20.22 ANALYSIS OF NEW BRIDGES IN FAULT RUPTURE ZONES

20.22.1 GENERAL

This BDM provides an analysis procedure for ordinary and recovery bridges that cross a surface fault rupture and are required to be evaluated for fault rupture offset.

Surface faults can vary from a well-defined single trace to a poorly-defined zone of disruption and from a horizontal to a nearly vertical ground displacement, as depicted in Figure 20.22.1-1. The location of the fault, or fault zone, with respect to the structure, and a determination of the design fault offset should be included in the project's foundation report. The bridge designer uses this information to meet the performance requirements in Caltrans Seismic Design Criteria (SDC).

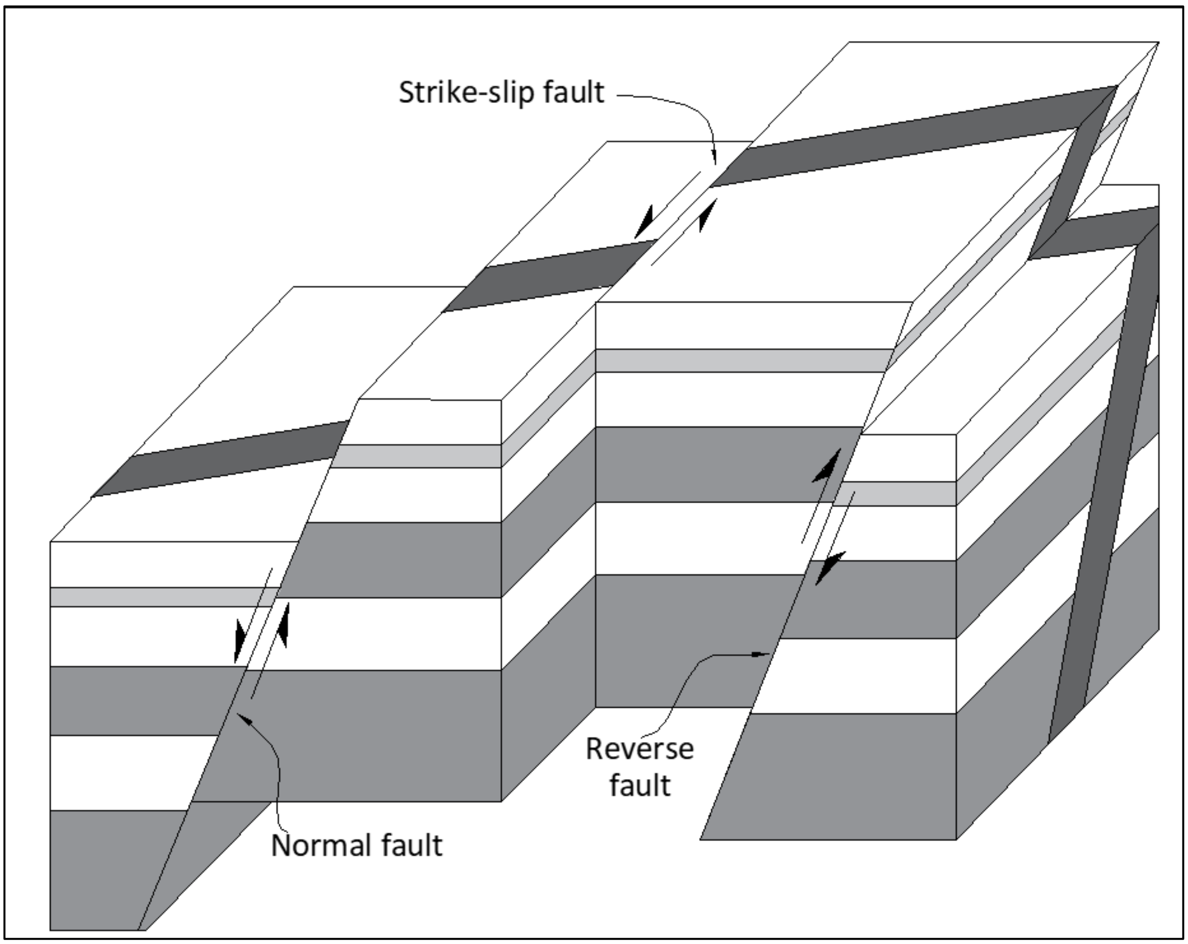


Figure 20.22.1-1 Schematic of main types of faults

20.22.2 DEFINITIONS

Dip-Slip Movement — The motion of rocks along a fault plane, either upward (reverse fault) or downward (normal fault) relative to the footwall.

