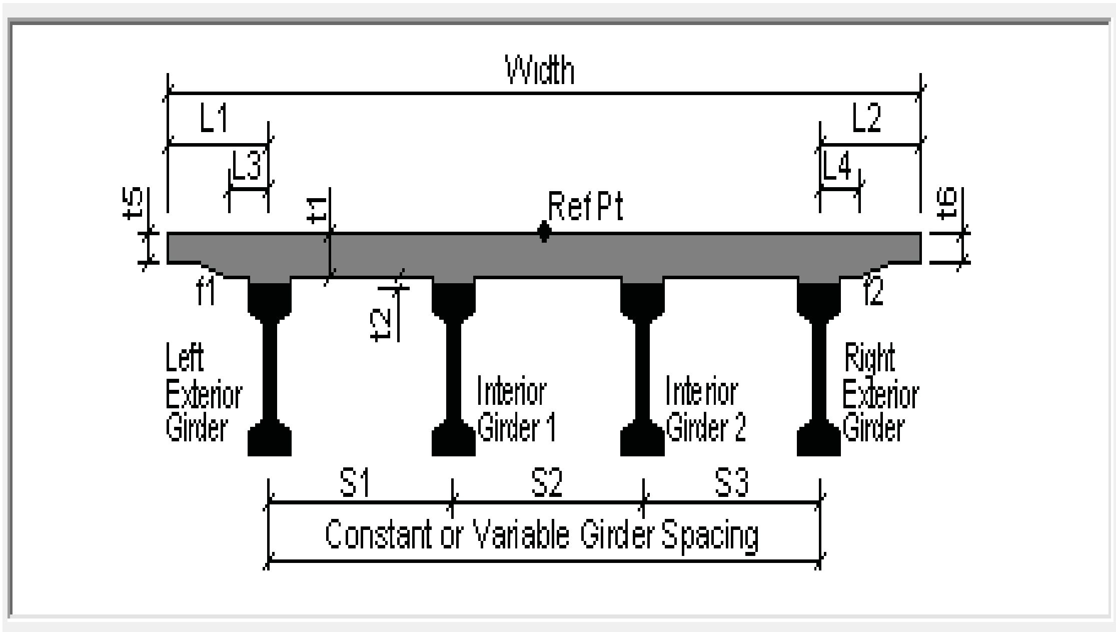

Figure 4. Snap from CSI Bridge software

Final model of bridge:

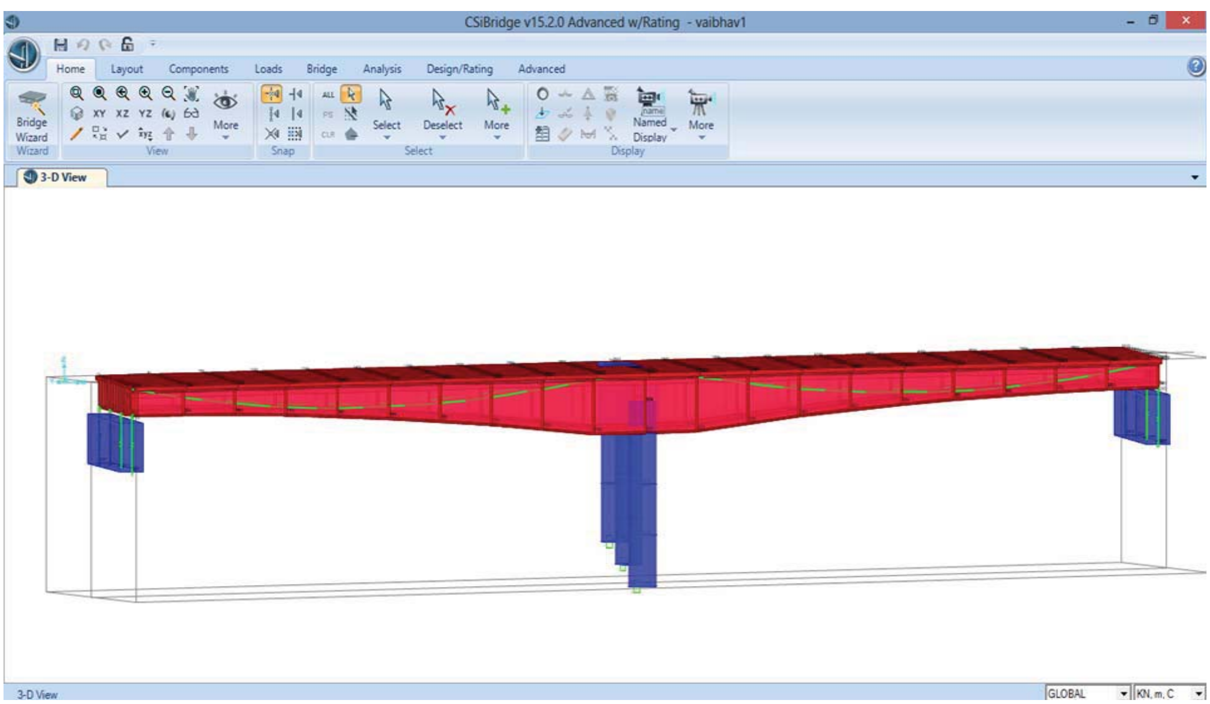

Figure 5. Varying thickness deck with prestressing cables and column bent at mid span as a non-linearity

Note that ends are restrained using abutment property while modelling. End abutments may be rotated for additional non linearity. The bent is at mid span with initial angle of 15 degrees. We can change as necessary. The foundation is given foundation spring property and is fixed in all degrees of freedom since we do not want foundation to move in any direction. The bearings are provided and they support the girders. The bearing property can be changed according to the degrees of freedom we want to specify for our bridge. While calculating live loads we use IRC Class AA loading. If designing bridge manually we need to find live load distribution factors to distribute live loads on girders using Morice Little method. Here software calculates the live load as well as live load bending moments. Live load distribution factors calculated by are given in *figure10.* For manually calculating the live load distribution factors the Morice little method is used since the number of longitudinal girders is more than 2. Morice Little method expresses flexure property of bridge deck as a whole in single parameter theta (θ). Since we are using software, we accept software evaluated live load distribution factors.

IV. LINEAR STATIC ANALYSIS

The base shear observed in software analysis is 7850kN. Let maximum lateral force at which column rebar reaches yield be=10000kN (set performance criterion more than base shear). Maximum lateral displacement observed on software analysis is= 1590mm.

Hence effective stiffness $K_{eff} = \frac{10000}{1590}$=6.28kN/mm

Approximate fundamental time period $T_f = 0.32X\sqrt{\frac{W}{Keff}}$

$= 0.32X\sqrt{\frac{98.7}{6.28}}$

= 1.26 sec.

Time period comparison:

By approximate formula=1.26 sec

By software linear static analysis=1.35 sec for first mode of vibration.

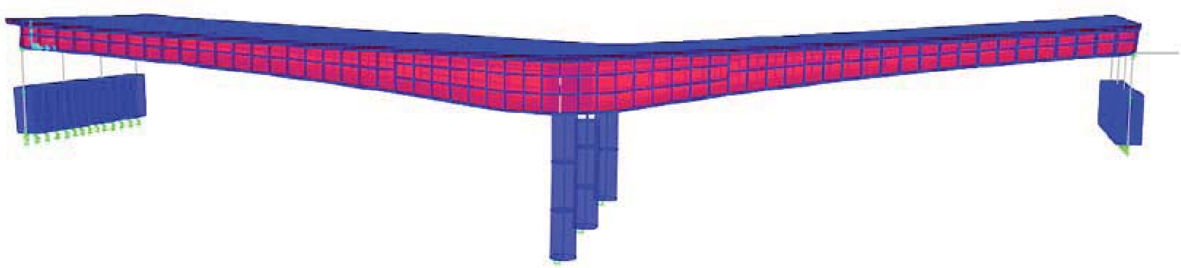

Figure 6. Deflected shape, first mode of vibration for linear static analysis.

