# Contents

# Chapter 4 Seismic Design and Retrofit

## 4.1 General

Seismic design of new bridges and bridge widenings shall conform to AASHTO *Guide Specifications for LRFD Seismic Bridge Design* as modified by Sections 4.2 and 4.3. Analysis and design of seismic retrofits for existing bridges shall be completed in accordance with Section 4.4. Seismic design of retaining walls shall be in accordance with Section 4.5. For nonconventional bridges, bridges that are deemed critical or essential, or bridges that fall outside the scope of the Guide Specifications for any other reasons, project specific design requirements shall be developed and submitted to the WSDOT Bridge Design Engineer for approval.

The importance classifications for all highway bridges in Washington State are classified as “Normal” except for special major bridges. Special major bridges fitting the classifications of either “Critical” or “Essential” will be so designated by either the WSDOT Bridge and Structures Engineer or the WSDOT Bridge Design Engineer.

The performance objective for “normal” bridges is life safety. Bridges designed in accordance with AASHTO Guide Specifications are intended to achieve the life safety performance goals.

## 4.2 WSDOT Additions and Modifications to AASHTO Guide Specifications for LRFD Seismic Bridge Design

WSDOT amendments to the AASHTO *Guide Specifications for LRFD Seismic Bridge Design* are as follows:

### 4.2.1 Definitions

**Guide Specifications Article 2.1 –** Add the following definitions:

- **Oversized Pile Shaft –** A drilled shaft foundation that is larger in diameter than the supported column and has a reinforcing cage larger than and independent of the columns. The size of the shaft shall be in accordance with Section 7.8.2.
- **Owner –** Person or agency having jurisdiction over the bridge. For WSDOT projects, regardless of delivery method, the term "Owner" in these Guide Specifications shall be the WSDOT Bridge Design Engineer or/and the WSDOT Geotechnical Engineer.

### 4.2.2 Earthquake Resisting Systems (ERS) Requirements for Seismic Design Catagories (SDCs) C and D

**Guide Specifications Article 3.3 –** WSDOT Global Seismic Design Strategies:

- **Type 1 –** Ductile Substructure with Essentially Elastic Superstructure. This category is permissible.
- **Type 2 –** Essentially Elastic Substructure with a Ductile Superstructure. This category is not permissible.
- **Type 3 –** Elastic Superstructure and Substructure with a Fusing Mechanism Between the Two. This category is permissible with WSDOT Bridge Design Engineer's approval.

With the approval of the Bridge Design Engineer, for Type 1 ERS for SDC C or D, if columns or pier walls are considered an integral part of the energy dissipating system but remain elastic at the demand displacement, the forces to use for capacity design of other components are to be a minimum of 1.2 times the elastic forces resulting

from the demand displacement in lieu of the forces obtained from overstrength plastic hinging analysis. Because maximum limiting inertial forces provided by yielding elements acting at a plastic mechanism level is not effective in the case of elastic design, the following constraints are imposed. These may be relaxed on a case by case basis with the approval of the Bridge Design Engineer.

1. Unless an analysis that considers redistribution of internal structure forces due to inelastic action is performed, all substructure units of the frame under consideration and of any adjacent frames that may transfer inertial forces to the frame in question must remain elastic at the design ground motion demand.
2. Effective member section properties must be consistent with the force levels expected within the bridge system. Reinforced concrete columns and pier walls should be analyzed using cracked section properties. For this purpose, in absence of better information or estimated by Figure 5.6.2-1, a moment of inertia equal to one half that of the un-cracked section shall be used.

3. Foundation modeling must be established such that uncertainties in modeling will not cause the internal forces of any elements under consideration to increase by more than 10 percent.
4. When site specific ground response analysis is performed, the response spectrum ordinates must be selected such that uncertainties will not cause the internal forces of any elements under consideration to increase by more than 10 percent.
5. Thermal, shrinkage, prestress or other forces that may be present in the structure at the time of an earthquake must be considered to act in a sense that is least favorable to the seismic load combination under investigation.
6. P-Delta effects must be assessed using the resistance of the frame in question at the deflection caused by the design ground motion.
7. Joint shear effects must be assessed with a minimum of the calculated elastic internal forces applied to the joint.
8. Detailing as normally required in either SDC C or D, as appropriate, must be provided.

It is permitted to use expected material strengths for the determination of member strengths for elastic response of members.

The use of elastic design in lieu of overstrength plastic hinging forces for capacity protection described above shall only be considered if designer demonstrates that capacity design of Article 4.11 of the AASHTO Guide Specifications for LRFD Bridge Seismic Design is not feasible due to geotechnical or structural reasons.

Type 3 ERS may be considered only if Type 1 strategy is not suitable and Type 3 strategy has been deemed necessary for accommodating seismic loads. Use of isolation bearings needs the approval of WSDOT Bridge Design Engineer. Isolation bearings shall be designed per the requirement specified in Section 9.3

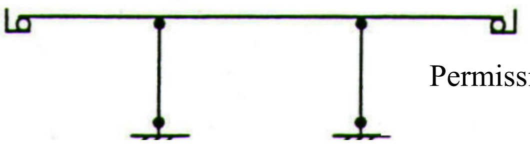
If the columns or pier walls are designed for elastic forces, all other elements shall be designed for the lesser of the forces resulting from the overstrength plastic hinging moment capacity of columns or pier walls and the unreduced elastic seismic force in all SDCs. The minimum detailing according to the bridge seismic design category shall be provided. Shear design shall be based on 1.2 times elastic shear force and nominal material strengths shall be used for capacities. Limitations on the use of ERS and ERE are shown in Figures 3.3-1a, 3.3-1b, 3.3-2, and 3.3-3.

- Figure 3.3-1b Type 6, connection with moment reducing detail should only be used at column base if proved necessary for foundation design. Fixed connection at base of column remains the preferred option for WSDOT bridges.
- The design criteria for column base with moment reducing detail shall consider all applicable loads at service, strength, and extreme event limit states.
- Figure 3.3-2 Types 6 and 8 are not permissible for non-liquefied configuration and permissible with WSDOT Bridge Design Engineer's approval for liquefied configuration

For ERSs and EREs requiring approval, the WSDOT Bridge Design Engineer's approval is required regardless of contracting method (i.e., approval authority is not transferred to other entities).

**1**

**Longitudinal Response**

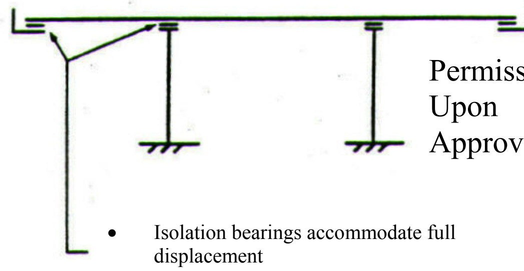


Permissible

- Plastic hinges in inspectable locations or elastic design of columns.
- Abutment resistance not required as part of ERS
- Knock-off backwalls permissible

**2**

**Longitudinal Response**

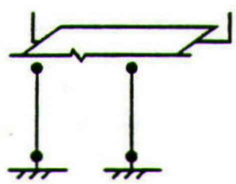


Permissible Upon Approval

- Isolation bearings accommodate full displacement
- Abutment not required as part of ERS

**3**

**Transverse Response**

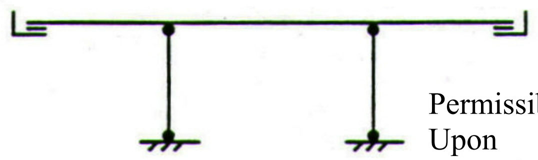


Permissible

- Plastic hinges in inspectable locations.
- Abutment not required in ERS, breakaway shear keys permissible with WSDOT Bridge Design Engineer's Approval

**4**

**Transverse or Longitudinal Response**



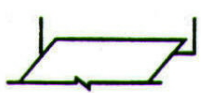
Permissible Upon Approval

- Plastic hinges in inspectable locations
- Isolation bearings with or without energy dissipaters to limit overall displacements

**5**

**Transverse or Longitudinal Response**

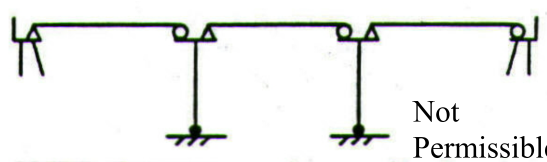
Permissible



- Abutment required to resist the design earthquake elastically
- Longitudinal passive soil pressure shall be less than 0.70 of the value obtained using the procedure given in BDM Article 4.2.11

**6**

**Longitudinal Response**



Not Permissible

- Multiple simply-supported spans with adequate support lengths
- Plastic hinges in inspectable locations or elastic design of columns

**Figure 3.3-1a Permissible Earthquake-Resisting Systems (ERSs)**
***BDM Figure 4.2.2-1***

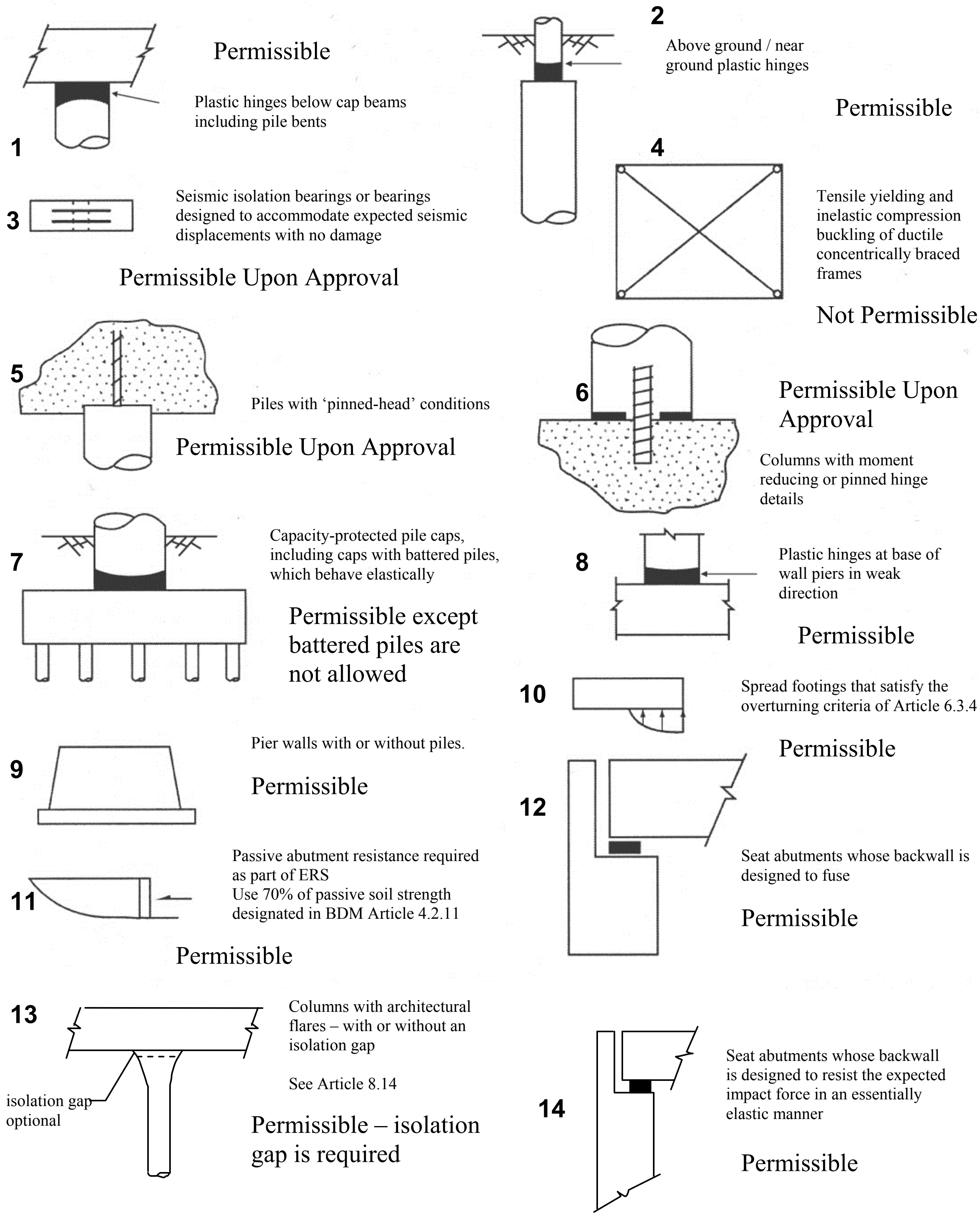



**Figure 3.3-1b Permissible Earthquake-Resisting Elements (EREs)**
***BDM Figure 4.2.2-2***

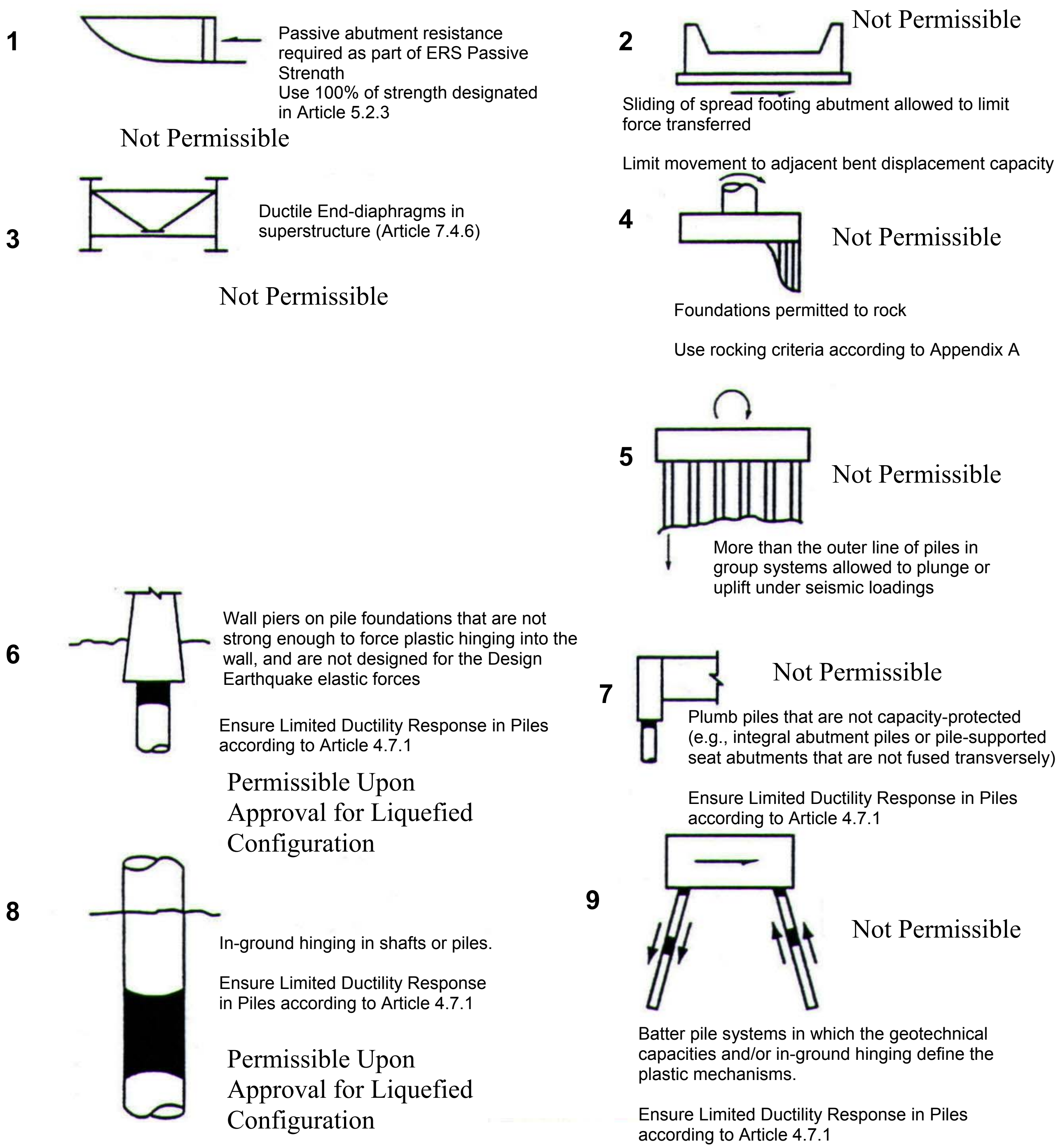



**Figure 3.3-2 Permissible Earthquake-Resisting Elements That Require Owner's Approval**
***BDM Figure 4.2.2-3***

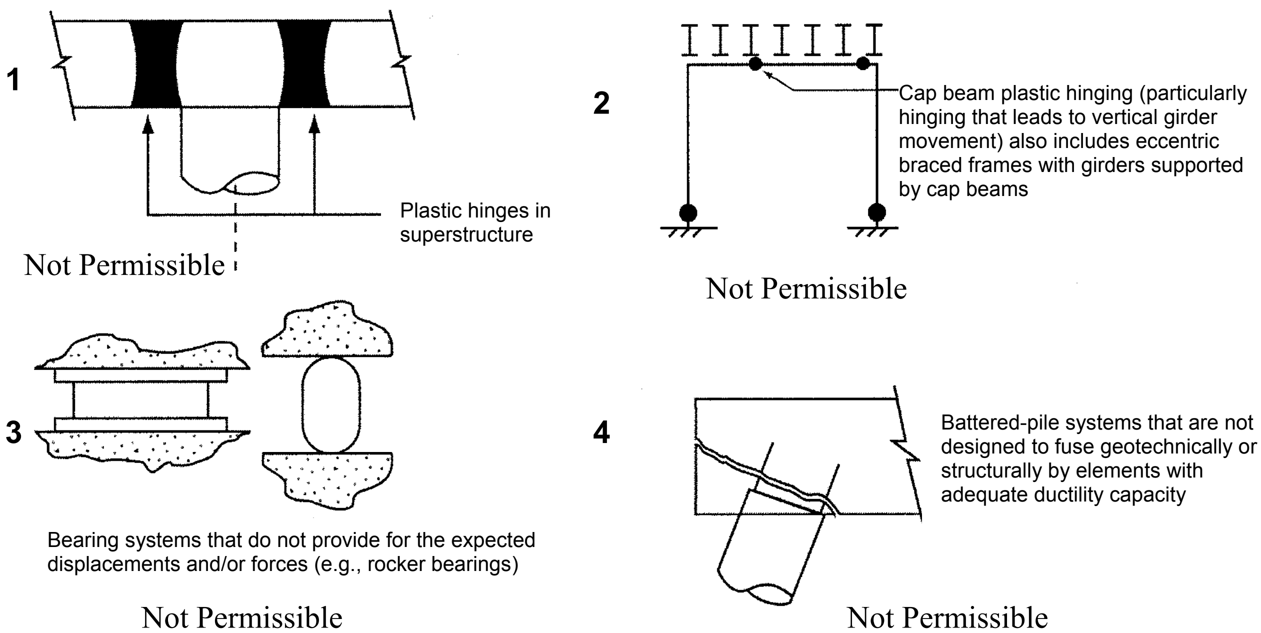



**Figure 3.3-3 Earthquake-Resisting Elements that Are Not Recommended for New Bridges**
***BDM Figure 4.2.2-4***

### *4.2.3 Seismic Ground Shaking Hazard*

**Guide Specifications Article 3.4** – For bridges that are considered critical or essential or normal bridges with a site Class F, the seismic ground shaking hazard shall be determined based on the WSDOT Geotechnical Engineer recommendations.

In cases where the site coefficients used to adjust mapped values of design ground motion for local conditions are inappropriate to determine the design spectra in accordance with general procedure of Article 3.4.1 (such as the period at the end of constant design spectral acceleration plateau ($T_s$) is greater than 1.0 second or the period at the beginning of constant design spectral acceleration plateau ($T_o$) is less than 0.2 second), a site-specific ground motion response analysis shall be performed.

### *4.2.4 Selection of Seismic Design Category (SDC)*

**Guide Specifications Article 3.5** – Pushover analysis shall be used to determine displacement capacity for both SDCs C and D.

### *4.2.5 Temporary and Staged Construction*

**Guide Specifications Article 3.6** – For bridges that are designed for a reduced seismic demand, the contract plans shall either include a statement that clearly indicatesthat the bridge was designed as temporary using a reduced seismic demand or show the Acceleration Response Spectrum (ARS) used for design.

### 4.2.6 Load and Resistance Factors

**Guide Specifications Article 3.7** – Revise as follows:

Use load factors of 1.0 for all permanent loads. The load factor for live load shall be 0.0 when pushover analysis is used to determine the displacement capacity. Use live load factor of 0.5 for all other extreme event cases. Unless otherwise noted, all $\phi$ factors shall be taken as 1.0.

### 4.2.7 Balanced Stiffness Requirements and Balanced Frame Geometry Recommendation

**Guide Specifications Articles 4.1.2 and 4.1.3** – Balanced stiffness between bents within a frame and between columns within a bent and balanced frame geometry for adjacent frames are required for bridges in both SDCs C and D. Deviations from balanced stiffness and balanced frame geometry requirements require approval from the WSDOT Bridge Design Engineer.

### 4.2.8 Selection of Analysis Procedure to Determine Seismic Demand

**Guide Specifications Article 4.2** – Analysis Procedures:

- Procedure 1 (Equivalent Static Analysis) shall not be used.
- Procedure 2 (Elastic Dynamic Analysis) shall be used for all “regular” bridges with two through six spans and “not regular” bridges with two or more spans in SDCs B, C, or D.
- Procedure 3 (Nonlinear Time History) shall only be used with WSDOT Bridge Design Engineer’s approval.

### 4.2.9 Member Ductility Requirement for SDCs C and D

**Guide Specifications Article 4.9** – In-ground hinging for drilled shaft and pile foundations may be considered for the liquefied configuration with WSDOT Bridge Design Engineer approval.

### 4.2.10 Longitudinal Restrainers

**Guide Specifications Article 4.13.1 –** Longitudinal restrainers shall be provided at the expansion joints between superstructure segments. Restrainers shall be designed in accordance with the FHWA Seismic Retrofitting Manual for Highway Structure (FHWA-HRT-06-032) Article 8.4 The Iterative Method. See the earthquake restrainer design example in the Appendix of this chapter. Restrainers shall be detailed in accordance with the requirements of Guide Specifications Article 4.13.3 and Section 4.4.5. Restrainers may be omitted for SDCs C and D where the available seat width exceeds the calculated support length specified in Equation C4.13.1-1.

Omitting restrainers for liquefiable sites shall be approved by the WSDOT Bridge Design Engineer.

Longitudinal restrainers shall not be used at the end piers (abutments).

### 4.2.11 Abutments

**Guide Specifications Article 5.2** – Diaphragm Abutment type shown in Figure 5.2.3.2-1 shall not be used for WSDOT bridges.

**Guide Specifications Article 5.2** – Abutments to be revised as follows:

**5.2.1 - General**

The participation of abutment walls in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges either to reduce column sizes or reduce the ductility demand on the columns. Damage to backwalls and wingwalls during earthquakes may be considered acceptable when considering no collapse criteria, provided that unseating or other damage to the superstructure does not occur. Abutment participation in the overall dynamic response of the bridge system shall reflect the structural configuration, the load transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of acceptable abutment damage. The capacity of the abutments to resist the bridge inertial loads shall be compatible with the soil resistance that can be reliably mobilized, the structural design of the abutment wall, and whether the wall is permitted to be damaged by the design earthquake. The lateral load capacity of walls shall be evaluated on the basis of a rational passive earth-pressure theory.

The participation of the bridge approach slab in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads may be considered permissible upon approval from both the WSDOT Bridge Design Engineer and the WSDOT Geotechnical Engineer.

The participation of the abutment in the ERS should be carefully evaluated with the Geotechnical Engineer and the Owner when the presence of the abutment backfill may be uncertain, as in the case of slumping or settlement due to liquefaction below or near the abutment.

**5.2.2 - Longitudinal Direction**

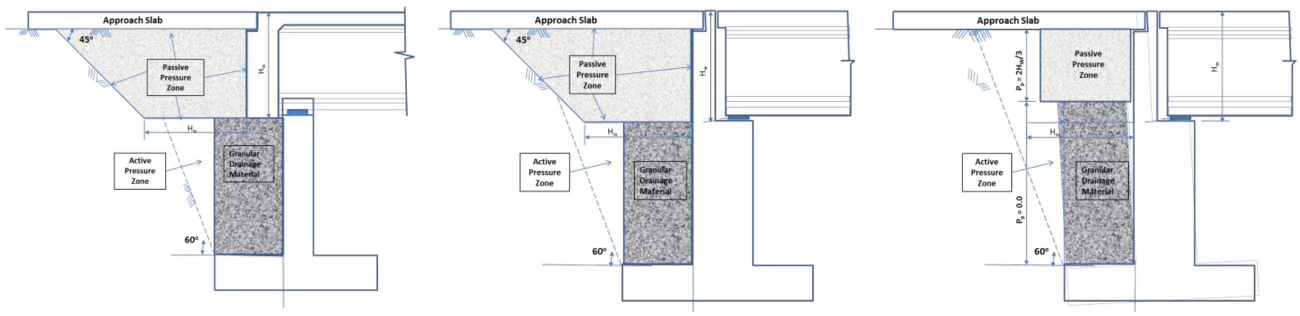
Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of two possible conditions:

- The dynamic active pressure condition as the wall moves away from the backfill, or
- The passive pressure condition as the inertial load of the bridge pushes the wall into the backfill.

The governing earth pressure condition depends on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration.

For semi-integral (Figure 5.2.2-a), L-shape abutment with backwall fuse

(Figure 5.2.2-b), or without backwall fuse (Figure 5.2.2-c), for which the expansion joint is sufficiently large to accommodate both the cyclic movement between the abutment wall and the bridge superstructure (i.e., superstructure does not push against abutment wall), the seismically induced earth pressure on the abutment wall shall be considered to be the dynamic active pressure condition. However, when the gap at the expansion joint is not sufficient to accommodate the cyclic wall/bridge seismic movements, a transfer of forces will occur from the superstructure to the abutment wall. As a result, the active earth pressure condition will not be valid and the earth pressure approaches a much larger passive pressure load condition behind the backwall. This larger load condition is the main cause for abutment damage, as demonstrated in past earthquakes. For semi-integral or L-shape abutments, the abutment stiffness and capacity under passive pressure loading are primary design concerns.




(a) Semi-integral Abutment

(b) L-shape Abutment Backwall Fuses

(c) L-shape Abutment Backwall Does Not Fuse

**Figure 5.2.2 - Abutment Stiffness and Passive Pressure Estimate**

***Figure 4.2.11-1***

Where the passive pressure resistance of soils behind semi-integral or L-shape abutments will be mobilized through large longitudinal superstructure displacements, the bridge may be designed with the abutments as key elements of the longitudinal ERS. Abutments shall be designed to sustain the design earthquake displacements. When abutment stiffness and capacity are included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally used for static service load design. This is illustrated schematically in Figures 4.2.11-1a and 4.2.11-1b. Dynamic active earth pressure acting on the abutment need not be considered in the dynamic analysis of the bridge. The passive abutment resistance shall be limited to 70 percent of the value obtained using the procedure given in Article 5.2.2.1.

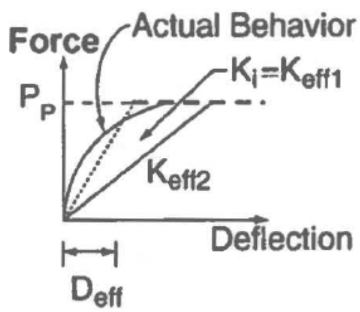
### 5.2.2.1 - Abutment Stiffness and Passive Pressure Estimate

Abutment stiffness, $K_{eff}$ in kip/ft, and passive capacity, $P_p$ in kips, should be characterized by a bilinear or other higher order nonlinear relationship as shown in Figure 5.2.2.1. When the motion of the back wall is primarily translation, passive pressures may be assumed uniformly distributed over the height ($H_w$) of the backwall or end diaphragm. The total passive force may be determined as:

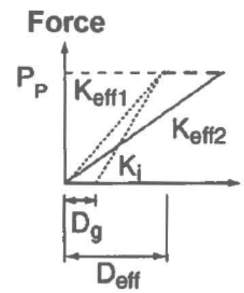
$$P_p = {}_{pp} H_w W_w \qquad (5.2.2.1\text{-}1)$$

Where:

$p_p$ = passive lateral earth pressure behind backwall or diaphragm (ksf)
$H_w$ = height of back wall or end diaphragm exposed to passive earth pressure (feet)
$W_w$ = width of back wall or diaphragm (feet)




(a) Semi-integral Abutment




(b) L-shape Abutment

**Figure 5.2.2.1 - Characterization of Abutment Capacity and Stiffness**

***Figure 4.2.11-2***

### 5.2.2.2 - Calculation of Best Estimate Passive Pressure $P_p$

If the strength characteristics of compacted or natural soils in the "passive pressure zone" are known, then the passive force for a given height, Hw, may be calculated using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire "passive pressure zone" as indicated in Figure 1. Therefore, the properties of

backfill present immediately adjacent to the wall in the active pressure zone may not be appropriate as a weaker failure surface can develop elsewhere in the embankment.

For L-shape abutments where the backwall is not designed to fuse, Hw shall conservatively be taken as the depth of the superstructure, unless a more rational soil- structure interaction analysis is performed.

If presumptive passive pressures are to be used for design, then the following criteria shall apply:

- Soil in the "passive pressure zone" shall be compacted in accordance with *Standard Specifications* Section 2-03.3(14)I, which requires compaction to 95 percent maximum density for all “Bridge Approach Embankments”.
- For cohesionless, nonplastic backfill (fines content less than 30 percent), the passive pressure pp may be assumed equal to 2Hw/3 ksf per foot of wall length.

For other cases, including abutments constructed in cuts, the passive pressures shall be developed by a geotechnical engineer.

### 5.2.2.3 - Calculation of Passive Soil Stiffness

Equivalent linear secant stiffness, $K_{eff}$ in kip/ft, is required for analyses. For semi-integral or L-shape abutments initial secant stiffness may be determined as follows:

$$K_{eff1} = \frac{P_p}{(F_w H_w)} \quad (5.2.2.3\text{-}1)$$

Where:

$P_p$ = passive lateral earth pressure capacity (kip)
$H_w$ = height of back wall (feet)
$F_w$ = the value of Fw to use for a particular bridge may be found in Table C3.11.1-1 of the AASHTO LRFD *Bridge Design Specifications*.

For L-shape abutments, the expansion gap should be included in the initial estimate of the secant stiffness as specified in:

$$K_{eff1} = \frac{P_p}{\left(F_w H_w + D_g\right)} \tag{5.2.2.3-2}$$

Where:

$D_g$ = width of gap between backwall and superstructure (feet)

For SDCs C and D, where pushover analyses are conducted, values of $P_p$ and the initial estimate of $K_{eff1}$ should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

**5.2.2.4 - Modeling Passive Pressure Stiffness in the Longitudinal Direction**

In the longitudinal direction, when the bridge is moving toward the soil, the full passive resistance of the soil may be mobilized, but when the bridge moves away from the soil no soil resistance is mobilized. Since passive pressure acts at only one abutment at a time, linear elastic dynamic models and frame pushover models should only include a passive pressure spring at one abutment in any given model. Secant stiffness values for passive pressure shall be developed independently for each abutment.

As an alternative, for straight or with horizontal curves up to 30 degrees single frame bridges, and compression models in straight multi-frame bridges where the

passive pressure stiffness is similar between abutments, a spring may be used at each abutment concurrently. In this case, the assigned spring values at each end need to

be reduced by half because they act in simultaneously, whereas the actual backfill passive resistance acts only in one direction and at one time. Correspondingly, the actual peak passive resistance force at either abutment will be equal to the sum of the peak forces developed in two springs. In this case, secant stiffness values for passive pressure shall be developed based on the sum of peak forces developed in each spring. If computed abutment forces exceed the soil capacity, the stiffness should be softened iteratively until abutment displacements are consistent (within 30 percent) with the assumed stiffness.

**5.2.3 - Transverse Direction**

Transverse stiffness of abutments may be considered in the overall dynamic response of bridge systems on a case by case basis upon Bridge Design Engineer approval.

Upon approval, the transverse abutment stiffness used in the elastic demand models may be taken as 50 percent of the elastic transverse stiffness of the adjacent bent.

Girder stops are typically designed to transmit the lateral shear forces generated by small to moderate earthquakes and service loads and are expected to fuse at the design event earthquake level of acceleration to limit the demand and control the damage in the abutments and supporting piles/shafts. Linear elastic analysis cannot capture the inelastic response of the girder stops, wingwalls or piles/shafts. Therefore, the forces generated with elastic demand assessment models should not be used to size the abutment girder stops. Girder stops for abutments supported on a spread footing shall be designed to sustain the lesser of the acceleration coefficient, As, times the superstructure dead load reaction at the abutment plus the weight of abutment and its footing or sliding friction forces of spread footings. Girder stops for pile/shaft supported foundations shall be designed to sustain the sum of 75 percent total lateral capacity of the piles/shafts and shear capacity of one wingwall.

The elastic resistance may be taken to include the use of bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing supported abutment, or pile resistance provided by piles acting in their elastic range.

The stiffness of fusing or breakaway abutment elements such as wingwalls (yielding or non-yielding), elastomeric bearings, and sliding footings shall not be relied upon to reduce displacement demands at intermediate piers.

Unless fixed bearings are used, girder stops shall be provided between all girders regardless of the elastic seismic demand. The design of girder stops should consider that unequal forces that may develop in each stop.

When fusing girder stops, transverse shear keys, or other elements that potentially release the restraint of the superstructure are used, then adequate support length meeting the requirements of Article 4.12 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design must be provided. Additionally, the expected redistribution of internal forces in the superstructure and other bridge system element must be considered. Bounding analyses considering incremental release of transverse restraint at each end of the bridge should also be considered.

**5.2.4 - Curved and Skewed Bridges**

Passive earth pressure at abutments may be considered as a key element of the ERS of straight and curved bridges with abutment skews up to 20 degrees. For larger skews, due to a combination of longitudinal and transverse response, the span has a tendency to rotate in the direction of decreasing skew. Such motion will tend to cause binding in the obtuse corner and generate uneven passive earth pressure forces on the abutment, exceeding the passive pressure near one end of the backwall, and providing little or no resistance at other end. This requires a more refined analysis to determine the amount of expected movement. The passive pressure resistance in soils behind semi-integral or L-shape abutments shall be based on the projected width of the abutment wall normal to the centerline of the bridge. Abutment springs shall be included in the local coordinate system of the abutment wall.

### *4.2.12 Foundation – General*

**Guide Specifications Article 5.3.1** – The required foundation modeling method (FMM) and the requirements for estimation of foundation springs for spread footings, pile foundations, and drilled shafts shall be based on the WSDOT Geotechnical Engineer's recommendations.

### *4.2.13 Foundation – Spread Footing*

**Guide Specifications Article C5.3.2** – Foundation springs for spread footings shall be determined in accordance with Section 7.2.7, *Geotechnical Design Manual* Section 6.5.1.1 and the WSDOT Geotechnical Engineer's recommendations.

### 4.2.14 Procedure 3: Nonlinear Time History Method

**Guide Specifications Article 5.4.4** – The time histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the WSDOT Geotechnical Engineer and the WSDOT Bridge Design Engineer.

### 4.2.15 $I_{eff}$ for Box Girder Superstructure

**Guide Specifications Article 5.6.3** – Gross moment of inertia shall be used for box girder superstructure modeling.

### 4.2.16 Foundation Rocking

**Guide Specifications Article 6.3.9** – Foundation rocking shall not be used for the design of WSDOT bridges.

### 4.2.17 Drilled Shafts

**Guide Specifications Article C6.5** – For WSDOT bridges, the scale factor for p-y curves or subgrade modulus for large diameter shafts shall not be used unless approved by the WSDOT Geotechnical Engineer and WSDOT Bridge Design Engineer.

### 4.2.18 Longitudinal Direction Requirements

**Guide Specifications Article 6.7.1 –** Case 2: Earthquake Resisting System (ERS) with abutment contribution may be used provided that the mobilized longitudinal passive pressure is not greater than 70 percent of the value obtained using procedure given in Article 5.2.2.1.

### 4.2.19 Liquefaction Design Requirements

**Guide Specifications Article 6.8** – Soil liquefaction assessment shall be based on the WSDOT Geotechnical Engineer's recommendation and *Geotechnical Design Manual* Section 6.4.2.8.

### 4.2.20 Reinforcing Steel

**Guide Specifications Article 8.4.1** – Reinforcing bars, deformed wire, cold-draw wire, welded plain wire fabric and welded deformed wire fabric shall conform to the material standards as specified in AASHTO LRFD *Bridge Design Specifications*.

ASTM A 615 reinforcement shall not be used in WSDOT Bridges. Only ASTM A 706 Grade 60 reinforcing steel shall be used in members where plastic hinging is expected for SDCs B, C, and D. ASTM A 706 Grade 80 reinforcing steels may be used for capacity-protected members as specified in Article 8.9. ASTM A 706 Grade 80 reinforcing steel shall not be used for oversized shafts where in ground plastic hinging is considered as a part of ERS.

Deformed welded wire fabric may be used with the WSDOT Bridge Design Engineer's approval.

Wire rope or strands for spirals and high strength bars with yield strength in excess of 75 ksi shall not be used.

**Guide Specifications Article C8.4.1** – Add the following paragraph to Article C8.4.1.

The requirement for plastic hinging and capacity protected members do not apply to the structures in SDC A, therefore use of ASTM A706 Grade 80 reinforcing steel is permitted in SDC A.

For SDCs B, C, and D, the moment-curvature analyses based on strain compatibility and nonlinear stress strain relations are used to determine the plastic moment capacities of all ductile concrete members. Further research is required to establish the shape and model of the stress-strain curve, expected reinforcing strengths, strain limits, and the stress-strain relationships for concrete confined by lateral reinforcement made with ASTM A 706 Grade 80 reinforcing steel.

### *4.2.21 Concrete Modeling*

**Guide Specifications Article 8.4.4- Revise the last paragraph as follows:**

Where in-ground plastic hinging approved by the WSDOT Bridge Design Engineer is part of the ERS, the confined concrete core shall be limited to a maximum compressive strain of 0.008. The clear spacing between the longitudinal reinforcements and between spirals and hoops in drilled shafts shall not be less than 6 inches or more than 8 inches when tremie placement of concrete is anticipated.

### *4.2.22 Expected Nominal Moment Capacity*

**Guide Specifications Article 8.5** – Add the following paragraphs after third paragraph.

The expected nominal capacity of capacity protected member using ASTM A 706 Grade 80 reinforcement shall be determined by strength design based on the expected concrete strength and yield strength of 80 ksi when the concrete reaches 0.003 or the reinforcing steel strain reaches 0.090 for #10 bars and smaller, 0.060 for #11 bars and larger.

Replace the definition of $\lambda_{mo}$ with the following:

$\lambda_{mo}$ = overstrength factor
= 1.2 for ASTM A 706 Grade 60 reinforcement
= 1.4 for ASTM A 615 Grade 60 reinforcement

### *4.2.23 Interlocking Bar Size*

**Guide Specifications Article 8.6.7** – The longitudinal reinforcing bar inside the interlocking portion of column (interlocking bars) shall be the same size of bars used outside the interlocking portion.

### *4.2.24 Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDCs C and D*

**Guide Specifications Article 8.8.3** – The splicing of longitudinal column reinforcement outside the plastic hinging region shall be accomplished using mechanical couplers that are capable of developing the tensile strength of the spliced bar. Splices shall be staggered at least 2 ft. Lap splices shall not be used. The design engineer shall clearly identify the locations where splices in longitudinal column reinforcement are permitted on the plans. In general where the length of the rebar cage is less than 60 ft (72 ft for No. 14 and No. 18 bars), no splice in the longitudinal reinforcement shall be allowed.

### 4.2.25 Development Length for Column Bars Extended into Oversized Pile Shafts f or SDCs C and D

**Guide Specifications Article 8.8.10** – Extending column bars into oversized shaft shall be per Section 7.4.4.C, based on TRAC Report WA-RD 417.1 "Non-Contact Lap Splice in Bridge Column-Shaft Connections."

### 4.2.26 Lateral Confinement for Oversized Pile Shaft for SDCs C and D

**Guide Specifications Article 8.8.12** – The requirement of this article for shaft lateral reinforcement in the column-shaft splice zone may be replaced with Section 7.8.2 K.

### 4.2.27 Lateral Confinement for Non-Oversized Strengthened Pile Shaft for SDCs C and D

**Guide Specifications Article 8.8.13** – Non oversized column shaft (the cross section of the confined core is the same for both the column and the pile shaft) is not permissible unless approved by the WSDOT Bridge Design Engineer.

### 4.2.28 Requirements for Capacity Protected Members

**Guide Specifications Article 8.9** – Add the following paragraphs:

For SDCs C and D where liquefaction is identified, with the WSDOT Bridge Design Engineer's approval, pile and drilled shaft in-ground hinging may be considered as an ERE. Where in-ground hinging is part of ERS, the confined concrete core should be limited to a maximum compressive strain of 0.008 and the member ductility demand shall be limited to 4.

Bridges shall be analyzed and designed for the non-liquefied condition and the liquefied condition in accordance with Article 6.8. The capacity protected members shall be designed in accordance with the requirements of Article 4.11. To ensure the formation of plastic hinges in columns, oversized pile shafts shall be designed for an expected nominal moment capacity, Mne, at any location along the shaft, that is, equal to 1.25 times moment demand generated by the overstrength column plastic hinge moment and associated shear force at the base of the column. The safety factor of 1.25 may be reduced to 1.0 depending on the soil properties and upon the WSDOT Bridge Design Engineer's approval.

The design moments below ground for extended pile shaft may be determined using the nonlinear static procedure (pushover analysis) by pushing them laterally to the displacement demand obtained from an elastic response spectrum analysis. The point of maximum moment shall be identified based on the moment diagram. The expected plastic hinge zone shall extend 3D above and below the point of maximum moment. The plastic hinge zone shall be designated as the "no splice" zone and the transverse steel for shear and confinement shall be provided accordingly.

### 4.2.29 Superstructure Capacity Design for Transverse Direction (Integral Bent Cap) for SDCs C and D

**Guide Specifications Article 8.11** – Revise the last paragraph as follows:

For SDCs C and D, the longitudinal flexural bent cap beam reinforcement shall be continuous. Splicing of cap beam longitudinal flexural reinforcement shall be accomplished using mechanical couplers that are capable of developing a minimum tensile strength of 85 ksi. Splices shall be staggered at least 2 ft. Lap splices shall not be used.

### 4.2.30 Superstructure Design for Non Integral Bent Caps for SDCs B, C, and D

**Guide Specifications Article 8.12** – Non integral bent caps shall not be used for continuous concrete bridges in SDC B, C, and D except at the expansion joints between superstructure segments.

### 4.2.31 Joint Proportioning

**Guide Specifications Article 8.13.4.1.1** – Revise the last bullet as follows:

Exterior column joints for box girder superstructure and other superstructures if the cap beam extends the joint far enough to develop the longitudinal cap reinforcement.