Fault Offset —The relative movement across a surface fault rupture.

Strike-Slip Fault — A fault in which rock strata are displaced mainly in a horizontal direction, parallel to the fault trace.

Vertical Offset — The vertical displacement between the two sides of a fault

20.22.3 NOTATION

A_{max} = Peak spectral acceleration coefficient

A_i = Response spectrum acceleration coefficient corresponding to the vibration period of mode *i*

F_i = Modal load vector of mode *i* (kip)

F_{LSA} = Transverse force applied at the superstructure support locations to estimate the elastic response of the bridge (kip)

Δ_{FR} = Fault offset at each ground nodes of the soil springs (in.)

g = Acceleration due to gravity (ft./sec^2)

m = Matrix of superstructure mass lumped at superstructure support locations with zero off-diagonal elements (superstructure tributary mass at the bents and abutments) (kip- sec^2/ft.)

$m_{eff,i}$ = Effective transverse modal mass for vibration mode i (kip- sec^2/ft.)

M_P = Column plastic moment (kip-ft.)

MR_i = Mass participation factor for vibration mode *i*

m_{tot} = Total superstructure mass computed as the sum of the elements of the mass vector *m* (kip- sec^2/ft.)

P_{dl} = Axial load attributed to dead load (kips)

Δ_{SUPER} = Superstructure displacement due to fault offset (in.)

Γ_i = Fault rupture modal participation factor for vibration mode *i*

Δ_c = Displacement capacity at each superstructure support location (in.)

$\Delta_{cP\Delta}$ = Maximum displacement capacity based on P-Δ limit (in.)

Δ_N = Fault rupture influence vector (normalized superstructure transverse displacements due to a unit fault offset)

Δ_u = Total displacement demand at each superstructure support location (in.)

$\Delta_{u,FR}$ = Net superstructure displacement at the support location (in.)

$\Delta_{u,LDA}$ = Displacement demands from the Linear Dynamic Analysis method (in.)

$\Delta_{u,LSA}$ = Displacement demands from the Linear Static Analysis method (in.)

ϕ_i = Vector of transverse components for vibration mode *i*

CMR = Cumulative effective modal mass ratio

NTHA = Nonlinear time history analysis

SRSS = Square root of the sum of the squares

20.22.4 INTRODUCTION

A bridge that crosses an active fault is subject to a combination of seismic hazards. The ground shaking component can cause a large dynamic deformation demand due to near fault effects such as directivity and the velocity pulse. The surface rupture component can cause an additional large quasi-static deformation demand due to the fault offset and angle of rupture relative to the bridge. While a nonlinear time history analysis (NTHA) using multiple time history records can be used to estimate the seismic demands for bridges crossing faults, an alternative method is presented here with simplified procedures appropriate for bridge designers addressing typical strike-slip fault crossings, which are the most common faulting situations in California. The analysis presented in this document combines these seismic hazards in a simplified approach. This simplified procedure may be used for new ordinary and recovery bridge projects with fault crossing in lieu of an NTHA.

Depending on the magnitude of the design fault offset, the resulting deformation can be addressed using ductile columns with adequate displacement capacity and supports with adequate support length. For some design fault offsets, hinges and isolation bearings can be designed to handle the resulting displacements. For relatively large design fault offsets, other strategies, such as designing the bridge with wide bents that can slide under the superstructure while continuing to support it, may be needed. When sacrificial abutment shear keys are used, they are typically ignored in the analysis since they are designed to fail during a large earthquake. However, if the abutment shear keys are not designed to fail, they should be included in the analysis model using springs as they can cause larger column displacements by restraining the ends of the superstructure.

20.22.5 ANALYSIS PROCEDURE AND CONSIDERATIONS

The procedure outlined in this memo and the companion STP 20.22 (Caltrans 2021) is based on research conducted by Chopra and Goel (2008). This procedure can be used for ordinary and recovery bridges. The design fault offset return period is 975 years (5 % probability of exceedance in 50 years) for both ordinary and recovery bridges.

