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SEISMIC PERFORMANCE STUDY OF URBAN BRIDGES USING NON-LINEAR STATIC ANALYSIS

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Abstract: The structure remains within elastic range employing linear method and resulting forces and the displacement are quite high. By introducing ductility in bridges, the load carrying capacity can be enhanced thus bridge is to be designed for lesser forces than obtained in elastic range. This requires to employ non-linear analysis (in-elastic range). Pushover analysis is an effective tool to evaluate the expected non-linear behavior and consequent failure pattern in different components of the bridge. In the present study, typical short and medium span bridges structure like a mono-pier, bent beam-pier frame (typical flyover) with and without elastic-foundation in the urban area are considered. Nonlinear push over analysis procedure as recommended by ATC-40 is adopted under various seismic demands. The hinge formation for expected performance level is recorded, and compared for different boundary conditions in terms of different soil types using soil-structure interaction, ground acceleration input, and various values of ductility factors. The response parameters like base shear and roof (top) displacement for each case are studied. Evaluation of performance points for the given structure is considered (important parameter) as per capacity-demand methodology.

Keywords: "Push over analysis, urban bridges, soil-structure interaction Plastic hinges, ATC-40"

I. INTRODUCTION

A bridge is an important component of the road transportation net work. Its performance during and even after an earthquake event is quite crucial for socio-economic considerations. Bridge often provides a vital link to earthquake ravaged areas as seen in the Bhuj earthquake and hence have a vital post-disaster operations. Therefore, critical bridge must remain functional even after the seismic event is over to provide relief as well as for security and defense purpose. Bridge consists of various components such as superstructure, substructure, foundation and the performance of the substructure, which is the link between ground and superstructure, influences the most towards performance of the bridge during an earthquake. Much of the substructure damage in past earthquake has occurred at columns (Tandon, 2001). Substructure may be of single or multiple column bents, reinforced concrete walls. The individual column may extend below the ground surface as a pile or caissons foundation or may be supported on a pile cap or spread footing. Therefore, the inclusion of influence of soil termed as soil-structure interaction (SSI) is desirable to carryout seismic performance study of bridges. The seismic performance of structure can be studied (Murthy and Jain, 2000) using simplified methods (response spectrum), time series method or non-linear static analysis. Non-linear static analysis is effective technique to study seismic response of structure like bridges.

In the present study, the performance of bridges in urban area under high seismicity, without mentioning specific route, typical structures resembling Metro piers i.e. Mono-piers, multi bent structures like flyover and short span bridges have been studied. The seismic region of greater impact i.e seismic zone IV and V (IRC:6 and IS:1893) have been considered. Soil-structure interaction (SSI) has also been considered. The hinge formation for expected performance level is recorded, and compared for different boundary conditions in terms of different soil types using soil-structure interaction, ground acceleration input, and various values of ductility factors. The response parameters like base shear and roof displacement for each case are studied. Evaluation of performance point (Sa, Sd) for the given structure is considered as per capacity-demand methodology. Structural analysis has been carried out using software SAP2000 in the present study. This would help assessing an existing bridge or for performance based design of bridges (Prakash, 2004).

II. CONCEPT OF PERFORMANCE STUDY OF BRIDGES

A. Performance Criteria

The basic design criterion, which any Earthquake-resistant structure must satisfy, is the following:

$$\text{Seismic demand} \leqslant \text{Computed capacity} \qquad \text{--- (1)}$$

For a design parameter such as shear or bending moment, 'Seismic demand' is the effect of the Earthquake on the structure. 'Computed capacity' is the structure's ability to resist that effect without failure.