The deflected shape as seen in *figure 6* is first mode of vibration with software calculated fundamental time period of 1.35 sec. The vertical deflection near both end abutments is 1.5 meters during peak earthquake excitation. This is more since bearings should not allow more than 0.5 meters. This indicates that the foundation spring property needs to be verified or the bearing check needs to be taken. Primary judgement says that the bearings have failed since there is no report of foundation failure. Note that we have modelled foundation in terms of foundation spring property whose behaviour is similar to that of actual foundation. This is called as software defined simulation of foundation

V. NON-LINEAR STATIC ANALYSIS

In push over analysis of bridge, computer model of bridge is subjected to lateral load of certain shape. (Parabolic, inverted, triangular or uniform). The intensity of lateral loads is slowly increased and sequence of cracks, yielding, plastic hinge formations and failure of various structural components is recorded. Pushover analysis gives us an idea about underperforming points in structure for seismic performance evaluation. No bridge can be pushed to infinity without failure. Performance point is where the seismic capacity and seismic demand curves meet. If performance point exists and damage state at point is acceptable, we have a bridge that satisfies pushover criteria.

Pushover curve or capacity spectrum:

The pushover curve for given bridge will be one of the curves shown in fig.7,

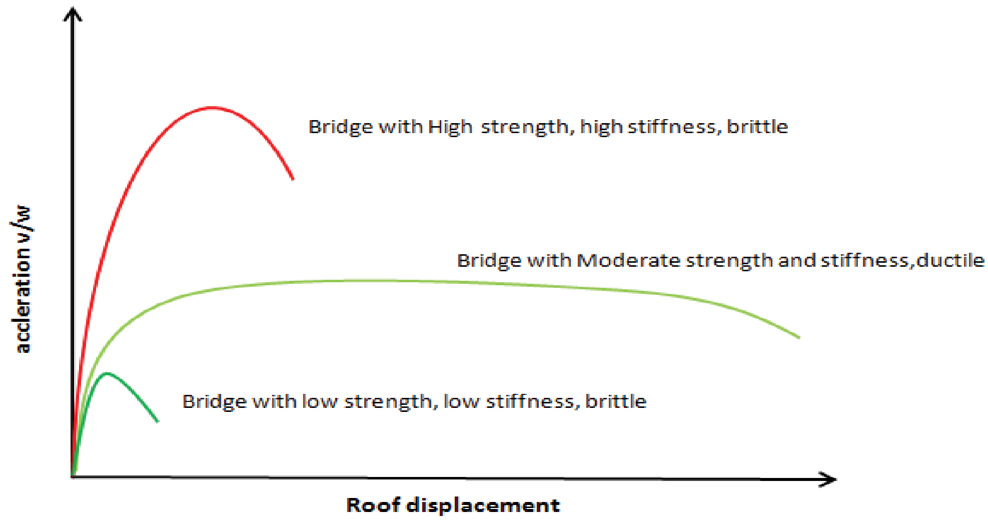

Figure 7. Capacity spectrum curve, in general

ATC-40 Non-linear static procedure:

1) First convert pushover curve into capacity curve
2) Plot elastic design spectra in acceleration-displacement format.
3) Plot demand diagram and capacity diagram together.
4) Point of intersection is displacement demand.