Seismic displacement demands at the superstructure support locations require the evaluation of two response quantities: (1) displacement demands due to static fault offset, i.e., static displacement demands, and (2) displacement demands due to ground shaking, i.e., dynamic displacement demands. The bridge foundations on one side of a strike–slip fault are subjected to half the fault offset in one direction while the foundations on the other side are subjected to half the fault offset in the opposite direction (See Figure 20.22.5-1). This fault offset is the static displacement component of the fault rupture demand. Subsequently, the bridge is subjected to the dynamic ground shaking hazard. The dynamic ground shaking hazard is analogous to the ground shaking hazard described in the SDC but captures the opposing directionality of shaking on each side of the fault rupture. The resulting static and dynamic displacements are combined to obtain the fault rupture seismic displacement demand. The response of interest is the relative displacement between the superstructure and the ground. Other responses, such as shear and flexural demands, are based on plastic hinging of the columns and are not the focus of this document; those aspects, including capacity protected member design and related force demands, are governed by the Caltrans Seismic Design Criteria (SDC).

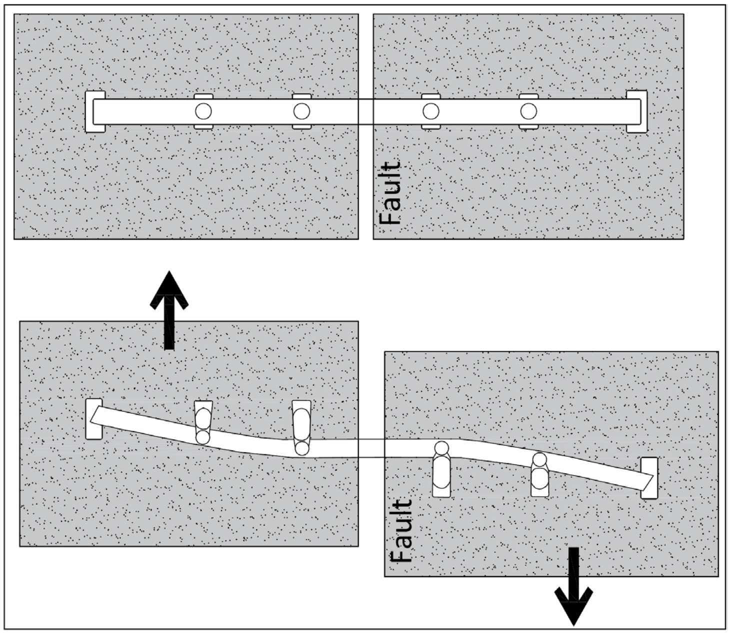
Figure 20.22.5-1 shows a fault that is normal to the longitudinal axis of the bridge. This is not likely to be the case. A more likely scenario is a fault that is at some angle off normal. When the horizontal rupture offset is not normal to the bridge's longitudinal axis, the horizontal rupture offset is resolved into two components, one parallel to the longitudinal axis of the bridge and one perpendicular to it. For the combined static and dynamic response, the horizontal fault rupture component that is normal to the bridge longitudinal axis should be used. The parallel horizontal fault rupture component and any vertical offset should be considered as static deformations to be applied to the bridge model in the pushover analysis.

20.22.6 STEPS OF PROCEDURE

20.22.6.1 Obtain the Design Fault Offset, Ground Shaking Hazard, and the Quasi-Static Response of the Structure

Obtain the predicted location, amount, and direction of displacement at the structure due to the fault offset.

A model of the bridge, including column plastic hinges, foundation soil springs, and springs for the shear keys and soil at the abutments based on the parameters provided in the SDC, is required to capture the behavior of the bridge for the design fault offset. The bridge model should include the foundations, which will have the net effect of reducing column drift demands. Gravity loads are applied to the bridge model, followed by foundation offsets due to the fault movement. The fault rupture displacement demand is the relative displacement between the center of gravity of the superstructure and the ground.



Note:

In this example the abutment shear keys do not break, and the bents closest to the fault have the most relative column displacement.

Figure 20.22.5-1. Plan view of a bridge crossing a right-lateral strike-slip fault that is perpendicular to the bridge

20.22.6.2 Obtain the Dynamic Response of the Structure

Chopra and Goel (2008) proposed procedures for computing the dynamic part of the response, including Linear Dynamic Analysis (LDA) and Linear Static Analysis (LSA) that can be implemented using structural analysis software, such as CSiBridge. LSA is easier to use, but it may be too conservative since it uses the peak acceleration of the response spectrum to estimate the dynamic response of the bridge (see below). Therefore, LDA should be used whenever possible.