### 4.2.32 Cast-in-Place and Precast Concrete Piles

**Guide Specifications Article 8.16.2** – Minimum longitudinal reinforcement of 0.75 percent of $A_g$ shall be provided for CIP piles in SDCs B, C, and D. Longitudinal reinforcement shall be provided for the full length of pile unless approved by the WSDOT Bridge Design Engineer.

# 4.3 Seismic Design Requirements for Bridge Modifications and Widening Projects

## 4.3.1 General

A bridge widening is defined as where substructure bents are modified and new columns or piers are added, or an increase of bridge deck width or widenings to the sidewalk or barrier rails of an existing bridge resulting in significant mass increase or structural changes.

Bridge widenings in Washington State shall be designed in accordance with the requirements of the current edition of the AASHTO LRFD Bridge Design Specifications. The seismic design shall be in accordance with the requirements of the AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO SGS), and WSDOT *Bridge Design Manual* (BDM). The widening portion (new structure) shall be designed to meet current WSDOT standards for new bridges. Seismic analysis is not required for single-span bridges and bridges in SDC A. However, existing elements of single span bridges shall meet the requirements of AASHTO SGS as applicable.

## 4.3.2 Bridge Widening Project Classification

Bridge widening projects are classified according to the scope of work as either minor or major widening projects.

### A. Minor Modification and Widening Projects

A bridge widening project is classified as a minor widening project if all of the following conditions are met:

- Substructure bents are not modified and no new columns or piers are added, while abutments may be widened to accommodate the increase of bridge deck width.
- The net superstructure mass increase is equal or less than 10% of the original superstructure mass.
- Fixity conditions of the foundations are unchanged.
- There are no major changes of the seismicity of the bridge site that can increase seismic hazard levels or reduce seismic performance of the structure since the initial screening or most recent seismic retrofit.
- No change in live load use of the bridge

### B. Major Modifications and Widening Projects

A bridge widening project is classified as a major widening project if any of the following conditions are met:

- Substructure bents are modified and new columns or piers are added, excepting abutments, which may be widened to accommodate the increase of bridge deck width.
- The net superstructure mass increase is more than 20% of the original superstructure mass.
- Fixity conditions of the foundations are changed.

- There are major changes of the seismicity of the bridge site that can increase seismic hazard levels or reduce seismic performance of the structure since the initial screening or most recent seismic retrofit.
- Change in live load use of the bridge

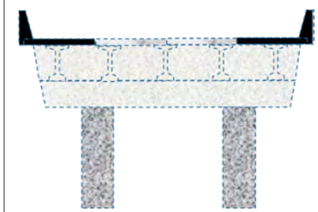
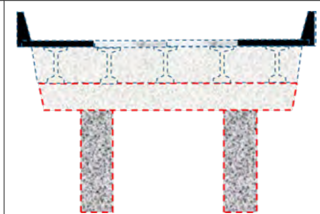
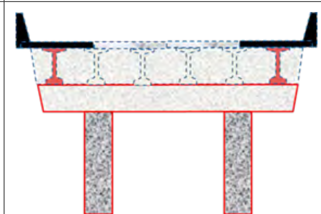
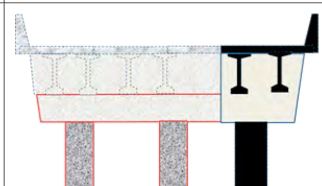
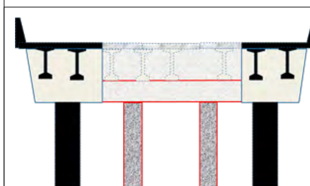
Major changes in seismicity include, but are not limited to, the following: near fault effect, significant liquefaction potential, or lateral spreading. If there are concerns about changes to the Seismic Design Response Spectrum at the bridge site, about a previous retrofit to the existing bridge, or an unusual imbalance of mass distribution resulting from the structure widening, the designer should consult the WSDOT Bridge and Structures Office.

### 4.3.3 Seismic Design Guidance:

The Seismic Design guidance for Bridge Modifications and Widening are as follows:

1. Bridge widening projects classified as Minor Widening projects do not require either a seismic evaluation or a retrofit of the structure. If the conditions for Minor Widening project are met, it is anticipated that the widened/modified structure will not draw enough additional seismic demand to significantly affect the existing substructure elements.
2. If the net superstructure mass increase is between10% to 20% of the original superstructure mass, and if all the other bulleted criteria listed for Minor Widening projects are met, then the “Do No Harm” policy and professional judgment could be used upon approval of the Bridge Design Engineer. The "Do No Harm" policy requires the designer to compare the C/D ratios of the existing bridge elements in the before widening condition to those of the after widening condition. If the C/D ratios are not decreased, the widening can be designed and constructed without retrofitting existing deficient bridge elements. Elements of the existing structure with C/D ratios made worse by the widening/modification work shall be retrofitted to restore their C/D ratios to before-widening values, at a minimum. Foundation elements with seismic deficiencies (C/D ratios made worse by the widening/modification work) shall be deferred to the Seismic Retrofit Program for rehabilitation.
3. Seismic analysis is required for all Major Modifications and Widening projects at project scoping level. A complete seismic analysis is required for bridges in Seismic Design Category (SDC) B, C, and D for major modifications and widening projects as described below. A project geotechnical report (including any unstable soil or liquefaction issues) shall be available to the structural engineer for seismic analysis. Seismic analysis shall be performed for both existing and widened structures. Capacity/Demand (C/D) ratios are required for existing bridge elements including foundation.
4. The widening portion of the structure shall be designed for liquefiable soils condition in accordance to the AASHTO SGS, and WSDOT BDM, unless soils improvement is provided to eliminate liquefaction.

5. Seismic improvement of existing columns and crossbeams to C/D > 1.0 is required. The cost of seismic improvement shall be paid for with widening project funding (not from the Retrofit Program). The seismic retrofit of the existing structure shall conform to the BDM, while the newly widened portions of the bridge shall comply with the AASHTO SGS, except for balanced stiffness criteria, which may be difficult to meet due to the existing bridge configuration. However, the designer should strive for the best balanced frame stiffness for the entire widened structure that is attainable in a cost effective manner. Major Widening Projects require the designer to determine the seismic C/D ratios of the existing bridge elements in the final widened condition. If the C/D ratios of columns and crossbeam of existing structure are less than 1.0, the improvement of seismically deficient elements is mandatory and the widening project shall include the improvement of existing seismically deficient bridge elements to C/D ratio of above 1.0 as part of the widening project funding. The C/D ratio of 1.0 is required to prevent the collapse of the bridge during the seismic event as required for life safety. Seismic improvement of the existing foundation elements (footings, pile caps, piles, and shafts to C/D ratios > 1.0) could be deferred to the Bridge Seismic Retrofit Program.

| Modifications or Widening | Alterations | Seismic Design Guidance | Illustration |
|---|---|---|---|
| **Minor Modifications**<br>• Deck Rehabilitations<br>• Traffic Barrier Replacements<br>• sidewalk addition/ rehabilitation<br>• No change in LL use | • Superstructure mass increase is less than 10%<br>• Fixity conditions are not changed | • Do not Require seismic evaluation<br>• Do not require retrofit of the structure |  |
| **Major Modifications**<br>Minor Modifications PLUS<br>• Replacing/adding girder and slab<br>• Change in LL use | • Superstructure mass increase between 10% to 20% and/or<br>• Fixity conditions are changed | • Seismic evaluation of the structure is required.<br>• Do-No-Harm is required for substructure.<br>• Do-No-Harm is required for foundation. |  |
| **Major Widening – Case 1**<br>Minor Modifications PLUS<br>• Superstructure or Bent Widening | • Superstructure mass increase is more than > 20% and/or<br>• Substructure/bents modified and/or<br>• Fixity conditions are changed | • Seismic evaluation of the structure is required.<br>• C/D ratio of equal or greater than 1.0 is required for substructure.<br>• Do-No-Harm could be used for Foundation. |  |
| **Major Widening – Case 2**<br>• widening on one side | • Substructure or bents are modified. Columns are added on one side. | • Seismic evaluation of the structure is required.<br>• C/D ratio of equal or greater than 1.0 is required for substructure.<br>• Do-No-Harm could be used for Foundation. |  |
| **Major Widening – Case 3**<br>• widening on both sides | • Substructure or bents are modified. Columns are added on both sides. | • Seismic evaluation of the structure is required.<br>• C/D ratio of equal or greater than 1.0 is required for substructure.<br>• Do-No-Harm could be used for Foundation. |  |

**Seismic Design Criteria for Bridge Modifications and Widening**
***Figure 4.3-1***

### 4.3.4 Scoping for Bridge Widening and Liquefaction Mitigation

The Region project manager should contact the Bridge Office for bridge widening and retaining wall scoping assistance before project funding commitments are made to the legislature and the public. The Bridge Office will work with the Geotechnical Office to assess the potential for liquefaction or other seismic hazards that could affect the cost of the proposed structures. The initial evaluation design time and associated costs for the Geotechnical and Bridge Offices shall be considered at the scoping phase.

### 4.3.5 Design and Detailing Considerations

**Support Length** – The support length at existing abutments, piers, in-span hinges, and pavement seats shall be checked. If there is a need for longitudinal restrainers, transverse restrainers, or additional support length on the existing structure, they shall be included in the widening design.

**Connections Between Existing and New Elements** – Connections between the new elements and existing elements should be designed for maximum over-strength forces. Where yielding is expected in the crossbeam connection at the extreme event limit state, the new structure shall be designed to carry live loads independently at the Strength I limit state. In cases where large differential settlement and/or a liquefaction-induced loss of bearing strength are expected, the connections may be designed to deflect or hinge in order to isolate the two parts of the structure. Elements subject to inelastic behavior shall be designed and detailed to sustain the expected deformations.

Longitudinal joints between the existing and new structure are not permitted.

**Differential Settlement** – The geotechnical designer should evaluate the potential for differential settlement between the existing structure and widening structure. Additional geotechnical measures may be required to limit differential settlements to tolerable levels for both static and seismic conditions. The bridge designer shall evaluate, design, and detail all elements of new and existing portions of the widened structure for the differential settlement warranted by the WSDOT Geotechnical Engineer. Angular distortions between adjacent foundations greater than 0.008 (RAD) in simple spans and 0.004 (RAD) in continuous spans should not be permitted in settlement criteria.

The horizontal displacement of pile and shaft foundations shall be estimated using procedures that consider soil-structure interaction (see *Geotechnical Design Manual* Section 8.12.2.3). Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement with consideration of the column length and stiffness. Tolerance of the superstructure to lateral movement will depend on bridge seat widths, bearing type(s), structure type, and load distribution effects.

**Foundation Types** – The foundation type of the new structure should match that of the existing structure. However, a different type of foundation may be used for the new structure due to geotechnical recommendations or the limited space available between existing and new structures. For example, a shaft foundation may be used in lieu of spread footing.

**Existing Strutted Columns** – The horizontal strut between existing columns may be removed. The existing columns shall then be analyzed with the new unbraced length and retrofitted if necessary.

**Non Structural Element Stiffness** – Median barrier and other potentially stiffening elements shall be isolated from the columns to avoid any additional stiffness to the system.

Deformation capacities of existing bridge members that do not meet current detailing standards shall be determined using the provisions of Section 7.8 of the *Retrofitting Manual for Highway Structures: Part 1 – Bridges*, FHWA-HRT-06-032. Deformation capacities of existing bridge members that meet current detailing standards shall be determined using the latest edition of the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*.

Joint shear capacities of existing structures shall be checked using *Caltrans Bridge Design Aid*, 14-4 Joint Shear Modeling Guidelines for Existing Structures.

In lieu of specific data, the reinforcement properties provided in Table 4.3.2-1 should be used.

| Property | Notation | Bar Size | ASTM A706 | ASTM A615 Grade 60 | ASTM A615 Grade 40* |
|---|---|---|---|---|---|
| Specified minimum yield stress (ksi) | $f_y$ | No. 3 - No. 18 | 60 | 60 | 40 |
| Expected yield stress (ksi) | $f_{ye}$ | No. 3 - No. 18 | 68 | 68 | 48 |
| Expected tensile strength (ksi) | $f_{ue}$ | No. 3 - No. 18 | 95 | 95 | 81 |
| Expected yield strain | $\varepsilon_{ye}$ | No. 3 - No. 18 | 0.0023 | 0.0023 | 0.00166 |
| Onset of strain hardening | $\varepsilon_{sh}$ | No. 3 - No. 8 | 0.0150 | 0.0150 | 0.0193 |
| | | No. 9 | 0.0125 | 0.0125 | |
| | | No. 10 & No. 11 | 0.0115 | 0.0115 | |
| | | No. 14 | 0.0075 | 0.0075 | |
| | | No. 18 | 0.0050 | 0.0050 | |
| Reduced ultimate tensile strain | $\varepsilon_{su}^{R}$ | No. 4 - No. 10 | 0.090 | 0.060 | 0.090 |
| | | No. 11 - No. 18 | 0.060 | 0.040 | 0.060 |
| Ultimate tensile strain | $\varepsilon_{su}$ | No. 4 - No. 10 | 0.120 | 0.090 | 0.120 |
| | | No. 11 - No. 18 | 0.090 | 0.060 | 0.090 |

* ASTM A615 Grade 40 is for existing bridges in widening projects.

**Stress Properties of Reinforcing Steel Bars**
***Table 4.3.2-1***

**Isolation Bearings** – Isolation bearings may be used for bridge widening projects to reduce the seismic demand through modification of the dynamic properties of the bridge. These bearings are a viable alternative to strengthening weak elements or non-ductile bridge substructure members of the existing bridge. Use of isolation bearings needs the approval of WSDOT Bridge Design Engineer. Isolation bearings shall be designed per the requirements specified in Section 9.3.

## 4.4 Seismic Retrofitting of Existing Bridges

Seismic retrofitting of existing bridges shall be performed in accordance with the FHWA publication FHWA-HRT-06-032, *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges* and WSDOT amendments as follows:

- Article 7.4.2 Seismic Loading in Two or Three Orthogonal Directions

Revise the first paragraph as follows:

When combining the response of two or three orthogonal directions the design value of any quantity of interest (displacement, bending moment, shear or axial force) shall be obtained by the 100-30 percent combination rule as described in AASHTO *Guide Specifications* Article 4.4.

- Delete Eq. 7.44 and replace with the following:

$$L_p = \text{the maximum of } [(8800\varepsilon_y d_b) \text{ or } (0.08L + 4400\varepsilon_y d_b)] \tag{7-44}$$

- Delete Eq. 7.49 and replace with the following:

$$\phi_p = \left(5\left(\frac{V_i - V_m}{V_i - V_f}\right) + 2\right)\phi_y \tag{7.49}$$

- Delete Eq. 7.51 and replace with the following:

$$\phi_p = \left(4\left(\frac{V_{ji} - V_{jh}}{V_{ji} - V_{jf}}\right) + 2\right)\phi_y \tag{7.51}$$

### 4.4.1 Seismic Analysis Requirements

The first step in retrofitting a bridge is to analyze the existing structure to identify seismically deficient elements. The initial analysis consists of generating capacity/ demand ratios for all relevant bridge components. Seismic displacement and force demands shall be determined using the multi-mode spectral analysis of Section 5.4.2.2 (at a minimum). Prescriptive requirements, such as support length, shall be considered a demand and shall be included in the analysis. Seismic capacities shall be determined in accordance with the requirements of the *Seismic Retrofitting Manual*. Displacement capacities shall be determined by the Method D2 – Structure Capacity/ Demand (Pushover) Method of Section 5.6. For most WSDOT bridges, the seismic analysis need only be performed for the upper level (1,000 year return period) ground motions with a life safety seismic performance level.

### 4.4.2 Seismic Retrofit Design

Once seismically deficient bridge elements have been identified, appropriate retrofit measures shall be selected and designed. Table 1-11, Chapters 8, 9, 10, 11, and Appendices D thru F of the *Seismic Retrofitting Manual* shall be used in selecting and designing the seismic retrofit measures. The WSDOT Bridge and Structure Office Seismic Specialist will be consulted in the selection and design of the retrofit measures.

### 4.4.3 Computer Analysis Verification

The computer results will be verified to ensure accuracy and correctness. The designer should use the following procedures for model verification:

- Using graphics to check the orientation of all nodes, members, supports, joint, and member releases. Make sure that all the structural components and connections correctly model the actual structure.
- Check dead load reactions with hand calculations. The difference should be less than 5 percent.
- Calculate fundamental and subsequent modes by hand and compare results with computer results.
- Check the mode shapes and verify that structure movements are reasonable.
- Increase the number of modes to obtain 90 percent or more mass participation in each direction. GTSTRUDL/SAP2000 directly calculates the percentage of mass participation.
- Check the distribution of lateral forces. Are they consistent with column stiffness? Do small changes in stiffness of certain columns give predictable results?

### 4.4.4 Earthquake Restrainers

Longitudinal restrainers shall be high strength steel rods conform to ASTM F 1554 Grade 105, including Supplement Requirements S2, S3 and S5. Nuts, and couplers if required, shall conform to ASTM A 563 Grade DH. Washers shall conform to AASHTO M 293. High strength steel rods and associated couplers, nuts and washers shall be galvanized after fabrication in accordance with AASHTO M 232. The length of longitudinal restrainers shall be less than 24 feet.

### 4.4.5 Isolation Bearings

Isolation bearings may be used for seismic retrofit projects to reduce the demands through modification of the dynamic properties of the bridge as a viable alternative to strengthening weak elements of non-ductile bridge substructure members of existing bridge. Use of isolation bearings needs the approval of WSDOT Bridge Design Engineer. Isolation bearings shall be designed per the requirements specified in Section 9.3.

# 4.5 Seismic Design Requirements for Retaining Walls

## 4.5.1 General

All retaining walls shall include seismic design load combinations. The design acceleration for retaining walls shall be determined in accordance with the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*. Once the design acceleration is determined, the designer shall follow the applicable design specification requirements listed in Appendix 8.1-A1:

Exceptions to the cases described in Appendix 8.1-A1 may occur with approval from the WSDOT Bridge Design Engineer and/or the WSDOT Geotechnical Engineer.

# 4.6 Appendices

## Design Example – Restrainer Design

FHWA-HRT-06-032 *Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges*, Example 8.1 Restrainer Design by Iterative Method

| | | | |
|---|---|---|---|
| $N$ = | 12.00 " | Seat Width (inch) | |
| $d_c$ = | 2.00 " | concrete cover on vertical faces at seat (inch) | |
| "G" = | 1.00 " | expansion joint gap (inch). For new structures, use maximum estimated opening. | |
| F.S. = | 0.67 | safety factor against the unseating of the span | |
| $F_y$ = | 176.00 ksi | restrainer yield stress (ksi) | |
| $E$ = | 10,000 | restrainer modulus of elasticity (ksi) | |
| $L$ = | 18.00 ' | restrainer length (ft.) | |
| $D_{rs}$ = | 1.00 " | restrainer slack (inch) | |
| $W_1$ = | 5000.00 | the weight of the less flexible frame (kips) (Frame 1) | |
| $W_2$ = | 5000.00 | the stiffness of the more flexible frame (kips) (Frame 2) | |
| $K_1$ = | 2040 | the stiffness of the less flexible frame (kips/in) (Frame 1) | |
| $K_2$ = | 510 | the stiffness of the more flexible frame (kips/in) (Frame 2) | OK |
| $\mu_d$ = | 4.00 | Target displacement ductility of the frames | |
| $g$ = | 386.40 | acceleration due to gravity (in/sec$^2$) | |
| $\xi$ = | 0.05 | design spectrum damping ratio | |
| $S_{DS}$ = | 1.75 | short period coefficient | $S_{DS} = F_a S_s$ |
| $S_{D1}$ = | 0.70 | long period coefficient | $S_{D1} = F_v S_1$ |
| $A_s$ = | 0.28 | effective peak ground acceleration coefficient | |
| $\Delta_{tol}$ = | 0.05 " | converge tolerance | |

Calculate the period at the end of constant design spectral acceleration plateau (sec)

$$T_s = \frac{S_{D1}}{S_{DS}}$$ = 0.7 / 1.75 = 0.4 sec

Calculate the period at beginning of constant design spectral acceleration plateau (sec)

$$T_o = 0.2T_s$$ = 0.2 * 0.4 = 0.08 sec

Step 1: Calculate Available seat width, $D_{as}$

$D_{as}$ = 12 - 1 - 2 * 2 = 7 "    0.67 * 7 = 4.69 "

Step 2: Calculate Maximum Allowable Expansion Joint Displacement and compare to the available seat width.

$D_r$ = 1 + 176 * 18 * 12 / 10000 = 4.8 "    > = 4.69 "    NG

Step 3: Compute expansion joint displacement without restrainers

The effective stiffness of each frame are modified due to yielding of frames.

$K_{1,eff}$ = 2040 / 4 = 510 kip/in

$K_{2,eff}$ = 510 / 4 = 127.5 kip/in

The effective natural period of each frame is given by:

$$T_{1,eff} = 2\pi\sqrt{\frac{W_1}{gK_{1,eff}}}$$ = 2 * PI() *( 5000 / ( 386.4 * 510) )^0.5 = 1 sec.

$$T_{2,eff} = 2\pi\sqrt{\frac{W_2}{gK_{2,eff}}}$$ = 2 * PI() *( 5000 / ( 386.4 * 127.5) )^0.5 = 2 sec.

The effective damping and design spectrum correction factor is:

$\xi_{eff}$ = 0.05 + ( 1 - 0.95 /( 4 ) ^ 0.5 - 0.05 * (4 ) ^ 0.5 ) / PI() = 0.19

$c_d$ = 1.5 / ( 40 * 0.19 + 1 ) + 0.5 = 0.68

Determine the frame displacement from Design Spectrum

| | | | | |
|---|---|---|---|---|
| $T_{1,eff}$ = | 1.00 | sec. | $S_a(T_{1,eff})$= | 0.699 |
| $T_{2,eff}$ = | 2.00 | sec. | $S_a(T_{2,eff})$= | 0.350 |

Modified displacement for damping other than 5 percent damped bridges

$$D_1 = \left(\frac{T_{1,eff}}{2\pi}\right)^2 c_d\, S_a(T_{1,eff})\, g$$ = ( 1 / ( 2*PI()))^2 * 0.68* 0.699 * 386.4 = 4.65 "

$$D_2 = \left(\frac{T_{2,eff}}{2\pi}\right)^2 c_d\, S_a(T_{2,eff})\, g$$ = ( 2 / ( 2*PI()))^2 * 0.68* 0.35 * 386.4 = 9.3 "

The relative displacement of the two frames can be calculated using the CQC combination of the two frame displacement as given by equation (Eq. 3)

the frequency ratio of modes, $\beta = \frac{\omega_1}{\omega_2} = \frac{T_2}{T_1}$ = 2 / 1 = 2

The cross-correlation coefficient,

$$\rho_{12} = \frac{8\xi_{eff}^2(1+\beta)\beta^{3/2}}{(1-\beta^2)^2 + 4\xi_{eff}^2\beta(1+\beta)^2}$$

$\rho_{12}$ = (8 * 0.19 ^2)*(1 + 2 )*( 2 ^(3/2))/((1 - 2 ^2)^2 + 4* 0.19 ^2* 2 *(1+ 2 )^2) = 0.2

The initial relative hinge displacement

$D_{eq_o}$ = ( 4.65 ^2 + 9.3 ^2 - 2 * 0.2 * 4.65 * 9.3 ) ^ 0.5 = 9.52 "    >=2/3 Das = 4.69 "

Restrainers are required

Step 4: Estimate the Initial restrainer stiffness

$$K_{eff_{mod}} = \frac{K_{1,eff}K_{2,eff}}{K_{1,eff} + K_{2,eff}}$$ = ( 510 * 127.5 ) / ( 510 + 127.5 ) = 102 kip/in

$$K_{r_1} = \frac{K_{eff_{mod}}(D_{eq_o} - D_r)}{D_{eq_o}}$$ = 102 * ( 9.52 - 4.8 ) / 9.52 = 50.54 kip/in

Adjust restrainer stiffness to limit the joint displacement to a prescribed value $D_r$ .
This can be achieve by using Goal Seek on the Tools menu.

| | | | |
|---|---|---|---|
| | Goal Seek | | |
| | Set Cell | $J$104 | Cell Address for $\Delta = D_{eq} - D_r$ |
| | To Value | | |
| By Changing Cell | | $D$104 | Cell address for initial guess |

Apply the Goal Seek every time you use the spreadsheet and Click OK

$K_r$ = 193.21 kip/in (Input a value to start) $\Delta$ = 0.00 "

Step 5: Calculate Relative Hinge Displacement from modal analysis.

Frame 1 mass $m_1$ = 5000 / 386.4 = 12.94 kip. Sec$^2$ / in
Frame 2 mass $m_2$ = 5000 / 386.4 = 12.94 kip. Sec$^2$ / in
$K_{1,eff}$ = 510.00 kip/in $K_{2,eff}$ = 127.50 kip/in

Solve the following quadratic equation for natural frequencies

$$A(\omega_i^2)^2 + B(\omega_i^2) + C = 0$$

$A = m_1 m_2$ = 12.94 * 12.94 = 167.44
$B = -m_1(K_{2,eff} + K_r) - m_2(K_{1,eff} + K_r)$
= - 12.94 * ( 127.5 + 193.21 ) - 12.94 * ( 510 + 193.21 ) = -13249.52
$C = K_{1,eff}\,K_{2,eff} + (K_{1,eff} + K_{2,eff})\,K_r$
$C$ = 510 * 127.5 + ( 510 + 127.5 ) * 193.21 = 188197.22

The roots of this quadratic are
$\omega_1^2$ = (-( -13249.52) +(( -13249.52) ^2-4* 167.44 * 188197.22 )^0.5)/( 2*167.44 ) = 60.57

$\omega_2^2$ = (-(-13249.52-((-13249.52)^2-4*167.44*188197.22)^0.5)/(2*167.44) = 18.56

The natural frequencies are
$\omega_1$ = 7.78 rad/sec $\omega_2$ = 4.31 rad/sec
The corresponding natural periods are
$T = \frac{2\pi}{\omega}$ $T_{1,eff}$ = 0.81 sec. $T_{2,eff}$ = 1.46 sec.
For mode 1, $\omega_1$ = 7.78 rad/sec $\omega_1^2$ = 60.57
$K_{1,eff} + K_r - m_1\omega_1^2$ = 510 + 193.21 - 12.94 * 60.57 = -80.61

The relative value (modal shape) corresponding

$$\frac{\phi_{11}}{\phi_{21}} = \frac{K_r}{K_{1,eff} + K_r - m_1\omega_1^2}$$ = 193.21 / -80.61 = -2.397

It is customary to describe the normal modes by assigning a unit value to one of the amplitudes.
For the first mode, set $\phi_{21}$ = 1.00 then $\phi_{11}$ = -2.40
The mode shape for the first mode is

$$\{\phi_1\} = \begin{Bmatrix}\phi_{11}\\ \phi_{21}\end{Bmatrix} = \begin{Bmatrix}-2.40\\ 1.00\end{Bmatrix}$$

For mode 2, $\omega_2$ = 4.31 rad/sec $\omega_2^2$ = 18.56

$K_{1,eff} + K_r - m_1\omega_2^2$ = 510 + 193.21 - 12.94 * 18.56 = 463.11

The relative value

$$\frac{\phi_{12}}{\phi_{22}} = \frac{K_r}{K_{1,eff} + K_r - m_1\omega_2^2}$$ = 193.21 / 463.11 = 0.417

For the 2nd mode, set $\phi_{12}$ = 1.00 then $\phi_{22}$ = 2.40

The mode shape for the 2nd mode is

$$\{\phi_2\} = \begin{Bmatrix} \phi_{12} \\ \phi_{22} \end{Bmatrix} = \begin{Bmatrix} 1.00 \\ 2.40 \end{Bmatrix}$$

Calculate the participation factor for mode " 1 " $P_1 = \dfrac{\{\phi_1\}^T[M]\{1\}}{\{\phi_1\}^T[K]\{\phi_1\}}(\{a\}^T\{\phi_1\})$

$\{\phi_1\}^T[M]\{1\} = m_1\phi_{11} + m_2\phi_{21} =$ 12.94 * -2.4 + 12.94 * 1 = -18.08

$\{\phi_1\}^T[K]\{\phi_1\} = (K_{1,eff} + K_r)\phi_{11}^2 - 2K_r\phi_{11}\phi_{21} + (K_{2,eff} + K_r)\phi_{21}^2$

= ( 510 + 193.21 ) * (-2.4) ^2 - 2* 193.21 * -2.4 * 1 + ( 127.5 + 193.21 ) * ( 1 ) ^2 = 5286.98

$\{a\}^T\{\phi_1\} = \phi_{21} - \phi_{11} =$ 1 - -2.4 = 3.4

$P_1 =$ -18.08 / 5286.98 * 3.4 = -0.0116 sec.$^2$

Calculate the participation factor for mode " 2 " $P_2 = \dfrac{\{\phi_2\}^T[M]\{1\}}{\{\phi_2\}^T[K]\{\phi_2\}}(\{a\}^T\{\phi_2\})$

$\{\phi_2\}^T[M]\{1\} = m_1\phi_{12} + m_2\phi_{22} =$ 12.94 * 1 + 12.94 * 2.4 = 43.96

$\{\phi_2\}^T[K]\{\phi_1\} = (K_{1,eff} + K_r)\phi_{12}^2 - 2K_r\phi_{12}\phi_{22} + (K_{2,eff} + K_r)\phi_{22}^2$

= ( 510 + 193.21 ) * (1) ^2 - 2* 193.21 * 1 * 2.4 + ( 127.5 + 193.21 ) * ( 2.4 ) ^2 = 1619.53

$\{a\}^T\{\phi_2\} = \phi_{22} - \phi_{12} =$ 2.4 - 1 = 1.4

$P_2 =$ 43.96 / 1619.53 * 1.4 = 0.0379 sec.$^2$

Determine the frame displacement from Design Spectrum

$T_{1,eff}$ = 0.81 sec. $S_a(T_{1,eff})$= 0.867

$T_{2,eff}$ = 1.46 sec. $S_a(T_{2,eff})$= 0.480

Calculate new relative displacement at expansion joint

$$D_{eq1} = P_1 c_d S_a (T_{1,eff}, 0.05)\, g =$$ -0.0116 * 0.68 * 0.867 * 386.4 = -2.64 "

$$D_{eq2} = P_2 c_d S_a (T_{2,eff}, 0.05)\, g =$$ 0.0379 * 0.68 * 0.48 * 386.4 = 4.77 "

The effective period ratio

$$\beta = \frac{\omega_1}{\omega_2} = \frac{T_{2,eff}}{T_{1,eff}}$$ = 1.46 / 0.81 = 1.81

The cross-correlation coefficient,

$\rho_{12}$ = (8 * 0.19 ^2)*(1+ 1.81 )*( 1.81 ^(3/2))/((1 -1.81 ^2)^2+4* 0.19 ^2* 1.81 *(1+1.81 )^2)

= 0.26

$D_{eq_1}$ = ( ( -2.64)^2 + ( 4.77)^2+2* 0.26 * ( -2.64 ) * (4.77))^0.5 = 4.8 "

>4.8 "

$\Delta = D_{eq} - D_r$ = 4.8 - 4.8 = 0 "

OK Go to Step 7 and calculate the number of restrainers

Step 7: Calculate number of restrainers

$$N_r = \frac{K_r D_r}{F_y A_r}$$

$D_r$ = 4.80 " $K_r$ = 193.21 kip/in $F_y$ = 176.00 ksi

$A_r$ = 0.222 in^2

$N_r$ = ( 193.21 * 4.8 ) / ( 176 * 0.222 ) = 23.74 restrainers

# 1. Introduction

This example serves to illustrate the procedure used to perform nonlinear static "pushover" analysis in both the longitudinal and transverse directions in accordance with the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* using SAP2000. A full model of the bridge is used to compute the displacement demand from a response-spectrum analysis. To perform the pushover analysis in the longitudinal direction, the entire bridge is pushed in order to include the frame action of the superstructure and adjacent bents. To perform the pushover analysis in the transverse direction, a bent is isolated using the SAP2000 "staged construction" feature. The example bridge is symmetric and has three spans. It is assumed the reader has some previous knowledge of how to use SAP2000. This example was created using SAP2000 version 14.2.0.

***Note:*** By producing this example, the Washington State Department of Transportation does not warrant that the SAP2000 software does not include errors. The example does not relieve Design Engineers of their professional responsibility for the software's accuracy and is not intended to do so. Design Engineers should verify all computer results with hand calculations.

**Brief Table of Contents of Example:**

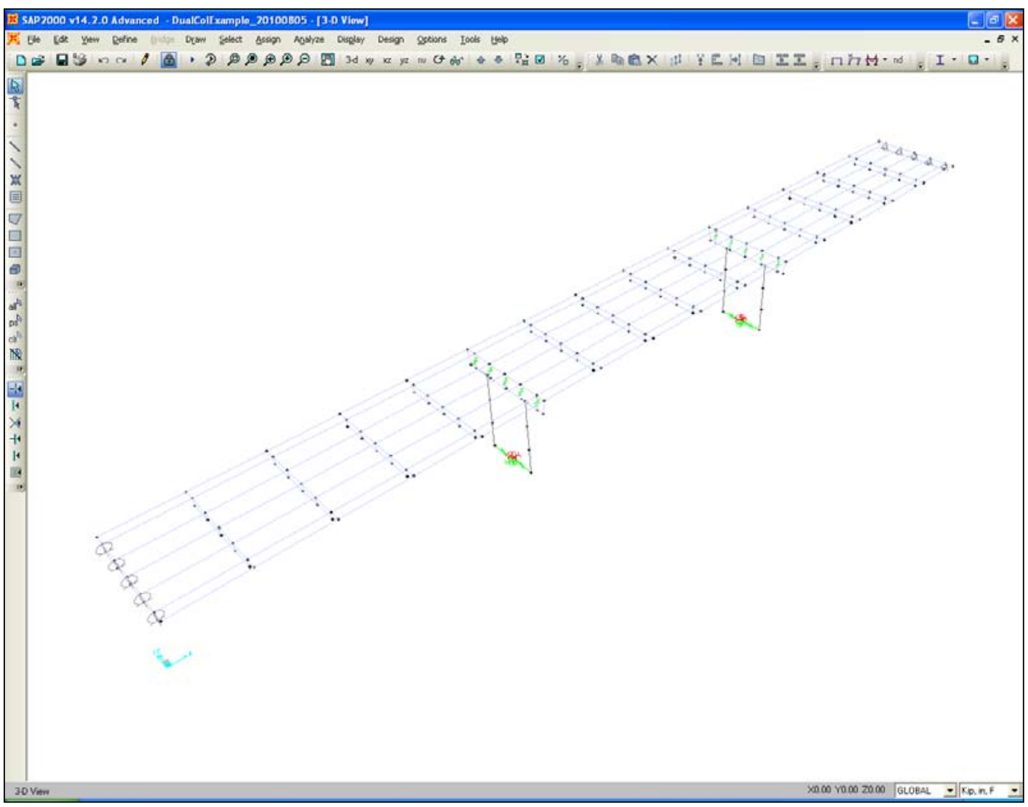
# 2. Model Setup

## 2.1 Overview of Model

This example employs SAP2000. The superstructure is modeled using frame elements for each of the girders and shell elements for the deck. Shell elements are also used to model the end, intermediate, and pier diaphragms. Non-prismatic frame sections are used to model the crossbeams since they have variable depth. The X-axis is along the bridge's longitudinal axis and the Z-axis is vertical. The units used for inputs into SAP2000 throughout this example are kip-in. The following summarizes the bridge being modeled:

- All spans are 145' in length
- (5) lines of prestressed concrete girders (WF74G) with 9'-6" ctc spacing
- 8" deck with 46'-11" to width
- Girders are continuous and fixed to the crossbeams at the intermediate piers
- (2) 5' diameter columns at bents
- Combined spread footings – 20'L x 40'W x 5'D at each bent
- Abutment longitudinal is free, transverse is fixed

Figure 2.1-1 shows a view of the model in SAP2000.

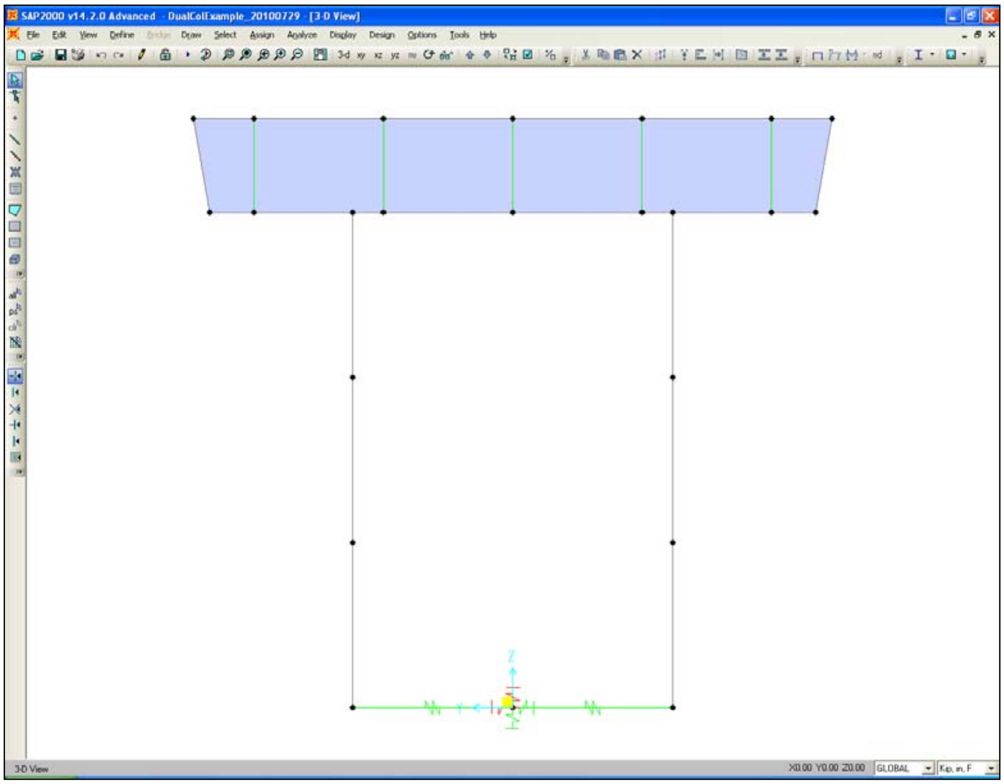



**Wireframe 3-D View of Model**
*Figure 2.1-1*

## 2.2 Foundations Modeling

### 2.2.1 Intermediate Piers

Each bent is supported by a combined spread footing that is 20'L x 40'W x 5'D. These footings are modeled using springs. Rigid links connect the bases of the columns to a center joint that the spring properties are assigned to as shown in Figure 2.2.1-1.



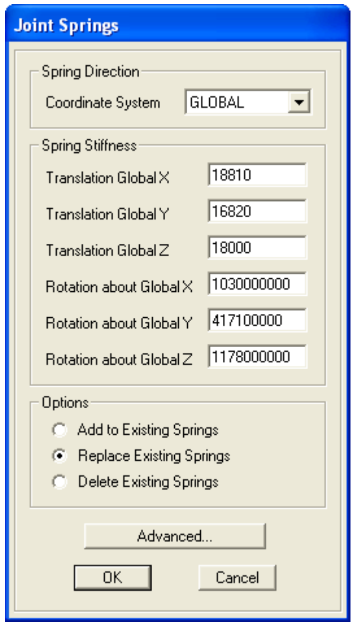
**Wireframe 2-D View of Bent**
***Figure 2.2.1-1***

The soil springs were generated using the method for spread footings outlined in Chapter 7 of the *Washington State Department of Transportation Bridge Design Manual*. The assumed soil parameters were G = 1,700 ksf and ν = 0.35. The spring values used in the model for the spread footings are shown in Table 2.2.1-1.

| Degree of Freedom | Stiffness Value |
|---|---|
| UX | 18,810 kip/in |
| UY | 16,820 kip/in |
| UZ | 18,000 kip/in |
| RX | 1,030,000,000 kip-in/rad |
| RY | 417,100,000 kip-in/rad |
| RZ | 1,178,000,000 kip-in/rad |

**Joint Spring Values for Spread Footings**
***Table 2.2.1-1***

Figure 2.2.1-2 shows the spread footing joint spring assignments (**Assign menu > Joint > Springs**).




**Spread Footing Joint Spring Assignments**
*Figure 2.2.1-2*

The springs used in the demand model (response-spectrum model) are the same as the springs used in the capacity model (pushover model). It is also be acceptable to conservatively use fixed-base columns for the capacity model.

## 2.2.2 Abutments

The superstructure is modeled as being free in the longitudinal direction at the abutments in accordance with the policies outlined in the *Washington State Department of Transportation Bridge Design Manual*. The abutments are fixed in the transverse direction in this example for simplification. However, please note that the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* require the stiffness of the transverse abutments be modeled. Since there are five girder lines instead of a spine element, the joints at the ends of the girders at the abutments all have joint restraints assigned to them. The girder joint restraint assignments at the abutments are listed in Table 2.2.2-1.

| Degree of Freedom | Fixity |
|---|---|
| UX | Free |
| UY | Fixed |
| UZ | Fixed |
| RX | Free |
| RY | Free |
| RZ | Free |

**Joint Fixity for Girder Joints at Abutments**
*Table 2.2.2-1*

Figure 2.2.2-1 shows the girder joint restraints at the abutments (**Assign menu > Joint > Restraints**).


