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**Authors:** Chung C. Fu, Ph.D., P.E., Director/Associate Professor, the Bridge Engineering Software & Technology (BEST) Center

Hamed AlAyed, Ph.D., Research assistant, the BEST Center

Department of Civil & Environmental Engineering

University of Maryland

College Park, MD 20742

Tel: 301-405-2011

Fax: 301-314-9129

E-mail: ccfu@eng.umd.edu

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# SEISMIC ANALYSIS OF BRIDGES USING DISPLACEMENT-BASED APPROACH

By Chung C. Fu and Hamed AlAyed, University of Maryland

## ABSTRACT

Nonlinear Static (Pushover) Procedure is specified in the guidelines for seismic rehabilitation of buildings presented by FEMA-273 (1997) as an analytical procedure that can be used in systematic rehabilitation of structures. However, those guidelines were presented to apply the Displacement Coefficient Method, which implements the well-known equal displacement rule with some modifications to estimate target (demand) displacement, only for buildings. This study is intended to evaluate the applicability of Nonlinear Static Procedure by implementing the Displacement Coefficient Method to bridges. For comparison purposes, the Nonlinear Dynamic Procedure (or nonlinear time-history analysis), which is considered to be the most accurate and reliable method of nonlinear seismic analysis, is also performed.

A three-span bridge of 97.5 meters (320 ft) in total length was analyzed using both the Nonlinear Static Procedure/Displacement Coefficient Method and nonlinear time-history. Nine time-histories were implemented to perform the nonlinear time-history analysis. Three load patterns were used to represent distribution of the inertia forces resulting from earthquakes. Demand (target) displacement, base shear, and deformation of plastic hinges obtained from the Nonlinear Static (Pushover) Procedure are compared with the corresponding values resulting from the nonlinear time history analysis. Analysis was performed using two levels of seismic load intensities (Design level and Maximum Considered Earthquake level). Performance of the bridge was evaluated against these two seismic loads. Comparison shows that the Nonlinear Static Procedure gives conservative results, compared to the nonlinear time history analysis, in the Design Level while it gives more conservative results in the Maximum Considered Earthquake level.

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## INTRODUCTION

A large number of bridges were designed and constructed at a time when bridge codes had no seismic design provisions, or when these provisions were insufficient according to current standards. Many of these bridges may suffer severe damage when struck by earthquakes, as evident by recent moderate earthquakes. Linearly elastic procedures are efficient as long as the structure behaves within elastic limits. If the structure responds beyond the elastic limits, linear analyses may indicate the location of first yielding but cannot predict failure mechanisms and account for redistribution of forces during progressive yielding. This fact makes the elastic procedures insufficient to perform assessment and retrofitting evaluation for those bridges in particular and structures in general. Nonlinear (static and dynamic) procedures can overcome this problem and show the performance level of the structures under any loading level.

The focus of seismic design in current building and bridge codes is one of life safety level without the ability to consider multiple levels of structural performance from various loading conditions. Economic losses due to recent earthquakes are estimated to be many billions of dollars and the numbers will be higher if the indirect losses are included. This fact lets codes committees and decision makers think beyond life safety, which is essential in design, to alleviate economic losses. This trend creates an increased interest in performance-based design for structures. One of the main advantages of performance-based design is its ability to show the performance situation of the structure and its components under different load intensities. The performance situation means that the damage level, if any, can be assessed and a judgment can be made as to which degree this structure can continue in service.

Many methods were presented to apply the Nonlinear Static Procedure (NSP) to structures. Those methods can be listed as (1) the Capacity Spectrum Method (CSM) (ATC, 1996); (2) the Displacement Coefficient Method (DCM) (FEMA-273, 1997); (3) the secant method [e.g., City of Los Angeles (COLA, 1995)]; and (4) Modal Pushover Analysis (MPA) Chopra and Goel (2001). However, these methods were developed to apply the NSP for buildings only. Bridge researchers and engineers are currently investigating similar concepts and procedures to develop simplified procedures for performance-based seismic evaluation of bridges (Barron, 2000; Dutta, 1999; Shinozuka, 2000).

Few studies were performed to apply the NSP for bridges. In those studies, the CSM was implemented to estimate the demand (target displacement). CSM needs many iterations while the DCM, in general, needs no iterations. In this study, the DCM will be implemented to estimate the target displacement and perform the pushover analysis. Also, the performance acceptance criteria proposed by FEMA-273 (1997) will be implemented to evaluate the performance levels.

## KEY ELEMENTS OF THE PUSHOVER ANALYSIS

Due to the nature of bridges, which extend horizontally rather than buildings extending vertically, some considerations and modifications should be taken into consideration to render the DCM applicable for bridges. The modifications and considerations should concentrate on the following key elements:

1. Definition of the control node: Control node is the node used to monitor displacement of the structure. Its displacement versus the base-shear forms the capacity (pushover) curve of the structure.
2. Developing the pushover curve, which includes evaluation of the force distributions: To have a displacement similar or close to the actual displacement due to earthquake, it is important to use a force distribution equivalent to the expected distribution of the inertia forces. Different formats of force distributions along the structure are implemented in this study to represent the earthquake load intensity.
3. Estimation of the displacement demand: This is a key element when using the pushover analysis. The control node is pushed to reach the demand displacement, which represents the maximum expected displacement resulting from the earthquake intensity under consideration.

4. Evaluation of the performance level: Performance evaluation is the main objective of a performance-based design. A component or an action is considered satisfactory if it meets a prescribed performance level. For deformation-controlled actions the deformation demands are compared with the maximum permissible values for the component. For force-controlled actions the strength capacity is compared with the force demand. If either the force demand in force-controlled elements or the deformation demand in deformation-controlled elements exceeds permissible values, then the element is deemed to violate the performance criteria.

## LOAD PATTERNS

Different load patterns were used to represent the load intensity produced by earthquake. The first pattern, which is the Uniform Pattern, is based on lateral forces that are proportional to the total mass assigned to each node. It can be applied to bridges as:

$$F_i = m_i * g \tag{1}$$

where $F_i$ = the lateral force at node $i$ ($i$ = 1, 2, …, $N$), $N$ = number of nodes, $m_i$ = mass assigned to node $i$, and $g$ is the ground acceleration. FEMA-273 (1997) requires using two load patterns (the Uniform Pattern and one of the other two load patterns) and takes the maximum value for each action. This load pattern is intended to emphasize the base shear rather than yielding high moment and deformations.