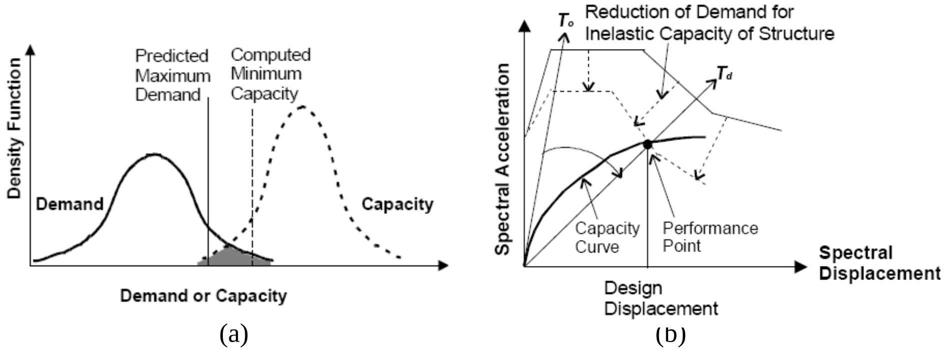

Figure 1. Probability distribution for capacity and demand concept (Naeim, 2000)

The shaded area in Figure 1(a) where both distributions overlap indicates that there is some probability of failure, in a situation capacity is less than demand. A process consisting of loading characterisation, acceptable performance level, providing adequate capacity, towards demand and strategy to meet the demand are mentioned schematically in Figure 2.

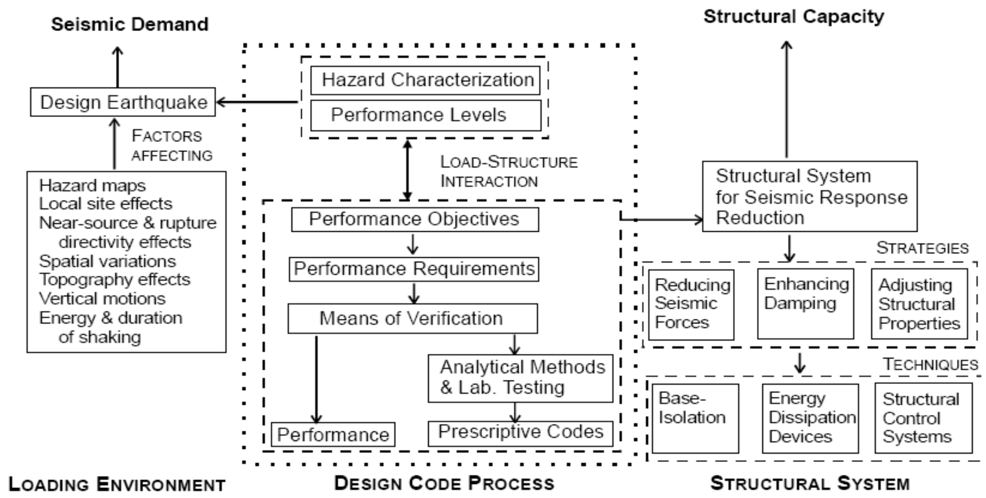

Figure 2. Performance Based Seismic Design Procedure (Priestly, 2000)

B. Performance Objective

Various performance levels are defined which specify consideration of specific seismicity and corresponding requirement during designing bridges as given in Table 1.

TABLE I

CONSIDERATION OF SEISMICITY AND PERFORMANCE LEVEL AS PER JAPANESE CODES

S. N.	Seismicity Considered	Requirement of Seismic Performance	Minimum Requirement for Design
1	Level 1: Frequent earthquakes	Performance 1: The structure should be functional without any repair	Normal: Design for Performance–1 with Level-1 of earthquake.
2	Level 2: Strong earthquake motions. Rare probability of occurrence during service life of the bridge	Performance 2: The structure should be functional within a few days. There may be minor repairs. There are two subtypes.	Severe: Design for Performance 2 or 3 with Level 2 of earthquake.
3	--	Performance 3: There may be extensive damage but no collapse.	--

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Inelastic seismic demand is based on inelastic capacity of structure. As inelastic displacements increase, the period of structure lengthens, damping increases and demand reduces. The Capacity Spectrum Method generates Performance Point where displacement is consistent with the implied damping. Design is based on displacement corresponding to the Performance Point (Figure 1 (b)), which implies a unique damage stage related to a specific hazard level. The parametric response related to certain performance objective are predefined related to type of structures such as for bridges given in Table 2 adopted from MCEER (2001).