20.22.6.2.1 Linear Static Analysis (LSA)

The dynamic response of the nonlinear bridge may be estimated with a linear static analysis. Furthermore, the elastic response of the bridge can be conservatively estimated by a static analysis of the structure due to the transverse forces F_{LSA} applied at the superstructure support locations. F_{LSA} is computed as:

$$F_{LSA} = mg\Delta_N A_{max} \tag{20.22.6.2.1-1}$$

20.22.6.2.2 Linear Dynamic Analysis (LDA)

LDA is carried out using the bridge vibration modes with the highest transverse mass participation factors for fault rupture. Enough modes are selected such that the cumulative transverse mass participation factor is at least 90%. The effective transverse modal mass for fault rupture is computed as:

$$m_{eff,i} = \frac{\left(\phi_i^T m \Delta_N\right)^2}{\phi_i^T m \phi_i} \qquad (20.22.6.2.2\text{-}1)$$

Once the modes are selected, compute the modal load vector, F_i, for each selected mode as:

$$F_i = \Gamma_i m g \phi_i A_i \qquad (20.22.6.2.2\text{-}2)$$

$$\Gamma_i = \left(\phi_i^T m \Delta_N\right) / \left(\phi_i^T m \phi_i\right) \qquad (20.22.6.2.2\text{-}3)$$

The next step is to determine modal displacement demands, $\Delta_{u,i}$, using static analysis of the bridge under the action of the modal load vectors, F_i. The load vector for each mode is assigned to a separate static load case in the analysis model. The result of load case *i* is the modal displacement demand, $\Delta_{u,i}$ of mode *i*. Superstructure dynamic displacement demand at each support location is obtained by the SRSS combination of the modal displacement demands.

20.22.6.3 Combine the Static and Dynamic Response to Obtain the Seismic Demand

The peak values of seismic demands are obtained by superposition of the peak values of the static and dynamic parts of the response to provide a generally conservative estimate of deck displacements, when compared to the results of the NTHA.

20.22.6.4 Perform Transverse Pushover Analysis to Obtain the Displacement Capacity at Each Bent

A transverse pushover analysis is performed on the bridge. This may be done one bent at a time or on the whole bridge. The displacement capacity at each bent shall be greater than the displacement demand obtained in Section 20.22.6.3. Gravity loads are applied first, followed by vertical and longitudinal ground offsets, if present, then the transverse pushover. The displacement capacity is evaluated per SDC 3.5 requirements, based on the curvature ductility of the columns. P-Δ moments based on displacement demands should not exceed 25% of the column plastic moment, M_p. A step-by-step analysis example of a bridge designed for fault rupture is presented in Attachment A.

20.22.7 REFERENCES

1. Caltrans (2025). *Seismic Design Criteria, Version 2.1.* California Department of Transportation, Sacramento, CA
2. Caltrans (2021). Structure Technical Policy 20.22 - *Analysis of New Bridges in Fault Rupture Zones*, California Department of Transportation, Sacramento, CA.
3. Chopra, A. K., and Geol, R. K. (2008). *Analysis of Ordinary Bridges Crossing Fault-Rupture Zones,* Report No. UCB/EERC-2008/01, University of California at Berkeley, CA.

20.22.8 APPENDIX A – DESIGN EXAMPLE

The 16th Street OC/OH Bridge on Route 215 in San Bernardino (Figure 20.22.8-1) is selected as the prototype bridge for this example. This bridge was designed for both horizontal and vertical fault offsets. The geotechnical engineer required that the design consider that the fault rupture can occur between any two supports. This example demonstrates the fault rupture analysis procedure of a fault rupture between Bents 4 and 5. The bridge designer should apply the same procedure to analyze different fault rupture locations and find the maximum seismic displacement demand. Figure 20.22.8-1 shows the bridge general plan. The 16th Street OC/OH Bridge is a six span CIP/PS Box Girder bridge supported on skewed 3 column bents. All 3 columns in each bent are supported on a single pile cap footing. The typical bent layout is presented in Figure 20.22.8.2.

The actual bridge upon which this design example is based was designed for a horizontal fault rupture offset of 19.7 inches and a vertical offset of 7.9 inches. However, this design example does not address vertical offset. The vertical offset is a static case that should be handled in the pushover analysis of the bridge. This example focuses on the horizontal offset and its impact on the seismic displacement demands at superstructure support locations, the bents, and abutments. The horizontal fault offset is assumed to occur in a direction normal to the deck centerline between Stations 22+16.144 and 24+29.868. The project’s Acceleration Response Spectrum (ARS) associated with the design fault offset is depicted in Figure 20.22.8-3.