The second load pattern for bridges, which is called the Modal Pattern in this study, can be written by using load pattern distribution according to the first mode as:

$$F_i = \left( m_i \phi_i \Big/ \sum_{i=1}^{N} m_i \phi_i \right) V \tag{2}$$

where $F_i$ = the lateral force at node $i$ ($i$ = 1, 2, …, $N$), $N$ = number of nodes, $m_i$ = mass assigned to node $i$, $\phi_i$ = amplitude of the fundamental mode at node $i$, and $V$ = base shear. This pattern may be used if more than 75% of the total mass participates in the fundamental mode of the direction under consideration (FEMA-273, 1997). The value of $V$ in the previous equation can be taken as an optional value since the distribution of forces is important while the values are increased incrementally until reaching the prescribed target displacement or collapse.

The third load pattern, which is called the Spectral Pattern in this study, should be used when the higher mode effects are deemed to be important. This load pattern is based on modal forces combined using SRSS (Square Root of Sum of the Squares) or CQC (Complete Quadratic Combination) method. It can be written as:

$$F_i = \left( m_i \delta_i \Big/ \sum_{i=1}^{N} m_i \delta_i \right) V \tag{3}$$

where $F_i$, $m_i$, $N$, and $V$ are the same as defined for the Modal Pattern (Eq. 2), and $\delta_i$ is the displacement of node $i$ resulted from response spectrum analysis of the structure (including a sufficient number of modes to capture 90% of the total mass), assumed to be linearly elastic. The appropriate ground motion spectrum should be used for the response spectrum analysis.

## CASE STUDY

A three-span bridge, which was presented by the Federal Highway Administration (FHWA, 1996) to illustrate the AASHTO requirements for seismic design of bridges, was chosen. The total length of the bridge is 97.5 m (320 ft) with spans of 30.5 (100), 36.5 (120), and 30.5 m (100 ft). All substructure elements are oriented at a 30-degree skew from a line perpendicular to a straight bridge centerline alignment. Fig. 1 shows the plan and the elevation of the bridge. The superstructure is a cast-in-place concrete box girder with two interior webs. The intermediate bents have a crossbeam integral with the box girder and two round columns that are pinned at the top of spread footing foundations. Fig. 2 shows a cross section through the bridge with an elevation of an intermediate bent. The seat-type abutments are on spread footings, as shown in Fig. 3, and the intermediate bents are all cast-in-place concrete. Framing of the box girder superstructure is shown in Fig. 4.

In the longitudinal direction, the intermediate bent columns are assumed to resist the entire longitudinal seismic force. The seat type abutments (Fig. 3) will allow free longitudinal movement of the superstructure and will not provide longitudinal restraint. In the transverse direction, the superstructure is assumed to act as a simply supported beam spanning laterally between the abutments with the maximum transverse displacement at the center of the middle span. The intermediate bents are assumed to participate in resisting the transverse seismic force along with the superstructure. A shear key provides transverse restraint to enable transfer of transverse seismic forces to the abutment.

**FINITE ELEMENT MODEL**

The structural analysis program, SAP2000- Version 7.4 (nonlinear) (CSI, 2000), was used to perform analyses. Geometric nonlinearity through considering P-Delta effect was applied to this bridge in addition to material nonlinearity. As shown in Fig. 5, the model includes a single line of three dimensional frame elements for the superstructure and elements for the intermediate bents.

The superstructure has been modeled with four elements per span and the work lines of the elements are located along the centroid of the superstructure. The total mass of the structure was lumped to the nodes of the superstructure (nodes 1-13 in Fig. 5). An additional load of 2.35 kips per linear foot (34.3 kN/m) of superstructure was considered to represent loads from traffic barriers and wearing surface overlay. The weight of the mid-span diaphragms was lumped to the nodes of the mid-spans. Weight of the cap beams and half weight of the bents were lumped to nodes of the superstructure corresponding to bents (nodes 5 and 9 in Fig. 5) since weight of the bent columns is not significant. Determination of the moment of inertia and torsional stiffness of the superstructure are based on uncracked cross sectional properties because the superstructure is expected to respond linearly to seismic loadings. The presence of skew is accounted for only in the orientation of the substructure elements, and is not considered in determination of the superstructure properties.

There are no elements to model the abutments; only support nodes are shown in Fig. 5. The bents are modeled with three-dimensional frame elements that represent the cap beams and individual columns. Fig. 6 shows the relationship between the actual bent and the “stick” model. Since columns are pinned to the column bases, two elements were used to model each column between the top of footing and the soffit of the box girder superstructure; the upper element represents the plastic hinge while the lower one represents the rest of the column. A rigid link was used to model the connection between the column top and the center of gravity for the cap (at the structure centroid) beam. Foundations are represented by a three-dimensional element with the same properties of the footing, which approximates a rigid link due to its high stiffness. The node at the top of the footing (X10) is released for rotation in both plan directions to model the pinned column base. Stiff elements (with increased stiffness properties) were used to model the cap beams for distribution of loads between the columns without having deformation to the cap beams in order to match the behavior of the superstructure. The moments of inertia for columns were calculated based on the cracked section using Moment-Curvature (M-$\phi$) curve.

The intermediate bent foundations were modeled with equivalent spring stiffnesses for the spread footing. Details of the spring supports are shown in Fig. 7. For this bridge, all of the intermediate bent footings use the same foundation springs. The stiffnesses are developed for the local bent supports and transformed to global support when input to SAP2000 program so as to have compatible results for the Nonlinear Static Procedure (NSP) and the Nonlinear Dynamic Procedure (NDP or nonlinear time-history analysis). Values of stiffnesses for foundation springs provided by FHWA (1996) are used in this study. The abutments have been modeled with a combination of full restraints (vertical translation and superstructure torsional rotation) and an equivalent spring stiffness (transverse translation), as shown in Fig. 7. Other degrees of freedom are released. Input files of this bridge for SAP2000 nonlinear version 7.4 can be found in AlAyed (2002) for both NSP and NDP.

**SEISMIC LOADING**

To perform analysis of structure, the next step after modeling is applying loads. Design response spectrum should be available in order to perform NSP. Also, it is necessary to simulate artificial time-histories or scaling actual time-history records. This bridge is to be built in the western United States in a seismic zone with an acceleration coefficient of (PGA = 0.3g) (FHWA, 1996). This estimation is based on the AASHTO (1995) specification for an earthquake of 10% probability of occurrence in 50 years, which is equivalent to a recurrence period of 475 years. This PGA is equivalent to the factor $S_{DS}$, presented in FEMA-302 (1997) and called Design Level in this study. The subsurface conditions consist of a 250-foot-deep glacial deposit of dense sand and gravel overlying rock. The $S_{D1}$ factor corresponding to the $S_{DS}$ = 0.3 is 0.407. The PGA for the Maximum Considered Earthquake (MCE), which is the earthquake that has a recurrence period of 2500 years, is equal to 0.45 and the corresponding $S_{D1}$ = 0.61. A 5%- damped response spectrum for both accelerations is shown in Fig. 8.