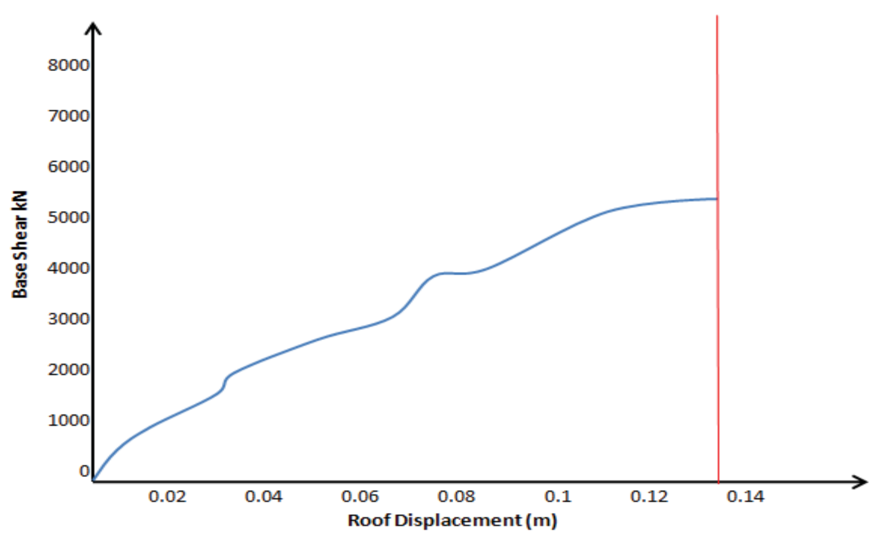

Figure 8. Pushover curve for bridge model

The pushover curve in *figure 8* indicates that bridge has moderate strength and stiffness is ductile and breaks at 0.135m, maximum roof displacement. The earlier computed value 0f 1.59 meters is for bridge as whole structure. The hinges are not seen in deck for lateral loading due to vast resisting depth. Hinge formation takes place either in pier or in bearing failure. If piers are of stone masonry like in old bridges, there will be no hinge formation since stones structure fail suddenly without any warnings. The bearings which connect sub structure with super structure fail in shear or overturning. We see the maximum lateral displacement of 1.59 meters due to shear failure as seen in *figure 6*. Hence bearings are to be redesigned.

Performance point (demand spectrum):

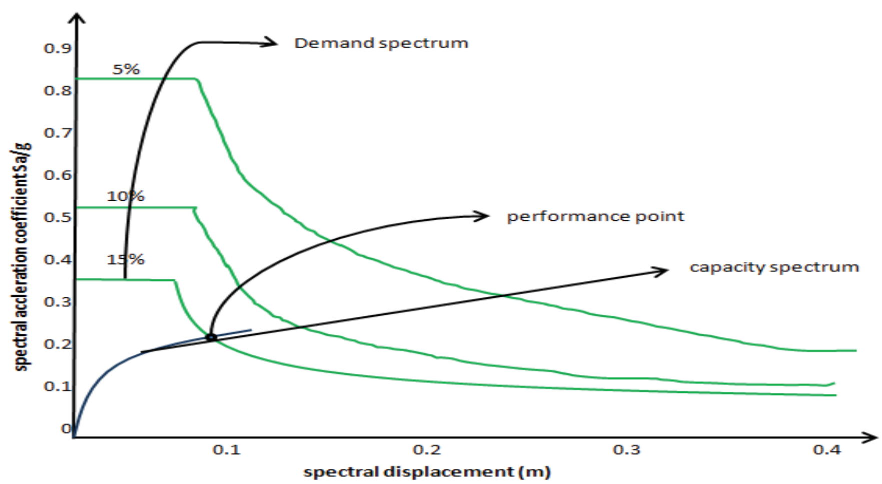

Figure 9. Performance point

Live load distribution factors calculated by software:

TABLE: Bridge Super Design 22 - AASHTOLRFD07 - PCCompShear-Prop										
DesReqName	BridgeObj	Station	Location	Girder	LLDFactV	LLDFactM	LLDFactSect	fysVRebar	fysLRebar	EsRebar
Text	Text	m	Text	Text	Unitless	Unitless	Unitless	KN/m2	KN/m2	KN/m2
DReq1 shear	bridge object	0	After	Left Exterior Girder	0.809146	0.809146	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	0	After	Interior Girder 1	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	0	After	Interior Girder 2	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	0	After	Interior Girder 3	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	0	After	Interior Girder 4	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	0	After	Right Exterior Girder	0.809146	0.809146	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	Before	Left Exterior Girder	0.809146	0.809146	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	Before	Interior Girder 1	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	Before	Interior Girder 2	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	Before	Interior Girder 3	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	Before	Interior Girder 4	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	Before	Right Exterior Girder	0.809146	0.809146	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	After	Left Exterior Girder	0.809146	0.809146	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	After	Interior Girder 1	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	After	Interior Girder 2	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	After	Interior Girder 3	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	After	Interior Girder 4	0.928711	0.795282	0.166667	413685.47	413685.47	199947978.8
DReq1 shear	bridge object	2.794	After	Right Exterior Girder	0.809146	0.809146	0.166667	413685.47	413685.47	199947978.8