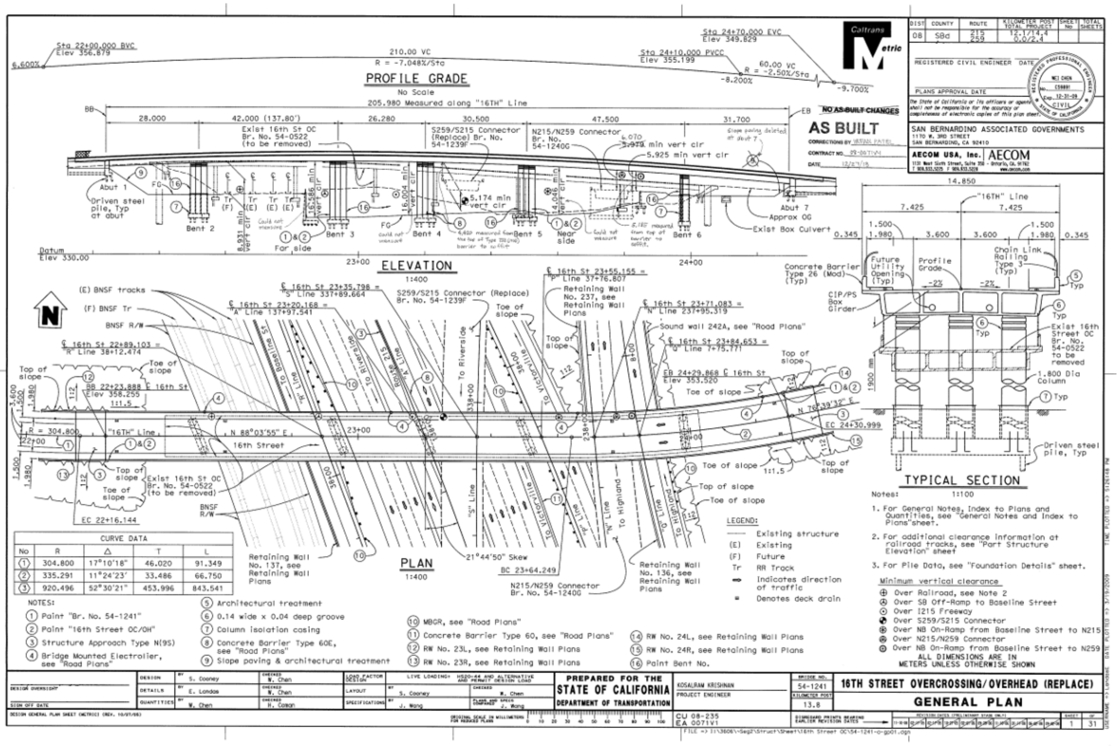


Figure 20.22.8-1 General Plan

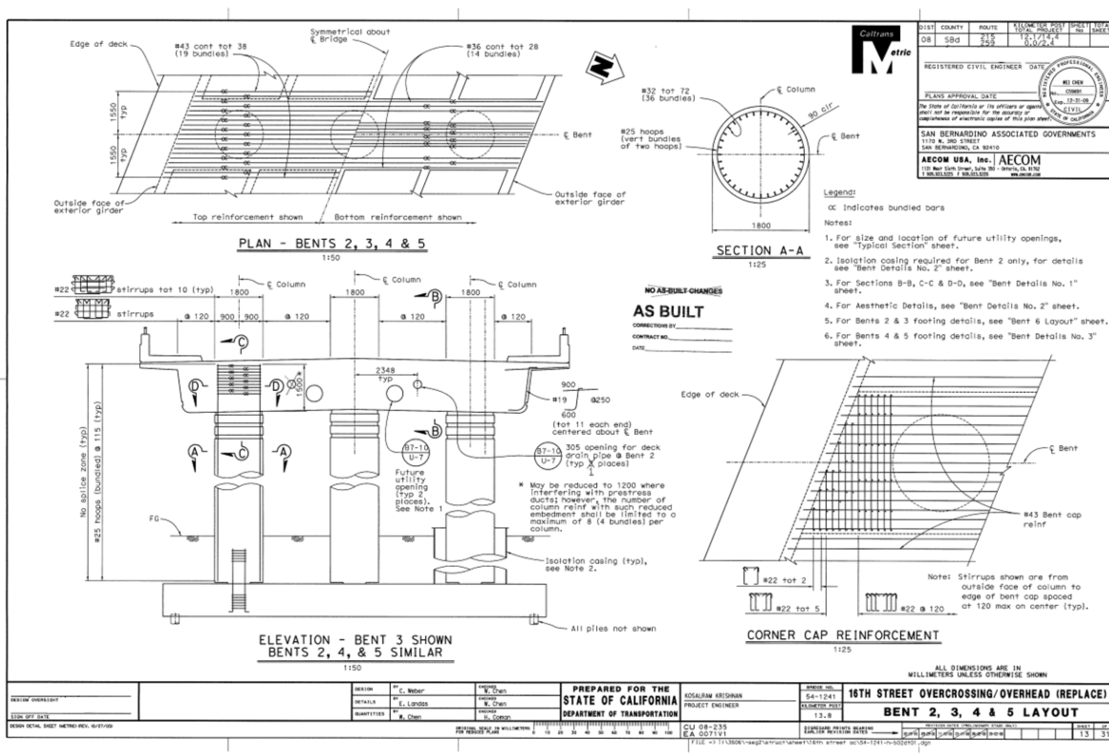


Figure 20.22.8-2 Typical Bent Layout

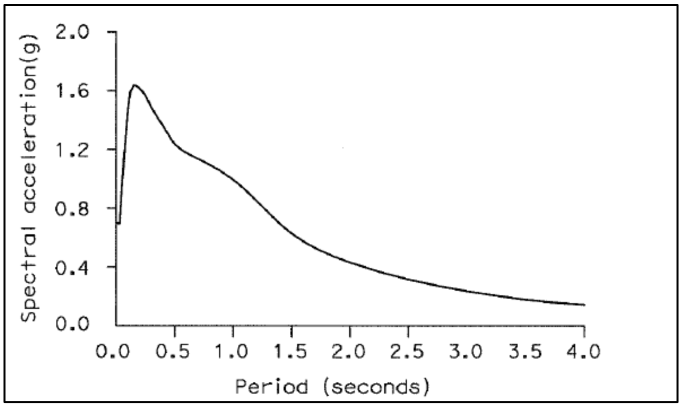


Figure 20.22.8-3 Project's Acceleration Response Spectrum Associated with Design Fault Offset

The total displacement demand is the sum of the static and dynamic components. A nonlinear static pushover analysis is used to establish static displacement demands. A CSiBridge model of the prototype bridge is used to illustrate the procedure for performing static pushover analysis, implementing LSA and LDA analyses. CSiBridge is used here; however, other finite element analysis programs may be used, as the procedure is not program dependent.

20.22.8.1 Structure Model

First, we begin by creating a spine model of the prototype bridge. The superstructure and bent caps are modeled using linear elastic frame elements. Columns are also modeled using frame elements. Columns in each bent are supported on a common footing using pin connections. P-M-M hinges are added at column tops, at potential plastic hinge locations. The bridge model is shown in Figure 20.22.8.1-1.

Footings are supported on 16.0 in. diameter driven pipe piles. The footing and piles are included in the model. Shell elements are used to model the footing. Pile to footing connections are pinned. Class S1 soil may be assumed for simplicity. However, soil-foundation-structure interaction of the footing pile group was modeled to more accurately capture the displacement demands. Nonlinear P-Y springs are attached to pile nodes at 5.0 ft. intervals, down to pile tips. Rollers are added to the bottom of piles. T-Z and Q-Z springs may also be used. A sample foundation model is shown in Figure 20.22.8.1.2.

The prototype bridge is supported on seat abutments with transverse shear keys on both sides of the superstructure. For this design example, the shear keys are assumed to break away under the design seismic hazard. The bridge is modeled with transverse rollers at the abutment locations, based on the assumption that shear keys will break in a major event, and that friction between the bridge superstructure and the abutment bearings is negligible. How the abutments are modeled in the analysis can have a significant impact on the response of the bridge. The roller model yields an upper bound solution for abutment seat displacement demands but may result in underestimated column demands at some locations. The bridge designer may choose to include a detailed abutment model if the shear keys are not expected to break away or if significant residual strength is expected. The soil behind the abutments is included in the model using nonlinear compression only springs. A simple bilinear elasto-plastic model consistent with SDC requirements is used. A two-inch longitudinal gap is assumed at the abutments.