To perform the Nonlinear Dynamic Procedure (NDP), or what usually is called Nonlinear Time-History Analysis (NTHA), acceleration time-history records should be available. Nine time-histories were implemented in this study; two of them are actual time-histories, which were adjusted to match the design response spectrum for each case, and the rest are artificial. The artificial time-histories were simulated using SIMQKE-1 code, which was developed by Venmarcke and Gasparini(1976) and modified by Blake and Park (1990). The actual acceleration time-histories are: (1) Northridge 01/17/94, Century City Lacc North, 090 (PEER, 2000-a), and (2) Loma Prieta 10/18/89, Gilroy # 2, 000 (PEER, 2000-b).

## RESULTS AND PARAMETRIC STUDY

Analyses were performed for two levels of seismic load intensity. For the first level (Design Level), PGA = 0.3g and for the second level (Maximum Considered Earthquake, MCE, Level), PGA = 0.45g. Comparison is performed for the maximum displacement, total base shear, and rotation of plastic hinges resulting from the NSP and the corresponding results from the nonlinear time-history.

### LONGITUDINAL DIRECTION

Period of the first mode in this direction is 0.97445 seconds and the modal participation mass ratio for this mode is 99.95% (Table 1). Pushover curve for this direction is shown in Fig. 9. The formula shown below (FEMA-273, 1997) is used to estimate the target displacement:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} \tag{4}$$

where $C_0$, $C_1$, $C_2$, and $C_3$ are modification factors to consider different parameters affect the control node displacement, and the rest of the formulae is the equal displacement rule. For more details about the definition of each of the previous parameters and how to estimate it, see FEMA-273 (1997).

The estimated target displacement is 98.5 mm (0.323 ft) for the Design Level and 147.8 mm (0.485 ft) for the MCE Level. The internal forces and deformations corresponding to each one of these displacements are determined in order to evaluate the performance level for each design level. In this study, maximum displacement, base shear, and rotation of plastic hinges resulting from the NSP are compared with the corresponding values resulting from the NDP (nonlinear time history), which is considered the most accurate and reliable procedure. To take care of the uncertainty associated with each time-history record, the average of forces and deformations resulting from the nine time-histories used in this study are implemented for the comparison with the NSP results.

Table 2 compares different parameters resulting from NSP with the corresponding parameters from the nonlinear time-history. In the NSP, where the DCM is implemented in this study, the Modal Pattern (M-Pattern) and Uniform Pattern (U-Pattern) are implemented while the Spectral Pattern (S-Pattern) is not used with the longitudinal direction since it gives the same results as the Modal Pattern (, where the participation mass ratio for the first mode is 99.95%). Since the participation mass ratio for the first mode is very close to 1.0, this bridge behaves similar to a single-degree-of-freedom system. Due to this behavior, the three load patterns give the same results in the longitudinal direction. Diff. (%) is defined as

$$\text{Diff. (\%)} = \frac{X_{NS} - X_{ND}}{X_{ND}} \times 100\% \tag{5}$$

where $X_{NS}$ is the value of the parameter from the NSP and $X_{ND}$ is the corresponding parameter from the NDP (nonlinear time-history).

As shown in Table 2, The NSP overestimates the target displacement by 14.1% for the Design Level and 37% for the MCE Level. As for the base shear, the difference is small in general since most of the base shear is developed in the elastic range. The difference is 3.7% for the Design Level and 9.4% for the MCE Level. Rotation of the plastic hinges is an important factor to define the performance level. The NSP overestimates the rotation of plastic hinges by 16% and 45% for the Design and MCE Levels, respectively.

### TRANSVERSE DIRECTION

Period of the first mode in this direction is 0.52764 seconds and the modal participation mass ratio for this mode is 87.34% (Table 1). Pushover curve for this direction is shown in Fig. 10. The same formula shown in Eq. 4 is implemented to estimate the target displacement. The estimated target displacement is 65.5 mm (0.215 ft) for the Design Level and 99.1 mm (0.325 ft) for the MCE Level.

Table 3 compares different parameters resulting from NSP with the corresponding parameters from the NDP. In the NSP, the three load patterns mentioned previously are implemented. As shown in Table 3, The NSP overestimates the target displacement by 3.9% for the Design Level and 21.1% for the MCE Level. As for the base shear, the Modal and Spectral Patterns give results close to the NDP results for the Design Level while these two patterns overestimate the base shear by 7.7% for the MCE Level. The Uniform Pattern, which intends to give conservative results for the base shear, overestimates the base shear by 13.8% for the Design Level and 24.5% for the MCE Level. As for the rotation of the plastic hinges, the NSP overestimates it by 11.3%, 15.1%, and 11.6% for the Modal, Uniform, and Spectral Patterns, respectively, in the Design Level. For the MCE Level, the NSP overestimates the rotation of plastic hinges by 39.9, 42.8, and 39.9% for the Modal, Uniform, and Spectral Patterns, respectively.

## EVALUATION OF PERFORMANCE LEVEL

Using acceptance criteria provided by FEMA-273 (1997) to evaluate performance level of this bridge, rotation of plastic hinges should not exceed the following values for the corresponding performance levels: 0.005 for immediate occupancy, 0.01 for life safety, and 0.017 for collapse prevention. In the longitudinal direction, the bridge satisfies life safety performance level for Design Level and collapse prevention performance level for Maximum Considered Earthquake (MCE) Level (Table 2). In the transverse direction, this bridge satisfies the immediate occupation performance level for Design Level and life safety for MCE Level (Table 3).

## SUMMARY AND CONCLUSIONS

Applicability of the Nonlinear Static Procedure (NSP) to bridges is investigated in this study using the Displacement Coefficient Method (DCM), which was presented by FEMA-273 (1997). A three-span bridge was presented and described as a case study. Comparison of results obtained from the Nonlinear Dynamic Procedure (NDP or nonlinear time-history), which is considered the most reliable method for nonlinear analysis, with the results of the NSP by implementing the DCM was performed to evaluate the validity of the latter procedure.