**Girder Joint Restraint Assignments at Abutments**
***Figure 2.2.2-1***

## 2.3 Materials Modeling

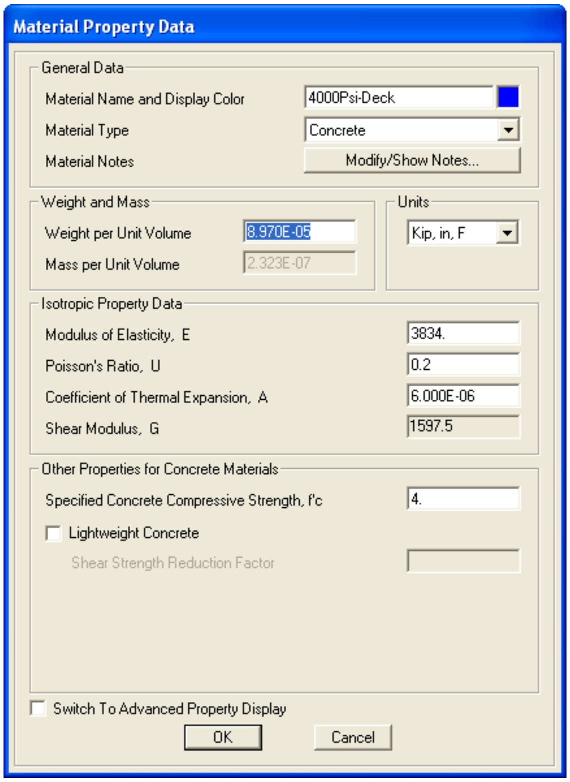
SAP2000's default concrete material properties have elastic moduli based on concrete densities of 144 psf. The elastic moduli of the concrete materials used in this example are based on the Washington State Department of Transportation's policy on concrete densities to be used in the calculations of elastic moduli. Please see the current WSDOT *Bridge Design Manual* and *Bridge Design Memorandums*. In Version 14 of SAP2000, nonlinear material properties for Caltrans sections are no longer defined in Section Designer and are now defined in the material definitions themselves. Table 2.3-1 lists the material definitions used in the model and the elements they are applied to (**Define menu > Materials**).

| Material Name | Material Type | Section Property Used For | Material Unit Weight (pcf) For Dead Load | Material Unit Weight (pcf) For Modulus of Elasticity |
|---|---|---|---|---|
| 4000Psi-Deck | Concrete | Deck | 155 | 150 |
| 4000Psi-Other | Concrete | Crossbeams & Diaphragms | 150 | 145 |
| 5200Psi-Column | Concrete | Columns | 150 | 145 |
| 7000Psi-Girder | Concrete | Girders | 165 | 155 |
| A706-Other | Rebar | Rebar Other Than Columns | 490 | - |
| A706-Column | Rebar | Column Rebar | 490 | - |

**Material Properties Used in Model**
***Table 2.3-1***

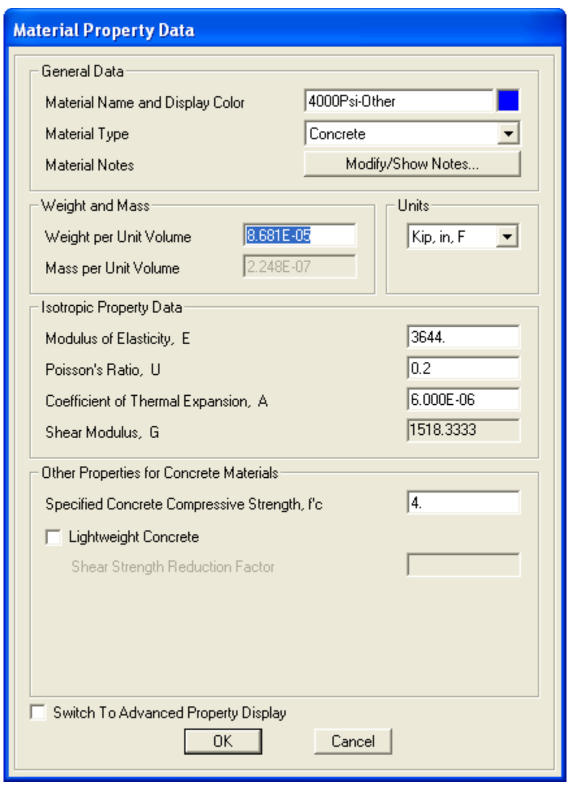
The "5200Psi-Column" and "A706-Column" material definitions are created to define the expected, nonlinear properties of the column section.

The Material Property Data for the material "4000Psi-Deck" is shown in Figure 2.3-1 (**Define menu > Materials > *select* 4000Psi-Deck > *click* Modify/Show Material *button***).



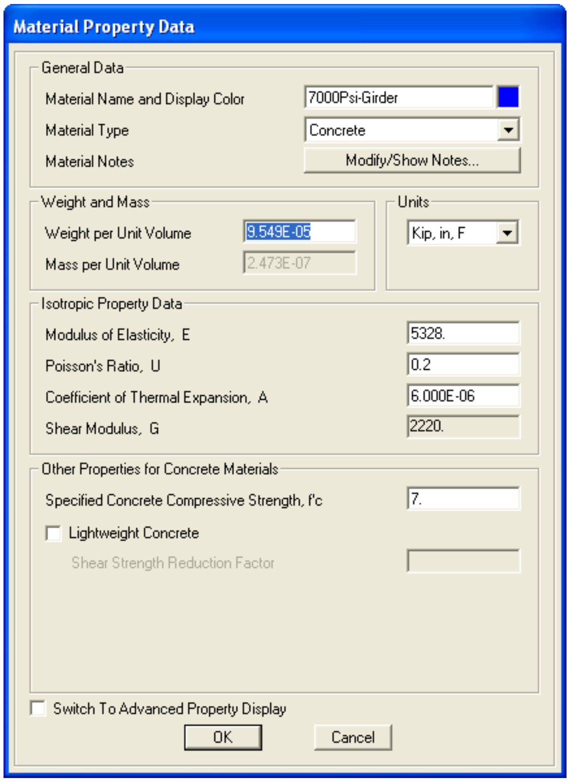

**Material Property Data for Material "4000Psi-Deck"**
*Figure 2.3-1*

The Material Property Data for the material "4000Psi-Other" is shown in Figure 2.3-2 (**Define menu** > **Materials** > *select* **4000Psi-Other** > *click* **Modify/Show Material** *button*).



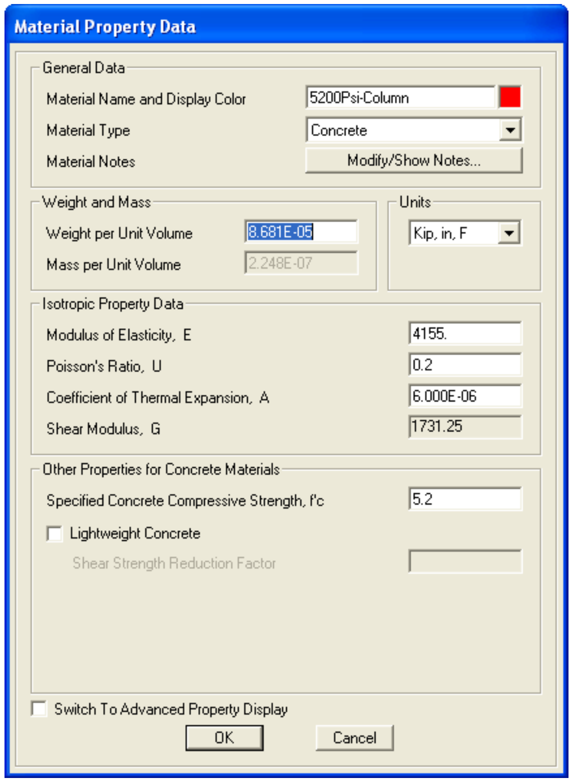

**Material Property Data for Material "4000Psi-Other"**
*Figure 2.3-2*

The Material Property Data for the material "7000Psi-Girder" is shown in Figure 2.3-3 (**Define menu** > **Materials** > ***select*** **7000Psi-Girder** > ***click*** **Modify/Show Material *button***).



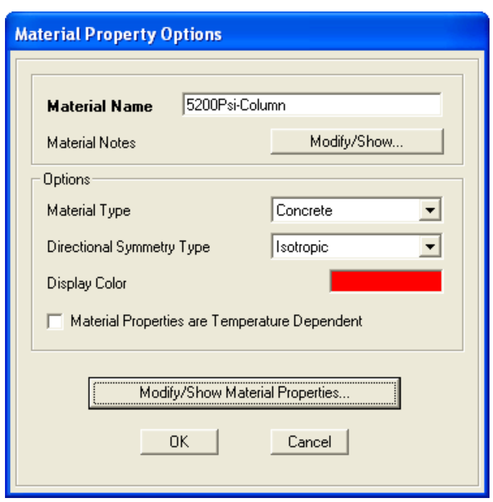

**Material Property Data for Material "7000Psi-Girder"**
***Figure 2.3-3***

The Material Property Data for the material "5200Psi-Column" is shown Figure 2.3-4 (**Define menu** > **Materials** > *select* **5200Psi-Column** > *click* **Modify/Show Material** *button*).



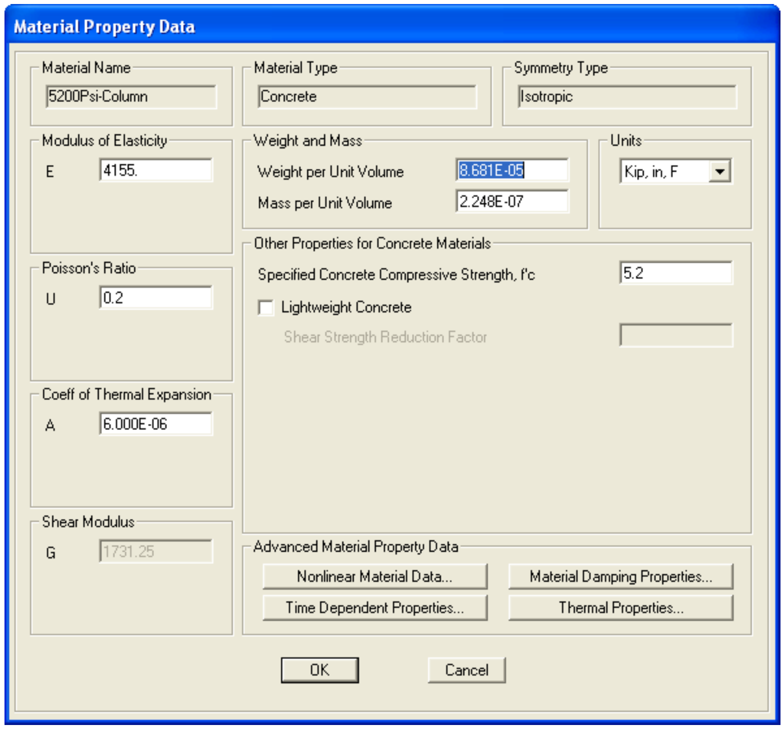

**Material Property Data for Material "5200Psi-Column"**
***Figure 2.3-4***

When the **Switch To Advanced Property Display** box shown in Figure 2.3-4 is checked , the window shown in Figure 2.3-5 opens.



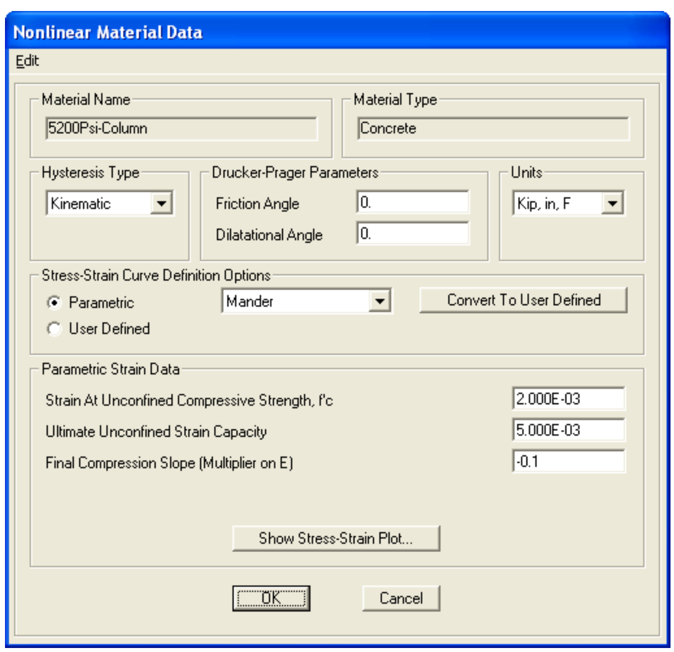

**Advanced Material Property Options for Material "5200Psi-Column"**
***Figure 2.3-5***

By clicking the **Modify/Show Material Properties** button in Figure 2.3-5, the window shown in Figure 2.3-6 opens.



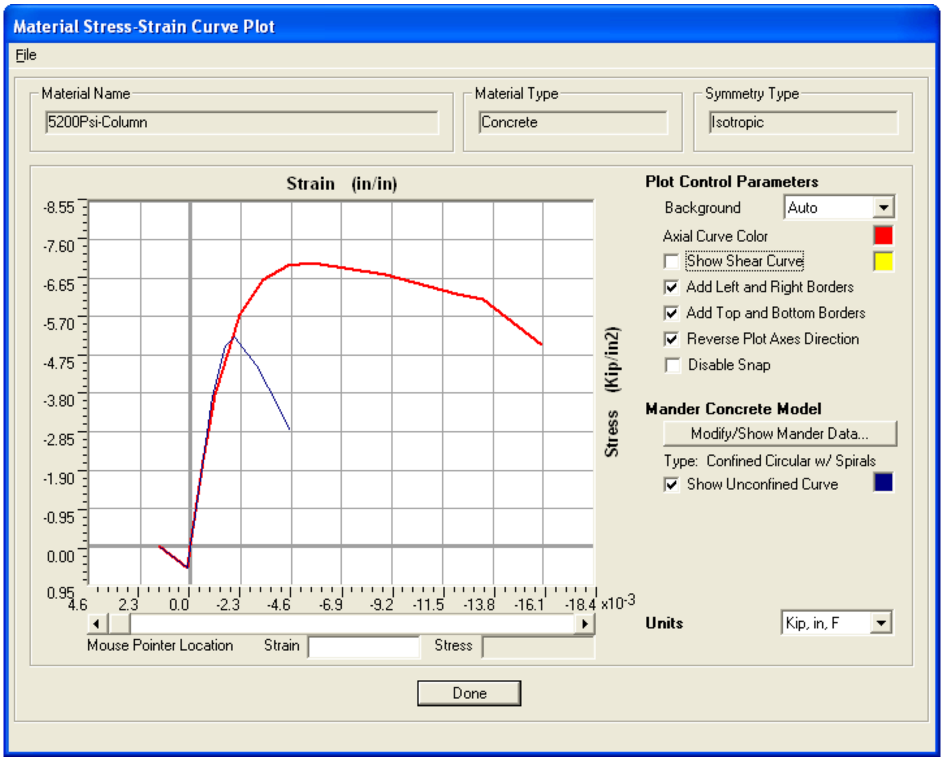

**Advanced Material Property Data for Material "5200Psi-Column"**
***Figure 2.3-6***

By clicking the **Nonlinear Material Data** button in Figure 2.3-6, the window shown in Figure 2.3-7 opens.




**Nonlinear Material Data for Material "5200Psi-Column"**
***Figure 2.3-7***

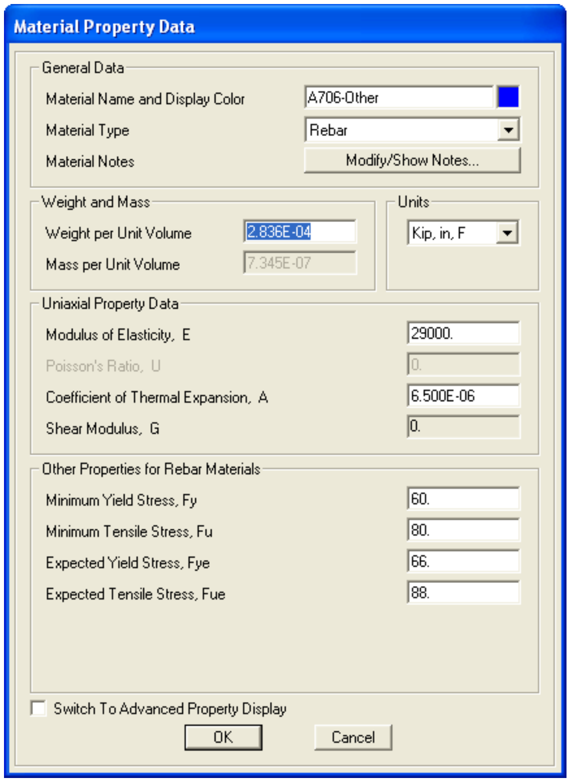
Note that in Figure 2.3-7 the *Strain At Unconfined Compressive Strength, f'c* and the *Ultimate Unconfined Strain Capacity* are set to the values required in Section 8.4.4 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. These unconfined properties are parameters used in defining the Mander confined concrete stress-strain curve of the column core. It is seen that under the *Stress-Strain Definition Options*, **Mander** is selected. By clicking the **Show Stress-Strain Plot** button in Figure 2.3-7, a plot similar to that shown Figure 2.3-8 is displayed.




**Material Stress-Strain Curve Plot for Material "5200Psi-Column"**
***Figure 2.3-8***

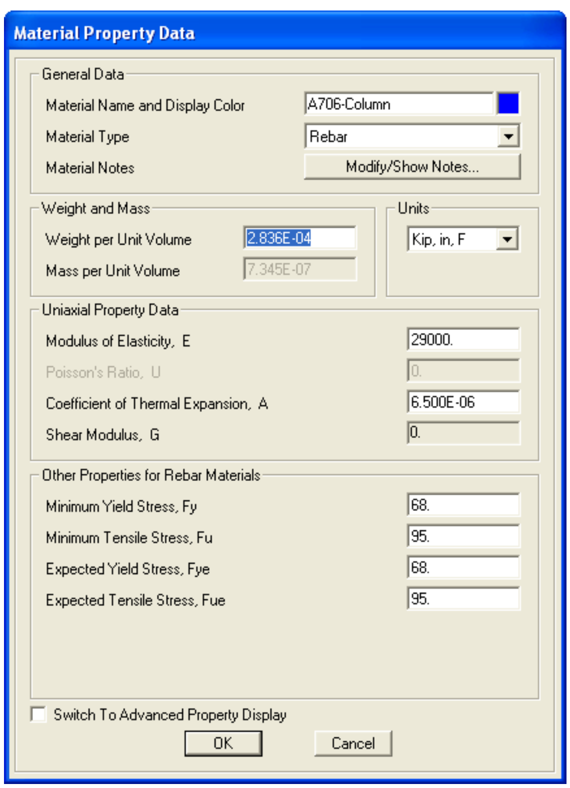
Figure 2.3-8 shows both the confined and unconfined nonlinear stress-strain relationships. The user should verify that the concrete stress-strain curves are as expected.

The Material Property Data for the material "A706-Other" is shown in Figure 2.3-9 (**Define menu** > **Materials** > ***select*** **A706-Other** > ***click*** **Modify/Show Material *button***).



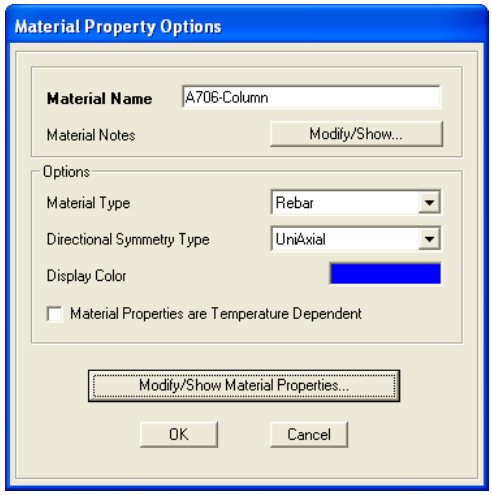

**Material Property Data for Material "A706-Other"**
***Figure 2.3-9***

The Material Property Data for the material "A706-Column" is shown in Figure 2.3-10 (**Define menu > Materials >** ***select*** **A706-Column >** ***click*** **Modify/Show Material** ***button***).



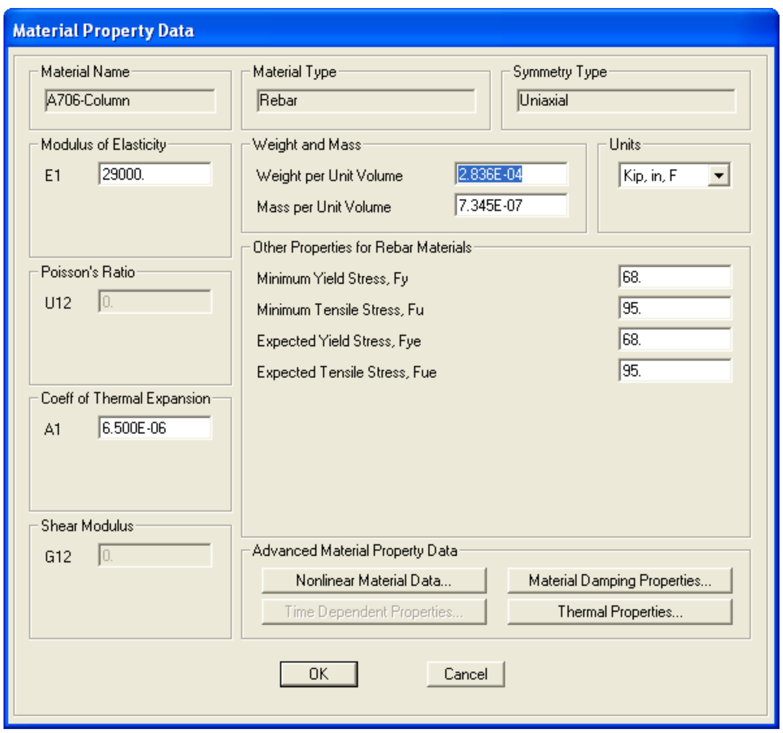

**Material Property Data for Material "A706-Column"**
***Figure 2.3-10***

When the **Switch To Advanced Property Display** box in Figure 2.3-10 is checked, the window shown in Figure 2.3-11 opens.



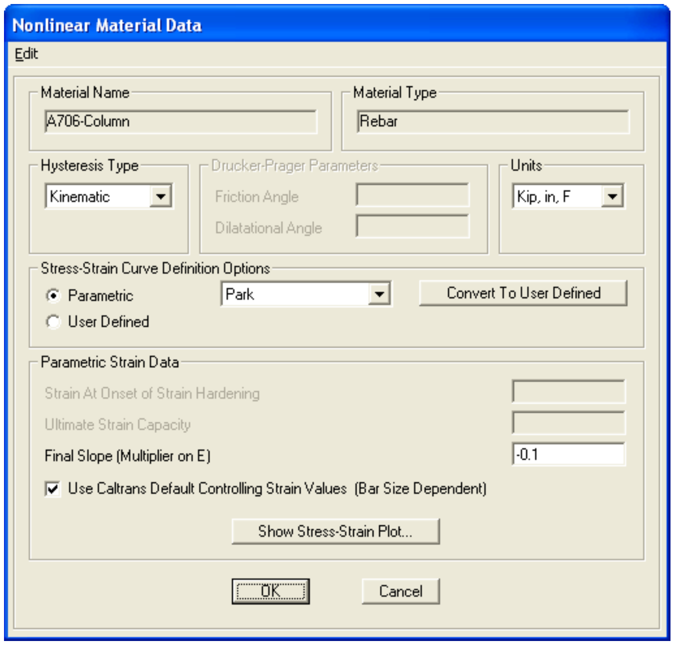

**Advanced Material Property Options for Material "A706-Column"**
***Figure 2.3-11***

By clicking the **Modify/Show Material Properties** button in Figure 2.3-11, the window shown in Figure 2.3-12 opens.



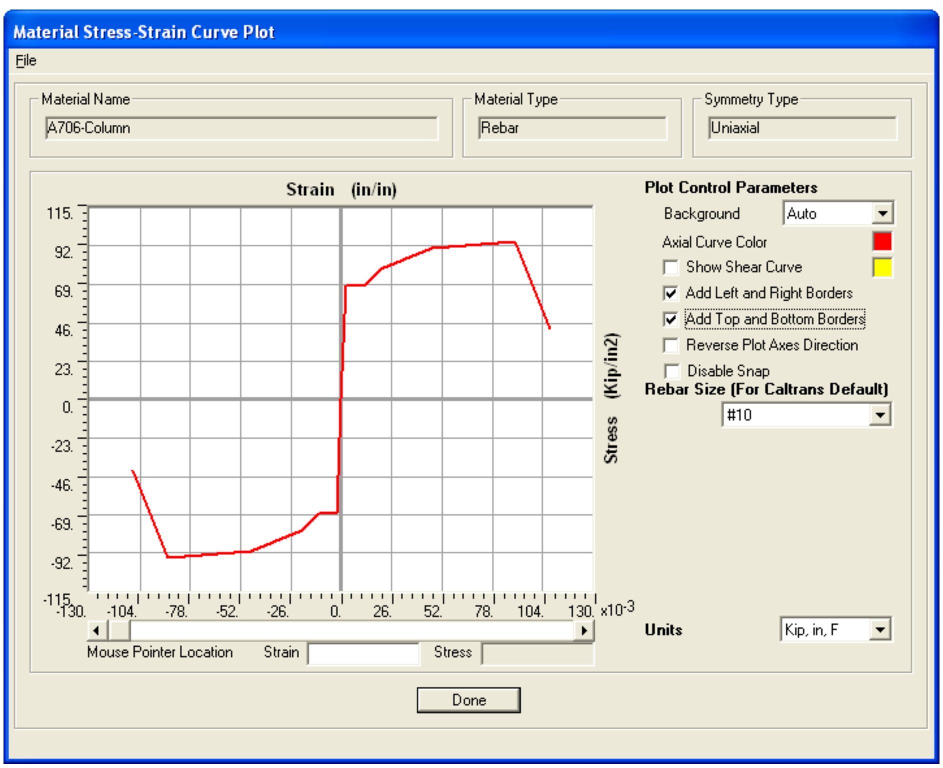

**Advanced Material Property Data for Material "A706-Column"**
***Figure 2.3-12***

In Figure 2.3-12, the *Minimum Yield Stress, Fy* = **68** ksi and the *Minimum Tensile Stress, Fu* = **95** ksi as required per Table 8.4.2-1 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. SAP2000 uses $F_y$ and $F_u$ instead of $F_{ye}$ and $F_{ue}$ to generate the nonlinear stress-strain curve. Therefore, the $F_{ye}$ and $F_{ue}$ inputs in SAP2000 do not serve a purpose for this analysis. By clicking the **Nonlinear Material Data** button in Figure 2.3-12, the window shown in Figure 2.3-13 opens.




**Nonlinear Material Data for Material "A706-Column"**
*Figure 2.3-13*

In Figure 2.3-13, it is seen that under the *Stress-Strain Curve Definitions Options*, **Park** is selected. Also the box for *Use Caltrans Default Controlling Strain Values* is checked. By clicking the **Show Stress-Strain Plot** button in Figure 2.3-13 the plot shown in Figure 2.3-14 is displayed.




**Material Stress-Strain Curve Plot for Material "A706-Column"**
*Figure 2.3-14*

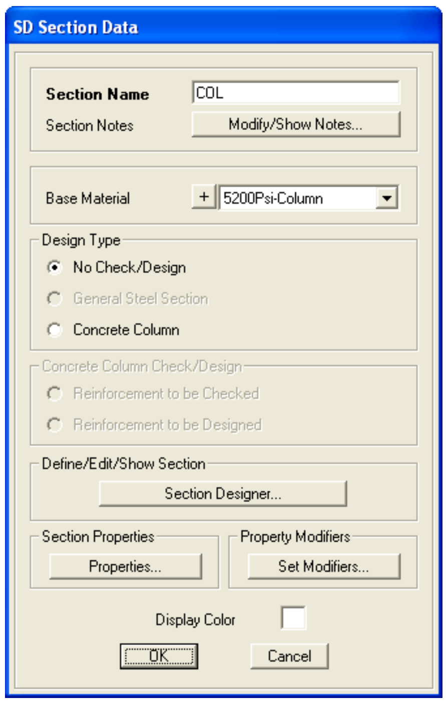
In Figure 2.3-14, the strain at which the stress begins to decrease is $\varepsilon^{R}_{su}$, which the user should verify for correctness.

## 2.4 Column Modeling

There are two columns at each bent. The columns are five feet in diameter and have (24) #10 bars for longitudinal steel, which amounts to a steel-concrete area ratio of about 1%. In the hinge zones, the columns have confinement steel consisting of #6 spiral bars with a 3.5 inch spacing.

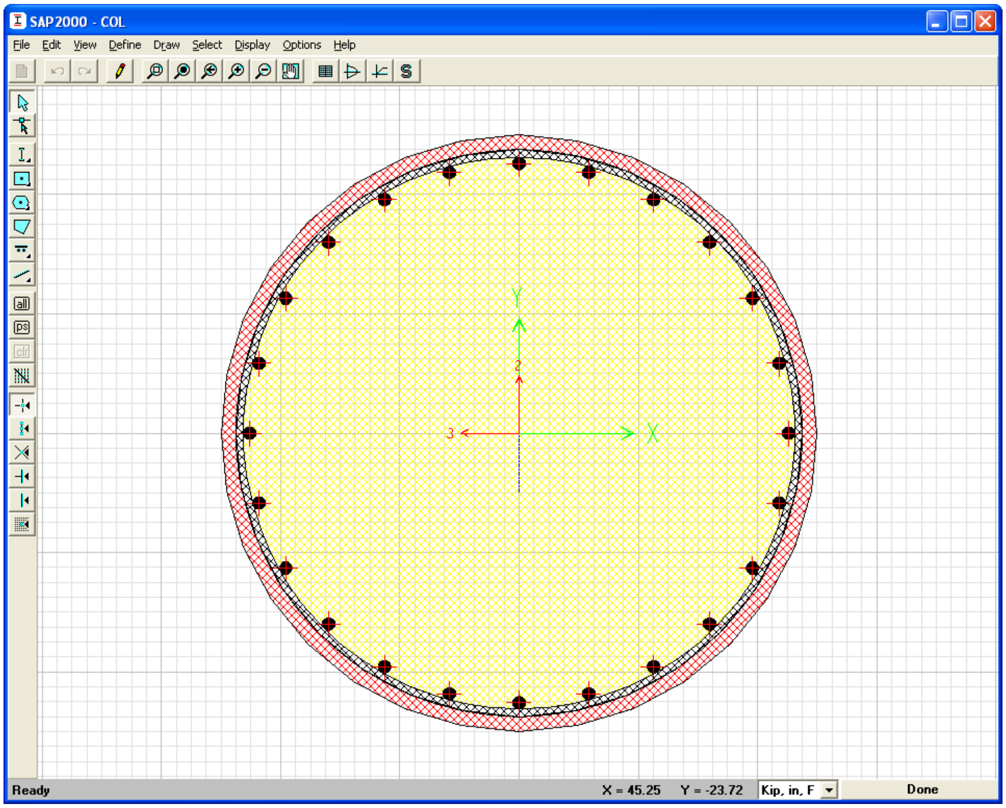
The column elements have rigid end offsets assigned to them at the footings and crossbeams. The net clear height of the columns is 29'-2". The columns are split into three frame elements. Section 5.4.3 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* requires that columns be split into a minimum of three elements.

Figure 2.4-1 shows the frame section property definition for the column elements (**Define menu > Section Properties > Frame Sections >** ***select*** **COL >** ***click*** **Modify/Show Property** ***button***).



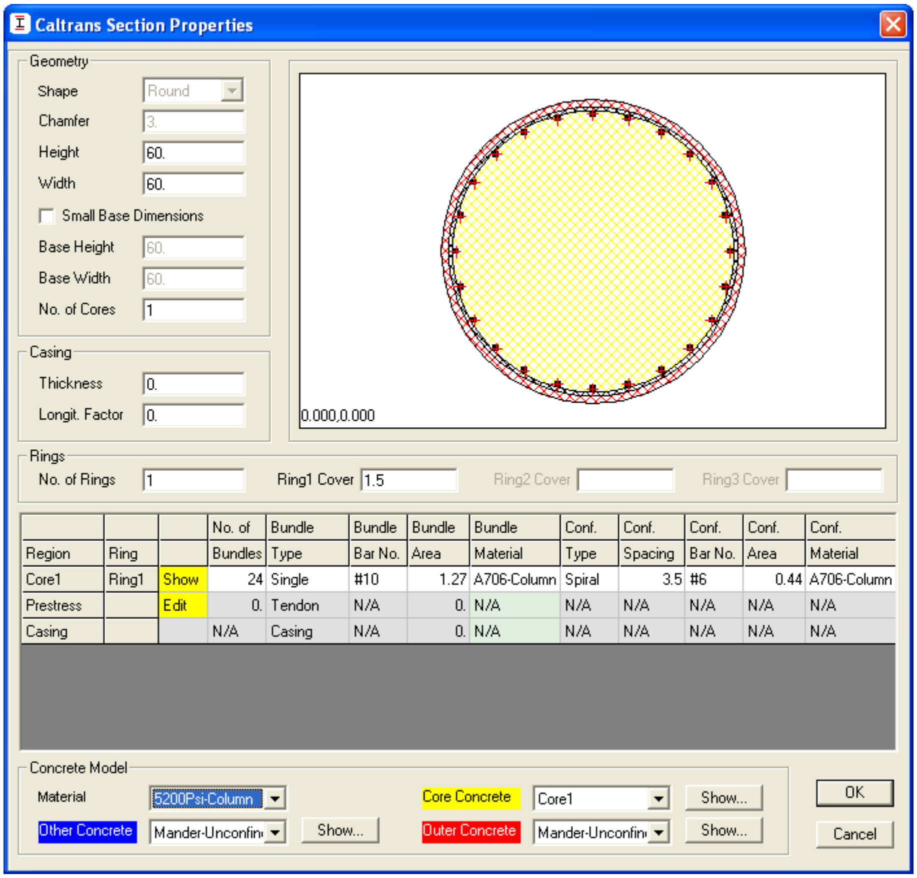

**Frame Section Property Definition for Frame Section "COL"**
***Figure 2.4-1***

By clicking the **Section Designer** button in Figure 2.4-1, the window shown in Figure 2.4-2 opens. The "COL" frame section is defined using a round Caltrans shape in Section Designer as shown in Figure 2.4-2.



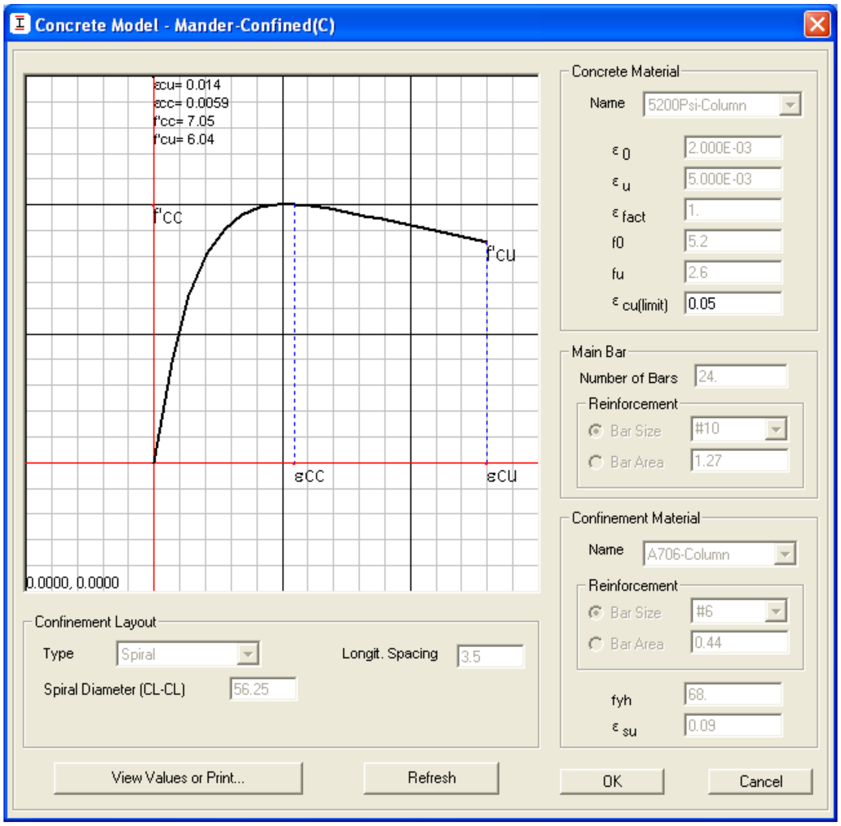

**Section Designer View of Frame Section "COL"**
***Figure 2.4-2***

By right-clicking on the section shown in Figure 2.4-2, the window shown in Figure 2.4-3 opens. Figure 2.4-3 shows the parameter input window for the Caltrans shape is shown in Figure 2.4-2.




**Caltrans Section Properties for Frame Section "COL"**
***Figure 2.4-3***

By clicking the **Show** button for the *Core Concrete* in Figure 2.4-3, the window shown in Figure 2.4-4 opens.

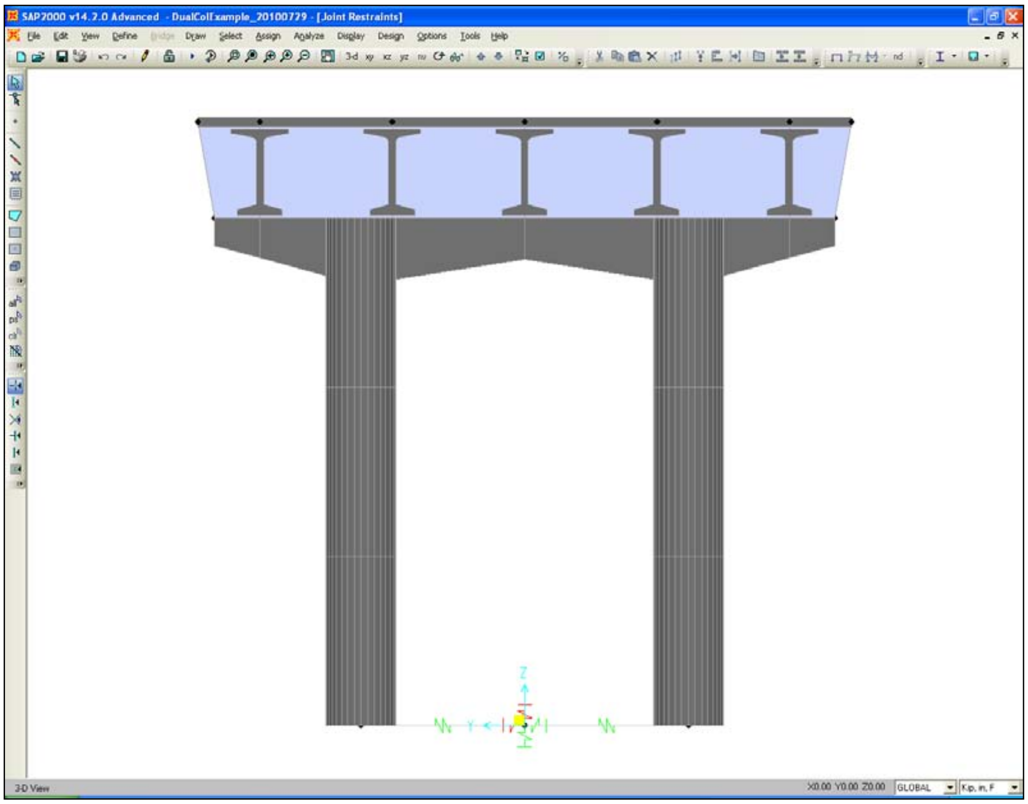



**Concrete Model for Core of Frame Section "COL"**
*Figure 2.4-4*

Figure 2.4-4 shows the Mander confined stress-strain concrete model for the core of the column. The user should verify that the concrete stress-strain curve is as expected.

## 2.5 Crossbeam Modeling

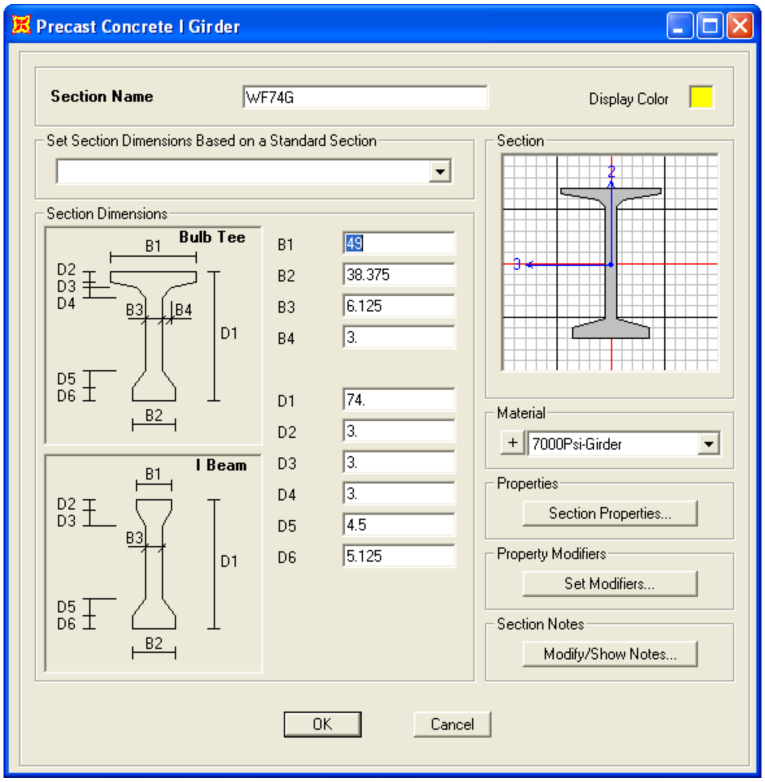
The crossbeams are modeled as frame elements with non-prismatic section properties due to the variable depth of the sections (**Define menu > Section Properties > Frame Sections**). The crossbeam elements have their insertion points set to the top center (**Assign menu > Frame > Insertion Point**). The pier diaphragm above the crossbeam is modeled with shell elements. An extruded view of the bent is shown in Figure 2.5-1.



**Extruded 2-D View of Bent**
*Figure 2.5-1*

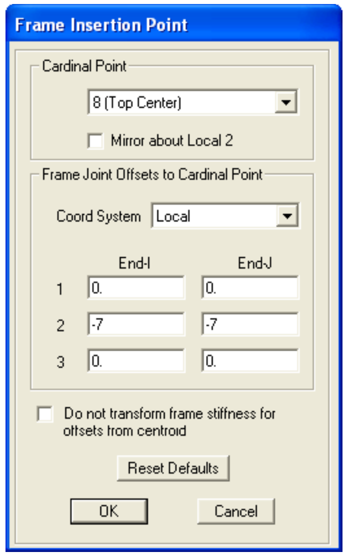
## 2.6 Superstructure Modeling

The girders are Washington State Department of Transportation WF74Gs. The frame section definition for section "WF74G" is shown in Figure 2.6-1 (**Define menu** > **Section Properties** > **Frame Sections** > ***select*** **WF74G** > ***click*** **Modify/Show Property** ***button***).