Table 10. Live load distribution factors

Bridge objects response display:

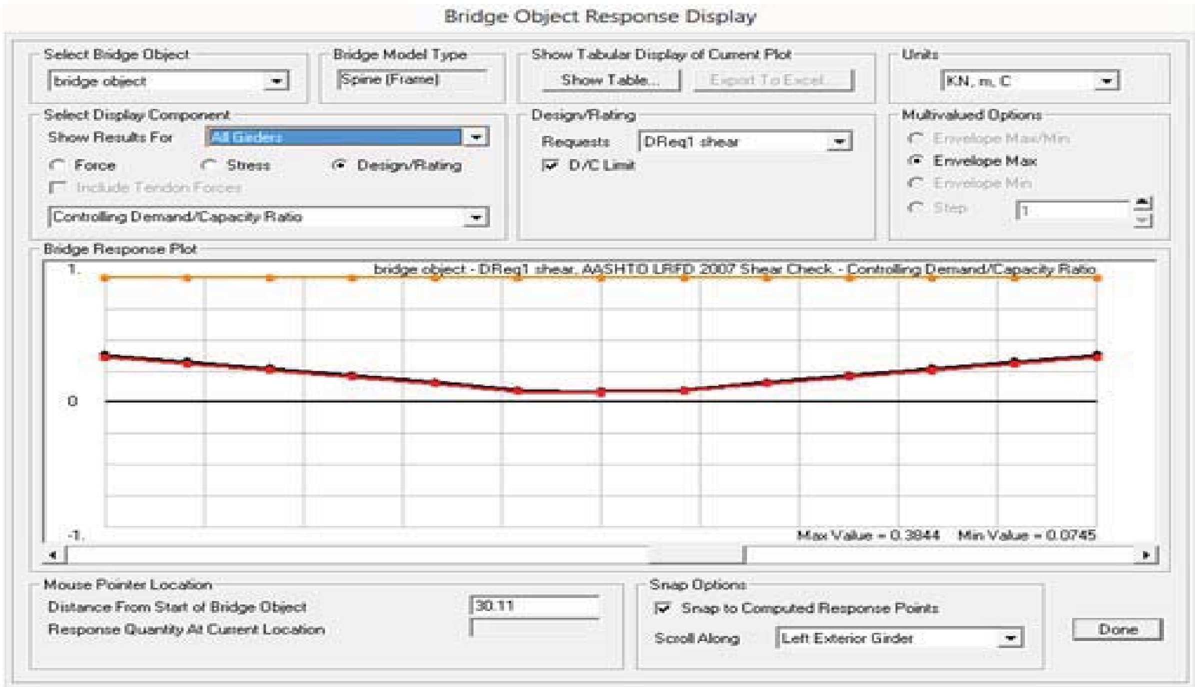

Figure 11. Shear check

VI. CONCLUSION

1) From performance point of view, effective time period comes to be 1.1 seconds as seen in above curve.(figure9). This is close to software calculated time period of 1.35 sec by linear static analysis and 1.2 sec as calculated by approximate formula.
2) Spectral displacement is close to 0.09m which is acceptable as no hinge was formed in structure.
3) The performance point is obtained at spectral acceleration of $0.298 m/s^2$.
4) For this model the spacing of girders was more, thus the deck slab was considerably thick. If deck slab thickness is to be reduced than girder spacing has to be reduced thereby increasing the number of girders.
5) Cross beams distributed live load evenly through girders under differential loading, and so this system of girder and cross beam was adopted after several model variations.
6) In deformed shape, bearings allowed more lateral displacement than necessary, hence bearings failed in shear however showed satisfactory results for overturning.

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