Target displacement for each case was estimated by using the DCM and implementing the three load patterns considered in this study. In the longitudinal direction, DCM gives conservative results for all the cases. It is clear that the difference in the target displacement increases as the structure is driven further into the inelastic range. This observation agrees with the conclusion made by Chopra et. al. (2001), that the Single-Degree-of-Freedom (SDF) estimate of roof displacement, which is used by FEMA-273 (1997), is biased and increases with the increase of the overall ductility. In the transverse direction, estimation of the target displacement by the DCM gives results close to the nonlinear time-history results for the Design Level. For the Maximum Considered Earthquake (MCE) Level, DCM overestimates the target displacement since the structure is driven further into the inelastic range. Rotation of plastic hinges is compatible with the target displacement in most of the cases. When the target displacement is overestimated, the rotation of plastic hinges is also overestimated but with a slightly higher difference. A similar trend was observed when the target displacement is underestimated.

As a result of the work that was completed in this study, the following conclusions were made:

1. Conservative results are obtained from the DCM in the longitudinal direction of the bridge and those results become to be over-conservative as the structure is driven farther into the inelastic range.
2. Reasonable results are obtained from the DCM in the transverse direction and those results become more conservative as the structure is driven further into the inelastic range.
3. DCM is applicable for bridges, in general. However, it inherits the same shortcoming associated with this method when it is implemented for buildings, which is the overestimation of target displacement (Chopra et. al., 2001).

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Table 1- Modal Participating Mass Ratios

M O D A L P A R T I C I P A T I N G M A S S R A T I O S

| MODE | PERIOD | INDIVIDUAL MODE (PERCENT) | | | CUMULATIVE SUM (PERCENT) | | |
|---|---|---|---|---|---|---|---|
| | | UX | UY | UZ | UX | UY | UZ |
| 1 | 0.974452 | 99.9455 | 0.0474 | 0.0000 | 99.9455 | 0.0474 | 0.000 |
| 2 | 0.527637 | 0.0544 | 87.3390 | 0.0000 | 99.9999 | 87.3864 | 0.000 |
| 3 | 0.210907 | 0.0000 | 0.0000 | 0.0000 | 99.9999 | 87.3864 | 0.000 |
| 4 | 0.125119 | 0.0000 | 12.3011 | 0.0000 | 99.9999 | 99.6874 | 0.000 |
| 5 | 0.081540 | 0.0000 | 0.0000 | 0.0000 | 99.9999 | 99.6874 | 0.000 |
| 6 | 0.068766 | 0.0000 | 0.0000 | 0.0000 | 99.9999 | 99.6874 | 0.000 |
| 7 | 0.048480 | 0.0000 | 0.3016 | 0.0000 | 99.9999 | 99.9890 | 0.000 |
| 8 | 0.034273 | 0.0000 | 0.0000 | 0.0000 | 100.0000 | 99.9890 | 0.000 |
| 9 | 0.030667 | 0.0000 | 0.0000 | 0.0000 | 100.0000 | 99.9890 | 0.000 |
| 10 | 0.024274 | 0.0000 | 0.0000 | 0.0000 | 100.0000 | 99.9890 | 0.000 |
| 11 | 0.022045 | 0.0000 | 0.0106 | 0.0000 | 100.0000 | 99.9996 | 0.000 |
| 12 | 0.018333 | 0.0000 | 0.0000 | 0.0000 | 100.0000 | 99.9996 | 0.000 |

Table 2 - Comparison of different parameters between NSP and NDP in the longitudinal direction

| Seismic Intensity | Analysis method | | | Displacement (mm) | Base shear (kN) | Rotation of plastic hinge in rad. (Element No.) 115 | 215 | 315 | 415 |
|---|---|---|---|---|---|---|---|---|---|
| Design Level | **NDP (Time-History)** (average) | | | **86.3** | **4742** | **0.00829** | **0.00828** | **0.00825** | **0.00823** |
| | NSP (DCM) $\delta_t$= 98.5 mm | M-Pattern | Value | 98.5 | 4915 | 0.00967 | 0.00962 | 0.00963 | 0.00955 |
| | | | Diff.(%) | 14.1 | 3.7 | 16.6 | 16.2 | 16.7 | 16.0 |
| | | U-Pattern | Value | 98.5 | 4915 | 0.00967 | 0.00962 | 0.00963 | 0.00955 |
| | | | Diff.(%) | 14.1 | 3.7 | 16.6 | 16.2 | 16.7 | 16.0 |
| MCE Level | **NDP (Time-History)** (average) | | | **107.9** | **4991** | **0.01169** | **0.01168** | **0.01163** | **0.0116** |
| | NSP (DCM) $\delta_t$= 147.8 mm | M-Pattern | Value | 147.8 | 5462 | 0.01695 | 0.01689 | 0.01686 | 0.01677 |
| | | | Diff.(%) | 37.0 | 9.4 | 45.0 | 44.6 | 45.0 | 44.6 |
| | | U-Pattern | Value | 147.8 | 5462 | 0.01695 | 0.01689 | 0.01686 | 0.01677 |
| | | | Diff.(%) | 37.0 | 9.4 | 45.0 | 44.6 | 45.0 | 44.6 |

Table 3 - Comparison of different parameters between NSP and NDP in the transverse direction

| Seismic Intensity | Analysis method | | | Displacement (mm) | Base shear (kN) | Rotation of plastic hinge in rad. (Element No.) 115 & 415 | 215 & 315 |
|---|---|---|---|---|---|---|---|
| **Design Level** | **NDP (Time-History)** (average) | | | **63.1** | **13166** | **0.00288** | **0.00341** |
| | NSP (DCM) $\delta_t$= 65.5 mm | M-Pattern | Value | 65.5 | 12953 | 0.00321 | 0.00381 |
| | | | Diff.(%) | 3.9 | -1.6 | 11.3 | 11.6 |
| | | U-Pattern | Value | 65.5 | 14985 | 0.00332 | 0.00388 |
| | | | Diff.(%) | 3.9 | 13.8 | 15.1 | 13.6 |
| | | S-Pattern | Value | 65.5 | 13113 | 0.00322 | 0.00381 |
| | | | Diff.(%) | 3.9 | -0.4 | 11.6 | 11.6 |
| **MCE Level** | **NDP (Time-History)** (average) | | | **81.7** | **16234** | **0.00516** | **0.00594** |
| | NSP (DCM) $\delta_t$= 99.1 mm | M-Pattern | Value | 99.1 | 17476 | 0.00722 | 0.00813 |
| | | | Diff.(%) | 21.2 | 7.7 | 39.9 | 36.8 |
| | | U-Pattern | Value | 99.1 | 20207 | 0.00737 | 0.00822 |
| | | | Diff.(%) | 21.2 | 24.5 | 42.8 | 38.3 |
| | | S-Pattern | Value | 99.1 | 17485 | 0.00722 | 0.00813 |
| | | | Diff.(%) | 21.2 | 7.7 | 39.9 | 36.8 |