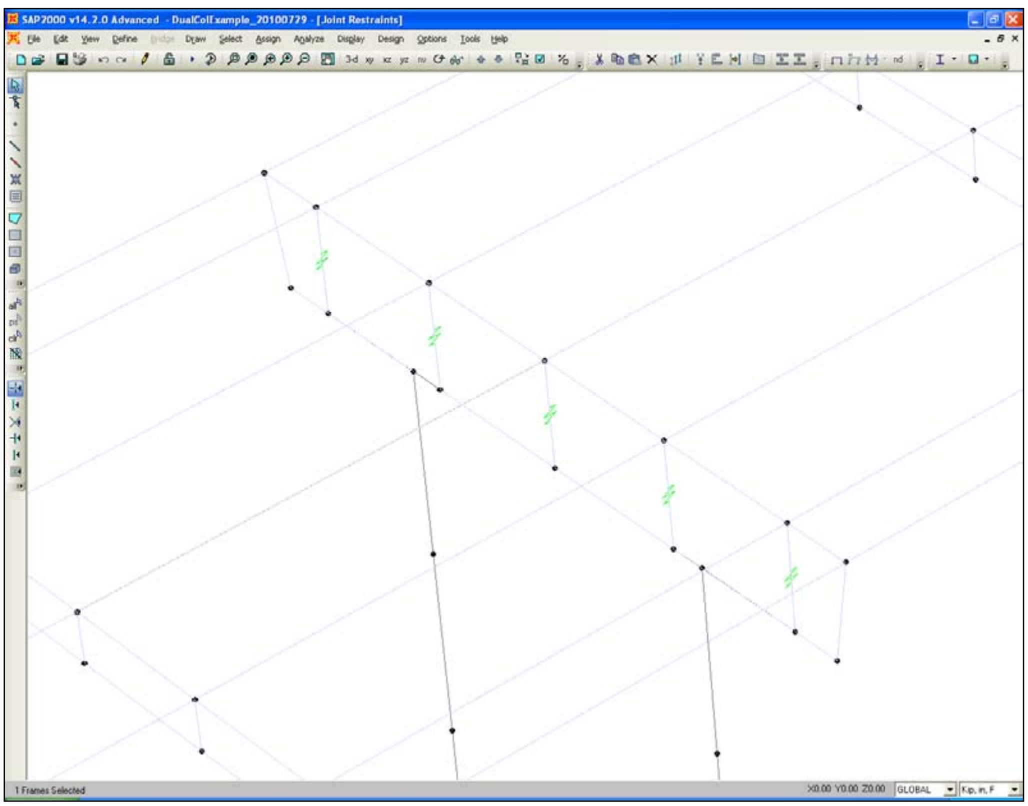

**Frame Section Parameter Input for Frame Section "WF74G"**
***Figure 2.6-1***

The girders are assigned insertion points such that they connect to the same joints as the deck elements but are below the deck. Since the deck is 8 inches thick and the gap between the top of the girder and the soffit of the deck is 3 inches, the insertion point is 7 inches (8 in./2 +3 in.) above the top of the girder. Figure 2.6-2 shows the girder frame element insertion point assignments (**Assign menu** > **Frame** > **Insertion Point**).




**Girder Frame Element Insertion Point Assignments**
*Figure 2.6-2*

Links connect the girders to the crossbeams which models the fixed connection between these elements. See the screen shot shown in Figure 2.6-3.

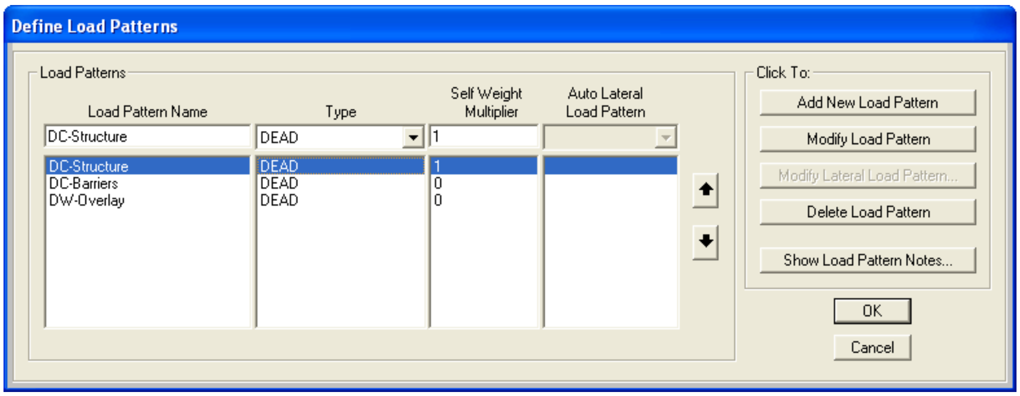



**Wireframe 3-D View of Bent and Superstructure Intersection**
*Figure 2.6-3*

The superstructure is broken into five segments per span. Section 5.4.3 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* requires that a minimum of four segments per span be used.

## 2.7 Gravity Load Patterns

There are three dead load patterns in the model: "DC-Structure", "DC-Barriers", and "DW-Overlay". The "DC-Structure" case includes the self weight of the structural components. The "DC-Barriers" case includes the dead load of the barriers, which is applied as an area load to the outermost deck shells. The "DW-Overlay" case includes the future overlay loads applied to the deck shells. The dead load pattern definitions are shown Figure 2.7-1 (**Define menu > Load Patterns**).




**Dead Load Pattern Definitions**
***Figure 2.7-1***

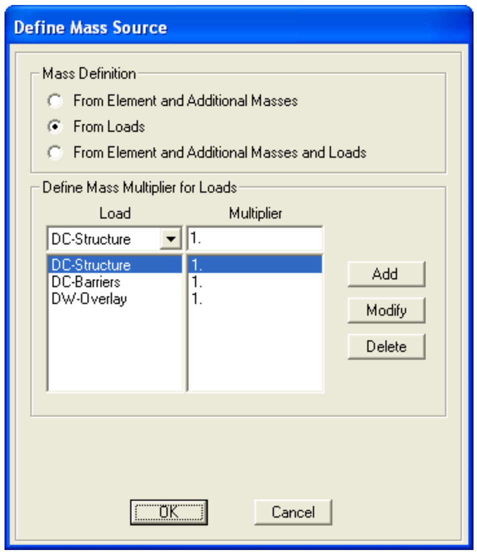
The designer should verify the weight of the structure in the model with hand calculations.

# 3. Displacement Demand Analysis

## 3.1 Modal Analysis

### 3.1.1 Mass Source

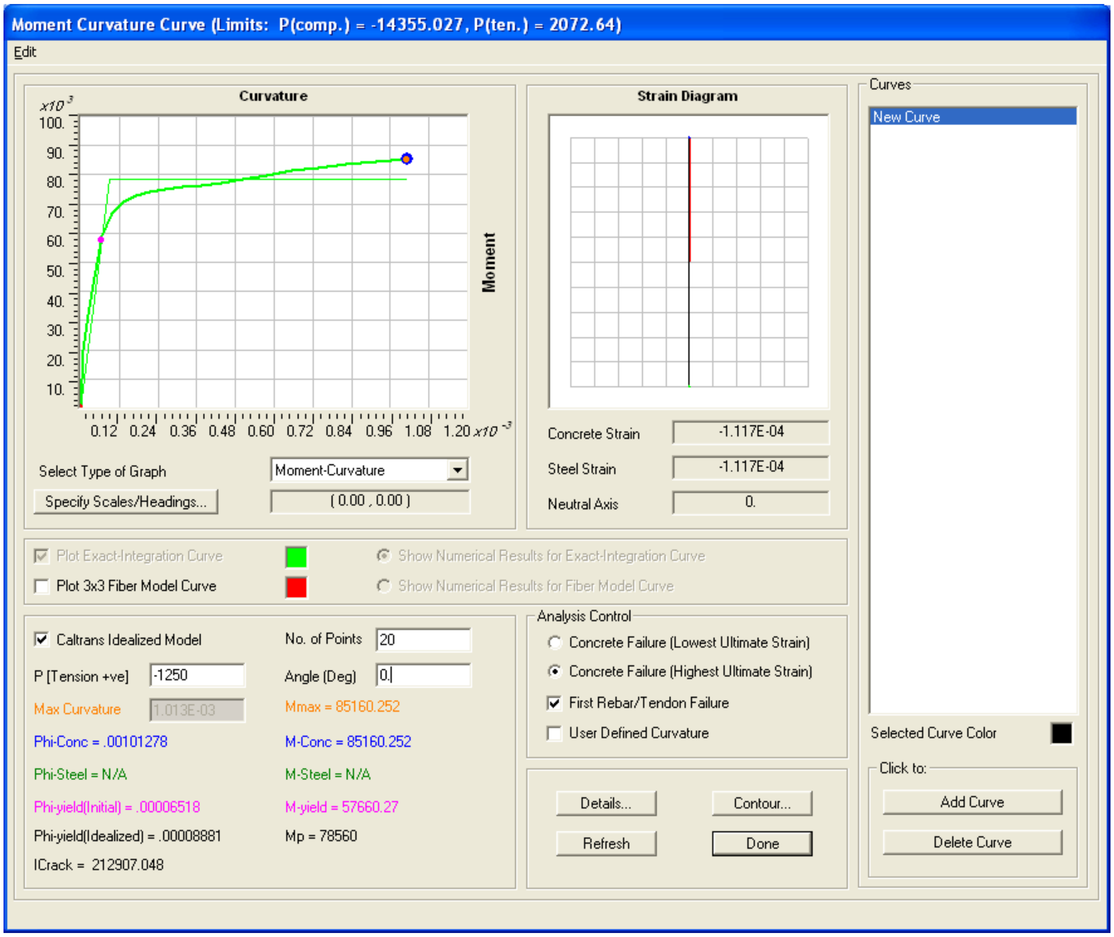
All of the dead loads are considered as contributing mass for the modal load case. A display of the mass source definition window from SAP2000 is shown in Figure 3.1.1-1 (**Define menu > Mass Source**).




**Mass Source Definition**
***Figure 3.1.1-1***

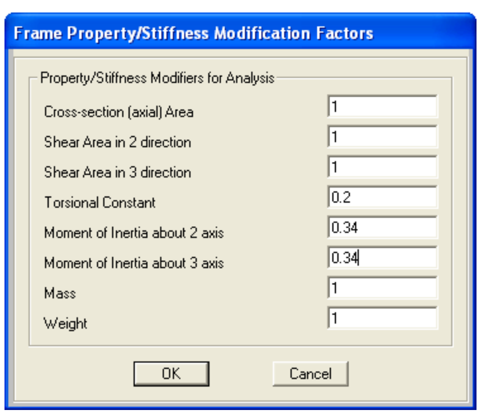
### 3.1.2 Cracking of Columns

Section 5.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* provides diagrams that can be used to determine the cracked section properties of the columns. However, SAP2000's Section Designer can be used to compute the effective section properties. If using Section Designer, the designer should verify that the method of calculation conforms to *AASHTO Guide Specifications of LRFD Seismic Bridge Design*. The column axial dead load at mid-height is approximately 1,250 kips without including the effects of the construction staging. For the bridge in this example, the inclusion of staging effects would cause the axial load in the columns to vary by less than ten percent. Such a small change in axial load would not significantly alter the results of this analysis. However, there are situations where the inclusion of construction sequence effects will significantly alter the analysis. Therefore, engineering judgment should be used when decided whether or not to include the effects of staging. By having Section Designer perform a moment-curvature analysis on the column section with an axial load of 1,250 kips, it is found that ICrack = 212,907 inch$^4$. The gross moment of inertia is 628,044 inch$^4$ (as calculated by SAP2000). Therefore, the ratio is 212,907/628,044 = 0.34. The moment-curvature analysis is shown in Figure 3.1.2-1 (**Define menu > Section Properties > Frame Sections >** ***select*** **COL >** ***click*** **Modify/Show Property** ***button*** **>** ***click*** **Section Designer** ***button*** **> Display menu > Show Moment-Curvature Curve**).




**Moment Curvature Curve for Frame Section "COL" at P = -1250 kips**
***Figure 3.1.2-1***

It can be seen in Figure 3.1.2-1 that concrete strain capacity limits the available plastic curvature. Designers should verify that SAP2000's bilinearization is acceptable. The property modifiers are then applied to the column frame elements as shown in Figure 3.1.2-2 (**Assign menu** > **Frame** > **Property Modifiers**).

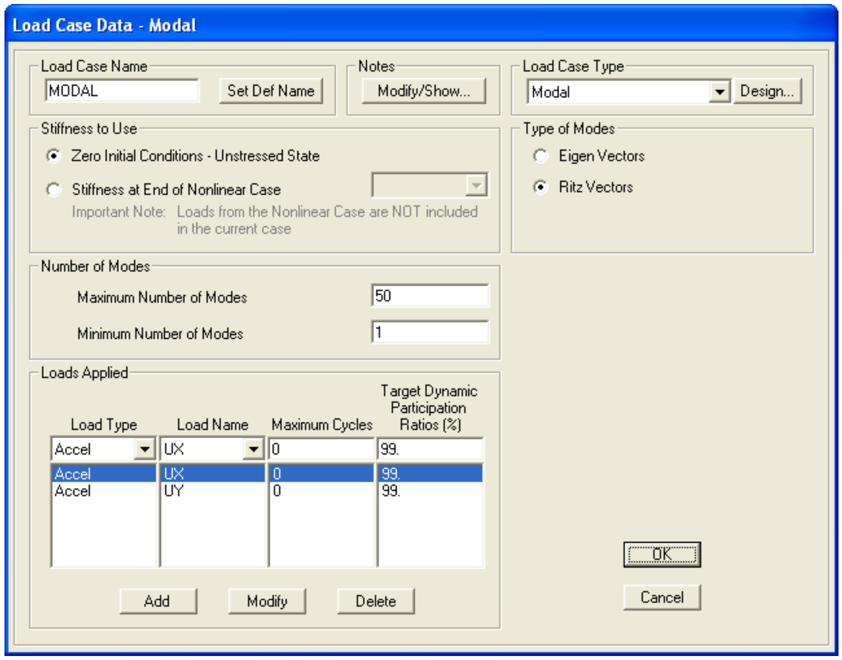



**Frame Property Modification Factor for Column Frame Elements**
***Figure 3.1.2-2***

The torsional constant modifier is 0.2 for columns as required by Section 5.6.5 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

### 3.1.3 Load Case Setup

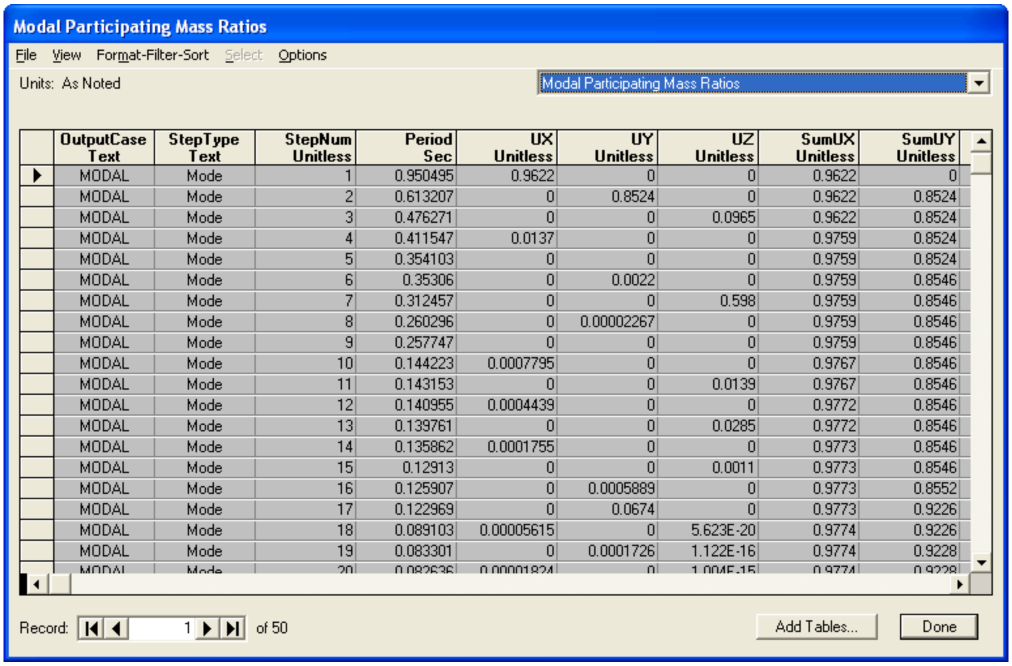
The "MODAL" load case uses Ritz vectors and is defined in SAP2000 as shown in Figure 3.1.3-1 (**Define menu > Load Cases >** ***select*** **MODAL >** ***click*** **Modify/Show Load Case** ***button***).




**Load Case Data for Load Case "MODAL"**
***Figure 3.1.3-1***

### 3.1.4 Verification of Mass Participation

Section 5.4.3 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* requires a minimum of 90% mass participation in both directions. For this example, the mass is considered to be the same in both directions even though the end diaphragms are free in the longitudinal direction and restrained in the transverse direction. By displaying the *Modal Participating Mass Ratios* table for the "MODAL" load case it is found that the X-direction (longitudinal) reaches greater than 90% mass participation on the first mode shape, while the Y-direction (transverse) reaches greater than 90% mass participation by the seventeenth mode shape. This implies that the minimum code requirements could be met by including only seventeen mode shapes. The *Modal Participating Mass Ratios* table is shown in Figure 3.1.4-1 (**Display menu > Show Tables >** ***check*** **Modal Participating Mass Ratios >** ***click*** **OK** ***button***).



Modal Participating Mass Ratios

File View Format-Filter-Sort Select Options

Units: As Noted

Modal Participating Mass Ratios

| OutputCase Text | StepType Text | StepNum Unitless | Period Sec | UX Unitless | UY Unitless | UZ Unitless | SumUX Unitless | SumUY Unitless |
|---|---|---|---|---|---|---|---|---|
| MODAL | Mode | 1 | 0.950495 | 0.9622 | 0 | 0 | 0.9622 | 0 |
| MODAL | Mode | 2 | 0.613207 | 0 | 0.8524 | 0 | 0.9622 | 0.8524 |
| MODAL | Mode | 3 | 0.476271 | 0 | 0 | 0.0965 | 0.9622 | 0.8524 |
| MODAL | Mode | 4 | 0.411547 | 0.0137 | 0 | 0 | 0.9759 | 0.8524 |
| MODAL | Mode | 5 | 0.354103 | 0 | 0 | 0 | 0.9759 | 0.8524 |
| MODAL | Mode | 6 | 0.35306 | 0 | 0.0022 | 0 | 0.9759 | 0.8546 |
| MODAL | Mode | 7 | 0.312457 | 0 | 0 | 0.598 | 0.9759 | 0.8546 |
| MODAL | Mode | 8 | 0.260296 | 0 | 0.00002267 | 0 | 0.9759 | 0.8546 |
| MODAL | Mode | 9 | 0.257747 | 0 | 0 | 0 | 0.9759 | 0.8546 |
| MODAL | Mode | 10 | 0.144223 | 0.0007795 | 0 | 0 | 0.9767 | 0.8546 |
| MODAL | Mode | 11 | 0.143153 | 0 | 0 | 0.0139 | 0.9767 | 0.8546 |
| MODAL | Mode | 12 | 0.140955 | 0.0004439 | 0 | 0 | 0.9772 | 0.8546 |
| MODAL | Mode | 13 | 0.139761 | 0 | 0 | 0.0285 | 0.9772 | 0.8546 |
| MODAL | Mode | 14 | 0.135862 | 0.0001755 | 0 | 0 | 0.9773 | 0.8546 |
| MODAL | Mode | 15 | 0.12913 | 0 | 0 | 0.0011 | 0.9773 | 0.8546 |
| MODAL | Mode | 16 | 0.125907 | 0 | 0.0005889 | 0 | 0.9773 | 0.8552 |
| MODAL | Mode | 17 | 0.122969 | 0 | 0.0674 | 0 | 0.9773 | 0.9226 |
| MODAL | Mode | 18 | 0.089103 | 0.00005615 | 0 | 5.623E-20 | 0.9774 | 0.9226 |
| MODAL | Mode | 19 | 0.083301 | 0 | 0.0001726 | 1.122E-16 | 0.9774 | 0.9228 |
| MODAL | Mode | 20 | 0.082636 | 0.00001824 | 0 | 1.004E-15 | 0.9774 | 0.9228 |

Record: 1 of 50 Add Tables... Done

**Modal Participating Mass Ratios for Load Case "MODAL"**
***Figure 3.1.4-1***

Figure 3.1.4-1 also shows that the first mode in the X-direction (longitudinal) has a period of 0.95 seconds and the first mode in the Y-direction (transverse) has period of 0.61 seconds. The designer should verify fundamental periods with hand calculations. The designer should also visually review the primary mode shapes to verify they represent realistic behavior.

# 3.2 Response-Spectrum Analysis

## 3.2.1 Seismic Hazard

The bridge is located in Redmond, Wash. The mapped spectral acceleration coefficients are:

$PGA = 0.396$ g
$S_s = 0.883$ g
$S_1 = 0.294$ g

A site class of E is assumed for this example and the site coefficients are:

$F_{PGA} = 0.91$
$F_a = 1.04$
$F_v = 2.82$

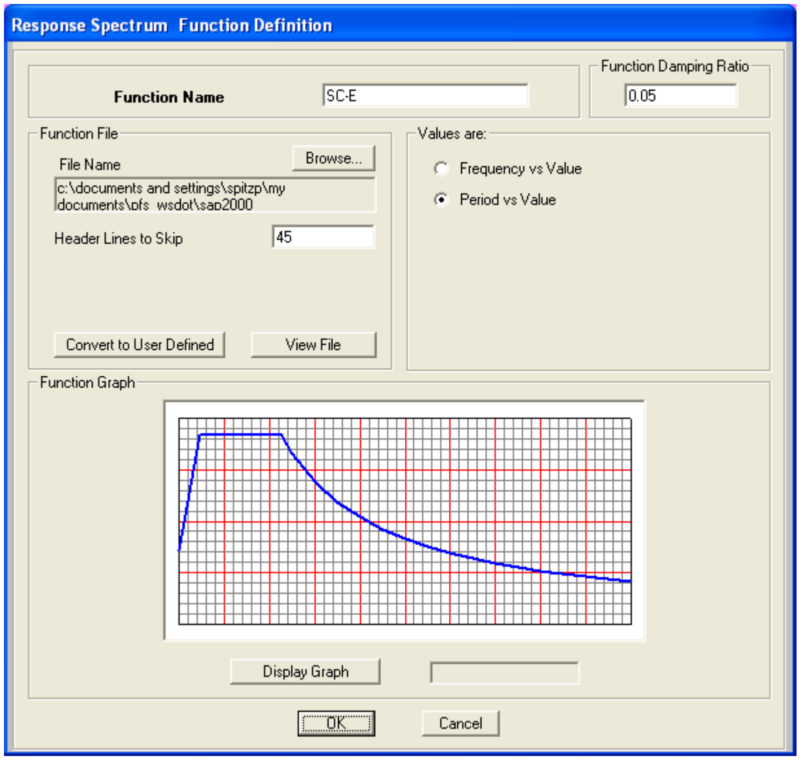
Therefore, the response-spectrum is generated using the following parameters:

$A_s = F_{PGA}*PGA = 0.361$ g
$S_{DS} = F_a*S_s = 0.919$ g
$S_{D1} = F_v*S_1 = 0.830$ g

Since $S_{D1}$ is greater than or equal to 0.50, per Table 3.5-1 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* the Seismic Design Category is D.

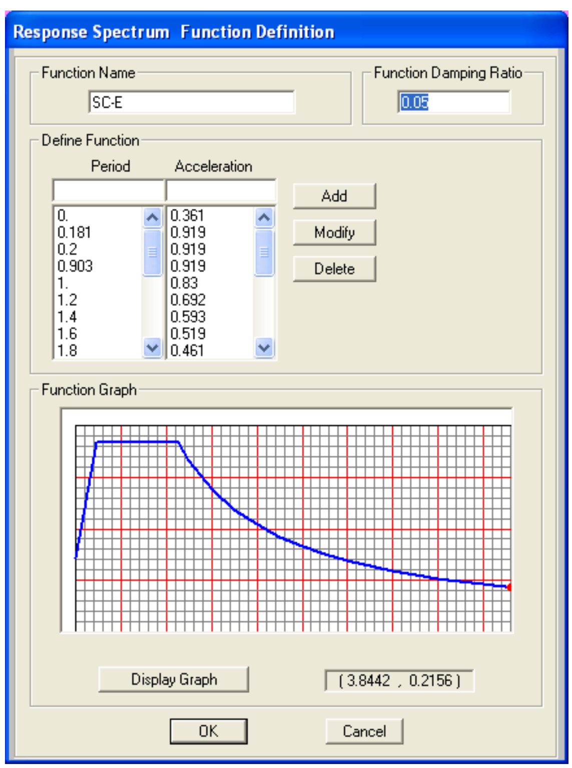
## 3.2.2 Response-Spectrum Input

The spectrum is defined from a file created using the AASHTO Earthquake Ground Motion Parameters tool. A screen shot of the response-spectrum as inputted in SAP2000 is shown in Figure 3.2.2-1 (**Define menu > Functions > Response Spectrum >** ***select*** **SC-E >** ***click*** **Show Spectrum** ***button***).




**Response Spectrum Function Definition from File for Function "SC-E"**
***Figure 3.2.2-1***

When the **Convert to User Defined** button is clicked, the function appears as shown in Figure 3.2.2-2.




**Response Spectrum Function Definition for Function "SC-E"**
***Figure 3.2.2-2***

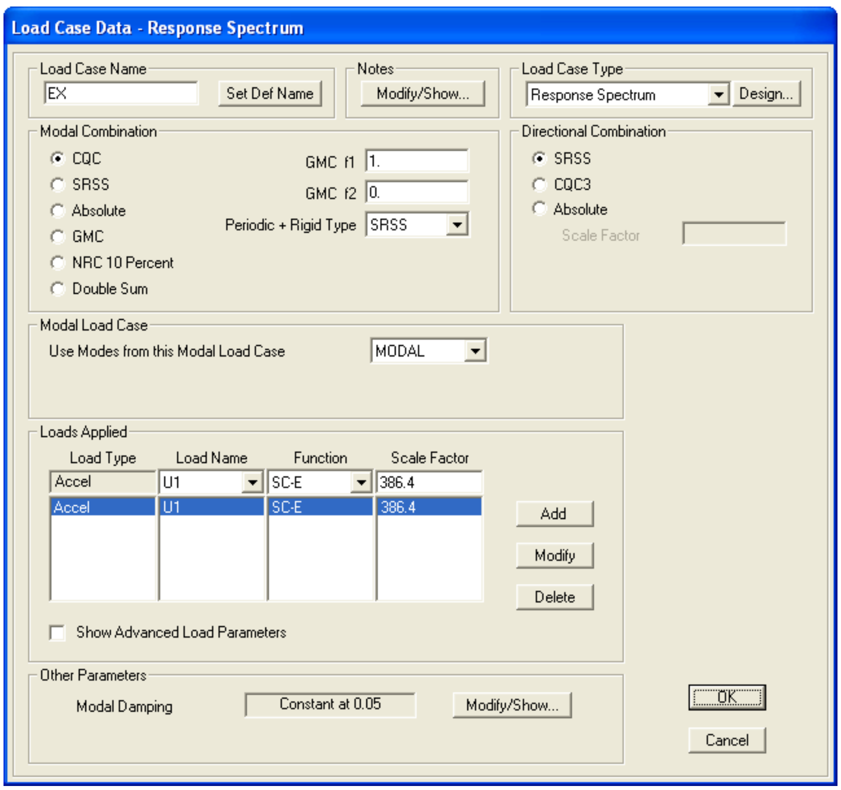
Having the response-spectrum function stored as “User Defined” is advantageous because the data is stored within the .SDB file. Therefore, if the .SDB file is transferred to a different location (different computer), the response-spectrum function will also be moved.

## 3.2.3 Load Case Setup

Two response-spectrum analysis cases are setup in SAP2000: one for each orthogonal direction.

### 3.2.3.1 Longitudinal Direction

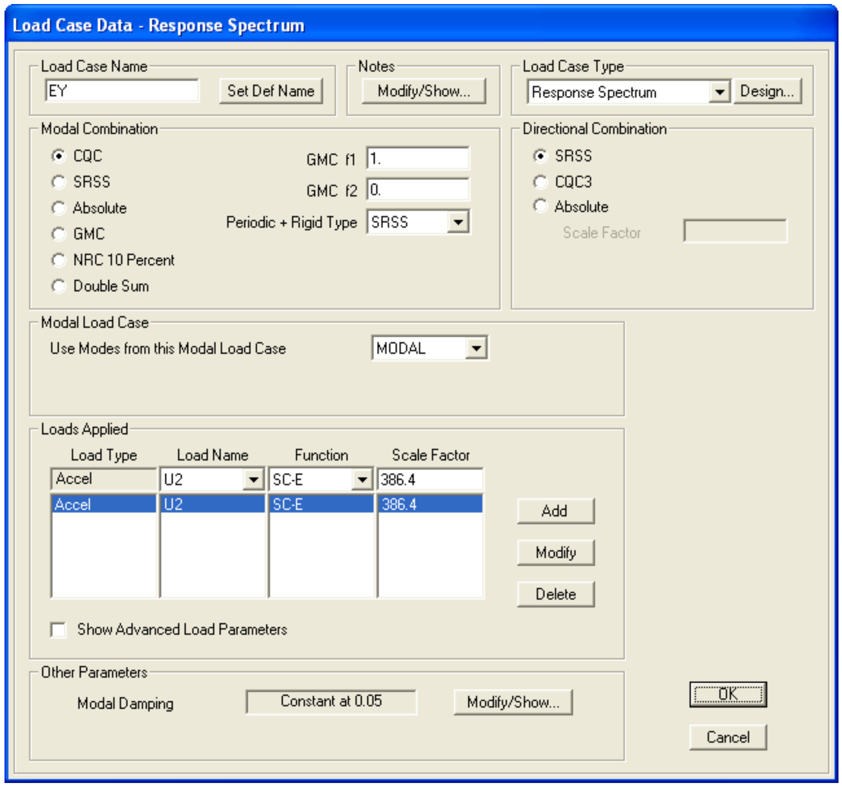
The load case data for the X-direction is shown in Figure 3.2.3.1-1 (**Define menu > Load Cases > *select* EX > *click* Modify/Show Load Case *button***).




**Load Case Data for Load Case “EX”**
***Figure 3.2.3.1-1***

### 3.2.3.2 Transverse Direction

The load case data for the Y-direction is shown Figure 3.2.3.2-1 (**Define menu > Load Cases > *select* EY > *click* Modify/Show Load Case *button***).



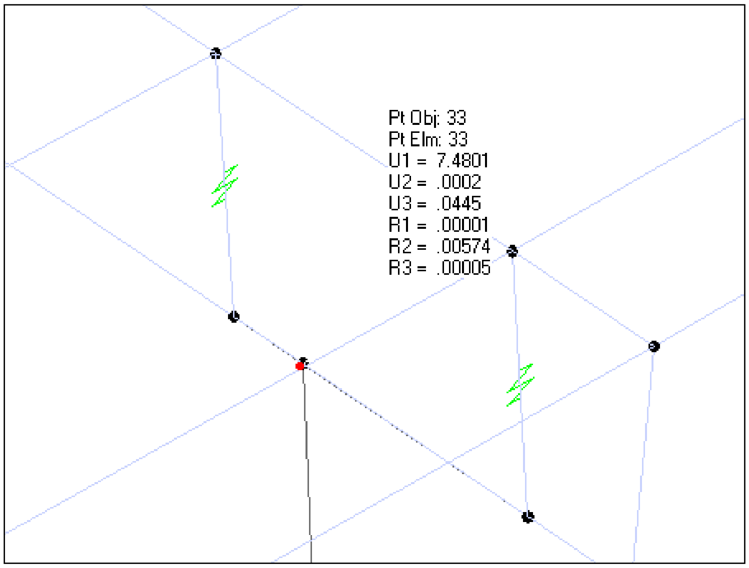

**Load Case Data for Load Case "EY"**
***Figure 3.2.3.2-1***

## 3.2.4 Response-Spectrum Displacements

The column displacements in this example are tracked at Joint 33, which is located at the top of a column. Since the bridge is symmetric, all of the columns have the same displacements in the response-spectrum analyses.

### 3.2.4.1 Longitudinal Direction

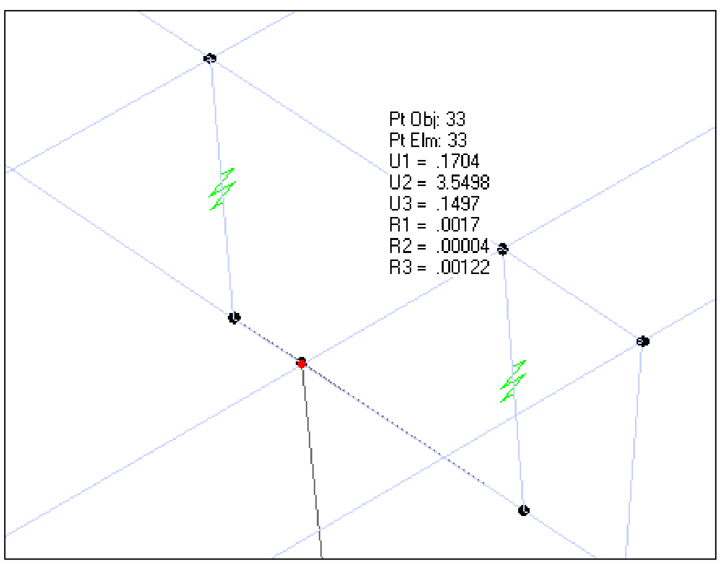
The horizontal displacements at the tops of the columns from the EX analysis case are U1 = 7.48 inches and U2 = 0.00 inches. This is shown in Figure 3.2.4.1-1 as displayed in SAP2000 (**Display menu > Show Deformed Shape >** ***select*** **EX >** ***click*** **OK** ***button***).




**Joint Displacement at Joint 33 for Load Case "EX"**
*Figure 3.2.4.1-1*

### 3.2.4.2 Transverse Direction

The horizontal displacements at the tops of the columns from the EY analysis case are U1 = 0.17 inches and U2 = 3.55 inches. This is shown in Figure 3.2.4.2-1 as displayed in SAP2000 (**Display menu > Show Deformed Shape >** ***select*** **EY >** ***click*** **OK** ***button***).




**Joint Displacement at Joint 33 for Load Case "EY"**
*Figure 3.2.4.2-1*

# 3.3 Displacement Demand

## 3.3.1 Displacement Magnification

Displacement magnification must be performed in accordance with Section 4.3.3 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

Compute $T_s$ and $T^*$:

$T_s$ $= S_{D1} / S_{DS}$
$= 0.830 / 0.919$
$= 0.903$ sec.

$T^*$ $= 1.25\ T_s$
$= 1.25 * 0.903$
$= 1.13$ sec.

### 3.3.1.1 Longitudinal Direction

Compute magnification for the X-direction (Longitudinal):

$T_{Long}$ $= 0.95$ sec. (see section 3.1.4)

$T^* / T_{Long}$ $= 1.13 / 0.95$
$= 1.19 > 1.0 \Rightarrow$ Magnification is required

$R_{d\_Long}$ $= (1 – 1/\mu_D) * (T^* / T) + 1 / \mu_D$
$= (1 – 1/6) * (1.19) + 1/6$ (Assume $\mu_D = 6$)
$= 1.16$

### 3.3.1.2 Transverse Direction

Compute magnification for the Y-direction (Transverse):

$T_{Trans}$ $= 0.61$ sec. (see section 3.1.4)

$T^* / T_{Trans}$ $= 1.13 / 0.61$
$= 1.85 > 1.0 \Rightarrow$ Magnification is required

$R_{d\_Trans}$ $= (1 – 1/\mu_D) * (T^* / T) + 1 / \mu_D$
$= (1 – 1/6) * (1.85) + 1/6$ (Assume $\mu_D = 6$)
$= 1.71$

## 3.3.2 Column Displacement Demand

Section 4.4 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* requires that 100% plus 30% of the displacements from each orthogonal seismic load case be combined to determine the displacement demands. The displacements are tracked as Joint 33, which is located at the top of a column.

### 3.3.1.1 Longitudinal Direction

For the X-direction (100EX + 30EY):

UX (due to EX) = 7.48 in.

UX (due to EY) = 0.17 in.

Therefore,

$$\Delta^{L}_{D\_Long} = 1.0 * R_{d\_Long} * 7.48 + 0.3 * R_{d\_Trans} * 0.17$$
$$= 1.0 * 1.16 * 7.48 + 0.3 * 1.71 * 0.17$$
= 8.76 in. => This is the displacement demand for the X-Dir

### 3.3.1.2 Transverse Direction

For the Y-direction (100EY + 30EX):

UY (due to EY) = 3.55 in.

UY (due to EX) = 0.00 in.

Therefore,

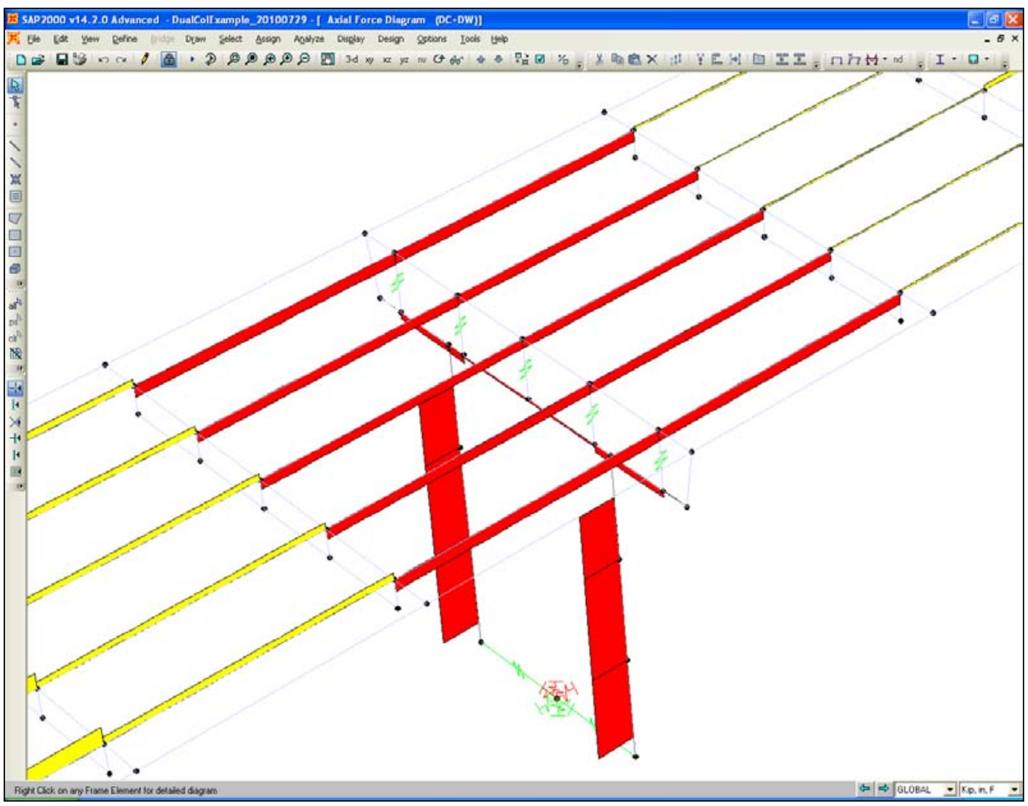
$$\Delta^{L}_{D\_Trans} = 1.0 * R_{d\_Trans} * 3.55 + 0.3 * R_{d\_Long} * 0.00$$
$$= 1.0 * 1.71 * 3.55 + 0.3 * 1.16 * 0.00$$
= 6.07 in. => This is the Displacement Demand for the Y-Dir

# 4. Displacement Capacity Analysis

## 4.1 Plastic Hinge Definitions and Assignments

### 4.1.1 Column Inflection Points

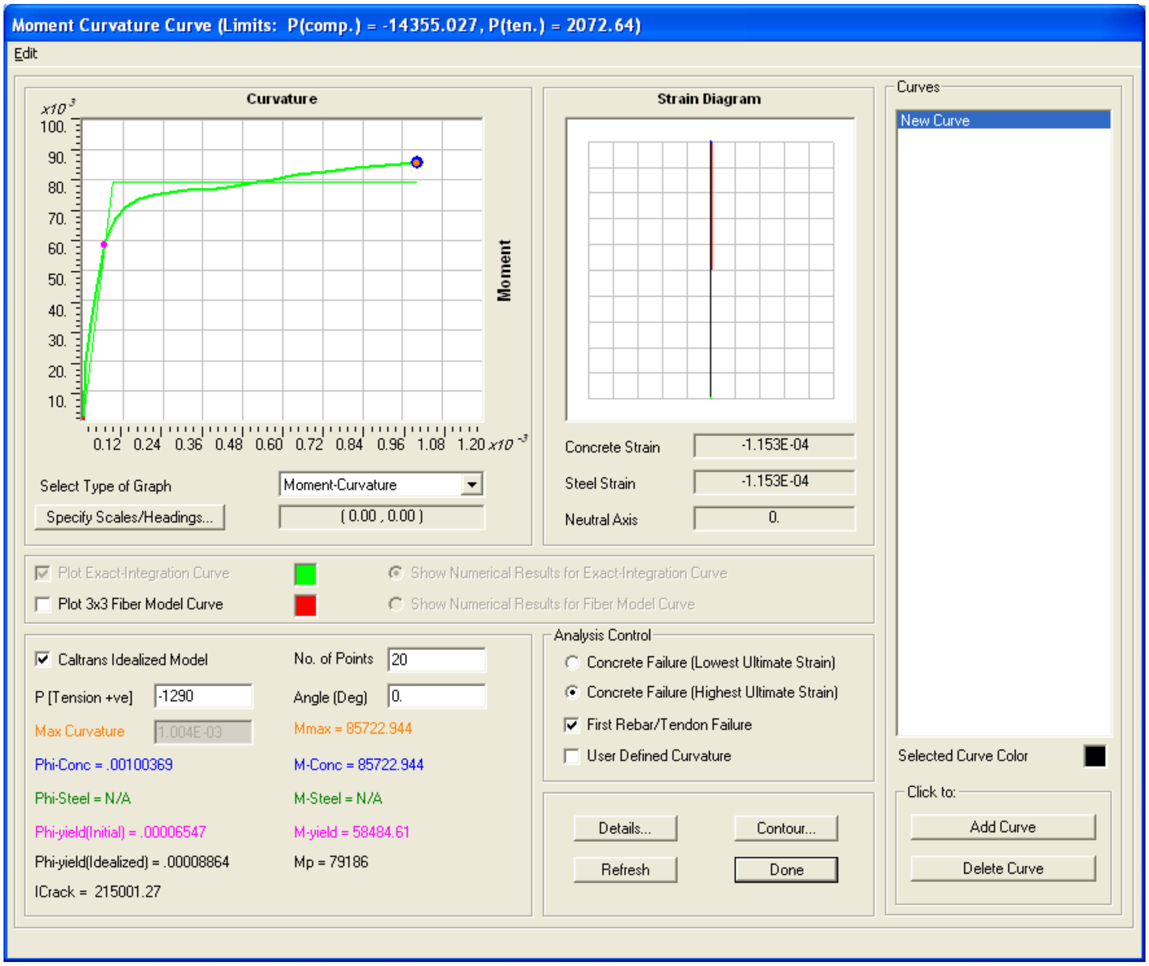
The tops and bottoms of all columns have enough moment fixity in all directions to cause plastic hinging, which means the columns will exhibit behavior similar to a fixed-fixed column. The plastic moment capacities of the columns under dead loads will be used to approximate the location of the column inflection points. Therefore, the axial loads (due to dead load) at the top and bottom of the columns must be determined. Due to the symmetry of the bridge in this example, the axial loads are the same for all of the columns, which will not be true for most bridges. Figure 4.1.1-1 shows the axial force diagram for the DC+DW load case as displayed in SAP2000 (**Display menu > Show Forces/Stresses > Frames/Cables >** ***select*** **DC+DW >** ***select*** **Axial Force >** ***click*** **OK** ***button***).