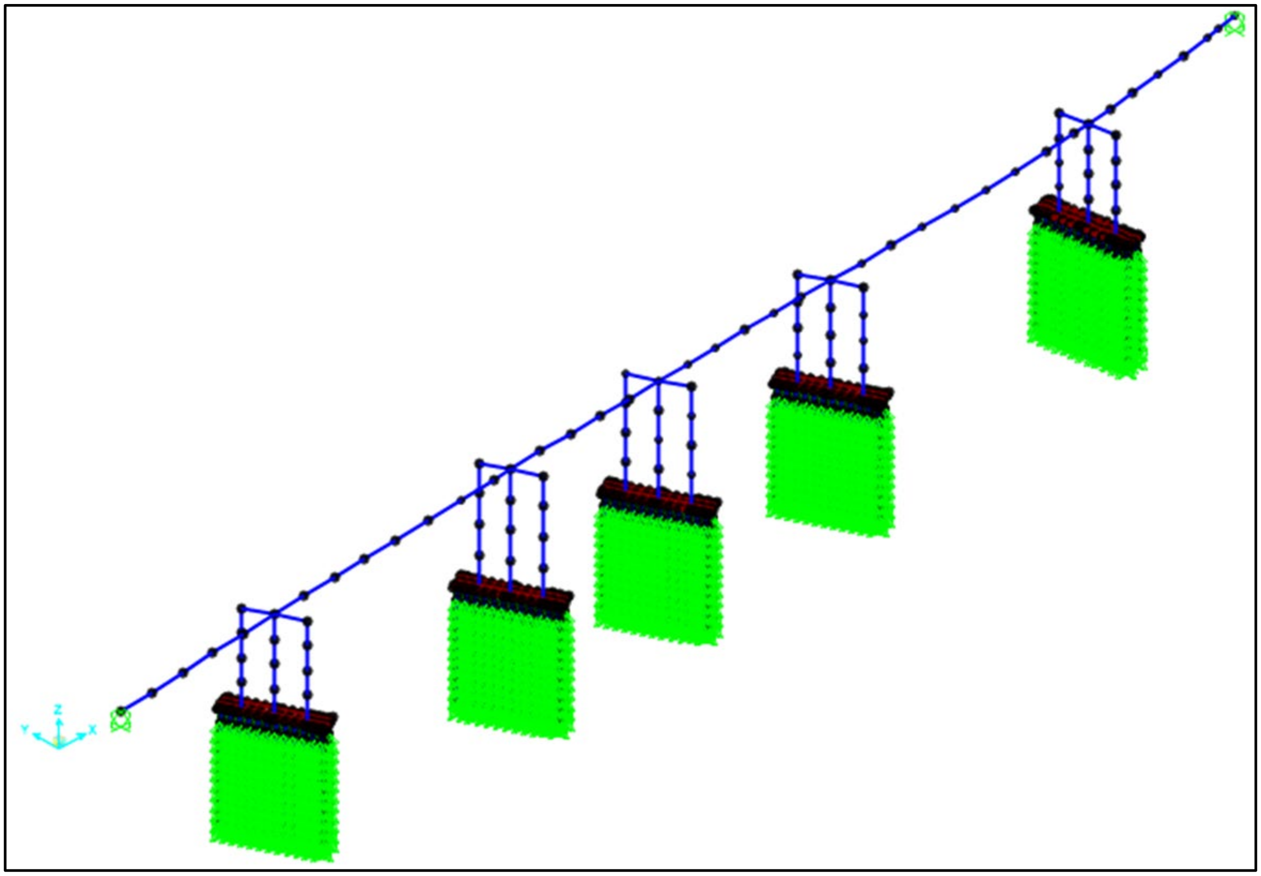


Figure 20.22.8.1-1 CSiBridge Model

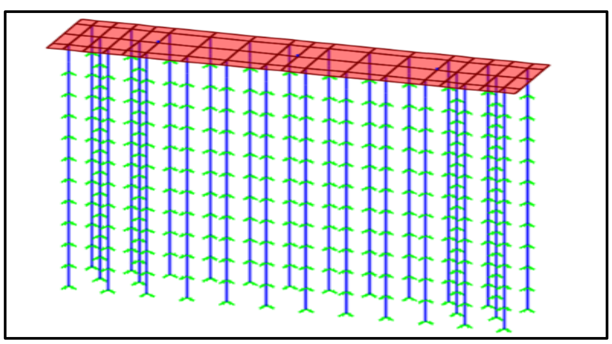


Figure 20.22.8.1-2 Typical Foundation Model in CSiBridge

20.22.8.2 Dynamic Displacement Demands Due to Ground Shaking

Both LSA and LDA procedures are illustrated step by step in the following sections. The first step in applying either procedure is to determine the fault rupture influence vector, Δ_N.