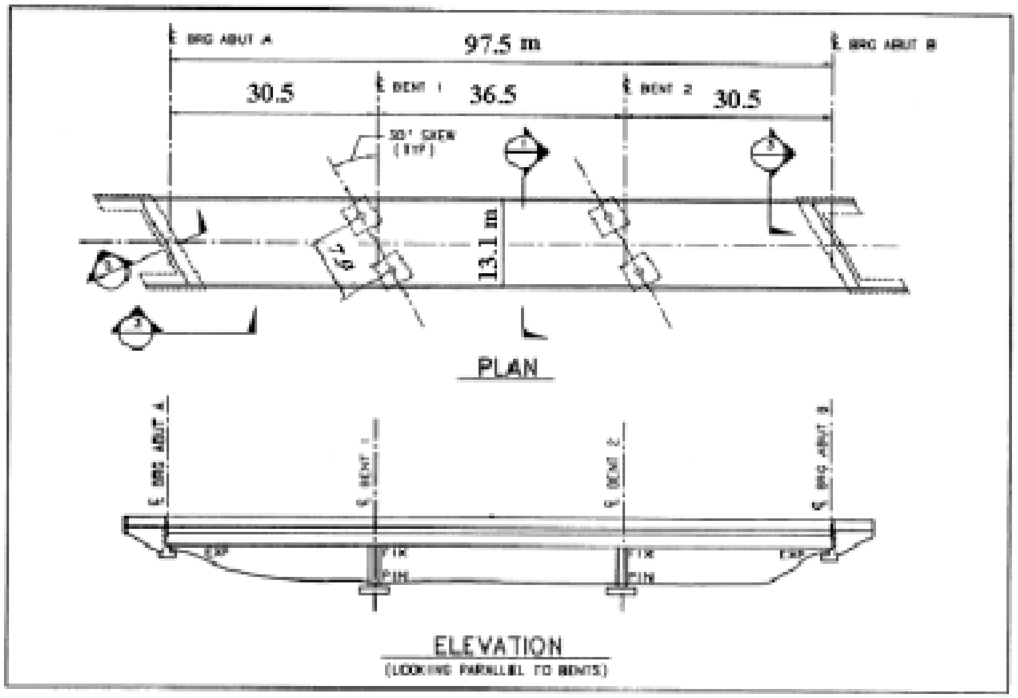


Fig. 1 - Plan and elevation of the bridge

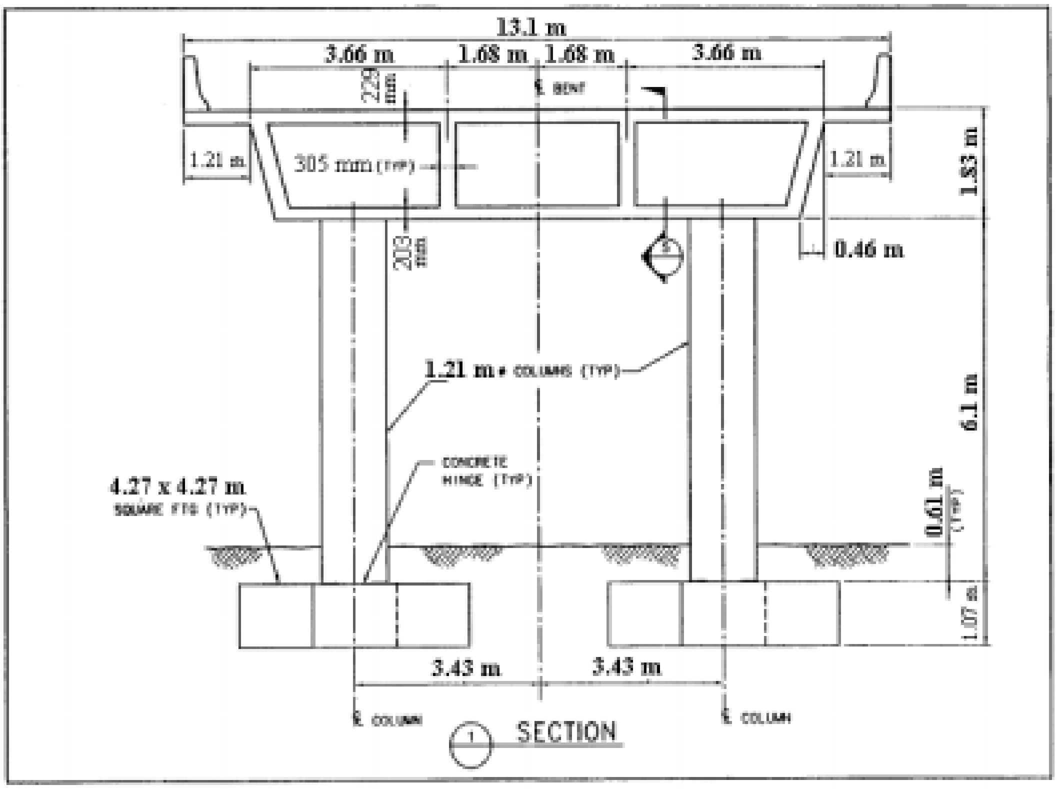


Fig. 2 - Typical cross section of the bridge

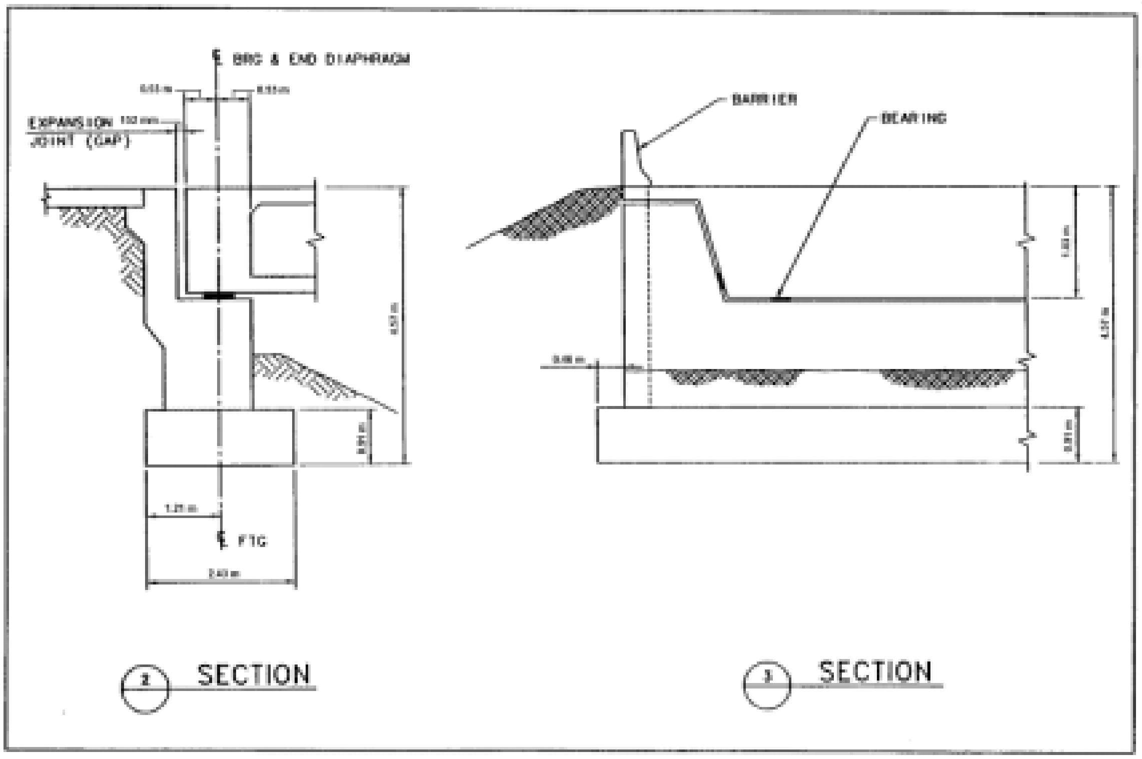