**Frame Axial Force Diagram for Load Case "DC+DW"**
***Figure 4.1.1-1***

From the axial loads displayed for the DC+DW load case it is determined that the axial force at the bottom of the column is approximately 1,290 kips and the axial force at the top of the column is approximately 1,210 kips (see section 3.1.2 of this example for a discussion on the inclusion of construction sequence effects on column axial loads). It is expected that the difference in axial load between the tops and bottoms of the columns will not result in a significant difference in the plastic moment. However, on some bridges the axial loads at the tops and bottoms of the columns may be substantially different or the column section may vary along its height producing significantly different plastic moments at each end.

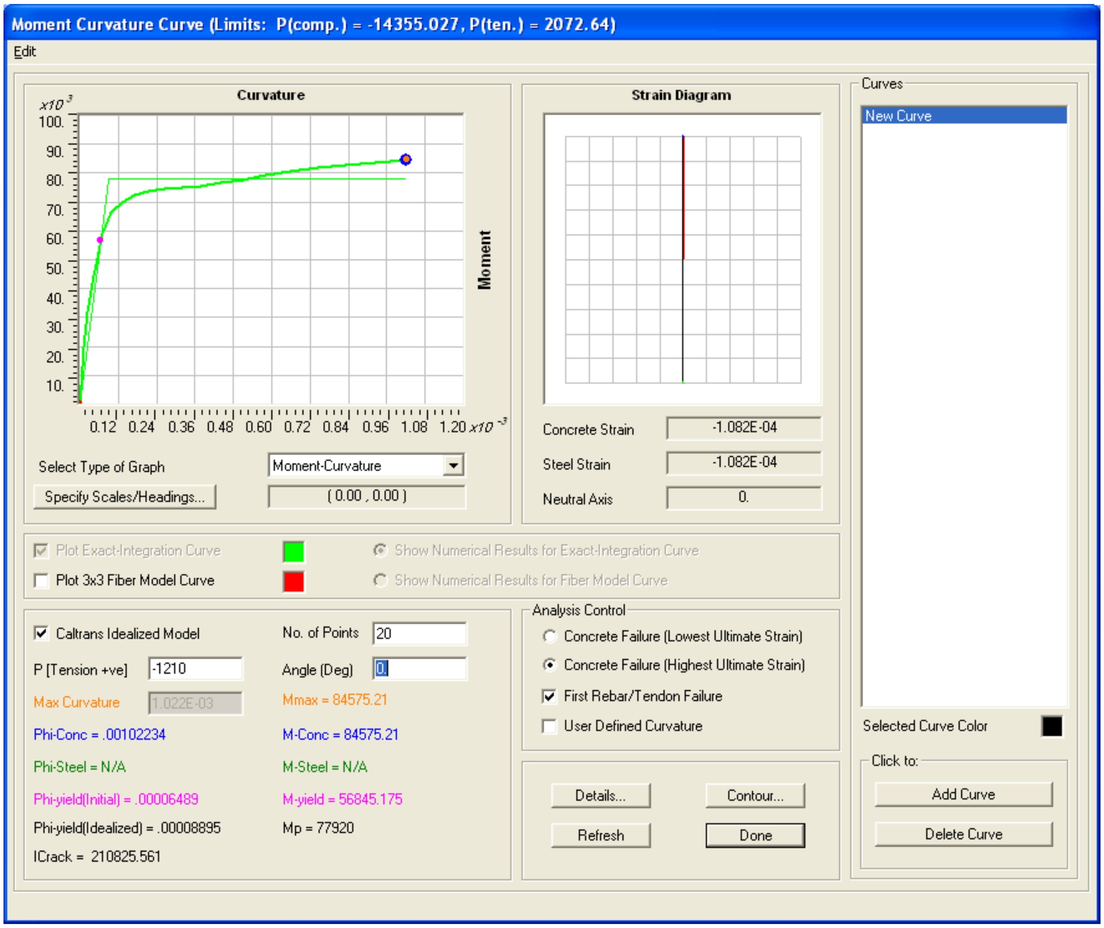
The moment-curvature analysis of the column base is shown in Figure 4.1.1-2 (**Define menu** > **Section Properties** > **Frame Sections** > ***select*** **COL** > ***click*** **Modify/Show Property *button*** > ***click*** **Section Designer *button*** > **Display menu** > **Show Moment-Curvature Curve**).




**Moment Curvature Curve for Frame Section "COL" at P = -1290 kips**
***Figure 4.1.1-2***

It is seen in Figure 4.1.1-2 that the plastic moment capacity at the base of the column is 79,186 kip-inches (with only dead load applied).

The moment-curvature analysis of the column top is shown in Figure 4.1.1-3 (**Define menu** > **Section Properties** > **Frame Sections** > ***select*** **COL** > ***click*** **Modify/Show Property *button*** > ***click*** **Section Designer *button*** > **Display menu** > **Show Moment-Curvature Curve**).




**Moment Curvature Curve for Frame Section "COL" at P = -1210 kips**
***Figure 4.1.1-3***

It is seen in Figure 4.1.1-3 that the plastic moment capacity at the top of the column is 77,920 kip-inches (with only dead load applied).

The clear height of the columns is 350 inches; therefore:

$L_1$ = Length from point of maximum moment at base of column to inflection point (in.)
$= 350 \times M_{p\_col\_base} / (M_{p\_col\_base} + M_{p\_col\_top})$
$= 350 \times 79186 / (79186 + 77920)$
$= 176$ in.

$L_2$ = Length from point of maximum moment at top of column to inflection point (in.)
$= 350 - L_1$
$= 350 - 176$
$= 174$ in.

## 4.1.2 Plastic Hinge Lengths

The plastic hinge lengths must be computed at both the tops and bottoms of the columns using the equations in Section 4.11.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. The hinge length is computed as follows:

$$L_p = 0.08L + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl}$$

Where:

$L$ = length of column from point of maximum moment to the point of moment contraflexure (in.)
= $L_1$ at the base of the columns ($L_{1Long} = L_{1Trans} = 176$ in.)
= $L_2$ at the top of the columns ($L_{2Long} = L_{2Trans} = 174$ in.)

$f_{ye}$ = expected yield strength of longitudinal column reinforcing steel bars (ksi)
= 68 ksi (ASTM A706 bars).

$d_{bl}$ = nominal diameter of longitudinal column reinforcing steel bars (in.)
= 1.27 in. (#10 bars)

$L_{p1}$ = Plastic hinge length at base of column
= $0.08*176 + 0.15*68*1.27 \geq 0.3*68*1.27$
= $27.03 \geq 25.91$
= 27.0 in.

$L_{p2}$ = Plastic hinge length at top of column
= $0.08*174 + 0.15*68*1.27 \geq 0.3*68*1.27$
= $26.87 \geq 25.91$
= 26.9 in.

In this example, the plastic hinge lengths in both directions are the same because the locations of the inflection points in both directions are the same. This will not always be the case, such as when there is a single column bent.

### 4.1.3 Assign Plastic Hinges

In order to assign the plastic hinges to the column elements, the relative locations of the plastic hinges along the column frame elements must be computed.

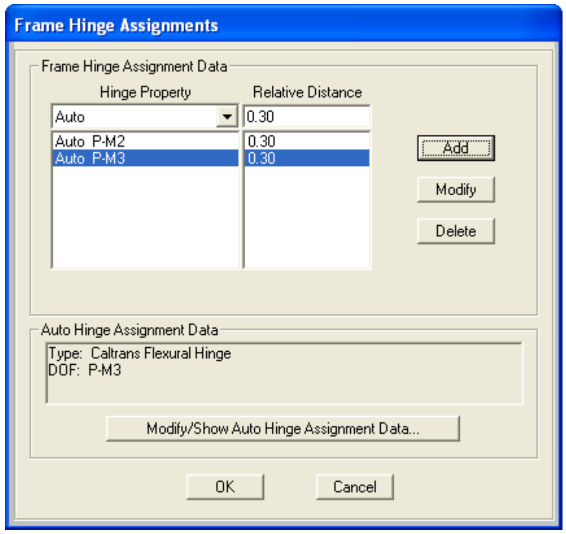
For the bases of the columns:

Relative Length = [Footing Offset + (Hinge Length / 2)] / Element Length
= [30 + (27.0 / 2)] / 146
= 0.30

For the tops of the columns:

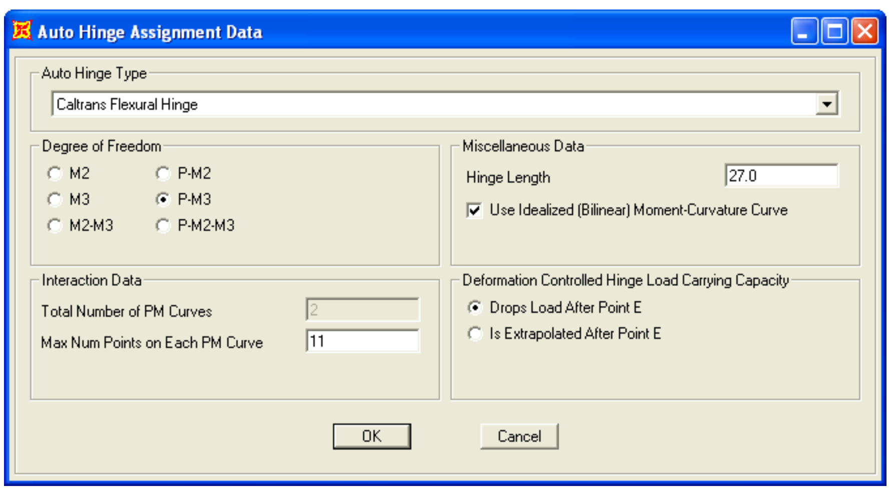
Relative Length = [Element Length – Xbeam Offset – (Hinge Length / 2)] / Element Length
= [146 – 58 – (26.9 / 2)] / 146
= 0.51

The hinges at the bases of the columns are assigned at relative distances as shown in Figure 4.1.3-1 (**Assign menu > Frame > Hinges**).



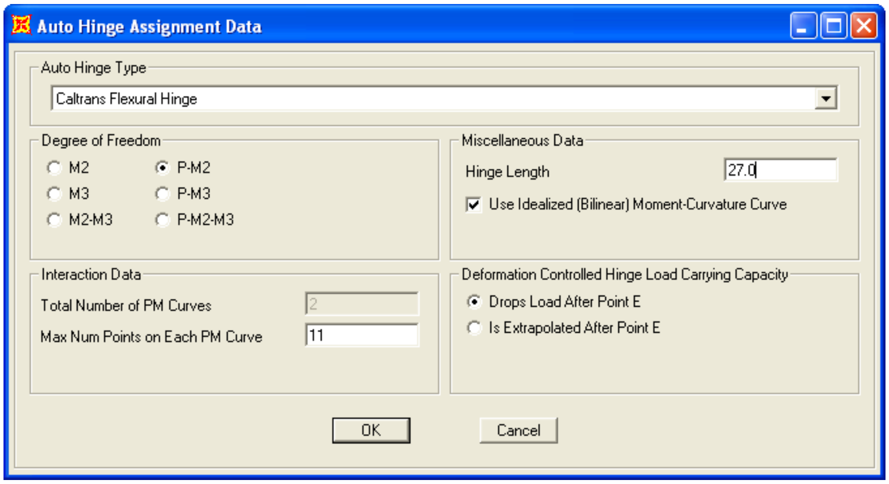

**Frame Hinge Assignments for Column Bases**
*Figure 4.1.3-1*

By selecting the **Auto P-M3** Hinge Property in Figure 4.1.3-1 and clicking the **Modify/Show Auto Hinge Assignment Data** button, the window shown in Figure 4.1.3-2 opens. Figure 4.1.3-2 shows the *Auto Hinge Assignment Data* form with input parameters for the hinges at the bases of the columns in the longitudinal direction. Due to the orientation of the frame element local axes, the P-M3 hinge acts in the longitudinal direction.




**Auto Hinge Assignment Data for Column Bases in Longitudinal Direction**
*Figure 4.1.3-2*

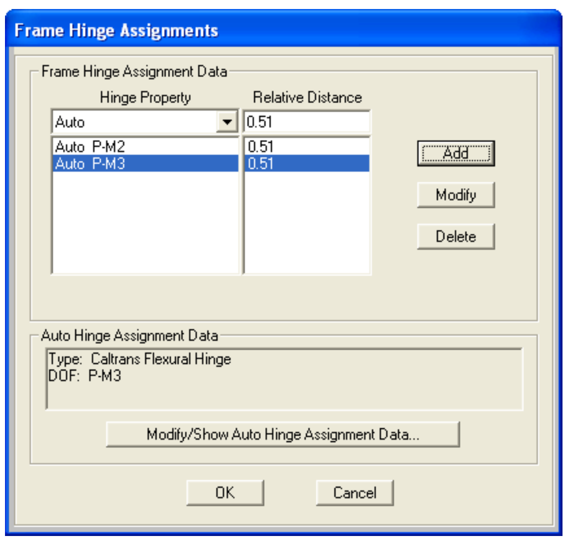
By selecting the **Auto P-M2** Hinge Property in Figure 4.1.3-1 and clicking the **Modify/Show Hinge Assignment Data** button in Figure 4.1.3-1, the window shown in Figure 4.1.3-3 opens. Figure 4.1.3-3 shows the *Auto Hinge Assignment Data* form with input parameters for the hinges at the bases of the columns in the transverse direction. Due to the orientation of the frame element local axes, the P-M2 hinge acts in the transverse direction.




**Auto Hinge Assignment Data for Column Bases in Transverse Direction**
***Figure 4.1.3-3***

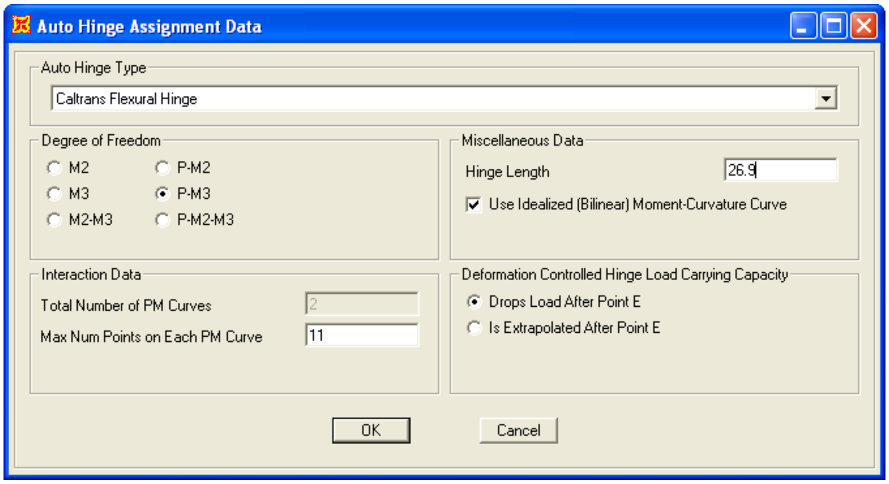
In Figures 4.1.3-2 and 4.1.3-3 it is seen that the *Hinge Length* is set to **27.0** inches, the **Use Idealized (Bilinear) Moment-Curvature Curve** box is checked, and the **Drops Load After Point E** option is selected.

The hinges at the tops of the columns are assigned at relative distances as shown in Figure 4.1.3-4 (**Assign menu** > **Frame** > **Hinges**).



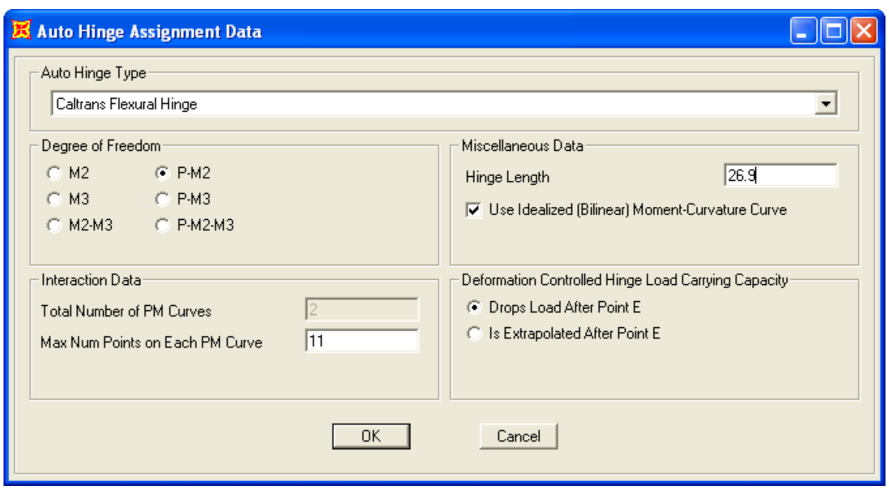

**Frame Hinge Assignments for Column Tops**
***Figure 4.1.3-4***

By selecting the **Auto P-M3** Hinge Property in Figure 4.1.3-4 and clicking the **Modify/Show Auto Hinge Assignment Data** button, the window shown in Figure 4.1.3-5 opens. Figure 4.1.3-5 shows the *Auto Hinge Assignment Data* form with input parameters for the hinges at the tops of the columns in the longitudinal direction. Due to the orientation of the frame element local axes, the P-M3 hinge acts in the longitudinal direction.




**Auto Hinge Assignment Data for Column Tops in Longitudinal Direction**
***Figure 4.1.3-5***

By selecting the **Auto P-M2** Hinge Property in Figure 4.1.3-4 and clicking the **Modify/Show Hinge Assignment Data** button, the window shown in Figure 4.1.3-6 opens. Figure 4.1.3-6 shows the *Auto Hinge Assignment Data* form with input parameters for the hinges at the tops of the columns in the transverse direction. Due to the orientation of the frame element local axes, the P-M2 hinge acts in the transverse direction.




**Auto Hinge Assignment Data for Column Tops in Transverse Direction**
***Figure 4.1.3-6***

In Figures 4.1.3-5 and 4.1.3-6 it is seen that the *Hinge Length* is set to **26.9** inches, the **Use Idealized (Bilinear) Moment-Curvature Curve** box is checked, and the **Drops Load After Point E** option is selected.

# 4.2 Pushover Analysis

## 4.2.1 Lateral Load Distributions

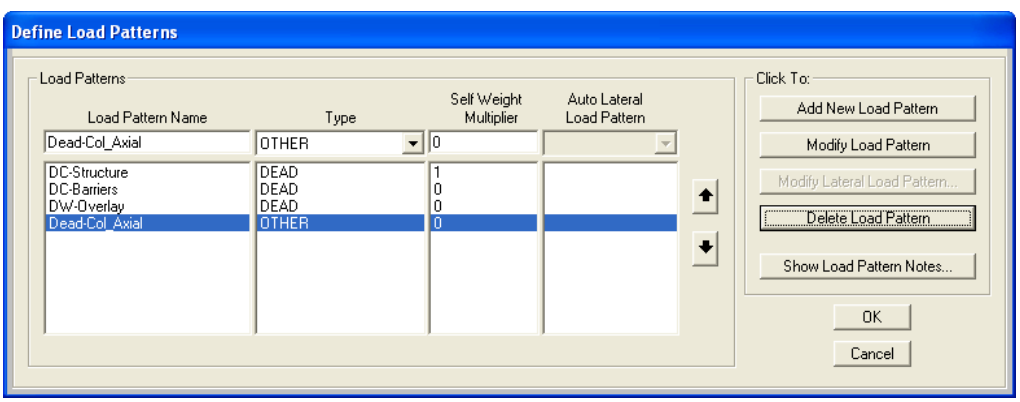
### 4.2.1.1 Longitudinal Direction

The lateral load distribution used in this example for the pushover analysis in the longitudinal direction is a direct horizontal acceleration on the structure mass. Also, the dead load can be applied as previously defined since the entire structure is present during the pushover analysis. It should be noted that a lateral load distribution proportional to the fundamental mode shape in the longitudinal direction is also acceptable provided that at least 75% of the structure mass participates in the mode. This recommendation is derived from provisions in *FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings*.

### 4.2.1.2 Transverse Direction

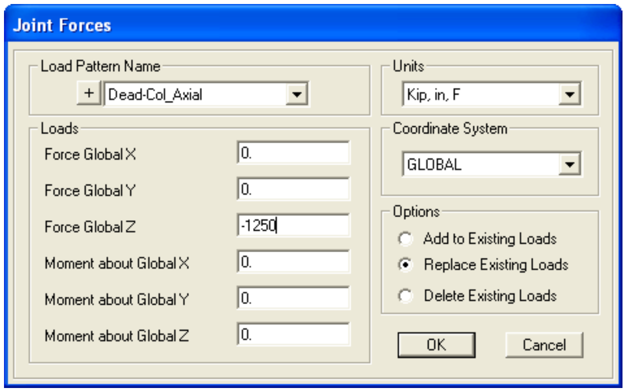
The lateral load distribution used in this example for the pushover analysis in the transverse direction consists of a horizontal load applied at the equivalent of the centroid of the superstructure. This load distribution is used to mimic a direct horizontal acceleration on the superstructure mass. The load is applied this way because the bent is isolated using staged construction and the superstructure is not present for the transverse pushover load case. As mentioned above, a lateral load distribution proportional to the fundamental mode shape in the transverse direction is also acceptable provided that at least 75% of the structure mass participates in the mode.

A special load pattern must be created for the column dead loads since the entire structure is not in place during the pushover analysis. A new load pattern called “Dead-Col_Axial” is added as shown in Figure 4.2.1.2-1 (**Define menu > Load Patterns**).



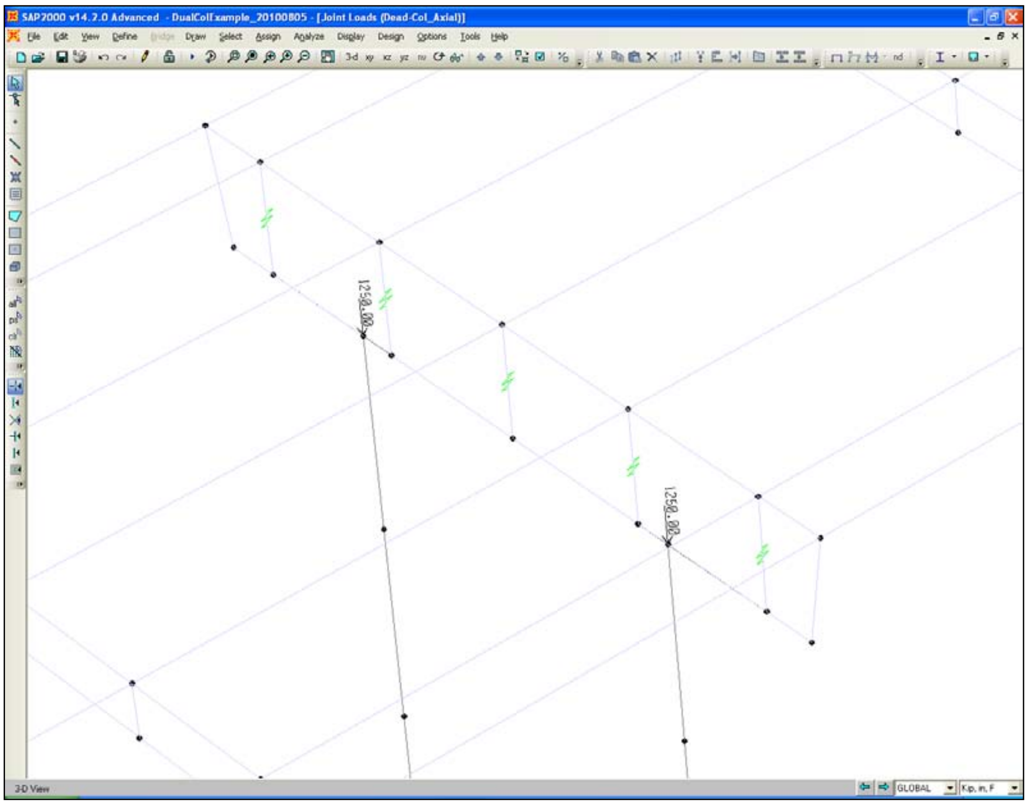

**“Dead-Col_Axial” Load Pattern Definition**
***Figure 4.2.1.2-1***

The column axial loads are 1,250 kips (average of top and bottom). The column dead load moments in the transverse direction are small and can be neglected. Figure 4.2.1.2-2 shows the joint forces assignment window for the “Dead-Col_Axial” load pattern (**Assign menu > Joint Loads > Forces**).




**Joint Force Assignment for Load Pattern "Dead-Col_Axial"**
***Figure 4.2.1.2-2***

After the forces defined in Figure 4.2.1.2-2 have been assigned, they can be viewed as shown in Figure 4.2.1.2-3.

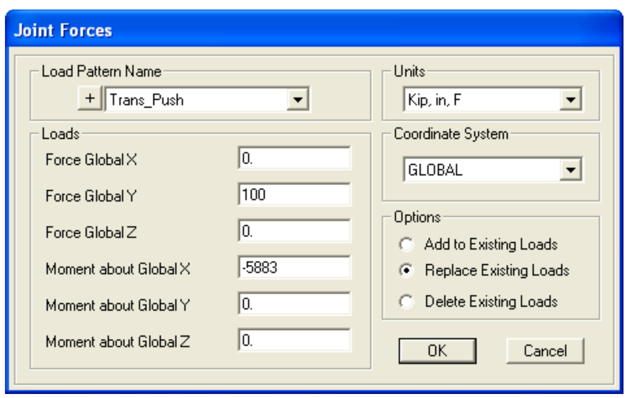



**Wireframe View of Assigned Forces for Load Pattern "Dead-Col_Axial"**
***Figure 4.2.1.2-3***

To define the transverse pushover analysis lateral load distribution, a new load pattern called "Trans_Push" is added as shown in Figure 4.2.1.2-4 (**Define menu > Load Patterns**).

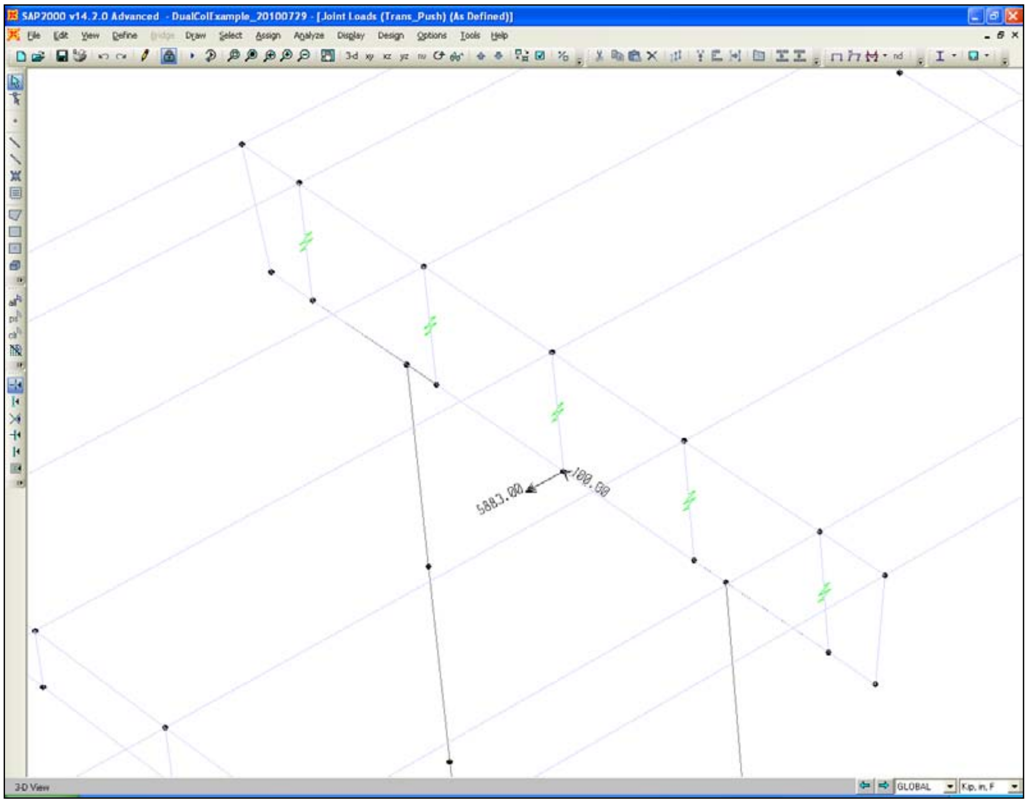

**"Trans_Push" Load Pattern Definition**
***Figure 4.2.1.2-4***

Since the superstructure is not defined as a spine element, there is no joint in the plane of the bent located at the centroid of the superstructure. Therefore, the load distribution for the transverse pushover analysis is an equivalent horizontal load consisting of a point load and a moment applied at the center crossbeam joint. The centroid of the superstructure is located 58.83 inches above the center joint. As a result, a joint force with a horizontal point load of 100 kips and a moment of 100*58.83 = 5,883 kip-inches is used. Special care should be taken to ensure that the shear and moment are applied in the proper directions. The joint forces are assigned to the crossbeam center joint as shown in Figure 4.2.1.2-5 (**Assign menu > Joint Loads > Forces**).




**Joint Force Assignment for Load Pattern "Trans_Push"**
***Figure 4.2.1.2-5***

After the forces defined in Figure 4.2.1.2-5 have been assigned, they can be viewed as shown in Figure 4.2.1.2-6.



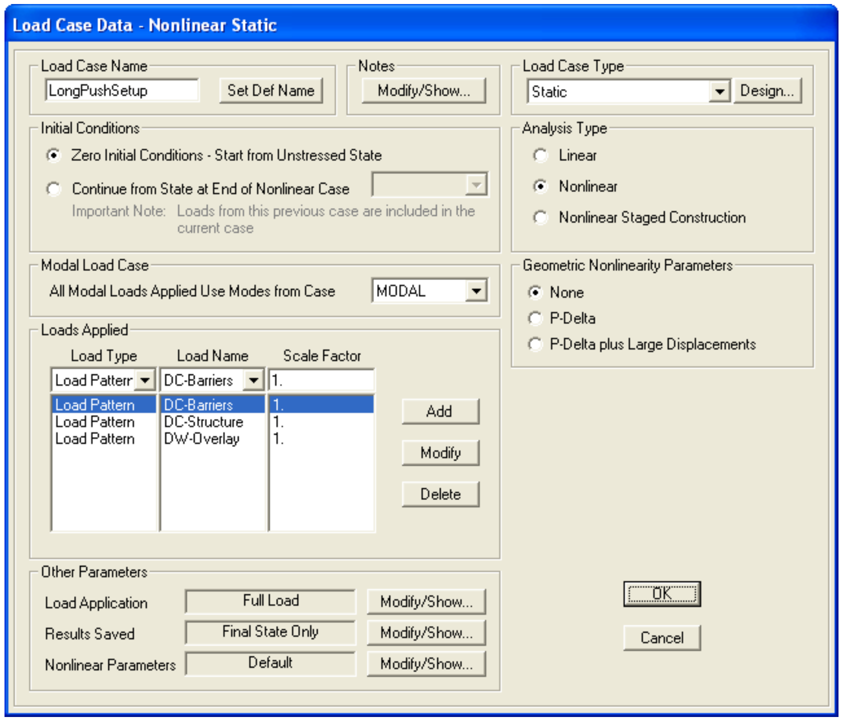

**Wireframe View of Assigned Forces for Load Pattern "Trans_Push"**
***Figure 4.2.1.2-6***

## 4.2.2 Load Case Setup

### 4.2.2.1 Longitudinal Direction

The dead load (DC+DW) must be applied prior to performing the pushover analysis. To do so in the longitudinal direction, a new load case is created called "LongPushSetup". In this load case, the dead load (DC+DW) is applied and the case is run as a nonlinear analysis. By running the load case as a nonlinear analysis type, another load case can continue from it with the loads stored in the structure.

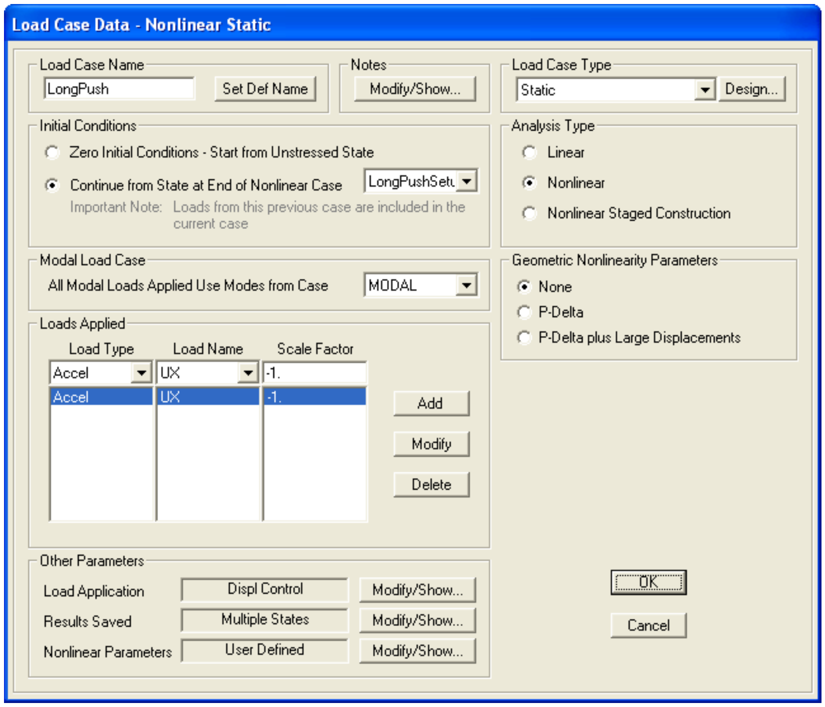
The *Load Case Data* form for the "LongPushSetup" load case is shown in Figure 4.2.2.1-1 (**Define menu > Load Cases >** ***select*** **LongPushSetup >** ***click*** **Modify/Show Load Case** ***button***).




**Load Case Data for Load Case "LongPushSetup"**
***Figure 4.2.2.1-1***

It is seen in Figure 4.2.2.1-1 that the *Initial Conditions* are set to **Zero Initial Conditions – Start from Unstressed State**, the *Load Case Type* is **Static**, the *Analysis Type* is set to **Nonlinear**, and the *Geometric Nonlinearity Parameters* are set to **None**.

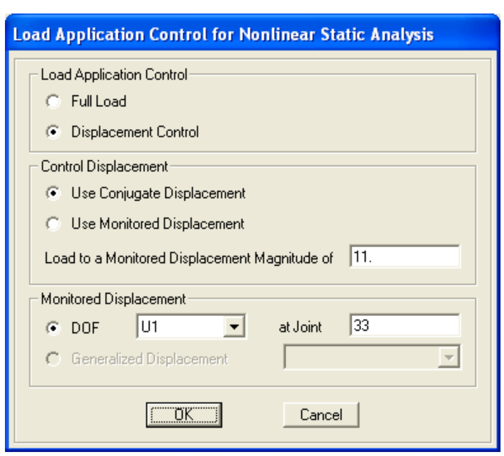
A new load case is now created called "LongPush", which will actually be the pushover analysis case. The *Load Case Data* form for the "LongPush" load case is shown in Figure 4.2.2.1-2 (**Define menu > Load Cases >** ***select*** **LongPush >** ***click*** **Modify/Show Load Case** ***button***).




**Load Case Data for Load Case "LongPush"**
***Figure 4.2.2.1-2***

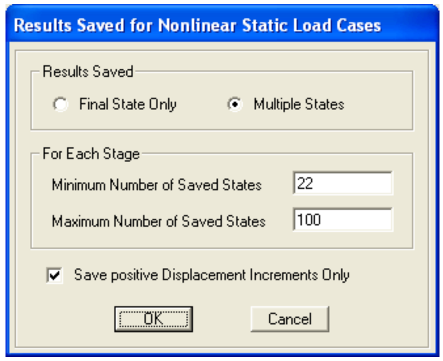
It is seen in Figure 4.2.2.1-2 that the *Initial Conditions* are set to **Continue from State at End of Nonlinear Case "LongPushSetup"**, the *Load Case Type* is **Static**, the *Analysis Type* is **Nonlinear**, and the *Geometric Nonlinearity Parameters* are set to **None**. Under *Loads Applied*, the *Load Type* is set to **Accel** in the **UX** direction with a *Scale Factor* equal to **-1**. Applying the acceleration in the negative X-direction results in a negative base shear and positive X-direction displacements.

By clicking the **Modify/Show** button for the *Load Application* parameters in Figure 4.2.2.1-2, the window shown in Figure 4.2.2.1-3 opens. It is seen in Figure 4.2.2.1-3 that the *Load Application Control* is set to **Displacement Control**, the *Load to a Monitored Displacement Magnitude of* value is set at **11** inches which is greater than the longitudinal displacement demand of 8.76 inches. Also, the *DOF* being tracked is **U1** *at Joint* **33**.




**Load Application Control for Load Case "LongPush"**
***Figure 4.2.2.1-3***

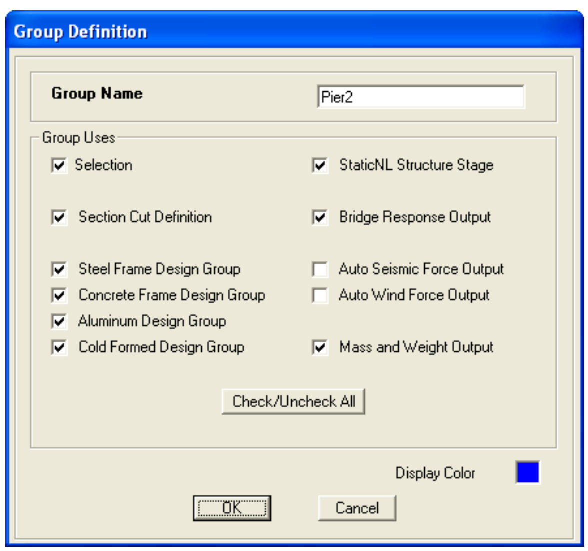
By clicking the **Modify/Show** button for the *Results Saved* in Figure 4.2.2.1-2, the window shown in Figure 4.2.2.1-4 opens. It is seen in Figure 4.2.2.1-4 that the *Results Saved* option is set to **Multiple States**, the *Minimum Number of Saved States* is set to **22**, which ensures that a step will occur for at least every half-inch of displacement. Also, the *Save positive Displacement Increments Only* box is checked.




**Results Saved for Load Case "LongPush"**
***Figure 4.2.2.1-4***

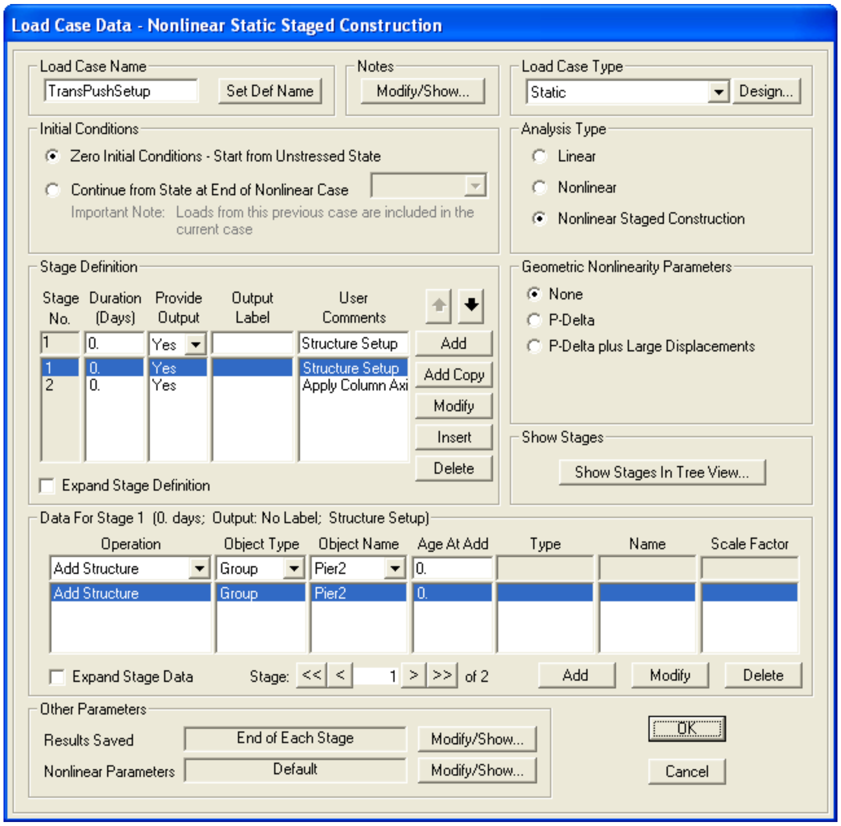
### 4.2.2.2 Transverse Direction

As with the longitudinal direction, the dead load must be applied prior to performing the pushover analysis in the transverse direction. However, for the transverse direction, a single bent will be isolated using staged construction prior to performing the pushover analysis. To do so, the elements at Pier 2 are selected and then assigned to a group (**Assign menu > Assign to Group**). Figure 4.2.2.2-1 shows the *Group Definition* for the group "Pier2" (**Define menu > Groups >** ***select*** **Pier2 >** ***click*** **Modify/Show Group** ***button***).



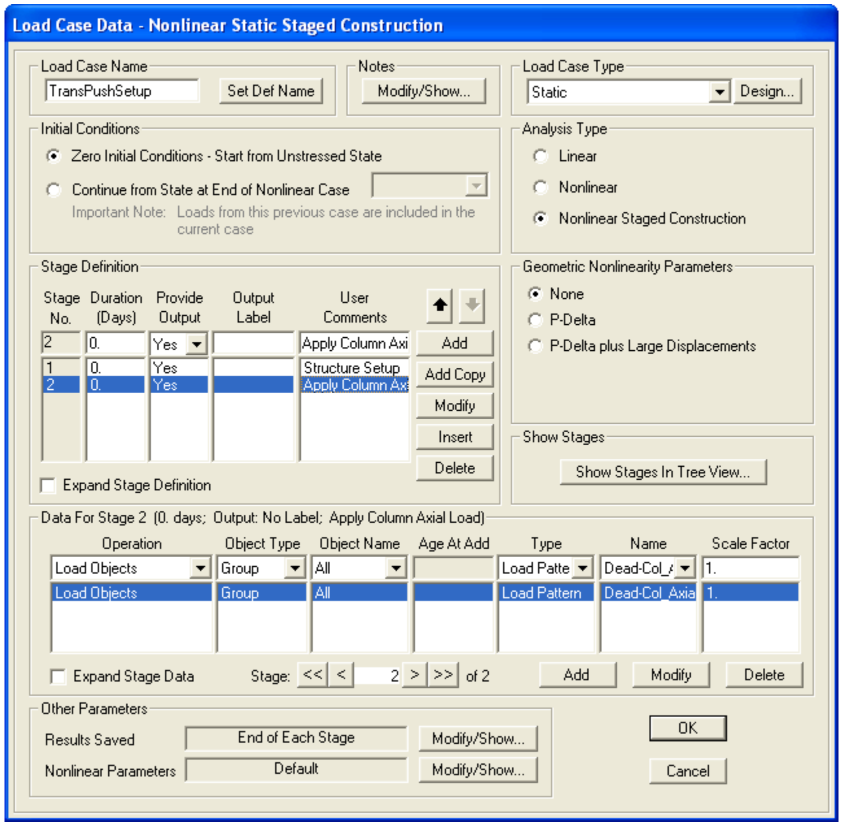

**Group Definition for Group "Pier2"**
***Figure 4.2.2.2-1***

To isolate the bent and apply the static loads to the columns, a staged construction load case called "TransPushSetup" is created (**Define menu > Load Cases >** ***select*** **TransPushSetup >** ***click*** **Modify/Show Load Case** ***button***). The "TransPushSetup" analysis case has two stages, one to isolate the bent, and one to apply the column axial loads. Note these two stages could be combined into one stage without altering the results. Stage 1 of the “TransPushSetup” load case definition is shown in Figure 4.2.2.2-2.