TABLE 2

PROPOSED DAMAGE FOR REQUIRED PERFORMANCE OBJECTIVE (MCEER)

Parameter	Life Safety	Immediate Occupancy
Column plastic hinge rotation	0.035 rad	0.01 rad
Vertical offset in girders	0.2 m	0.03 m

C. Nonlinear Procedures

Simplified nonlinear procedure is used for practical applications. Different simplified nonlinear procedures used to implement the pushover analysis are:

(i) Capacity Spectrum Method (CSM) (ATC-40, 1996)
(ii) Displacement Coefficient Method (DCM) (FEMA-273,1997)
(iii) The secant method and
(iv) Modal Pushover Analysis (MPA) (Chopra 2001)

Different methods used for evaluating the Nonlinear static procedure (NSP) may lead to similar results in most of the analysis but they differ in respect to simplicity, transparency and clarity of theoretical background. Nonlinear static analysis also called as pushover analysis is used to determine displacement capacity of structures and also to estimate available plastic rotational capacities to ensure satisfactory seismic performance. Seismic demands in pushover analysis are estimated by establishing the capacity curve for a structure by monotonically increasing the displacement at a control node until a prescribed displacement is reached or the structure collapses.

D. Capacity Spectrum Method (CSM) - ATC 40

The procedure for the CSM has been developed by ATC-40. In CSM, the design curve shown in Figure 1(b) is reduced by using spectral reduction factors to intersect the capacity curve shown in Figure 8 to find the performance point. The performance point indicated the seismic capacity of structure which will be equal to seismic demand imposed in structure by ground motion. In pushover analysis, the performance point or target displacement is based on the assumption that the fundamental mode or uniform mode of vibration is the predominant response of the structure and mode shapes remain unchanged until collapse occurs. The performance point must satisfy two relationships namely a)the point must lie on the capacity spectrum or capacity curve in order to represent a structure at given displacement and b)the point lie on the spectral demand curve, reduced from the elastic 5 percent-damped design spectrum (Abheysinghe, 2001).

III. STUDY OF TYPICAL URBAN BRIDGES

For studying the performance of bridges in urban area under high seismicity, typical structures resembling Metro piers i.e. mono-piers, multi-bent structures like flyover and short span bridges have been studied. The seismic region of greater impact i.e. seismic zone IV and V have been considered. Soil-structure interaction (SSI) has also been considered. Static (response spectrum) as well as nonlinear static procedure like push over analysis with parametric variation have been studied to understand their seismic performance. The structural details do not pertain to the actual bridges rather their relevance is considered during seismic performance study.

A. Study of Mono-Pier Bridge

The structural details and modeling are as follows.

Circular pier, Diameter- 1.8m, Height= 9m, Centre of Gravity (C.G.) of super structure= 2m from pier top (Fig.3). Concrete: M45 grade, Steel: FE500 grade, Reinforcement :1.35%, (Priestley, 1996). For Soil-structure Interaction (SSI) modeling, pile with diameter as 1.8m is taken up to 10m depth below ground level with adopted reinforcement value. Superstructure loading is taken for a span of 40 m.

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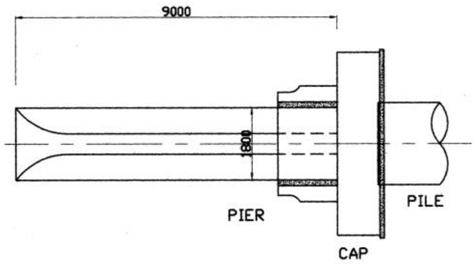