Fig. 3 - Seat type abutment

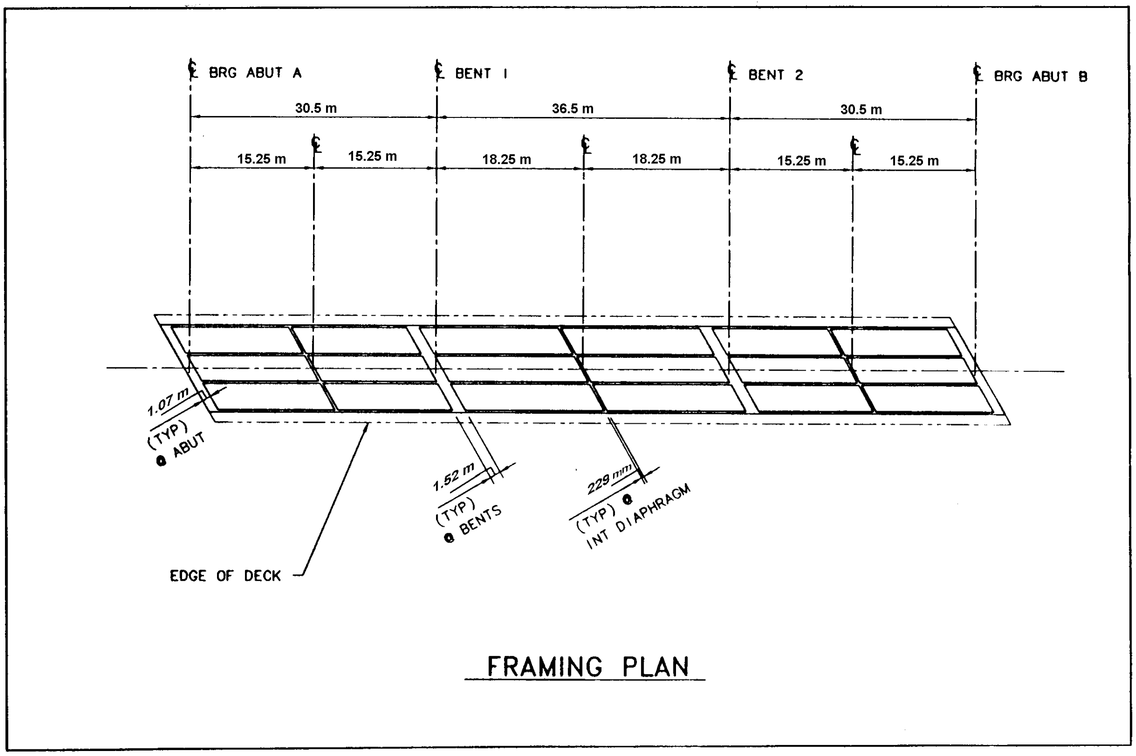


Fig. 4 - Box girder framing plan

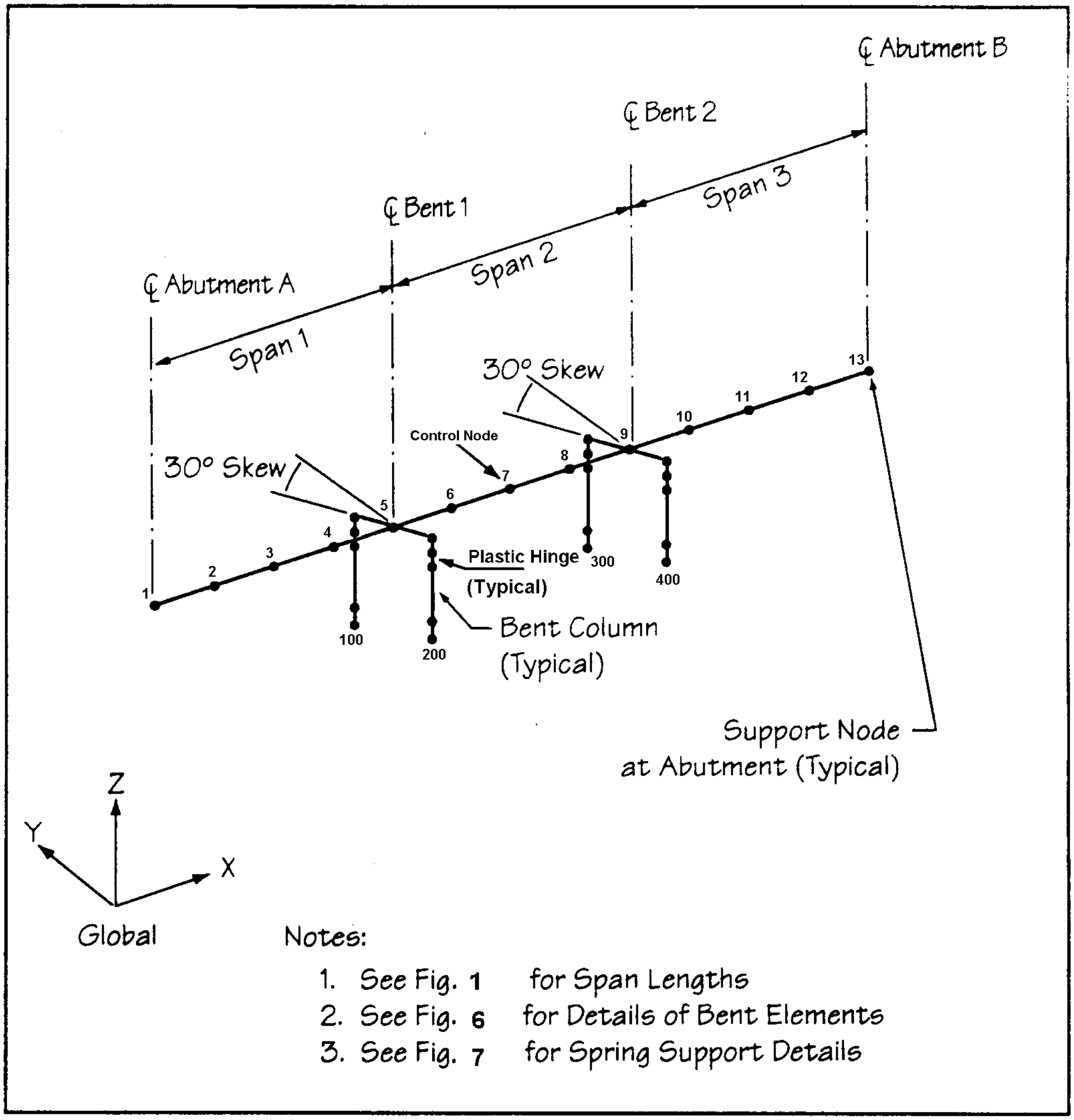


Fig. 5 - Structural model of the bridge