**Stage 1 Load Case Data for Load Case “TransPushSetup”**
***Figure 4.2.2.2-2***

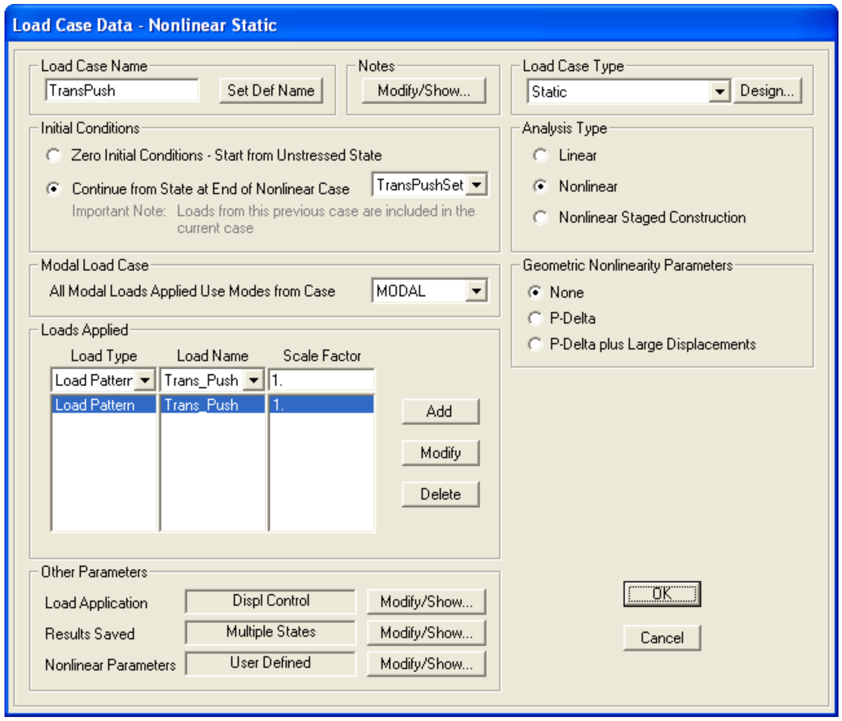
It is seen in Figure 4.2.2.2-2 that the only elements added are those in the group “Pier2”, the *Initial Conditions* are set to **Zero Initial Conditions – Start from Unstressed State**, the *Load Case Type* is **Static**, the *Analysis Type* is set to **Nonlinear Staged Construction**, and the *Geometric Nonlinearity Parameters* are set to **None**. Stage 2 of the “TransPushSetup” load case definition is shown in Figure 4.2.2.2-3.




**Stage 2 Load Case Data for Load Case "TransPushSetup"**
***Figure 4.2.2.2-3***

It is seen in Figure 4.2.2.2-3 that the load pattern "Dead-Col_Axial" is applied.

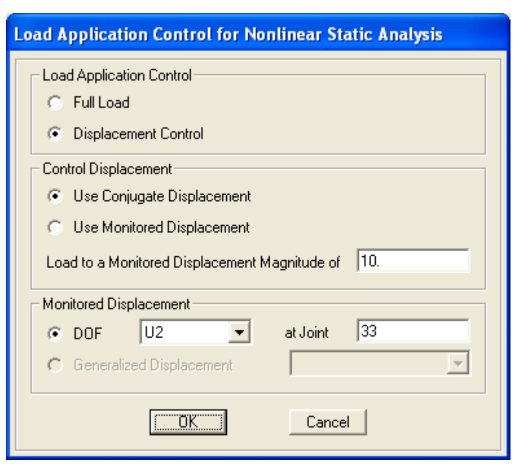
A new load case is now created called "TransPush", which will actually be the pushover analysis case. The *Load Case Data* form for the "TransPush" load case is shown in Figure 4.2.2.2-4 (**Define menu > Load Cases >** ***select*** **TransPush >** ***click*** **Modify/Show Load Case** ***button***).




**Load Case Data for Load Case "TransPush"**
***Figure 4.2.2.2-4***

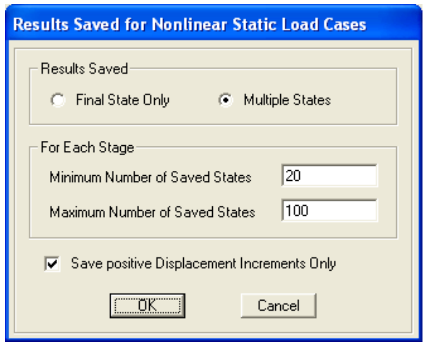
It is seen in Figure 4.2.2.2-4 that the *Initial Conditions* are set to **Continue from State at End of Nonlinear Case "TransPushSetup"**, the *Load Case Type* is **Static**, the *Analysis Type* is **Nonlinear**, and the *Geometric Nonlinearity Parameters* are set to **None**. Under *Loads Applied*, the *Load Type* is set to **Load Pattern** with the *Load Name* set to **Trans_Push** and the *Scale Factor* is equal to **1**.

By clicking the **Modify/Show** button for the *Load Application* parameters in Figure 4.2.2.2-4, the window shown in Figure 4.2.2.2-5 opens. It is seen in Figure 4.2.2.2-5 that the *Load Application Control* is set to **Displacement Control**, the *Load to a Monitored Displacement Magnitude of* value is set at **10** inches, which is larger than the transverse displacement demand of 6.07 inches. Also, the *DOF* being tracked is **U2** *at Joint* **33**.




**Load Application Control for Load Case "TransPush"**
***Figure 4.2.2.2-5***

By clicking the **Modify/Show** button for the *Results Saved* in Figure 4.2.2.2-4, the window shown in Figure 4.2.2.2-6 opens. It is seen in Figure 4.2.2.2-6 that the *Results Saved* option is set to **Multiple States**, the *Minimum Number of Saved States* is set to **20**, which ensures that a step will occur for at least every half-inch of displacement. Also, the *Save positive Displacement Increments Only* box is checked.

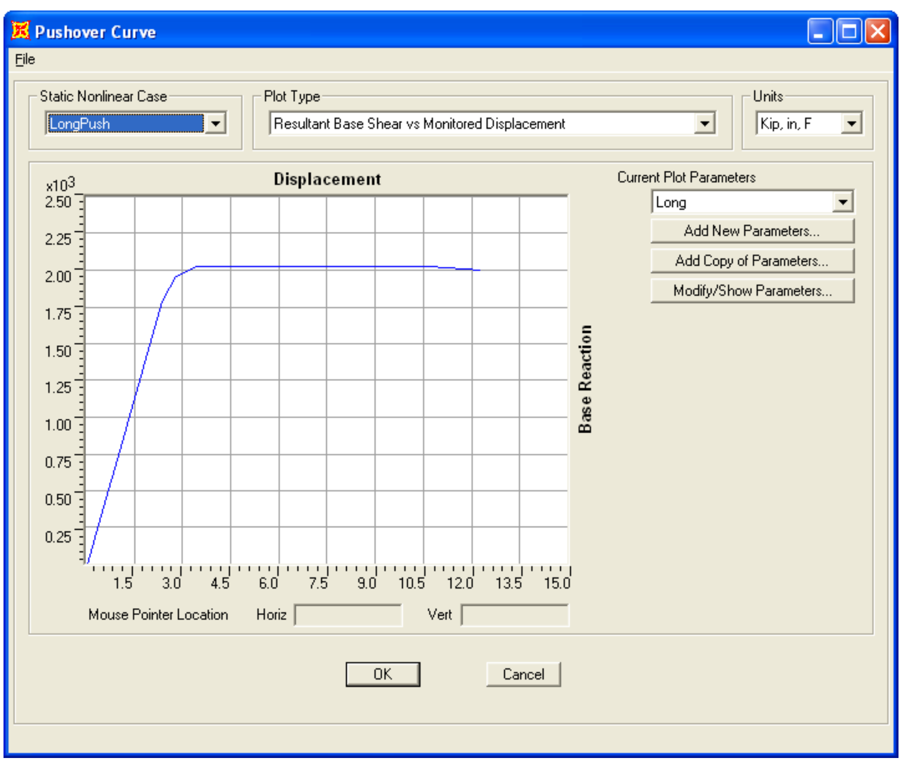



**Results Saved for Load Case "TransPush"**
***Figure 4.2.2.2-6***

## 4.2.3 Load Case Results

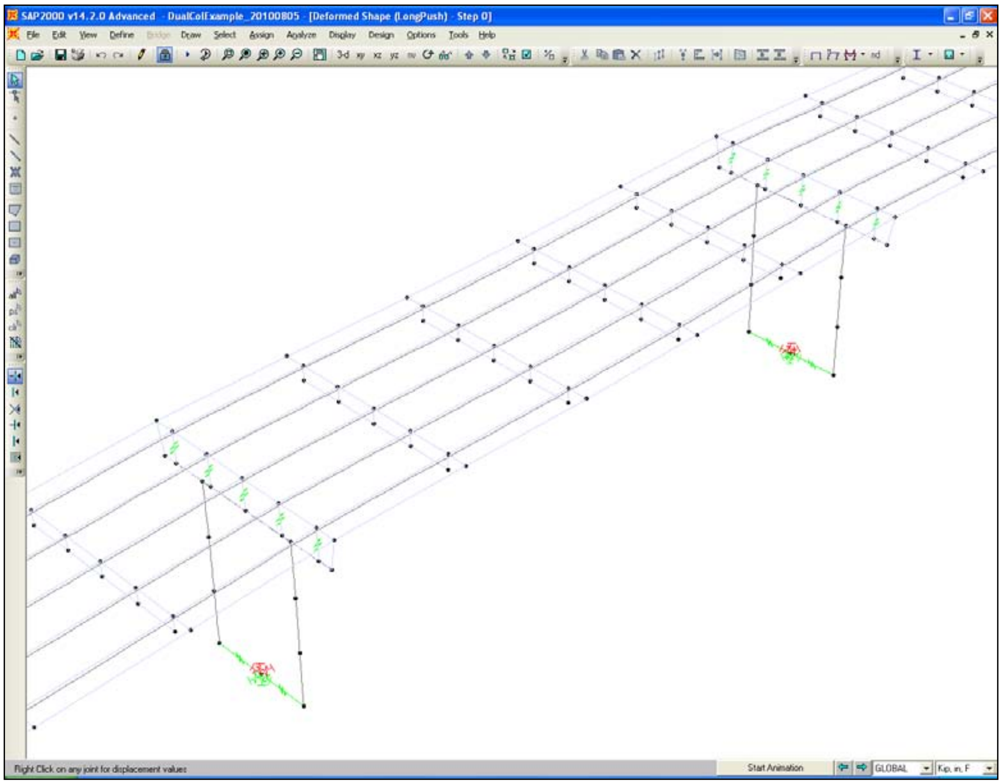
### 4.2.3.1 Longitudinal Direction

The system pushover curve for the longitudinal direction is shown in Figure 4.2.3.1-1 (**Display menu > Show Static Pushover Curve**). The point on the curve where the base shear begins to decrease indicates the displacement at which the first plastic hinge reaches its curvature limit state and is the displacement capacity of the structure.

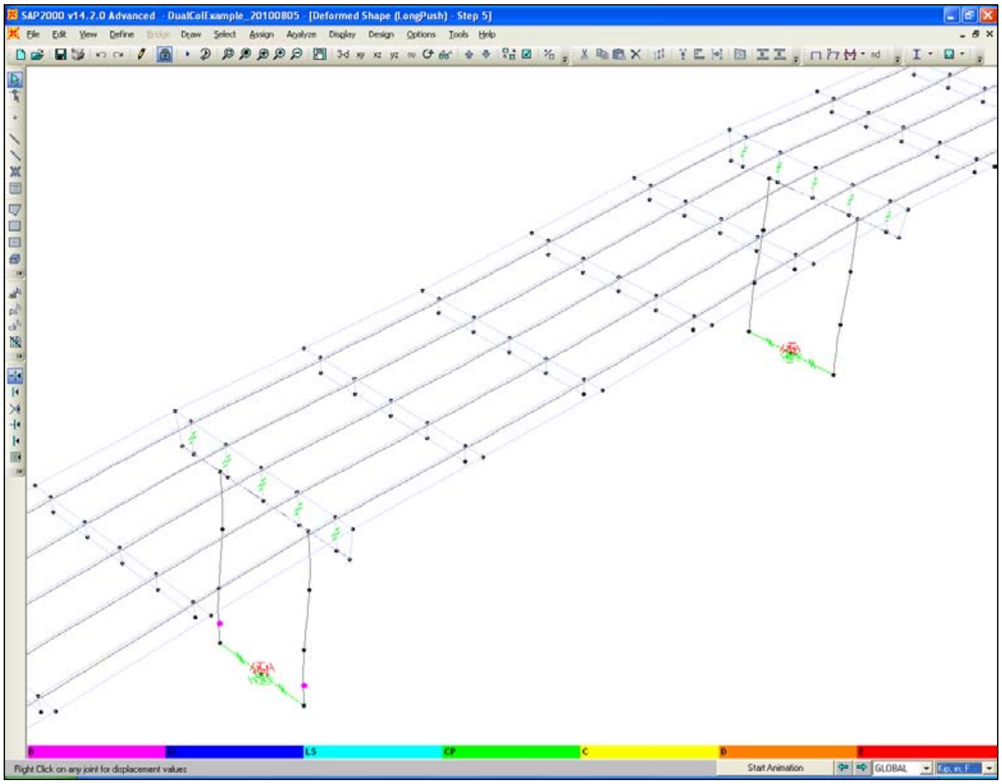



**Pushover Curve for Load Case "LongPush"**
***Figure 4.2.3.1-1***

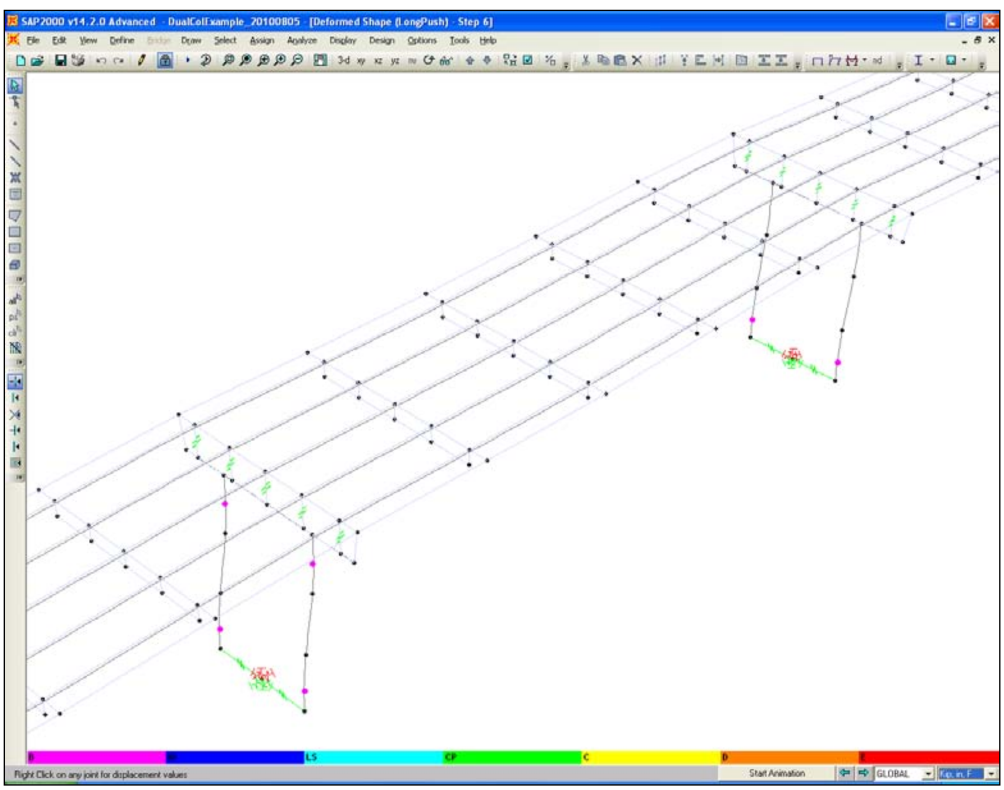
Figures 4.2.3.1-2 through 4.2.3.1-13 show the deformed shape of the structure at various displacements for the load case "LongPush" (**Display menu > Show Deformed Shape >** ***select*** **LongPush >** ***click*** **OK** ***button***). Note that the plastic hinge color scheme terms such as "IO", "LS", and "CP" are in reference to performance based design of building structures. However, for Caltrans plastic hinges, the colors are discretized evenly along the plastic deformation. Therefore, the color scheme still provides a visual representation of the hinge plastic strain progression that is useful.



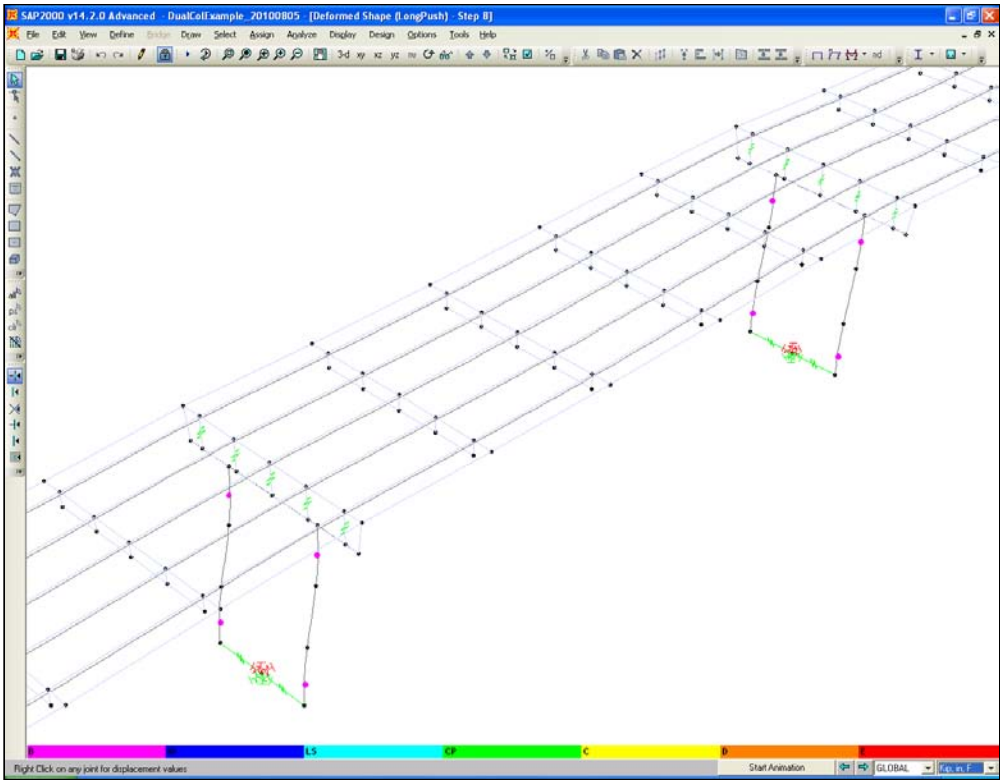

**View of Deformed Shape for the Load Case "LongPush" at UX = 0.0 in.**
***Figure 4.2.3.1-2***



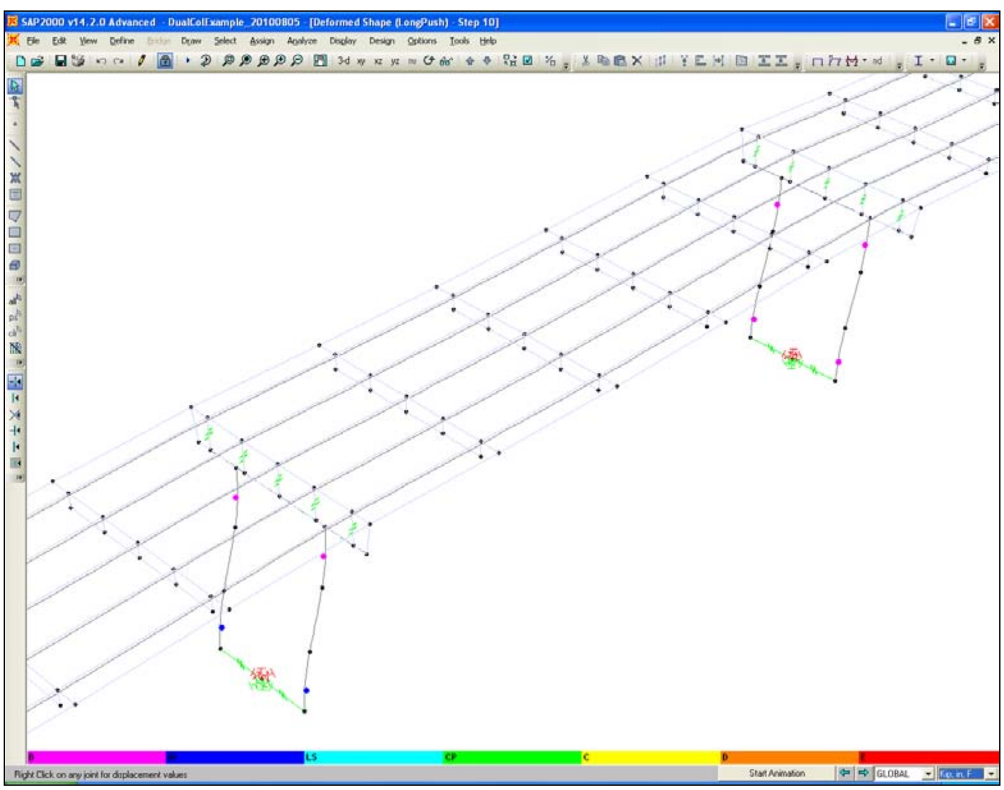

**View of Deformed Shape for the Load Case "LongPush" at UX = 2.3 in.**
*Figure 4.2.3.1-3*



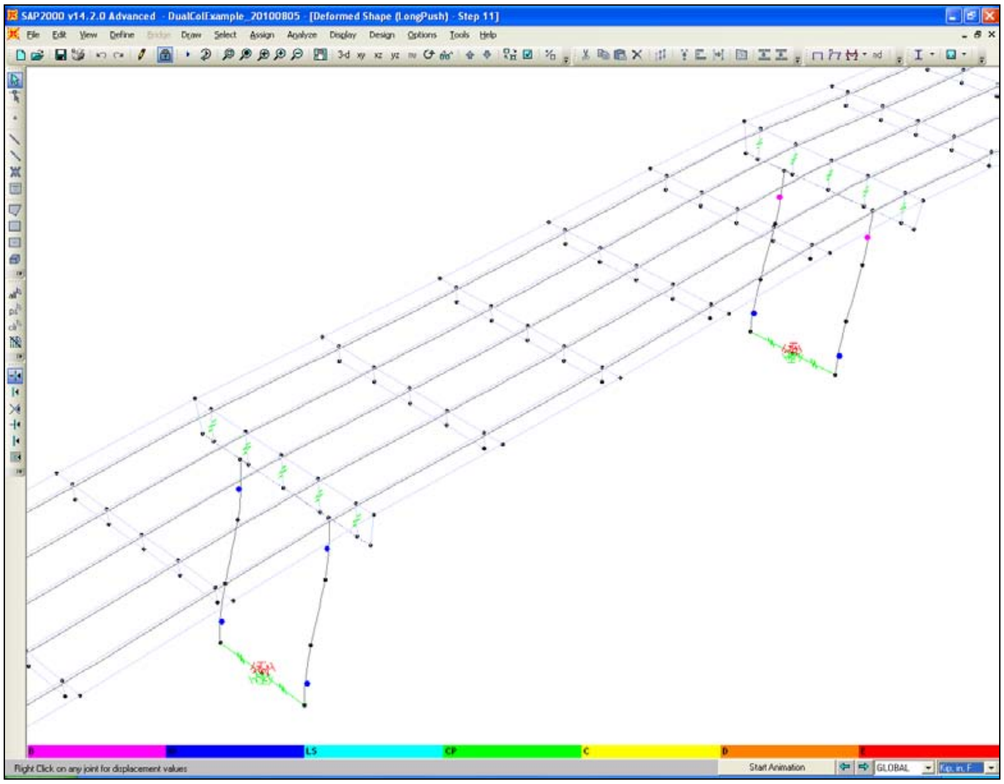

**View of Deformed Shape for the Load Case "LongPush" at UX = 2.8 in.**
*Figure 4.2.3.1-4*



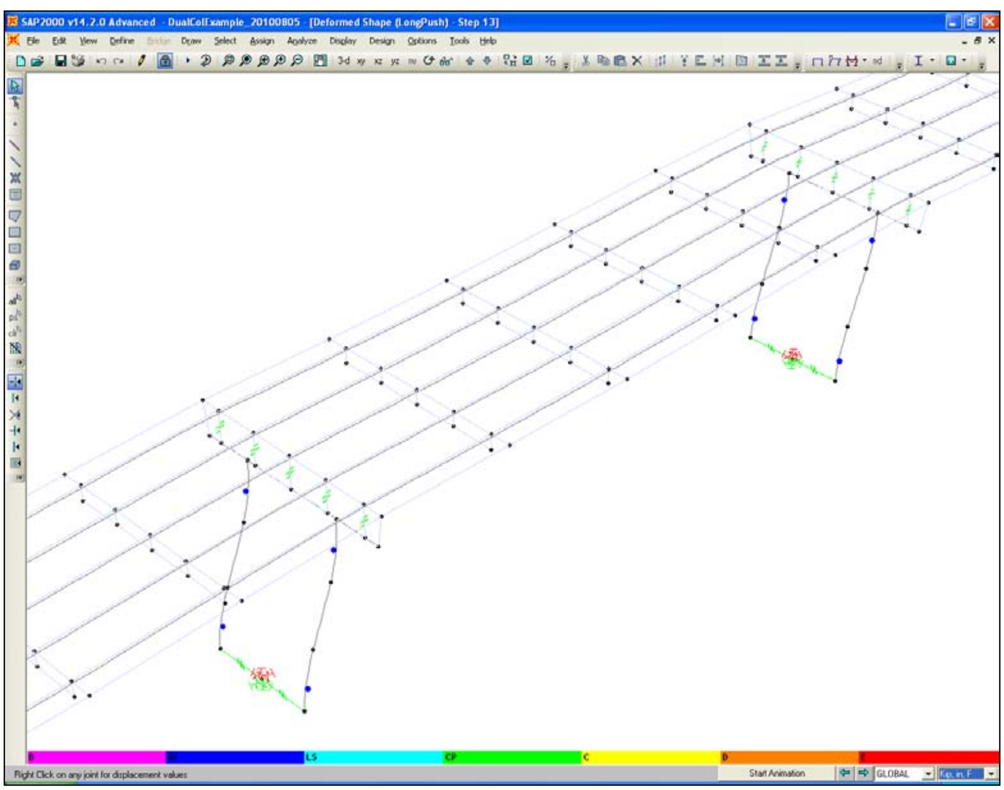

**View of Deformed Shape for the Load Case "LongPush" at UX = 3.5 in.**
*Figure 4.2.3.1-5*



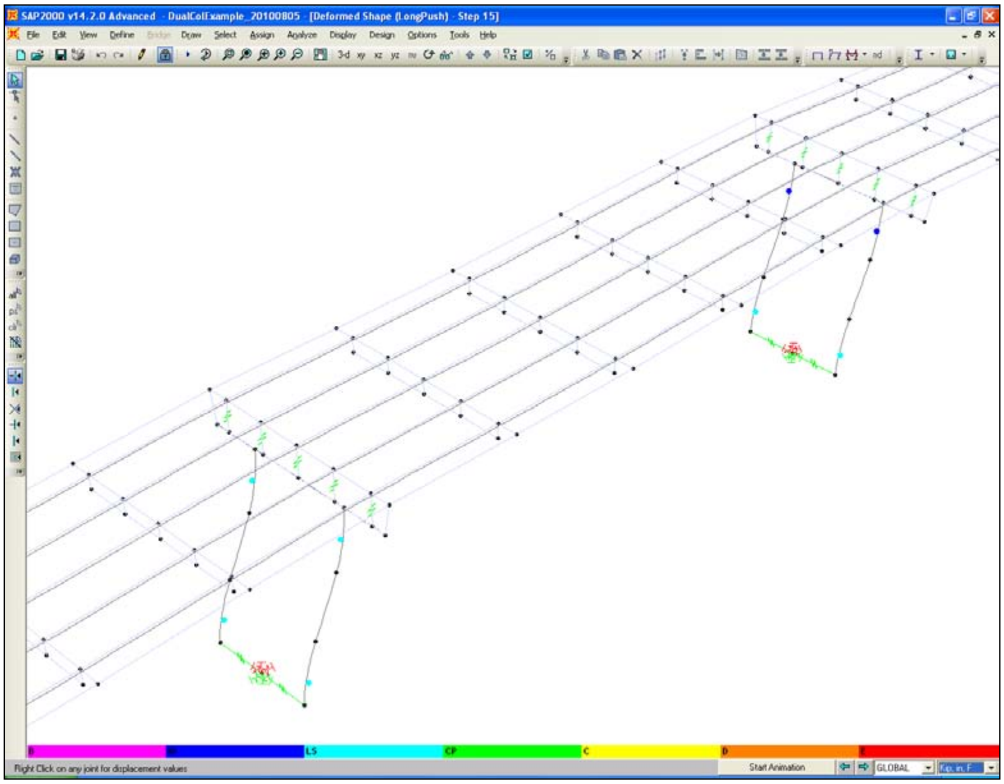

**View of Deformed Shape for the Load Case "LongPush" at UX = 4.4 in.**
*Figure 4.2.3.1-6*



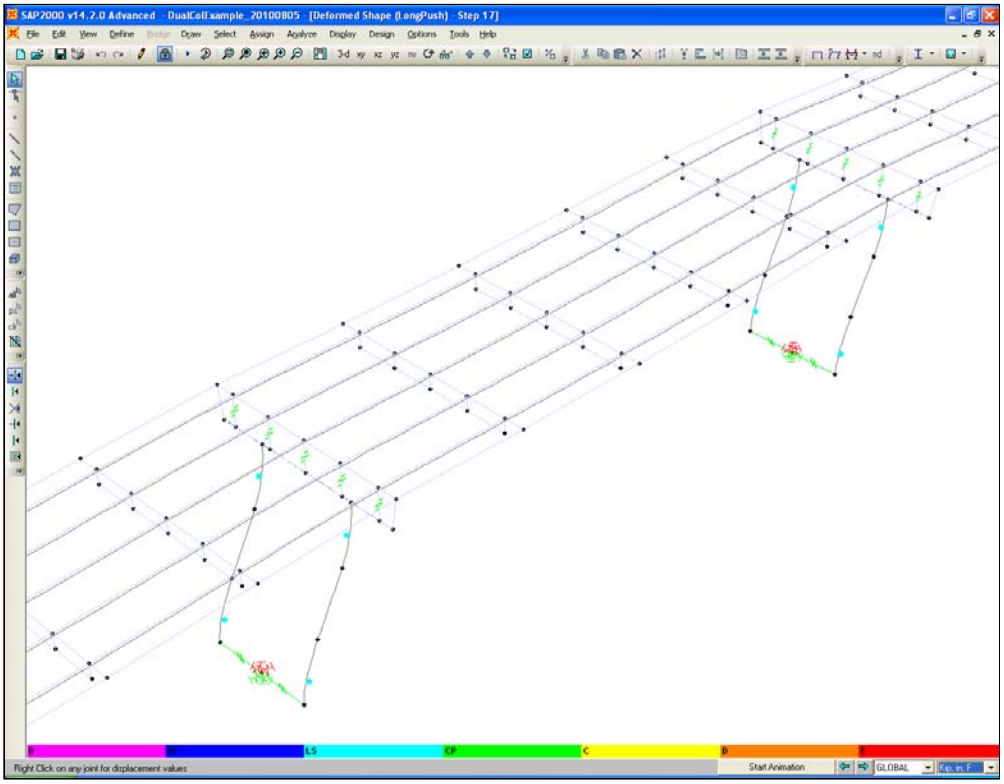

**View of Deformed Shape for the Load Case “LongPush” at UX = 4.9 in.**
*Figure 4.2.3.1-7*



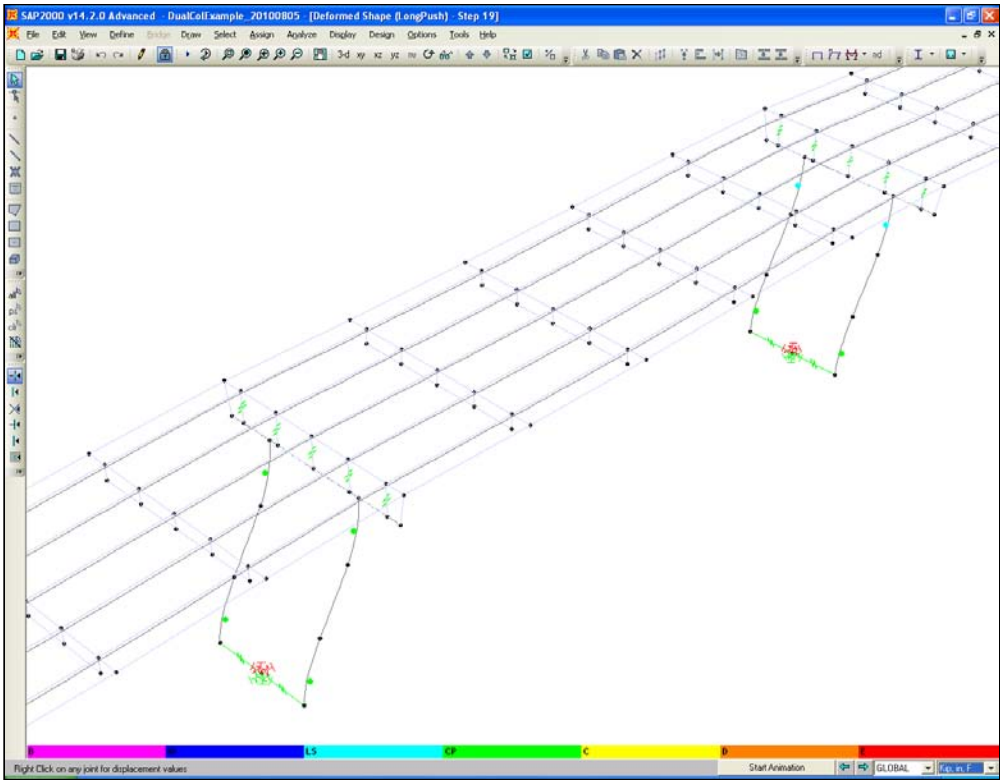

**View of Deformed Shape for the Load Case “LongPush” at UX = 5.9 in.**
*Figure 4.2.3.1-8*



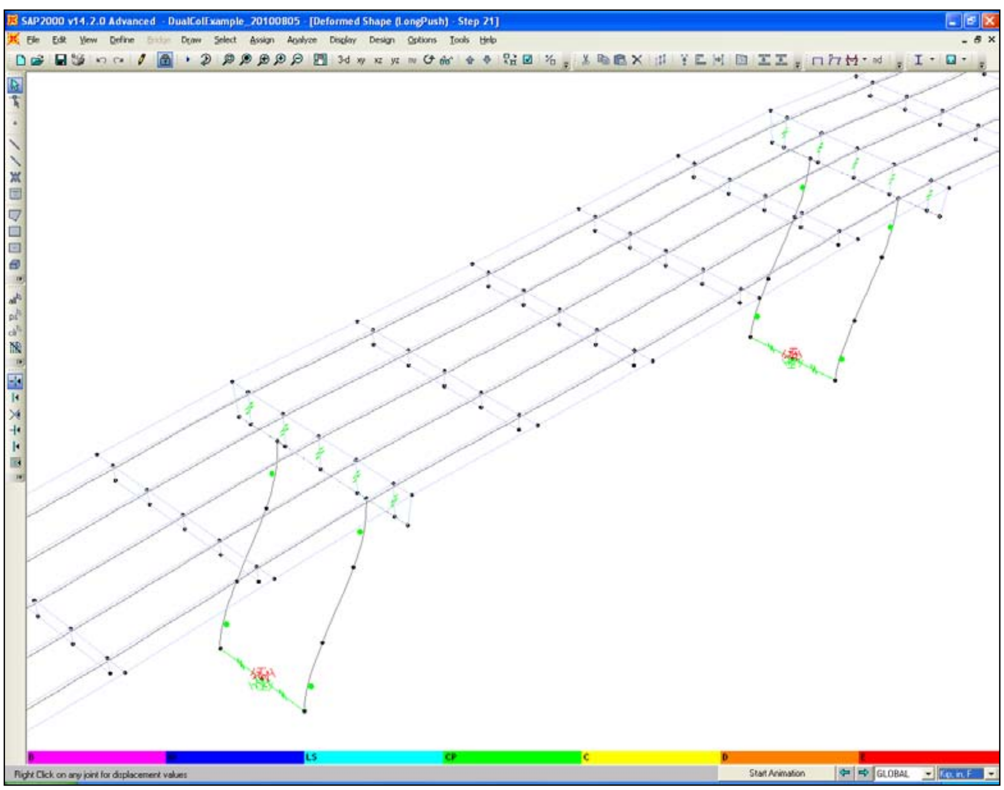

**View of Deformed Shape for the Load Case "LongPush" at UX = 6.9 in.**
*Figure 4.2.3.1-9*



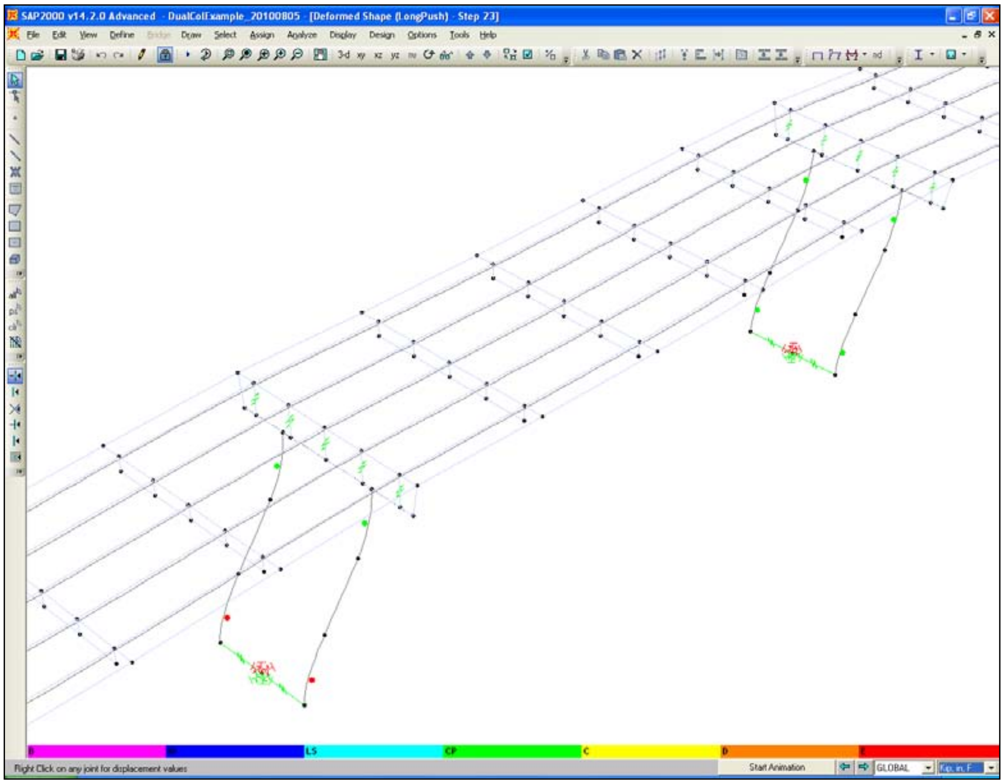

**View of Deformed Shape for the Load Case "LongPush" at UX = 7.9 in.**
*Figure 4.2.3.1-10*




**View of Deformed Shape for the Load Case “LongPush” at UX = 8.9 in.**
*Figure 4.2.3.1-11*



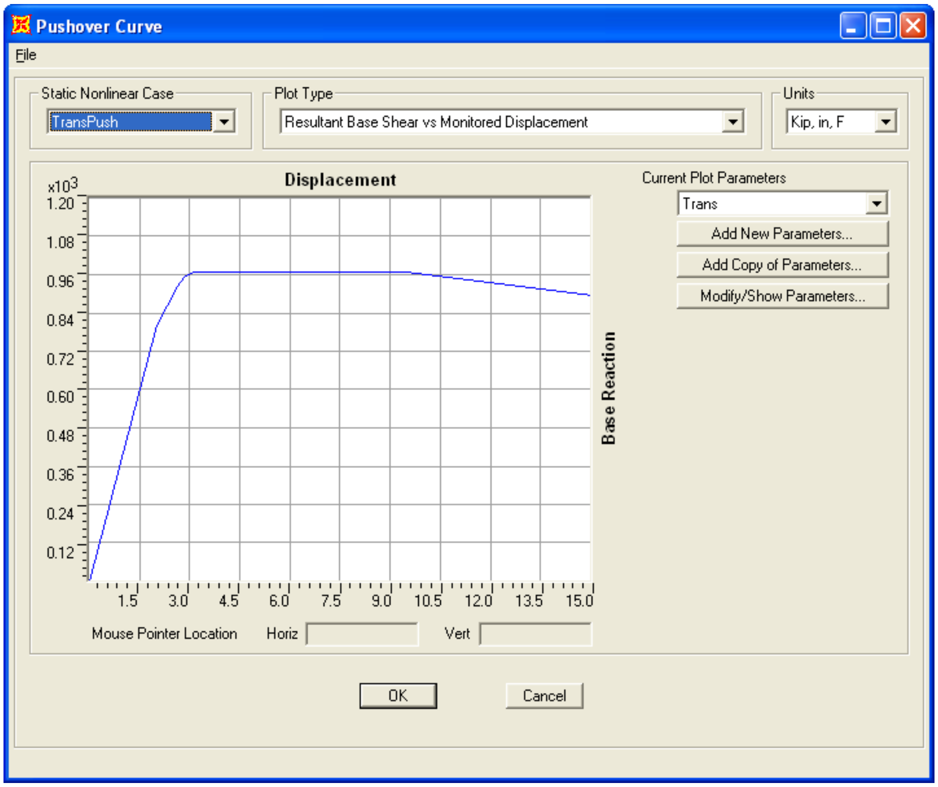

**View of Deformed Shape for the Load Case “LongPush” at UX = 9.9 in.**
*Figure 4.2.3.1-12*



**View of Deformed Shape for the Load Case "LongPush" at UX = 10.7 in.**
***Figure 4.2.3.1-13***

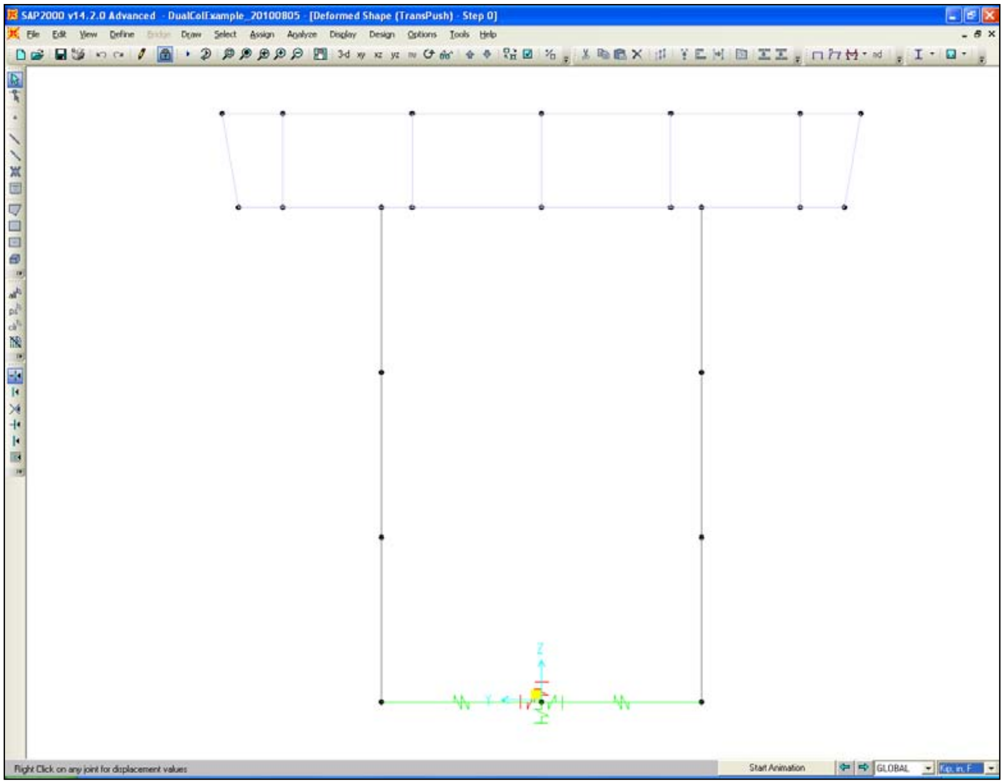
### 4.2.3.2 Transverse Direction

The system pushover curve for the transverse direction is shown in Figure 4.2.3.2-1 (**Display menu > Show Static Pushover Curve**). The point on the curve where the base shear begins to decrease indicates the displacement at which the first plastic hinge reaches its curvature limit state and is the displacement capacity of the structure.