Figure 3 Sketch of Typical Mono-Pier

Modeling is carried out using SAP2000 v.11.0.0. A simple beam element model with discretized node at 1m spacing is generated using the tool. Mass is lumped at the nodes at C.G. of superstructure. Two cases of base fixity are considered. The base is fixed at ground and base with elastic foundation having a Cast-in-drill-hole (CIDH) pile with diameter as of pier modeled up to 10m depth below ground level. Three type of soil are taken into consideration and modeled as Winkler springs. The three soil conditions of IS1893/IRC6 are represented as Type I- Rocky soil (coarse crushed stone), type II- Medium soil (very well compacted sand and clays soil with sand) type III- Soft soil (Fine or slightly compacted soil) having modulus of subgrade reaction, ks (MN/m3) as 225, 90, 15 respectively (Hemslay, 2000). The spring with spring constant proportionate to the depth of location at node modeled representing the soil stiffness, ki (kN/mm) and are for type-I as 25.31 to 1822, for type-II as 2.5 to 2880 for type III as 3.38 to 432.
Response Spectra analysis is carried out using IS:1893/IRC6:2002 spectra and methodology with varying values of response reduction factor, R. The Zone factor (Z) and Soil Type I (Rocky soil), Type II (Medium soil) and Type III (Soft soil) as mentioned above.
Further, for in-elastic analysis, the hinges are assigned to the mono-pier model at the bottom of fixed base and throughout the pile depth below ground level adjacent to springs. The Life safety performance level is chosen as a limit of failure. Respective performance point in terms of base shear and roof displacement are set up as limiting criteria for performance based study. The typical capacity curve for fixed base and mono pier-pile foundation model are discussed in next sections.

B. Seismic Performance of multi-bent bridge

A multiple bent bridge structure as shown in the Figure 4 is considered. The structure may have resemblance with typical short span bridge having four span with precast I-girders. Bent beam : 2m x 1.5m rectangular RCC beam. Concrete M25 grade and Steel FE 415 grade. Pier:1.3 m diameter circular RCC section, Concrete M45 grade and Steel FE 415 grade. Longitudinal Reinforcement 25 nos. of 25mm diameter bar. Transverse reinforcement 12mm dimeter spiral at 115mm c/c spacing. Superstructure details : Precast I-Girder section, there are 27 no. of girders each having cross sectional area of 0.6 m2. Deck slab is 150mm thick. Dead weight from crash barrier, median and wearing course also considered. Live load of 70 R 2-lane is taken.

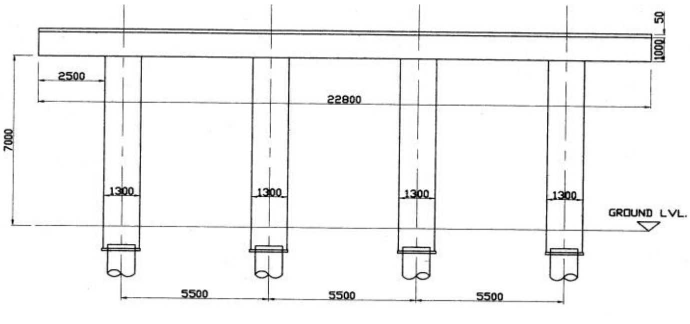

Figure 4 Cross-sectional Details of Multi-Bent Beam-Pier Frame System

For non-linear static analysis, a simple interior bent beam-pier frame is modeled by using SAP2000. Mass of 300 ton is lumped at each top node of beam-pier connection in three Global axes. Nonlinear static analysis (push over) using ATC 40‘s capacity spectrum method is carried out to find out the performance point. A displacement based pushover technique is used of 0.24g and 0.36g for ground acceleration for soil Type II and Type III in both longitudinal and transverse directions. In the case demand spectra from IS1893:2002 is used with varying values of response reduction factor (R). Hinges are assigned to the base of pier and in vicinity of bent beam-pier joint as this location is more prone to failure. A case of auto hinge generation of SAP with default properties is adopted in the study for more precise

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failure evaluation. Both the performance objectives as per MCEER (Table 2) are considered.
Nonlinear static analysis of multi-span bridge is also studied. The dimensional details are shown in Figure 5. Bridge Wizard module in SAP2000 is used to model the complete bridge and push over analysis is carried out.