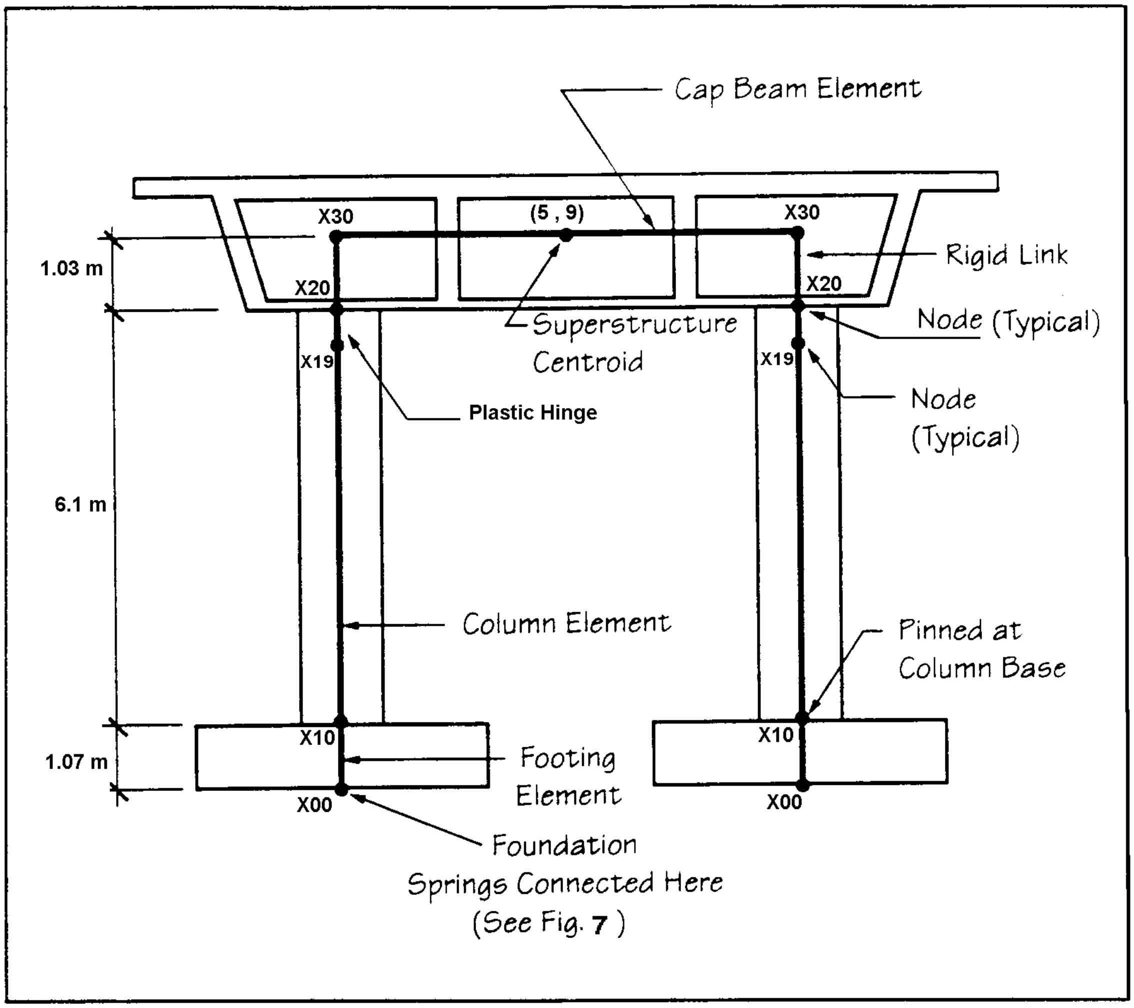


Fig. 6 - Details of bent elements

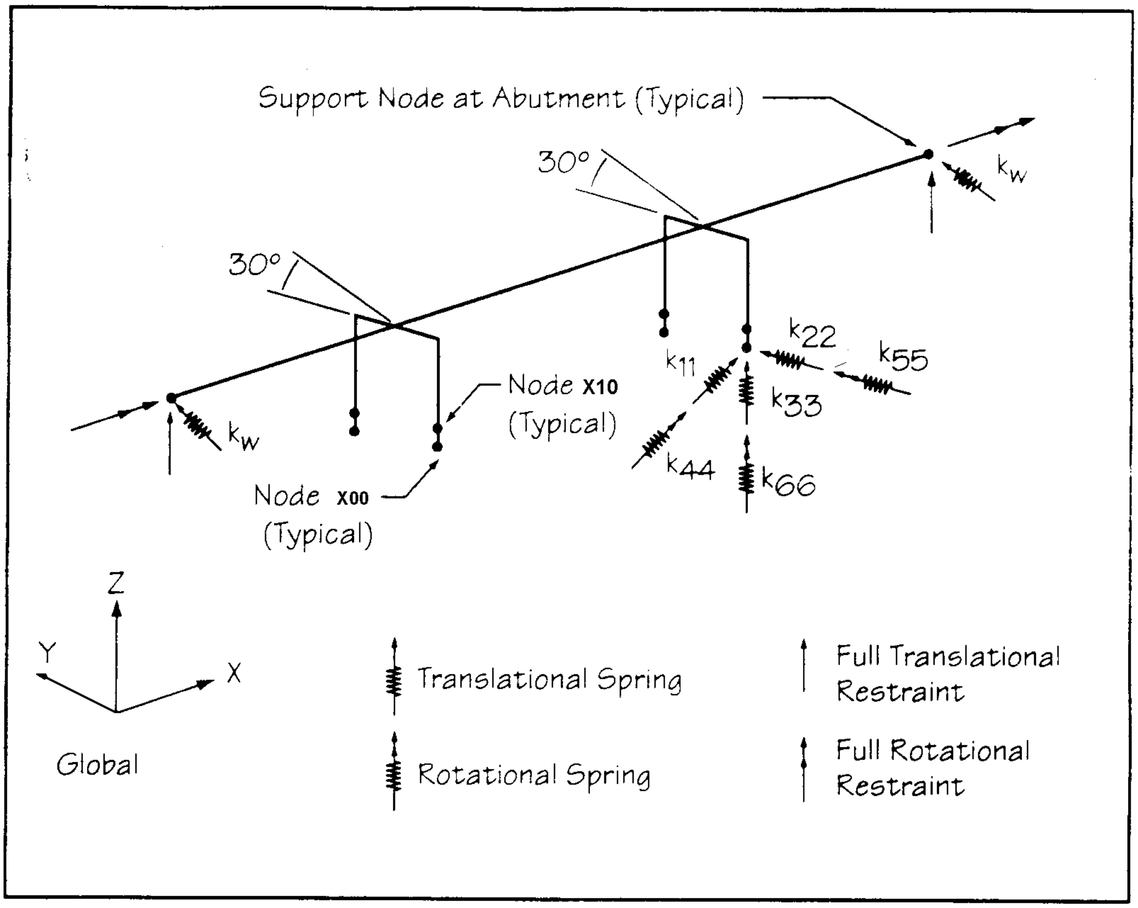


Fig. 7 - Details of spring supports

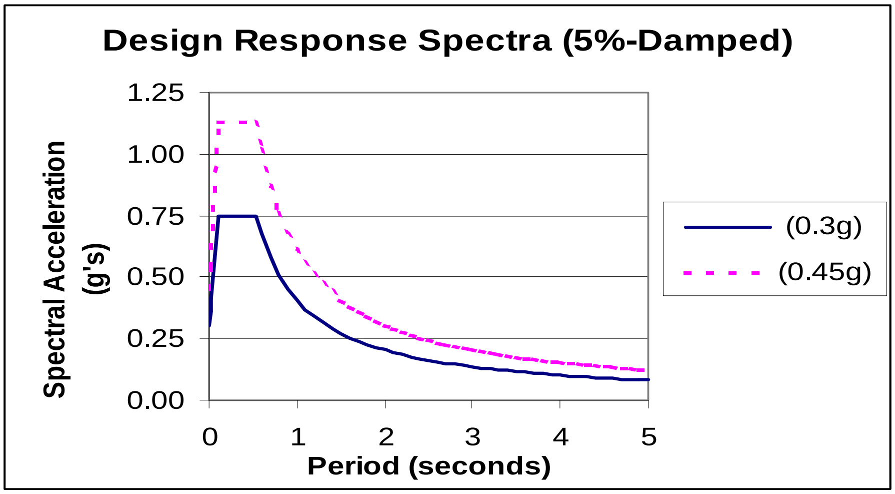