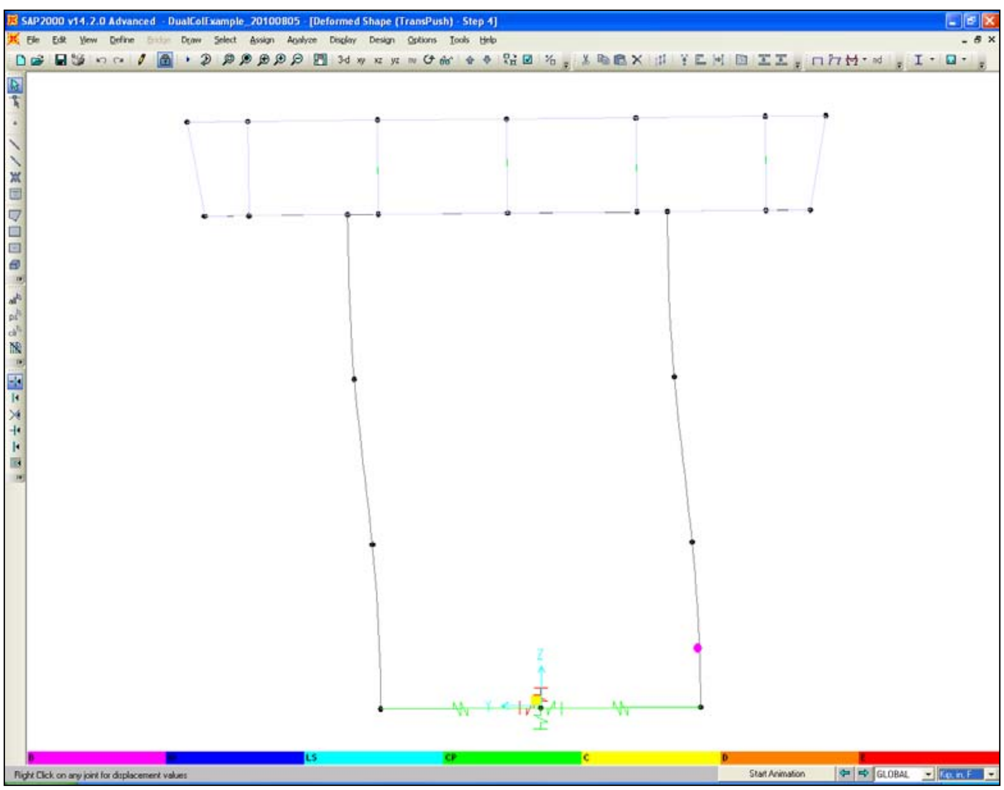



**Pushover Curve for Load Case "TransPush"**
***Figure 4.2.3.2-1***

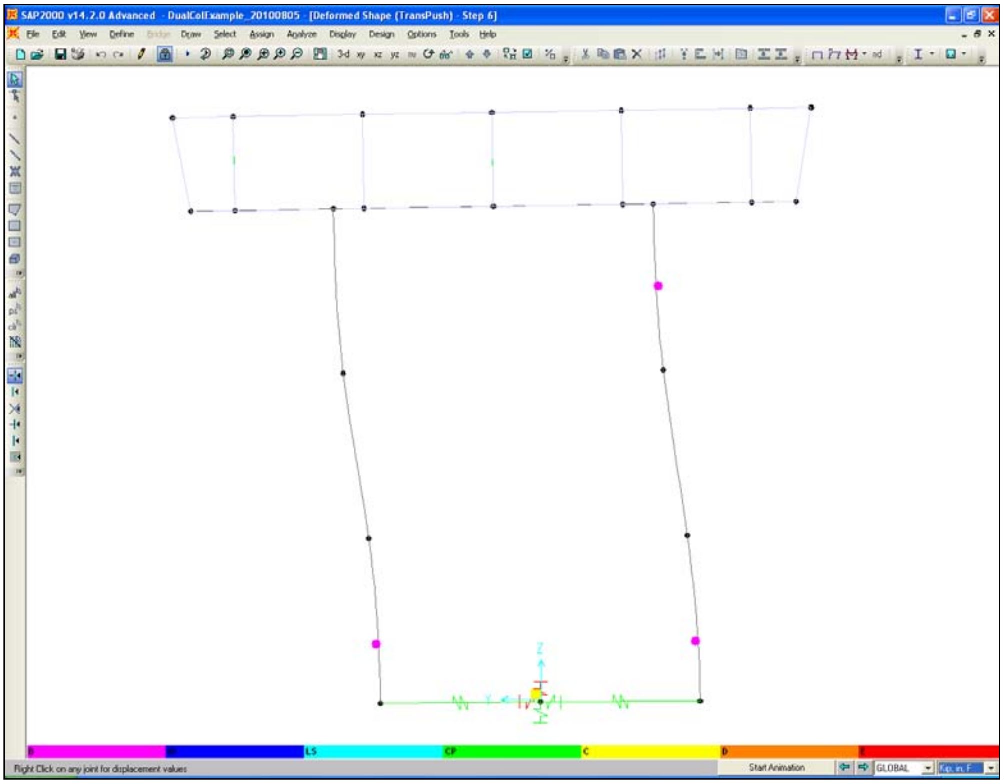
Figures 4.2.3.2-2 through 4.2.3.2-13 show the deformed shape of the structure at various displacements for the load case "TransPush" (**Display menu > Show Deformed Shape >** ***select*** **TransPush >** ***click*** **OK** ***button***). Note that the plastic hinge color scheme terms such as "IO", "LS", and "CP" are in reference to performance-based design of building structures. However, for Caltrans plastic hinges, the colors are discretized evenly along the plastic deformation. Therefore, the color scheme still provides a visual representation of the hinge plastic strain progression that is useful.



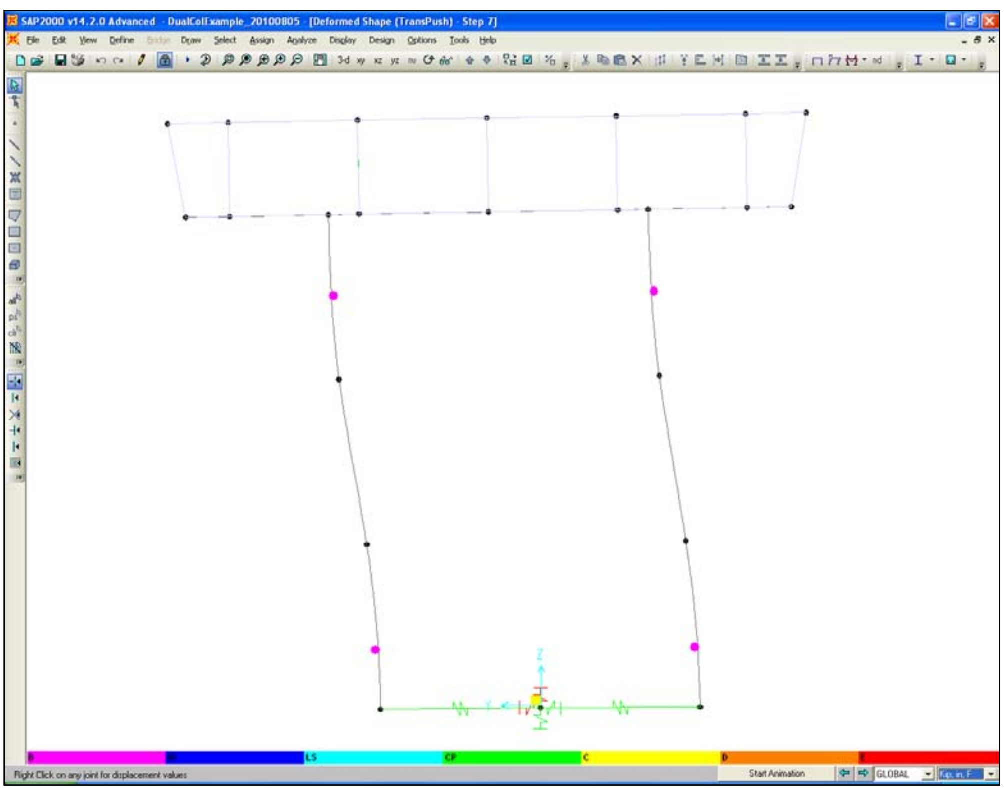
**View of Deformed Shape for the Load Case "TransPush" at UY = 0.0 in.**
*Figure 4.2.3.2-2*



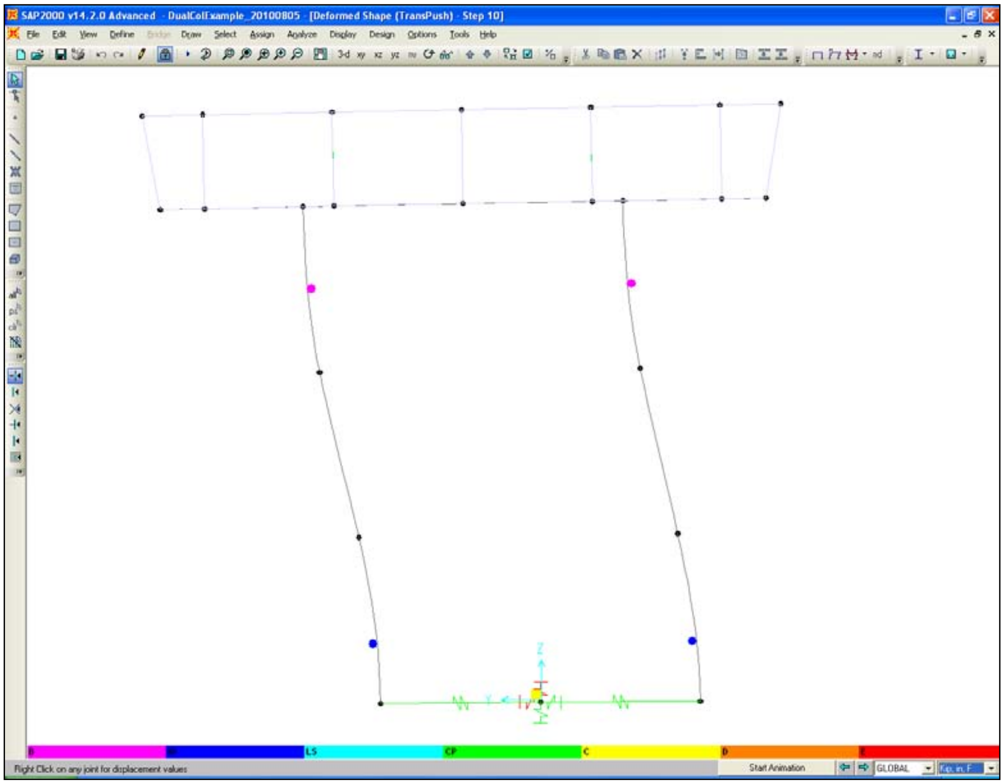
**View of Deformed Shape for the Load Case "TransPush" at UY = 2.0 in.**
*Figure 4.2.3.2-3*



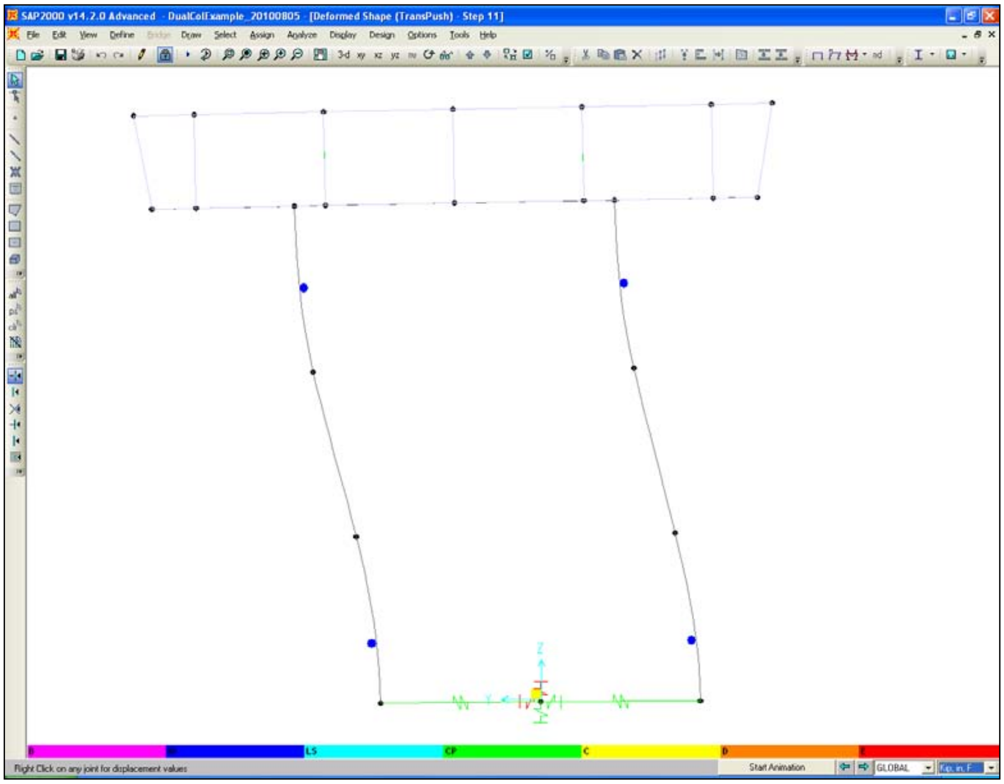

**View of Deformed Shape for the Load Case “TransPush” at UY = 2.8 in.**
*Figure 4.2.3.2-4*



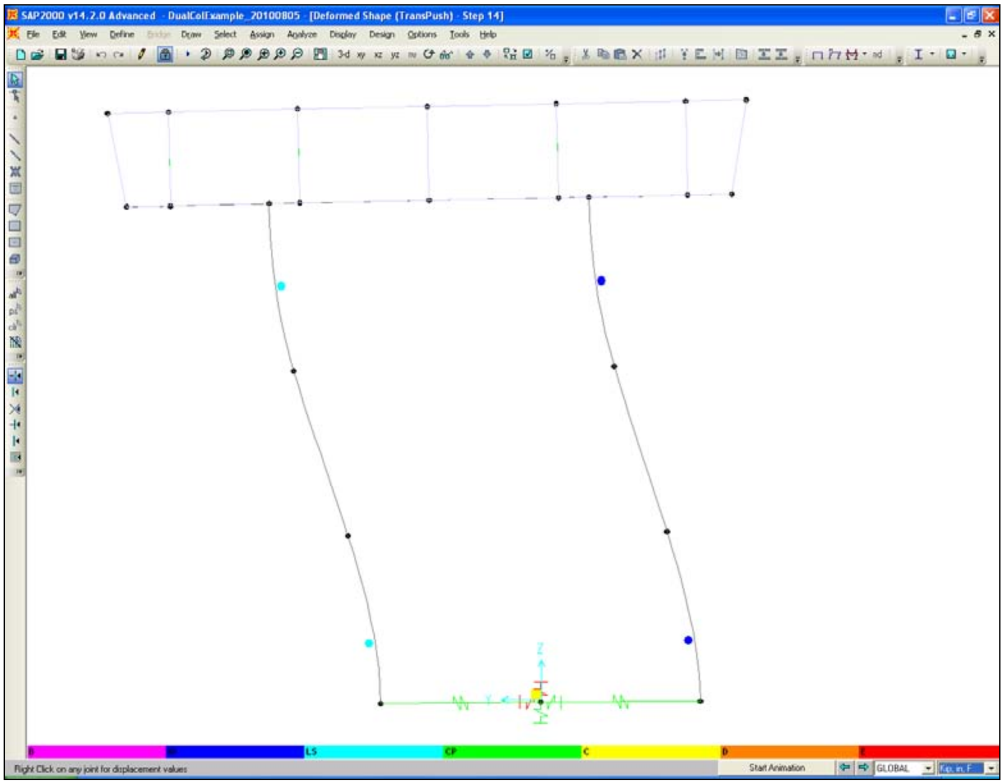

**View of Deformed Shape for the Load Case “TransPush” at UY = 3.1 in.**
*Figure 4.2.3.2-5*



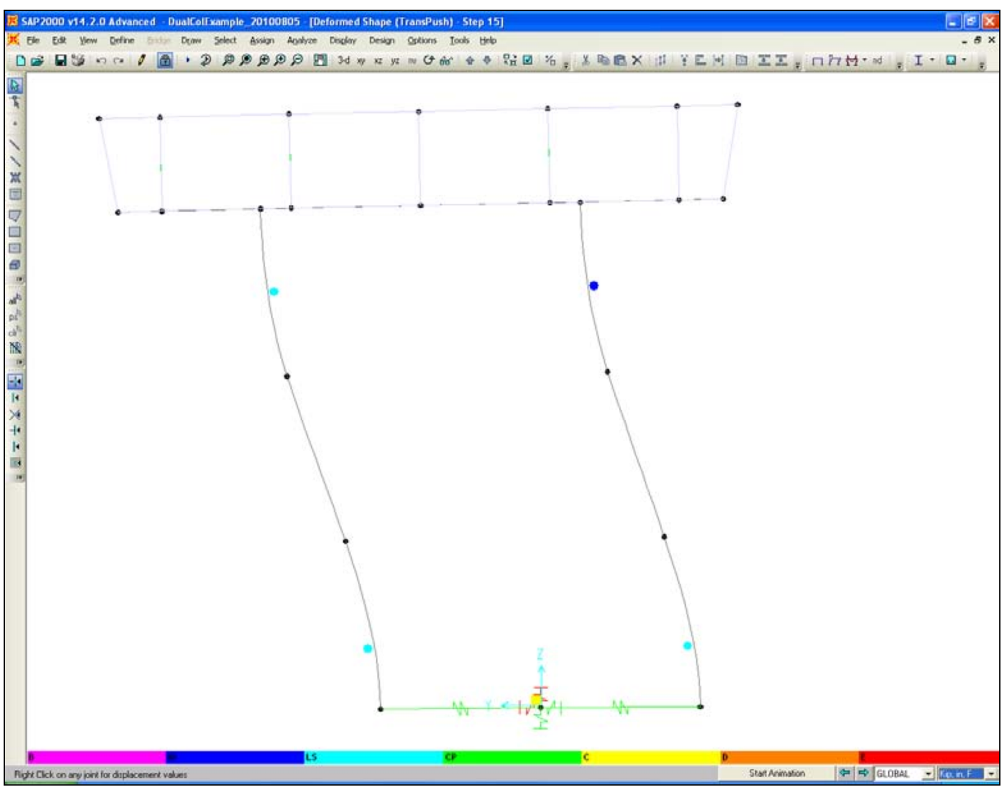

**View of Deformed Shape for the Load Case "TransPush" at UY = 4.6 in.**
*Figure 4.2.3.2-6*



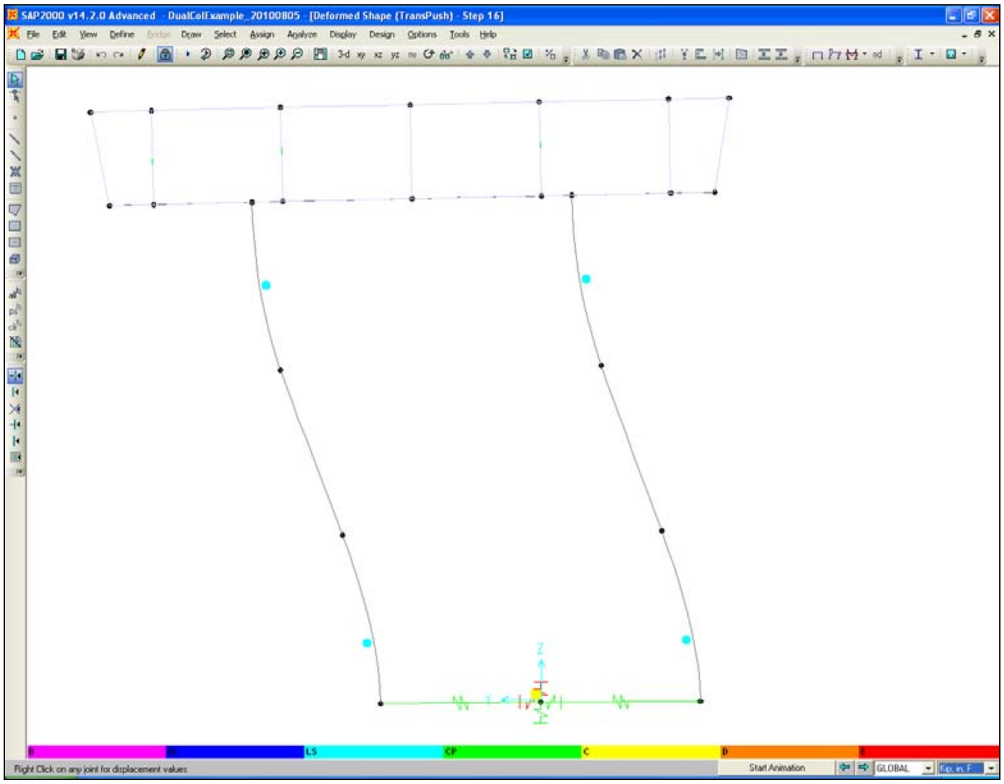

**View of Deformed Shape for the Load Case "TransPush" at UY = 5.1 in.**
*Figure 4.2.3.2-7*



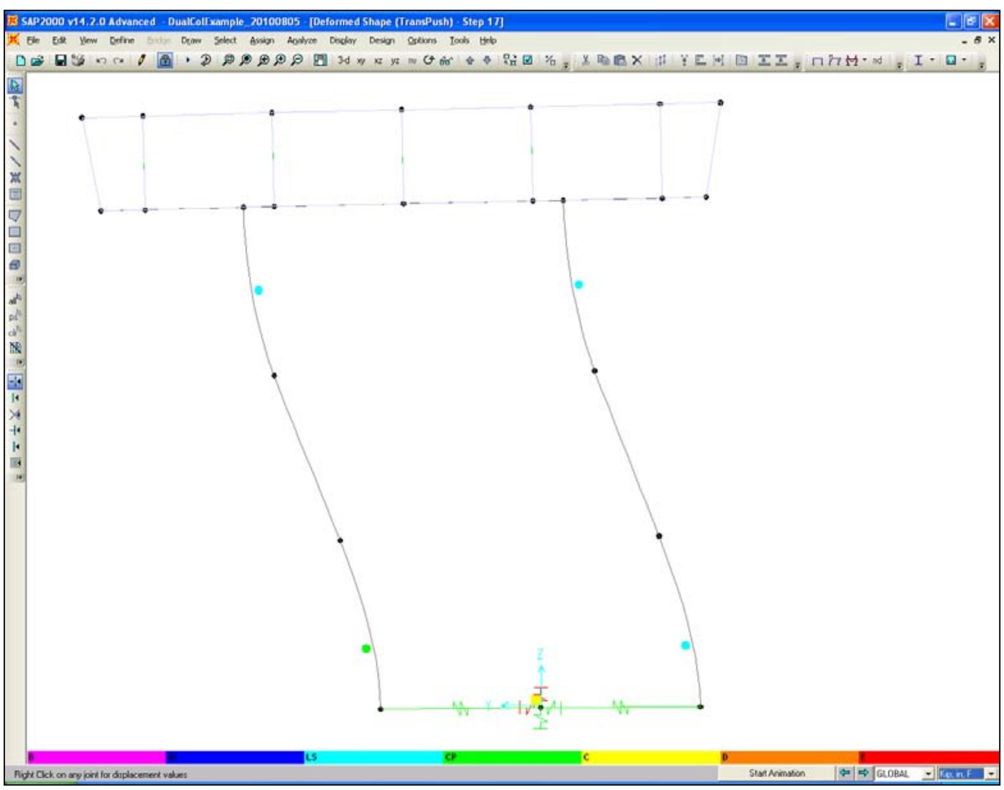

**View of Deformed Shape for the Load Case "TransPush" at UY = 6.6 in.**
*Figure 4.2.3.2-8*



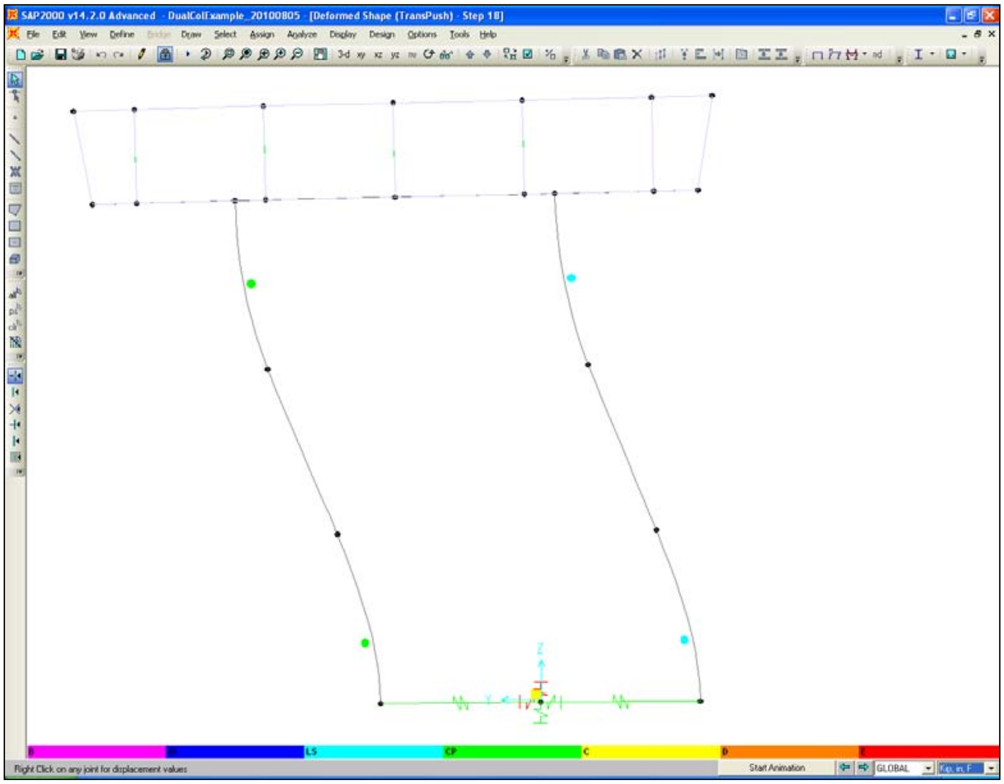

**View of Deformed Shape for the Load Case "TransPush" at UY = 7.1 in.**
*Figure 4.2.3.2-9*



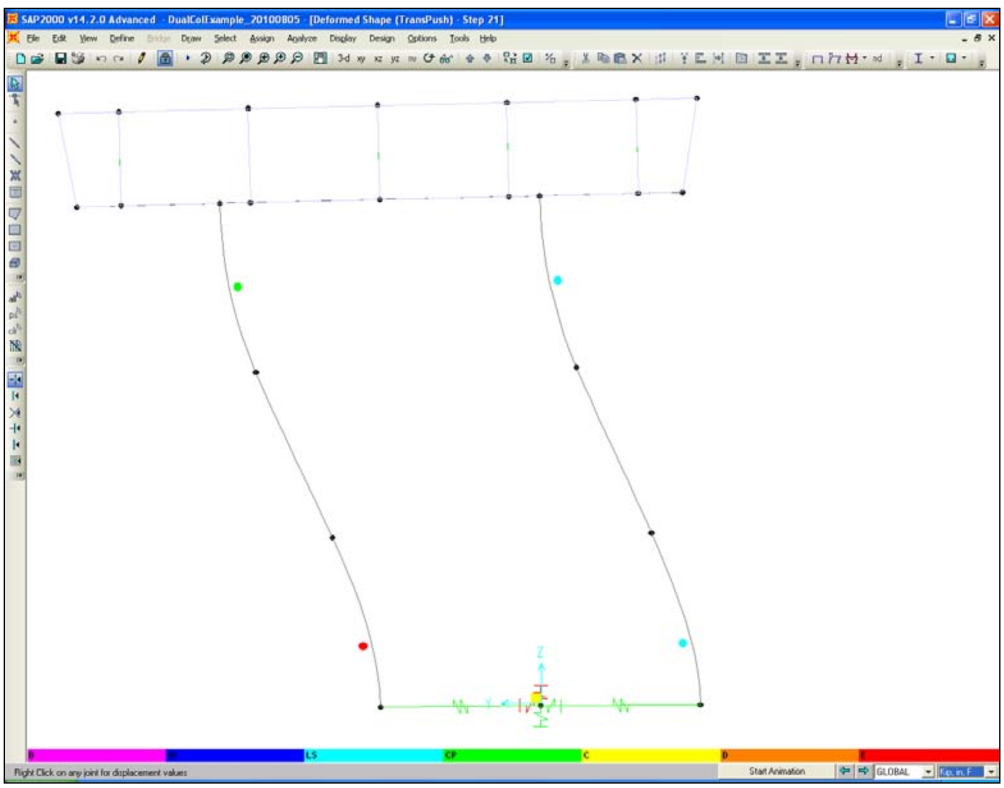

**View of Deformed Shape for the Load Case "TransPush" at UY = 7.6 in.**
*Figure 4.2.3.2-10*




**View of Deformed Shape for the Load Case "TransPush" at UY = 8.1 in.**
*Figure 4.2.3.2-11*




**View of Deformed Shape for the Load Case "TransPush" at UY = 8.6 in.**
*Figure 4.2.3.2-12*




**View of Deformed Shape for the Load Case "TransPush" at UY = 9.5 in.**
*Figure 4.2.3.2-13*

# 5. Code Requirements

## 5.1 P-Δ Capacity Requirement Check

The requirements of section 4.11.5 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* must be satisfied or a nonlinear time history analysis that includes P-Δ effects must be performed. The requirement is as follows:

$P_{dl}\Delta_r \leq 0.25\ M_p$

Where:

$P_{dl}$ = unfactored dead load acting on the column (kip)
= 1,250 kips

$\Delta_r$ = relative lateral offset between the point of contraflexure and the furthest end of the plastic hinge (in.)
= $\Delta^L{}_D$ / 2 (Assumed since the inflection point is located at approximately mid-height of the column. If the requirements are not met, a more advanced calculation of $\Delta_r$ will be performed)

$M_p$ = idealized plastic moment capacity of reinforced concrete column based upon expected material properties (kip-in.)
= 78,560 kip-in. (See Figure 3.1.2-1)

### 5.1.1 Longitudinal Direction

$0.25M_p$ = 0.25 * 78,560
= 19,640 kip-in.

$\Delta_r$ = $\Delta^L{}_{D\_Long}$ / 2
= 8.76 / 2
= 4.38 in.

$P_{dl}\Delta_r$ = 1,250 * 4.38
= 5,475 kip-in. < $0.25M_p$ = 19,640 kip-in. => Okay

### 5.1.2 Transverse Direction

$\Delta_r$ = $\Delta^L{}_{D\_Trans}$ / 2
= 6.07 / 2
= 3.04 in.

$P_{dl}\Delta_r$ = 1,250 kips * 3.04
= 3,800 kip-in. < $0.25M_p$ = 19,640 kip-in. => Okay

## 5.2 Minimum Lateral Strength Check

The requirements of Section 8.7.1 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* must be satisfied. The requirement is as follows:

$M_{ne} \geq 0.1\ P_{trib}\ (H_h + 0.5\ D_s) / \Lambda$

Where:

$M_{ne}$ = nominal moment capacity of the column based upon expected material properties as shown in Figure 8.5-1 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (kip-in.)

$P_{trib}$ = greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kip)

$H_h$ = the height from the top of the footing to the top of the column or the equivalent column height for a pile extension (in.)
= 34.0 * 12 (Top of footing to top of crossbeam)
= 408 in.

$D_s$ = depth of superstructure (in.)
= 7.083 * 12
= 85 in.

$\Lambda$ = fixity factor (See Section 4.8.1 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*)
= 2 for fixed top and bottom

Determine $P_{trib}$:

Since the abutments are being modeled as free in the longitudinal direction, all of the seismic mass is collected at the bents in the longitudinal direction. Therefore, the force associated with the tributary seismic mass collected at the bent is greater than the dead load per column and is computed as follows:

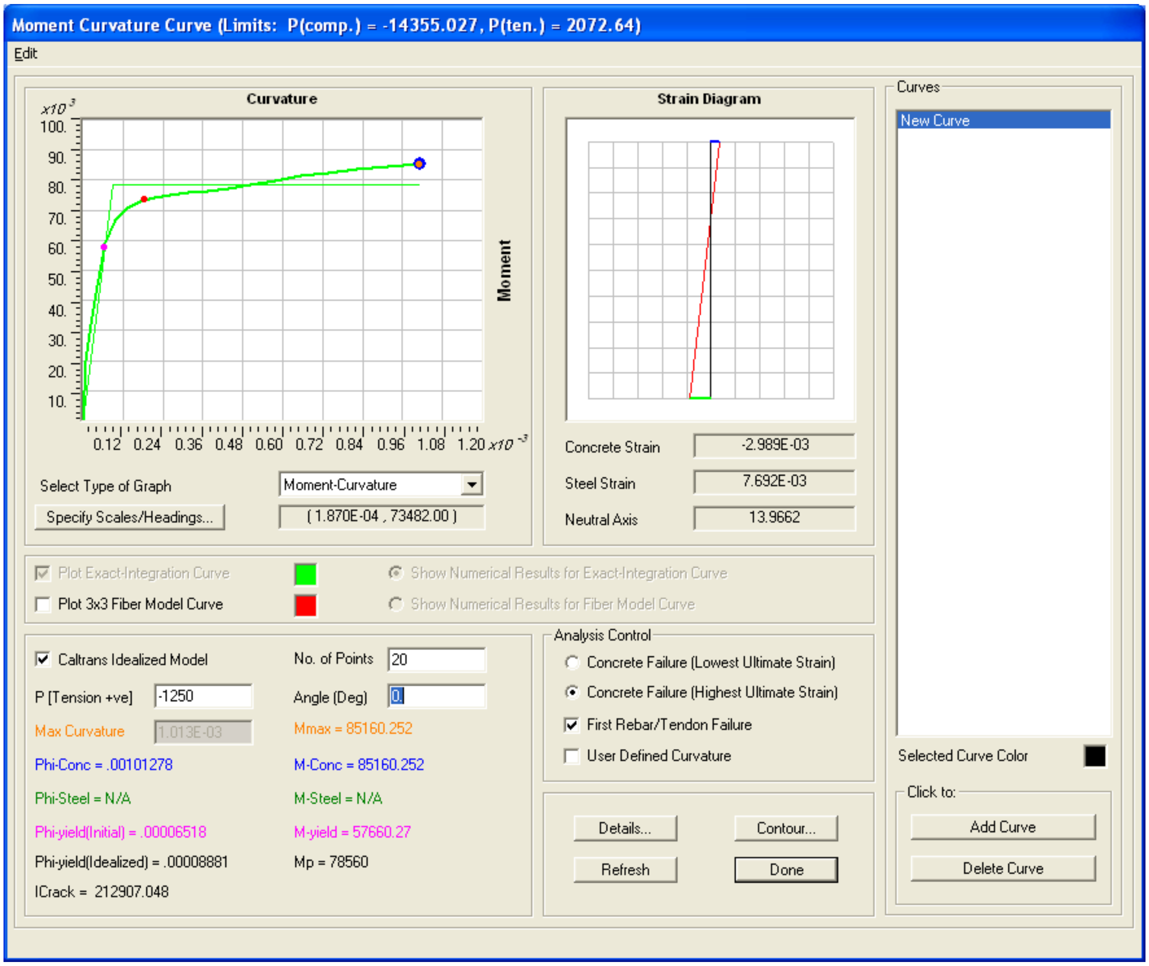
$P_{trib}$ = Weight of Structure / # of bents / # of columns per bent
= 6,638 / 2 / 2
= 1,660 kips

Note that a more sophisticated analysis to determine the tributary seismic mass would be necessary if the bridge were not symmetric and the bents did not have equal stiffness.

Determine $M_{ne}$:

Section 8.5 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* defines $M_{ne}$ as the expected nominal moment capacity based on the expected concrete and reinforcing steel strengths when the concrete strain reaches a magnitude of 0.003. Section Designer in SAP2000 can be used to determine $M_{ne}$ by performing a moment-curvature analysis and displaying the moment when the concrete reaches a strain of 0.003. The moment-curvature diagram for the column section is shown in Figure 5.2-1 with values displayed at a concrete strain of 0.002989 (**Define menu > Section Properties > Frame Sections > *select* COL > *click* Modify/Show**

**Property *button* > *click* Section Designer *button* > Display menu > Show Moment-Curvature Curve**).




**Moment-Curvature Curve for Frame Section "COL" at $\varepsilon_c$ = 0.003**
***Figure 5.2-1***

It is seen in Figure 5.2-1 that $M_{ne}$ = 73,482 kip-inches.

Perform Check:

$0.1\ P_{trib}\ (H_h + 0.5\ D_s) / \Lambda$ $= 0.1 * 1{,}660 * (408 + 0.5 * 85) / 2$
$= 37{,}392$ kip-in. $< 73{,}482$ kip-in. $= M_{ne}$ => Okay

## 5.3 Structure Displacement Demand/Capacity Check

The requirements of Section 4.8 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* must be satisfied. The requirement is as follows:

$$\Delta^L{}_D < \Delta^L{}_C$$

Where:

$\Delta^L{}_D$ = displacement demand taken along the local principal axis of the ductile member (in.)

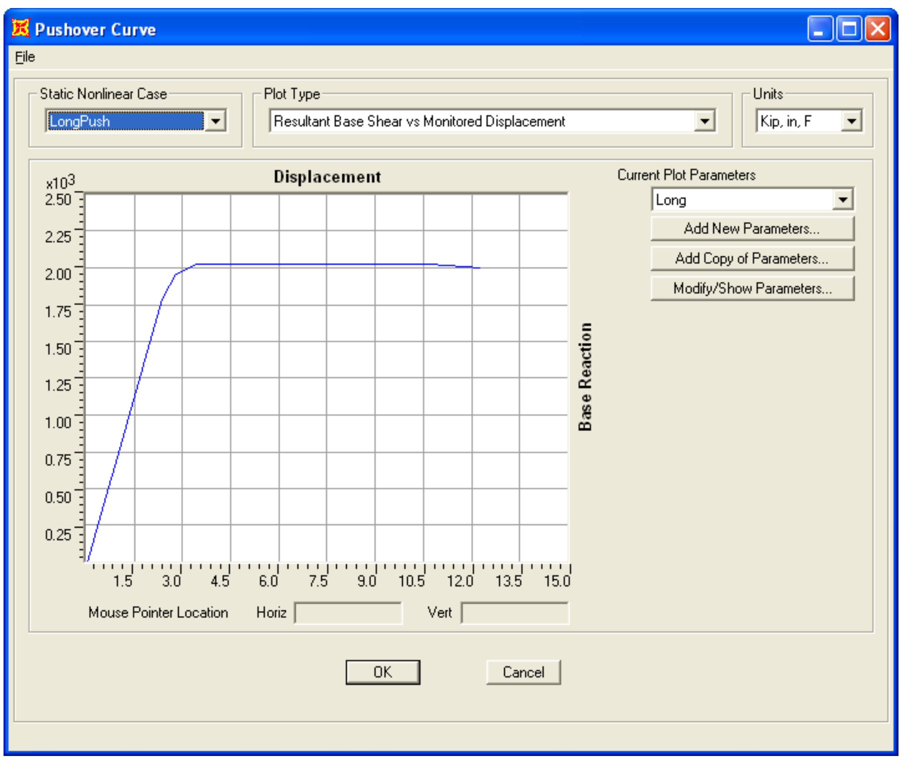
$\Delta^L{}_C$ = displacement capacity taken along the local principal axis corresponding to $\Delta^L{}_D$ of the ductile member (in.)

### 5.3.1 Longitudinal Direction

From section 3.3.2.1, the displacement demand in the longitudinal direction is $\Delta^L{}_{D\_Long}$ = 8.73 inches.

Determine $\Delta^L{}_{C\_Long}$:

The displacement capacity can be determined from the pushover curve as show in Figure 5.3.1-1 (**Display menu > Show Static Pushover Curve**).




**Pushover Curve for Load Case "LongPush"**
***Figure 5.3.1-1***

The displacement at which the first hinge ruptures (fails) is the displacement capacity of the structure and is also the point at which the base shear begins to decrease. It can be seen in Figure 5.3.1-1 that the base shear does not decrease until a displacement of approximately 11 inches. This suggests the displacement capacity of the bridge in the longitudinal direction is greater than the displacement demand. To confirm this, the table shown in Figure 5.3.1-2 can be displayed by clicking **File menu > Display Tables** in Figure 5.3.1-1.