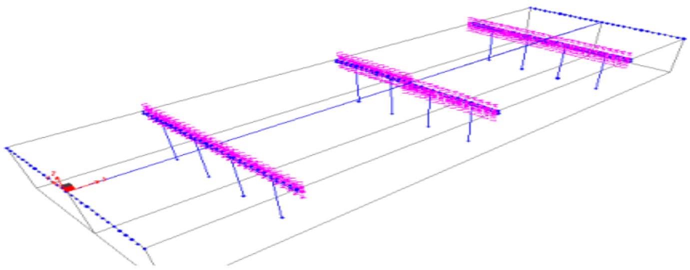

Figure 5 Full bridge model using SAP2000

IV. RESULTS AND DISCUSSION

Based on numerical study carried out on typical short and medium span bridges, important results are presented and discussed.

i) *Linear Static Analysis (Response spectra) for Mono-Pier System*

The relations between Base shear and roof displacement has been obtained from linear static analysis and is presented in Figure 6. For the Pier model, by varying R base shear reduces but at the same time Ductility of material in terms of roof displacement varies in 25-30%. For the different Earthquake ground motion for particular type of soil, it is observed that as R value increases the roof displacement reduces considerably. As R value increases there is corresponding reduction in the base shear also relative to particular earthquake intensity.

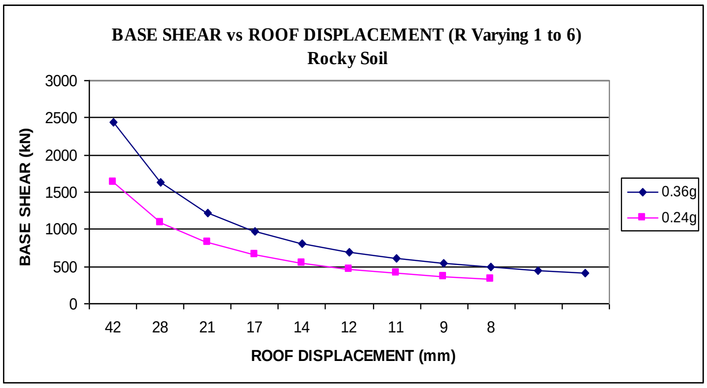

Figure 6 Base Shear vs Roof Displacement (Ground Acceleration Varies)

ii) *Nonlinear static analysis of mono-pier model using capacity spectrum method*

Non-linear static analysis (Pushover) has been compared with response spectrum for mono-pier with fixed base (Soil type II) under similar site condition for different values of R and a respective Base shear and roof displacement and is found that respective values are nearly similar.

iii) *Hinge Formation*

Push over analysis for the two system shows that the Life safety (LS) as a performance level chosen for pier with fixed foundation is achieved in step 23 and for flexible pier case Immediate occupancy (IO) is achieved in step 11 as shown in Figure 7. This indicates that Mono pier–pile system has more ductile behavior than that of fixed base system provided proper reinforcement detailing is adopted. Also the hinge formations are more liable to occur near ground level in pile system than fixed base pier. This highlights need of modeling soil with bridge

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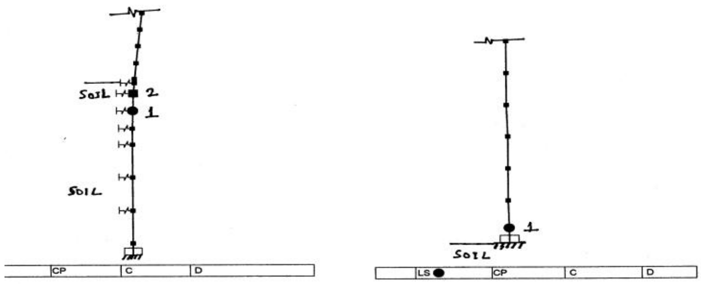

(A) Mono-Pier with Elastic Foundation, (B) Mono-Pier with Fixed Base.

Figure 7 Hinge formation.

iv) *Capacity Curve for Mono-Pier Fixed Base with Elastic Foundation*

The mono-pier with fixed base gives higher base shear but lower roof displacement at performance point compared to pier with elastic foundation as shown in Fig.8. Based on Pushover curve for mono-pier model with fixed base, the following performance points are obtained: performance point V, D as 1260.2,0.114, performance point Sa,Sd as 0.135g, 0.106m, performance point, Teff, Beff as 1.777, 0.202, respectively.