Fig. 8 - 5% damped response spectrum

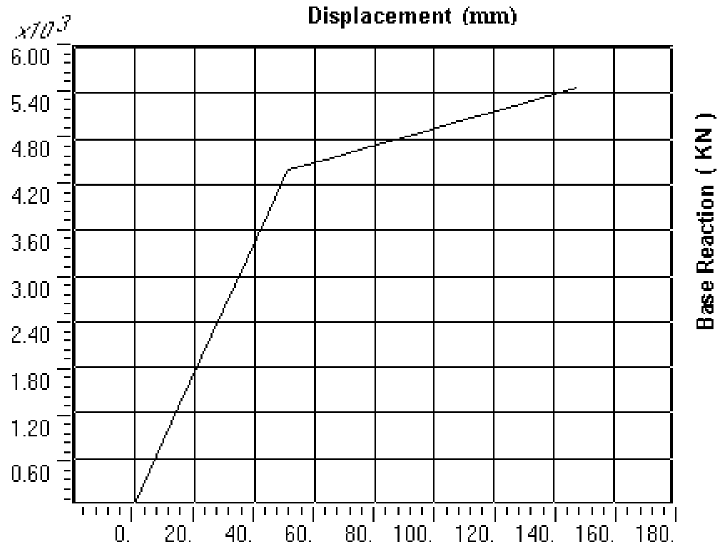


Fig. 9 - Pushover curve for longitudinal direction

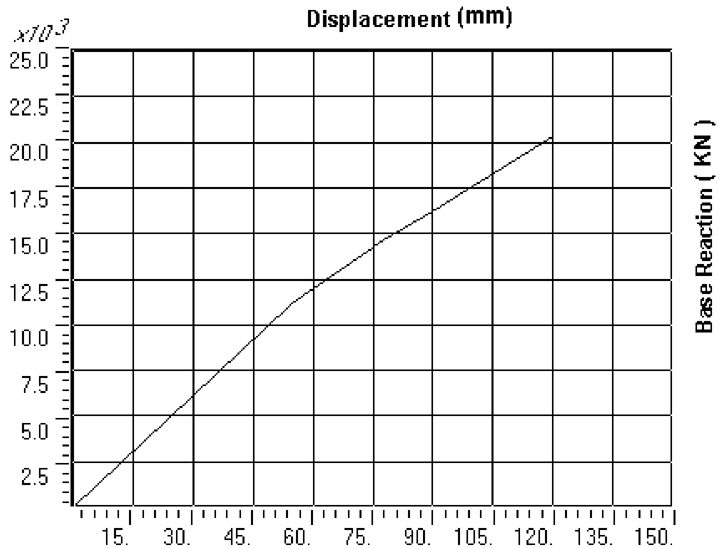


Fig. 10 - Pushover curve for transverse direction