Table Display

File Edit

Pushover Curve - LongPush

| Step | Displacemen in | BaseForce Kip | AtoB | BtoIO | IOtoLS | LStoCP | CPtoC | CtoD | DtoE | BeyondE | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0.041996 | 0.000 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 1 | 0.541996 | 386.976 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 2 | 1.041996 | 773.952 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 3 | 1.541996 | 1160.928 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 4 | 2.041996 | 1547.905 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 5 | 2.347536 | 1784.379 | 14 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 6 | 2.793085 | 1955.096 | 10 | 6 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 7 | 3.293085 | 2009.869 | 10 | 6 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 8 | 3.449424 | 2026.995 | 8 | 8 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 9 | 3.949424 | 2027.072 | 8 | 8 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 10 | 4.449424 | 2027.149 | 8 | 6 | 2 | 0 | 0 | 0 | 0 | 0 | 16 |
| 11 | 4.949424 | 2027.227 | 8 | 2 | 6 | 0 | 0 | 0 | 0 | 0 | 16 |
| 12 | 5.449424 | 2027.304 | 8 | 2 | 6 | 0 | 0 | 0 | 0 | 0 | 16 |
| 13 | 5.949424 | 2027.381 | 8 | 0 | 8 | 0 | 0 | 0 | 0 | 0 | 16 |
| 14 | 6.449424 | 2027.459 | 8 | 0 | 8 | 0 | 0 | 0 | 0 | 0 | 16 |
| 15 | 6.949424 | 2027.536 | 8 | 0 | 2 | 6 | 0 | 0 | 0 | 0 | 16 |
| 16 | 7.449424 | 2027.613 | 8 | 0 | 2 | 6 | 0 | 0 | 0 | 0 | 16 |
| 17 | 7.949424 | 2027.691 | 8 | 0 | 0 | 8 | 0 | 0 | 0 | 0 | 16 |
| 18 | 8.449424 | 2027.768 | 8 | 0 | 0 | 8 | 0 | 0 | 0 | 0 | 16 |
| 19 | 8.949424 | 2027.845 | 8 | 0 | 0 | 2 | 6 | 0 | 0 | 0 | 16 |
| 20 | 9.449424 | 2027.923 | 8 | 0 | 0 | 2 | 6 | 0 | 0 | 0 | 16 |
| 21 | 9.949424 | 2028.000 | 8 | 0 | 0 | 0 | 8 | 0 | 0 | 0 | 16 |
| 22 | 10.44942 | 2028.077 | 8 | 0 | 0 | 0 | 8 | 0 | 0 | 0 | 16 |
| 23 | 10.69028 | 2028.114 | 8 | 0 | 0 | 0 | 6 | 0 | 0 | 2 | 16 |
| 24 | 12.28880 | 2000.729 | 8 | 0 | 0 | 0 | 0 | 0 | 0 | 8 | 16 |

Current Sort String

Current Filter String

Done

**Pushover Curve Tabular Data for Load Case "LongPush"**
***Figure 5.3.1-2***

Figure 5.3.1-2 shows the step, displacement, base force, and hinge state data for the longitudinal pushover analysis. By definition, hinges fail if they are in the "Beyond E" hinge state. In Figure 5.3.1-2 it can be seen that step 23 is the first step any hinges reach the "Beyond E" hinge state. Therefore, $\Delta^{L}_{C\_Long}$ = 10.69 inches and the following can be stated:

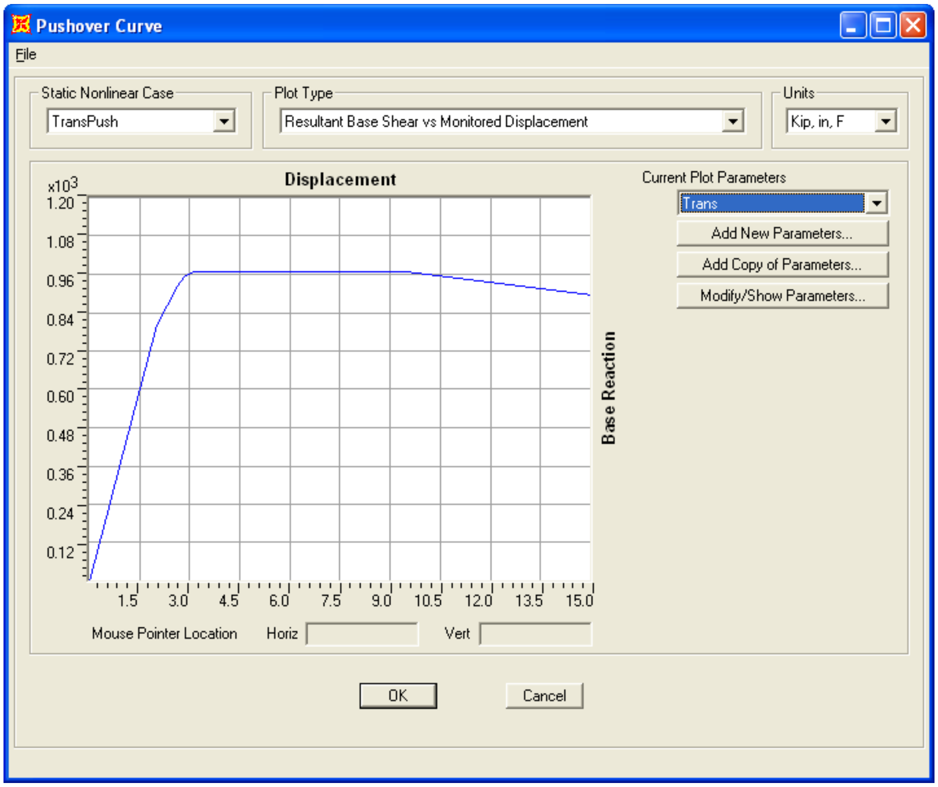
$\Delta^{L}_{C\_Long}$ = 10.69 in. > $\Delta^{L}_{D\_Long}$ = 8.76 in. => Longitudinal Displacement Demand/Capacity is Okay

## 5.3.2 Transverse Direction

From Section 3.3.2.2 of this example, the displacement demand in the transverse direction is $\Delta^{L}_{D\_Trans}$ = 6.07 inches.

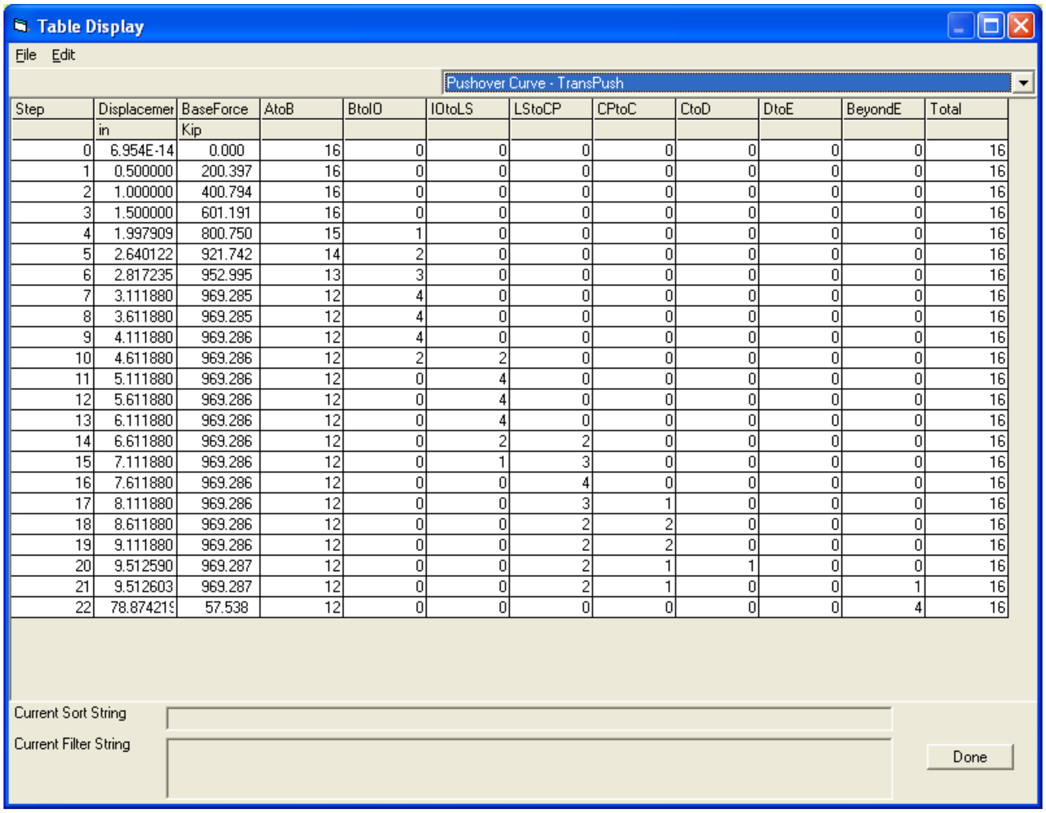
Determine $\Delta^{L}_{C\_Trans}$:

The displacement capacity can be determined from the pushover curve as show in Figure 5.3.2-1 (**Display menu > Show Static Pushover Curve**).




**Pushover Curve for Load Case "TransPush"**
***Figure 5.3.2-1***

As mentioned above, the displacement at which the first plastic hinge ruptures (fails) is the displacement capacity of the structure and is also the point at which the base shear begins to decrease. It can be seen in Figure 5.3.2-1 that the base shear does not decrease until a displacement of approximately 9.5 inches. This suggests the displacement capacity of the bridge in the transverse direction is greater than the displacement demand. To confirm this, the table shown in Figure 5.3.2-2 can be displayed by clicking **File menu > Display Tables** in Figure 5.3.2-1.




| Step | Displacemer in | BaseForce Kip | AtoB | BtoIO | IOtoLS | LStoCP | CPtoC | CtoD | DtoE | BeyondE | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 6.954E-14 | 0.000 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 1 | 0.500000 | 200.397 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 2 | 1.000000 | 400.794 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 3 | 1.500000 | 601.191 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 4 | 1.997909 | 800.750 | 15 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 5 | 2.640122 | 921.742 | 14 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 6 | 2.817235 | 952.995 | 13 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 7 | 3.111880 | 969.285 | 12 | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 8 | 3.611880 | 969.285 | 12 | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 9 | 4.111880 | 969.286 | 12 | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 16 |
| 10 | 4.611880 | 969.286 | 12 | 2 | 2 | 0 | 0 | 0 | 0 | 0 | 16 |
| 11 | 5.111880 | 969.286 | 12 | 0 | 4 | 0 | 0 | 0 | 0 | 0 | 16 |
| 12 | 5.611880 | 969.286 | 12 | 0 | 4 | 0 | 0 | 0 | 0 | 0 | 16 |
| 13 | 6.111880 | 969.286 | 12 | 0 | 4 | 0 | 0 | 0 | 0 | 0 | 16 |
| 14 | 6.611880 | 969.286 | 12 | 0 | 2 | 2 | 0 | 0 | 0 | 0 | 16 |
| 15 | 7.111880 | 969.286 | 12 | 0 | 1 | 3 | 0 | 0 | 0 | 0 | 16 |
| 16 | 7.611880 | 969.286 | 12 | 0 | 0 | 4 | 0 | 0 | 0 | 0 | 16 |
| 17 | 8.111880 | 969.286 | 12 | 0 | 0 | 3 | 1 | 0 | 0 | 0 | 16 |
| 18 | 8.611880 | 969.286 | 12 | 0 | 0 | 2 | 2 | 0 | 0 | 0 | 16 |
| 19 | 9.111880 | 969.286 | 12 | 0 | 0 | 2 | 2 | 0 | 0 | 0 | 16 |
| 20 | 9.512590 | 969.287 | 12 | 0 | 0 | 2 | 1 | 1 | 0 | 0 | 16 |
| 21 | 9.512603 | 969.287 | 12 | 0 | 0 | 2 | 1 | 0 | 0 | 1 | 16 |
| 22 | 78.87421[illegible] | 57.538 | 12 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 16 |

**Pushover Curve Tabular Data for Load Case "TransPush"**
***Figure 5.3.2-2***

Figure 5.3.2-2 shows the step, displacement, base force, and hinge state data for the transverse pushover analysis. Recall the transverse pushover analysis only includes a single bent. By definition, hinges fail if they are in the "Beyond E" hinge state. In Figure 5.3.2-2 it can be seen that step 21 is the first step any hinges reach the "Beyond E" hinge state. Therefore, $\Delta^{L}_{C\_Trans}$ = 9.51 inches and the following can be stated:

$\Delta^{L}_{C\_Trans} = 9.51$ in. $> \Delta^{L}_{D\_Trans} = 6.07$ in. => Transverse Displacement Demand/Capacity is Okay

## 5.4 Member Ductility Requirement Check

The requirements for hinge ductility demands in Section 4.9 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* must be met for all hinges in the structure. The member ductility demand may be computed as follows:

$\mu_D \quad \leq 6$ (for multiple column bents)

Where:

$\mu_D \quad$ = ductility demand
$= 1 + \Delta_{pd} / \Delta_{yi}$

$\Delta_{yi} \quad$ = idealized yield displacement (does not include soil effects) (in.)
$= \varphi_{yi} * L^2 / 3$

$L \quad$ = length from point of maximum moment to the inflection point (in.)

$\varphi_{yi} \quad$ = idealized yield curvature (1/in.)

$\Delta_{pd} \quad$ = plastic displacement demand (in.)
$= \theta_{pd} * (L - 0.5*L_p)$

$\theta_{pd} \quad$ = plastic rotation demand determined by SAP2000 (rad.)

$L_p \quad$ = plastic hinge length (in.)

Therefore:

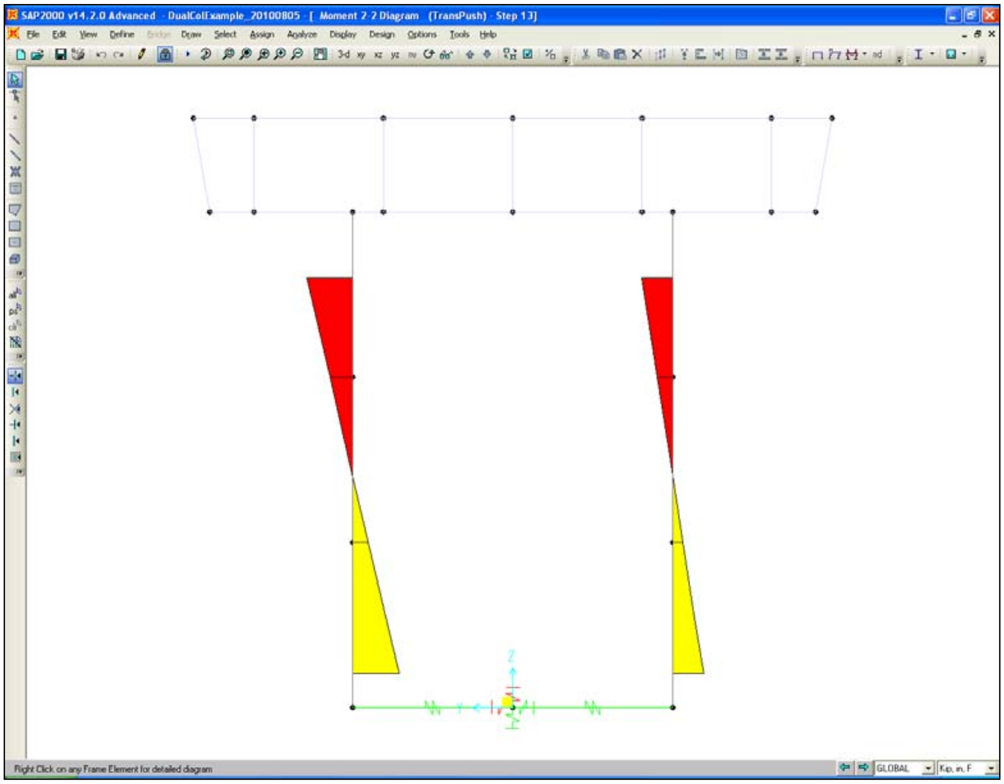
$\mu_D \quad = 1 + 3 * [\theta_{pd} / (\varphi_{yi} * L)] * (1 - 0.5 * L_p / L)$

This example will explicitly show how to compute the ductility demand for the lower hinge of the trailing column being deflected in the transverse direction. The ductility demands for the remaining hinges are presented in tabular format.

Determine L:

The locations of the inflection points were approximated previously to determine the hinge lengths. However, now that the pushover analysis has been performed, the actual inflection points can be determined.

Figure 5.3.2-2 shows that at step 13 the displacement is 6.11 inches, which is slightly greater than the displacement demand. Figure 5.4-1 shows the column moment 2-2 diagram at step 13 of the TransPush load case as displayed in SAP2000 (**Display menu > Show Forces/Stresses > Frames/Cables >** ***select*** **TransPush >** ***select*** **Moment 2-2 >** ***select*** **Step 13 >** ***click*** **OK** ***button***).



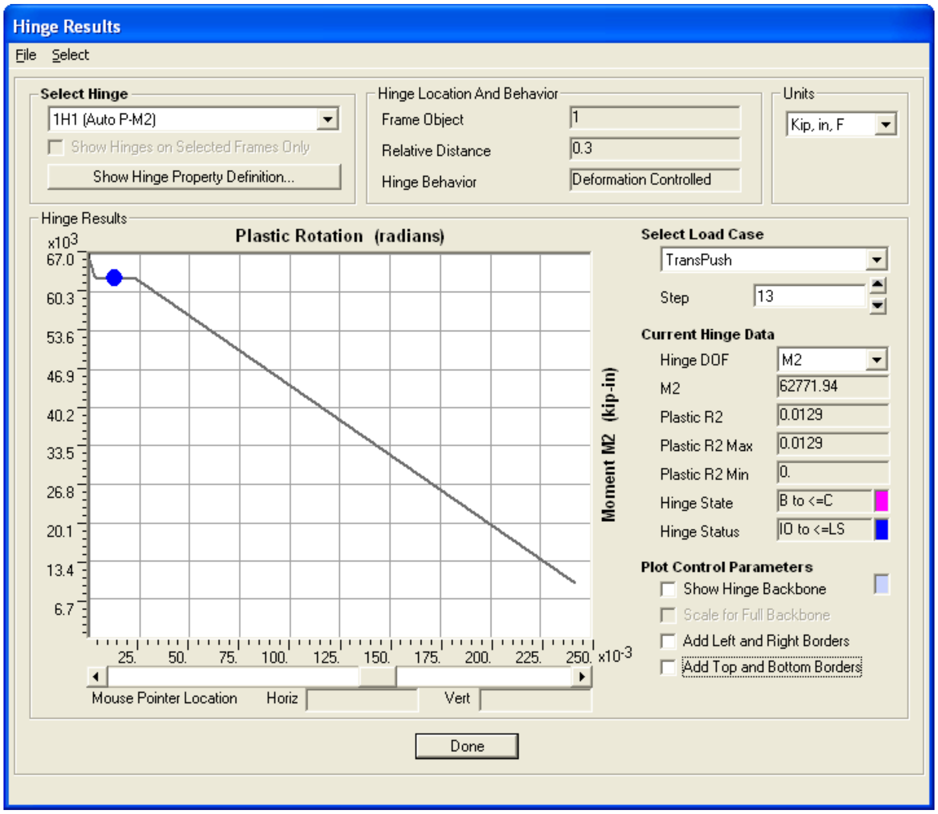
**Frame Moment 2-2 Diagram for Load Case "TransPush" at Step 13**
*Figure 5.4-1*

From this information it is found that the inflection point is 59 inches above the lower joint on the middle column element and the following is computed:

L = Length from point of maximum moment at base of column to inflection point
= Length of Lower Element – Footing Offset + 59
= 146 – 30 + 59
= 175 in.

Determine $\theta_{pd}$:

Since the displacement of the bent at step 13 is greater than the displacement demand, the plastic rotation at step 13 is greater than or equal to the plastic rotation demand. The plastic rotation at each step can be found directly from the hinge results in SAP2000. The name of the lower hinge on the trailing column is 1H1. Figure 5.4-2 shows the plastic rotation plot of hinge 1H1 at step 13 of the TransPush load case (**Display menu > Show Hinge Results >** ***select hinge*** **1H1 (Auto P-M2) >** ***select load case*** **TransPush >** ***select step*** **13 >** ***click*** **OK** ***button***).

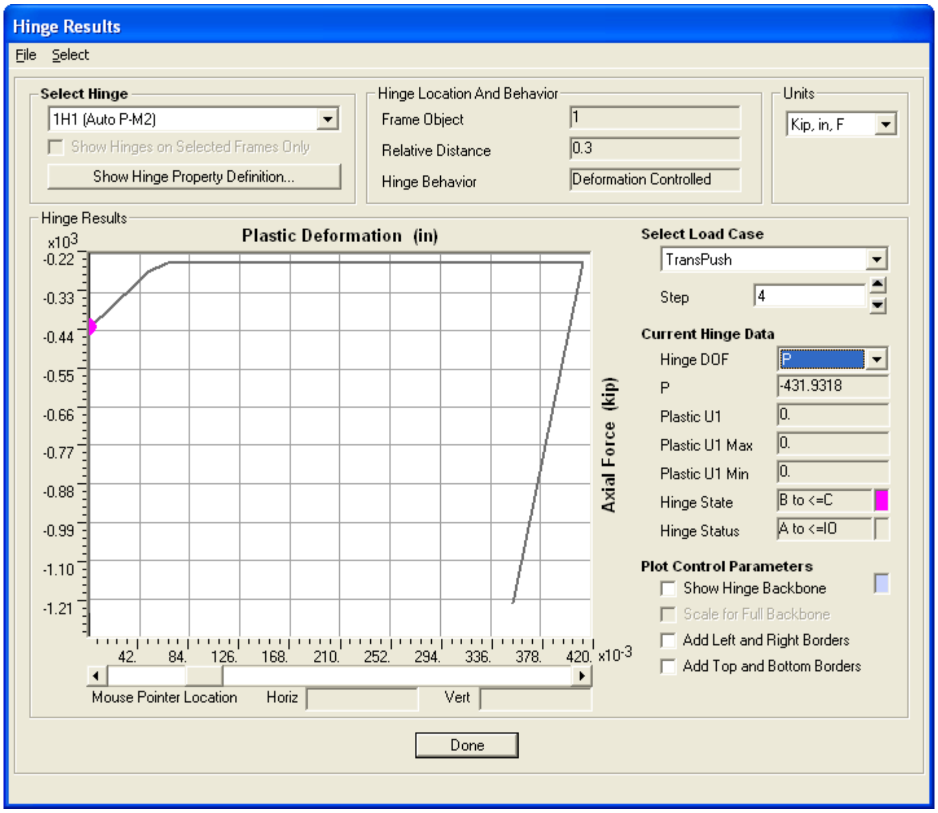



**Hinge "1H1" Plastic Rotation Results for Load Case "TransPush" at Step 13**
***Figure 5.4-2***

Figure 5.4-2 shows that the plastic rotation for hinge 1H1 is 0.0129 radians. Therefore $\theta_{pd}$ = 0.0129 radians.

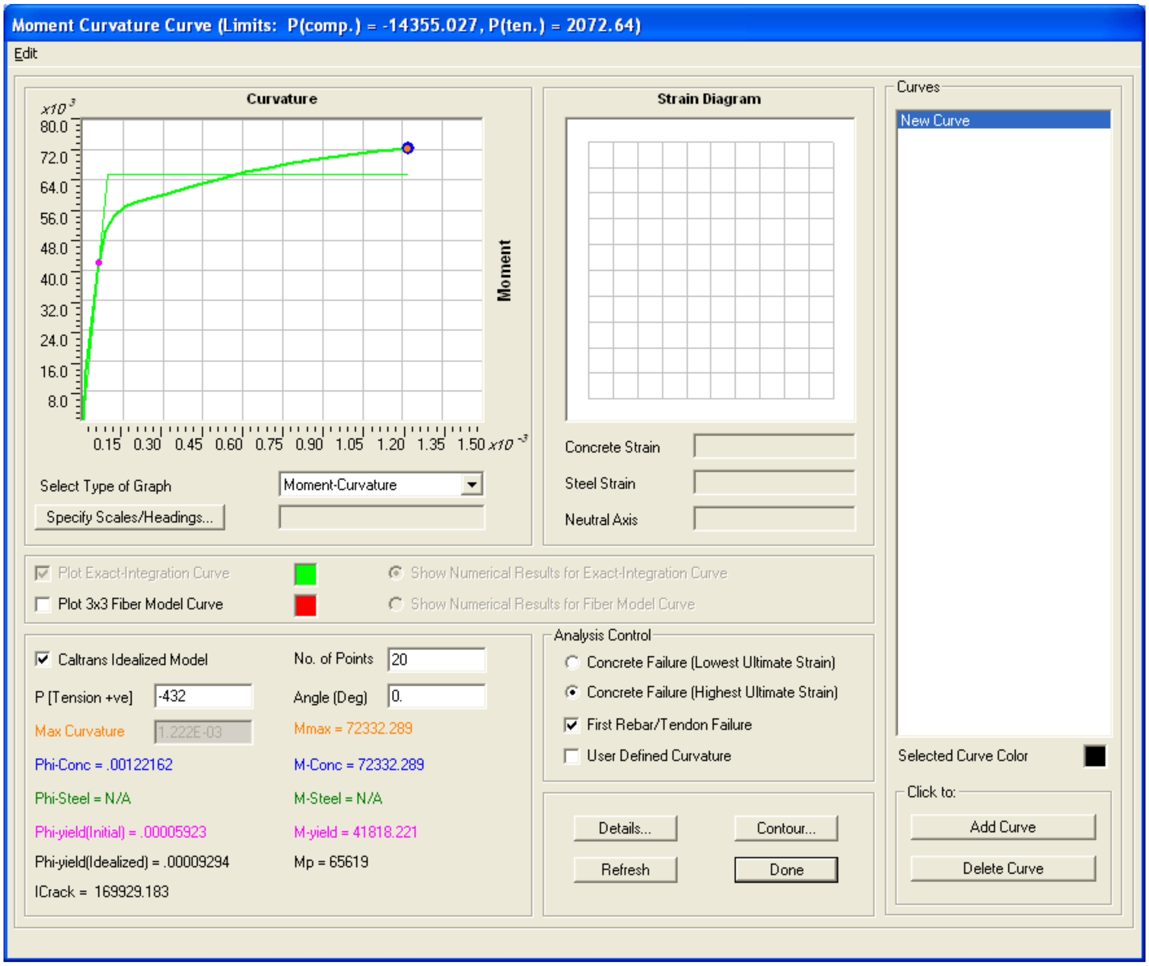
Determine $\varphi_{yi}$:

The idealized yield curvature will be found by determining the axial load in the hinge at first yield and then inputting that load into Section Designer. The axial load at yield can be found by viewing the hinge results at step 4 (when the hinge first yields). Figure 5.4-3 shows the axial plastic deformation plot of hinge 1H1 at step 4 of the TransPush load case (**Display menu > Show Hinge Results >** ***select hinge*** **1H1 (Auto P-M2) >** ***select load case*** **TransPush >** ***select step*** **4 >** ***select hinge DOF*** **P >** ***click*** **OK** ***button***).




**Hinge "1H1" Axial Plastic Deformation Results for Load Case "TransPush" at Step 4**
***Figure 5.4-3***

Figure 5.4-3 shows that the axial load in hinge 1H1 at step 4 of the TransPush load case is -432 kips. That load can now be entered into Section Designer to determine the idealized yield curvature, $\varphi_{yi}$. The moment-curvature diagram for the column section with P = -432 kips is shown in Figure 5.4-4 (**Define menu** > **Section Properties** > **Frame Sections** > ***select*** **COL** > ***click*** **Modify/Show Property** ***button*** > ***click*** **Section Designer** ***button*** > **Display menu** > **Show Moment-Curvature Curve**).




**Moment-Curvature Curve for Frame Section "COL" at P = -432 kips**
***Figure 5.4-4***

Figure 5.4-4 shows that Phi-yield(Idealized) = .00009294. Therefore: $\varphi_{yi} = 0.00009294$ inches$^{-1}$.

The ductility demand in the transverse direction for the lower hinge in the trailing column can now be calculated as follows:

$$\mu_D = 1 + 3 * [\theta_{pd} / (\varphi_{yi} * L)] * (1 – 0.5 * L_p / L)$$

Where:

$L$ = 175 in.

$\varphi_{yi}$ = 0.00009294 in.$^{-1}$

$\theta_{pd}$ = 0.0129 rad.

$L_p$ = 27.0 in.

Therefore:

$$\mu_D = 1 + 3 * [0.0129 / (0.00009294 * 175)] * (1 – 0.5 * 27.0 / 175)$$
$$= 3.2 < 6 \Rightarrow \text{okay}$$

The ductility demands and related values for all column hinges are shown in Table 5.4-1.

| Pushover Direction | Column and Hinge Location | Hinge Name | Yield Step | Axial Load at Yield | $\varphi_{yi}$ | $\theta_{pd}$ | $L_p$ | L | $\mu_D$ |
|---|---|---|---|---|---|---|---|---|---|
| (Long/Trans) | (-) | (-) | (#) | (kips) | (1/in.) | (rad.) | (in.) | (in.) | (-) |
| Longitudinal | Trailing Lower | 1H2 | 5 | -1222 | .0000889 | .0207 | 27.0 | 175 | 4.7 |
| Longitudinal | Trailing Upper | 3H2 | 6 | -1135 | .00008926 | .0204 | 26.9 | 175 | 4.6 |
| Longitudinal | Leading Lower | 7H2 | 6 | -1354 | .00008851 | .0195 | 27.0 | 175 | 4.5 |
| Longitudinal | Leading Upper | 9H2 | 8 | -1277 | .0000887 | .0168 | 26.9 | 175 | 4.0 |
| Transverse | Trailing Lower | 1H1 | 4 | -432 | .00009294 | .0129 | 27.0 | 175 | 3.2 |
| Transverse | Trailing Upper | 3H1 | 5 | -305 | .00009292 | .0118 | 26.9 | 175 | 3.0 |
| Transverse | Leading Lower | 4H1 | 6 | -2226 | .00009185 | .0104 | 27.0 | 175 | 2.8 |
| Transverse | Leading Upper | 6H1 | 7 | -2253 | .00009186 | .00902 | 26.9 | 175 | 2.6 |

**Ductility Demands for All Column Hinges**
***Table 5.4-1***

Table 5.4-1 shows that all hinge ductility demands are less than 6.

# 5.5 Column Shear Demand/Capacity Check

The column shear requirements in Section 8.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* must be met for all columns in the structure.

$\varphi_s V_n \geq V_u$

Where:

$\varphi_s = 0.9$
$V_n$ = nominal shear capacity (kips)
$= V_c + V_s$

Concrete Shear Capacity:

$V_c$ = concrete contribution to shear capacity (kips)
$= v_c A_e$

Where:

$A_e = 0.8\ A_g$
$A_g$ = gross area of member cross-section (in.$^2$)

$v_c$ if $P_u$ is compressive:

$v_c = 0.032\ \alpha'\ [1 + P_u / (2\ A_g)]\ f'_c{}^{1/2} \leq \min(\ 0.11\ f'_c{}^{1/2}\ ,\ 0.047\ \alpha'\ f'_c{}^{1/2})$

$v_c$ otherwise:

$v_c = 0$

For circular columns with spiral reinforcing:

$0.3 \leq \alpha' = f_s / 0.15 + 3.67 - \mu_D \leq 3$
$f_s = \rho_s f_{yh} \leq 0.35$
$\rho_s = (4\ A_{sp}) / (s\ D')$

Where:

$P_u$ = ultimate compressive force acting on section (kips)
$A_{sp}$ = area of spiral (in.$^2$)
$s$ = pitch of spiral (in.)
$D'$ = diameter of spiral (in.)
$f_{yh}$ = nominal yield stress of spiral (ksi)
$f'_c$ = nominal concrete strength (ksi)
$\mu_D$ = maximum local ductility demand of member

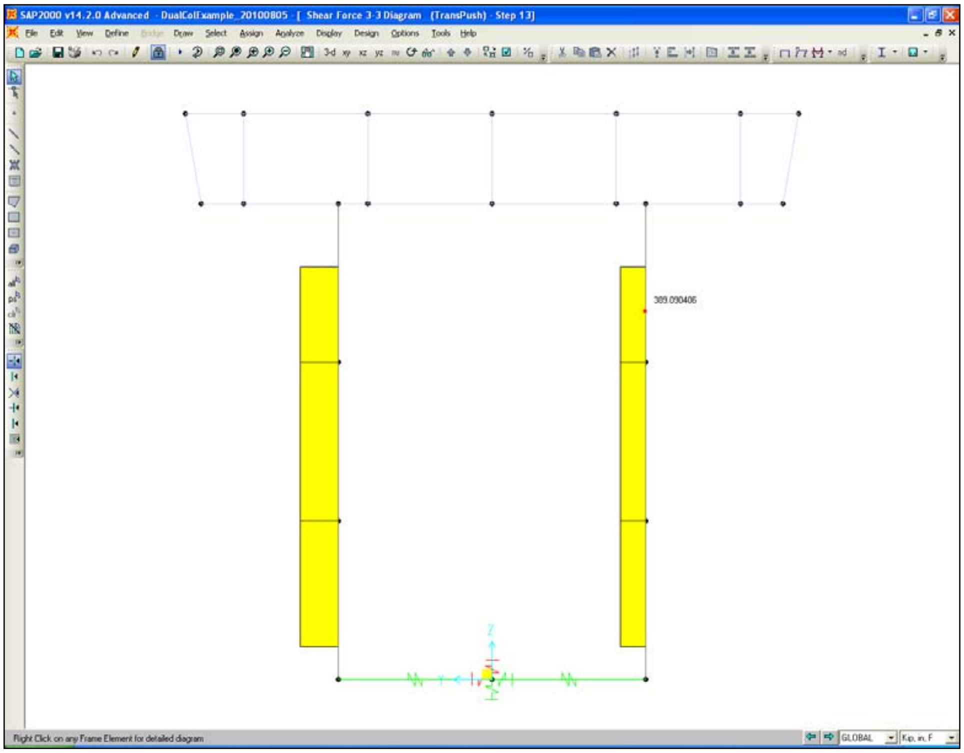
Steel Shear Capacity:

$V_s$ = steel contribution to shear capacity (kips)
$V_s = (\pi / 2)\ (A_{sp}\ f_{yh}\ D') / s$

This example will explicitly show how to perform the shear demand/capacity check for the trailing column being deflected in the transverse direction. The shear demand/capacity checks for the remaining columns are presented in tabular format.

Determine $V_u$:

Figure 5.5-1 shows the column shear diagram for the TransPush load case as displayed in SAP2000 (**Display menu > Show Forces/Stresses > Frames/Cables >** ***select*** **TransPush >** ***select*** **Shear 3-3 >** ***select*** **Step 13 >** ***click*** **OK** ***button***).




**Frame Shear 3-3 Diagram for Load Case "TransPush" at Step 13**
***Figure 5.5-1***

From Figure 5.5-1 it is determined that the plastic shear in the trailing column is 389 kips. Section 8.6.1 states that $V_u$ shall be determined on the basis of $V_{po}$, which is the shear associated with the overstrength moment, $M_{po}$, defined in Section 8.5 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. For ASTM A 706 reinforcement the overstrength magnifier is 1.2, and so the shear for the SAP2000 model must be multiplied by this factor.

Therefore:

$V_u \quad = \lambda_{mo} V_p$

Where:

$\lambda_{po} \quad = 1.2$

$V_p \quad = 389$ kips

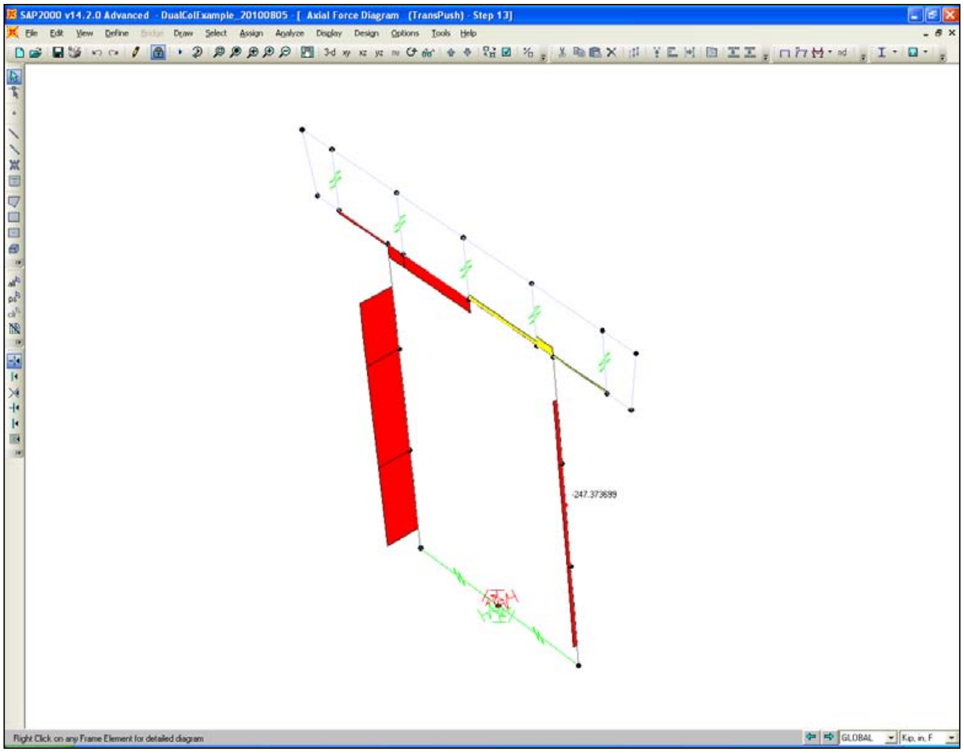
and

$V_u \quad = 1.2 * 389$

$\quad = 467$ kips

Determine $V_c$:

Figure 5.5-2 shows the column axial load diagram for the TransPush load case as displayed in SAP2000 (**Display menu > Show Forces/Stresses > Frames/Cables >** ***select*** **TransPush >** ***select*** **Axial Force >** ***select*** **Step 13 >** ***click*** **OK** ***button***).




**Frame Axial Force Diagram for Load Case "TransPush" at Step 13**
***Figure 5.5-2***

From Figure 5.5-2 it is determined that the axial force in the trailing column is -247 kips.

Therefore:

$P_u = 247$ kips

$A_g = \pi * 60^2 / 4$
$= 2827.4 \text{ in.}^2$

$A_e = 0.8\ A_g$
$= 0.8 * 2827.4$
$= 2262 \text{ in.}^2$

$A_{sp} = 0.44 \text{ in.}^2$
$s = 3.5$ in.

$D' = 60 – 1.5 – 1.5 – 0.75$
$= 56.25$ in.

$f_{yh} = 60$ ksi
$f'_c = 4$ ksi
$\rho_s = (4\ A_{sp}) / (s\ D')$
$= (4 * 0.44) / ( 3.5 * 56.25)$
$= 0.0089$

$f_s = \rho_s f_{yh} \leq 0.35$
$= 0.0089 * 60 \leq 0.35$
$= 0.54 \leq 0.35$
$= 0.35$ ksi

$\mu_D$ = 3.2 (see Section 5.4 of this example)

$0.3 \leq \alpha'$ $= f_s / 0.15 + 3.67 - \mu_D \leq 3$
$= 0.35 / 0.15 + 3.67 – 3.2 \leq 3$
$= 0.35 / 0.15 + 3.67 – 3.2 \leq 3$
$= 2.8 \leq 3$

$\alpha'$ = 2.8

$v_c$ $= 0.032\ \alpha'\ [1 + P_u / (2\ A_g)]\ f'_c{}^{1/2} \leq \min(\ 0.11\ f'_c{}^{1/2}\ , 0.047\ \alpha'\ f'_c{}^{1/2})$
$= 0.032 * 2.8 * [1 + 247 / (2 * 2827.4)]\ 4^{1/2} \leq \min(0.11 * 4^{1/2}, 0.047 * 2.8 * 4^{1/2})$
$= 0.187 \leq \min(0.22\ , 0.263)$
= 0.187 ksi

$V_c$ $= v_c A_e$
= 0.187 * 2262
= 423 kips

Determine $V_s$:
$V_s$ $= (\pi / 2)\ (A_{sp}\ f_{yh}\ D') / s$
$= (\pi / 2)\ (0.44 * 60 * 56.25) / 3.5$
= 666 kips

Determine $\varphi_s V_n$:

$\varphi_s V_n$ $= \varphi_s\ (V_c + V_s)$
= 0.9 * (423 + 666)
= 980 kips > $V_u$ = 467 kips => okay

The shear demands and capacities and related values for all columns are shown in Table 5.5-1.

| Pushover Direction | Column | $V_p$ | $V_u$ | $P_u$ | $\mu_D$ | $\alpha'$ | $v_c$ | $V_c$ | $V_s$ | $\varphi_s V_n$ |
|---|---|---|---|---|---|---|---|---|---|---|
| (Long/Trans) | (-) | (kips) | (kips) | (kips) | (-) | (-) | (ksi) | (kips) | (kips) | (kips) |
| Longitudinal | Trailing | 484 | 581 | 1175 | 4.7 | 1.3 | 0.10 | 228 | 666 | 804 |
| Longitudinal | Leading | 497 | 596 | 1320 | 4.5 | 1.5 | 0.12 | 268 | 666 | 841 |
| Transverse | Trailing | 389 | 467 | 247 | 3.2 | 2.8 | 0.19 | 423 | 666 | 980 |
| Transverse | Leading | 580 | 696 | 2253 | 2.8 | 3.0 | 0.22 | 498 | 666 | 1047 |

**Column Shear Demands and Capacities**
***Table 5.5-1***

Table 5.5-1 shows that the shear capacities are greater than the shear demands for all columns.

# 5.6 Balanced Stiffness and Frame Geometry Requirement Check

The balanced stiffness and balanced frame geometry requirements of Sections 4.1.2 and 4.1.3 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* must be met. Due to the symmetry of this example, these requirements are okay by inspection. However, on many bridges these requirements may highly influence the design.

## 4.99 References

AASHTO *LRFD Bridge Design Specifications*, 6th Edition, 2012

AASHTO *Guide Specifications for LRFD Seismic Bridge Design*, 2nd Edition, 2011

AASHTO *Guide Specifications for Seismic Isolation Design*, 3rd Edition, 2010

Caltrans *Bridge Design Aids* 14 4 Joint Shear Modeling Guidelines for Existing Structures, California Department of Transportation, August 2008

FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges*, Publication No. FHWA-HRT-06-032, January 2006

McLean, D.I. and Smith, C.L., *Noncontact Lap Splices in Bridge Column-Shaft Connections*, Report Number WA-RD 417.1, Washington State University

WSDOT *Geotechnical Design Manual* M 46-03, Environmental and Engineering Program, Geotechnical Services, Washington State Department of Transportation