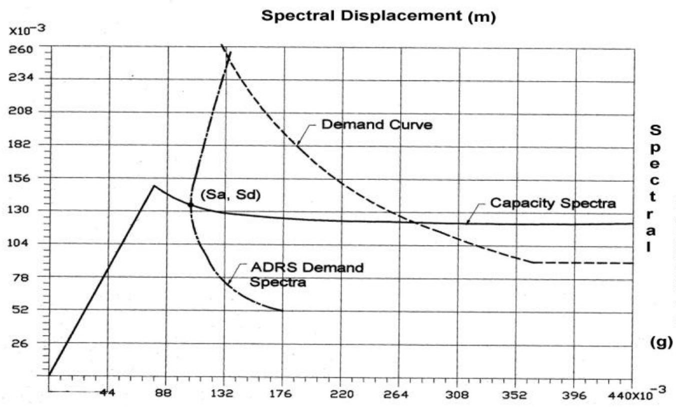

Figure 8 Pushover Curve for Mono-Pier Model with Elastic Foundation

v) *Nonlinear Static Analysis of Bent Beam–Pier Frame Model*

These values are observed to be similar to the values given by the response spectra analysis. Also the results indicate that at the transverse mode structure has to carry large amount of shear forces for comparatively small roof displacement compared to single column pier (Table 3).

TABLE 3

RESPONSE VALUE BY CAPACITY SPECTRUM METHOD (LONGITUDINAL & TRANSVERSE DIRECTION)

Seismic demand	Pushover in Longitudinal direction for Soil Type II, 0.24g		Pushover in Transverse direction for Soil Type II , 0.36g	
R	Base Shear (kN)	Roof displacement (mm)	Base Shear (kN)	Roof displacement (mm)
1	2093	83	3647.53	47
2.5	1725.96	23	3093	13
4	1684.94	17	2499.5	7
5	1605	14	1979.52	6
6	1332	11	1673	5

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For the longitudinal push case, the first hinge formation is observed at intermediate pier as shown in Figure 9. It indicates that the first hinge for Life safety level is achieved first at the intermediate pier suggesting this to be possible weakest part in this case.

Figure 9 Life Safety Level Hinge Locations in Two Directions (LONGITUDINAL & TRANSVERSE DIRECTION)

For the transverse push case, the first hinge formation is observed at outer cap-beam portion adjacent to pier-bent beam exterior joint pier as shown in Figure 9. However, for the life safety level, hinge is formed first in intermediate piers similar to the longitudinal case.

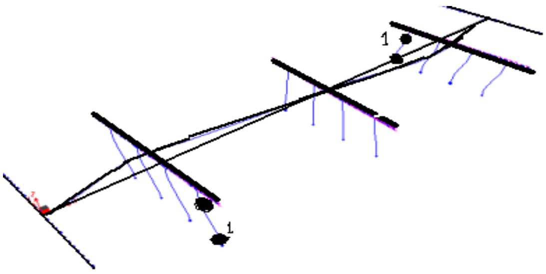

Figure 10 First Hinge in Longitudinal Direction for Multi-span Bridge

For multi-span bridge, under longitudinal push over, the exterior columns were observed to be weak as hinges were formed (Figure 10). Also for transverse direction higher steps are required.

V. CONCLUSION

The seismic study carried out on mono-pier, multi-bent and multi span bridge system with and without consideration of flexibility of foundation, and the important conclusions can be highlighted as follows:

Push over analysis for *the mono-pier system* shows that the Life safety (LS) as a performance level chosen for pier with fixed foundation is achieved in step 23. However, for Immediate occupancy (IO), in flexible pier is achieved earlier i.e. in step 11. This indicates that consideration of Mono pier–pile as flexible foundation system has more ductile behavior than that of fixed base system provided proper reinforcement detailing is adopted. Also the hinge formations are more liable to occur near ground level in pile system than fixed base pier. This highlights need of modeling soil with bridge. In case of *bent beam–pier frame model* the results indicate that in transverse mode case, the bridge has to carry large amount of shear forces for comparatively small roof displacement as compared to mono-pier system. However, for the longitudinal mode case, the first hinge formation is observed at intermediate pier suggesting this to be possible weakest part in this case. *For multi-span bridge*, under longitudinal push over, the exterior columns were observed to be weak as hinges were formed first.

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