The influence vector for fault rupture, Δ_N, is determined using a linear static analysis by applying a unit offset to the bridge model. A transverse displacement of 1.0 in. (along the y-axis) is applied to all fixed nodes at the backs of the soil springs at bents 2, 3, and 4. A negative transverse displacement of (-1.0 in.) is applied to all fixed soil spring nodes at bents 5 and 6. The abutments are free in the transverse direction. If transverse springs were used in the model to represent the shear keys, the unit displacements would have to be applied to the abutment springs, too. The resulting superstructure deflected shape is shown in Figure 20.22.8.2-1. Displacement results are in Table 20.22.8.2-1. The second row in Table 20.22.8.2-1 is the displacements at superstructure support locations from the unit offset load case. Superstructure displacements are normalized to the displacement at Abutment 1, the largest value, to determine the influence vector, Δ_N.

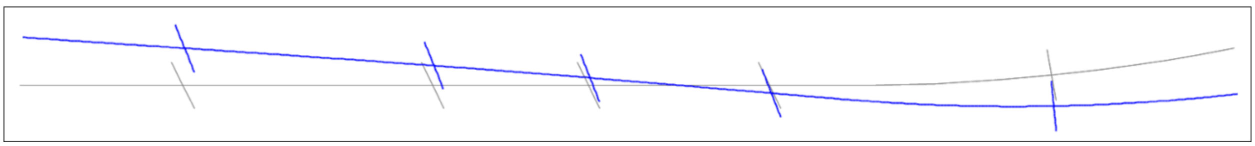


Figure 20.22.8.2-1 Superstructure Deflected Shape Resulting from the Unit Offset Load

Table 20.22.8.2-1 Influence Vector, Δ_N

Location	Abut 1	Bent 2	Bent 3	Bent 4	Bent 5	Bent 6	Abut 7
Superstructure Displacements (in.)	1.595	1.233	0.656	0.249	-0.258	-1.029	-1.518
Δ_N	1.000	0.773	0.411	0.156	-0.162	-0.645	-0.951

20.22.8.3 Compute Dynamic Response of the Bridge Using LSA

The dynamic response of the bridge can be estimated by LSA of the bridge subject to the force vector F_{LSA} applied to the superstructure support locations, as shown in Figure 20.22.8.3-1. F_{LSA} is computed as shown in Equation 20.22.6.2.1-1.

The peak spectral acceleration coefficient of the response spectrum, A_{max}, in this example is 1.637. Table 20.22.8.3-1 shows the applied force vector and resulting superstructure displacements at all support locations.

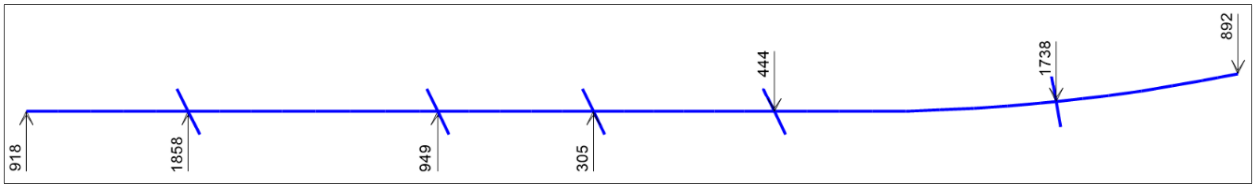


Figure 20.22.8.3-1 Application of F_{LSA} to the Superstructure

Table 20.22.8.3-1 LSA Displacement Demands

Location	Abut 1	Bent 2	Bent 3	Bent 4	Bent 5	Bent 6	Abut 7
m (kip-s²/ft)	17.42	45.62	43.79	37.08	52.11	51.15	17.80
Δ_N	1.000	0.773	0.411	0.156	-0.162	-0.645	-0.951
F_{LSA} *(kips)*	918	1858	949	305	-444	-1738	-892
$\Delta_{u,LSA}$ *(in.)*	**39.0**	**28.4**	**13.5**	**4.7**	**-5.7**	**-23.0**	**-35.7**

20.22.8.4 Compute Dynamic Response of the Bridge Using LDA

The LDA procedure is carried out using the bridge vibration modes with the highest transverse fault rupture effective modal mass. Enough modes are selected such that the cumulative effective modal mass for all selected modes is at least 90%. First, we run the modal analysis using Eigenvectors. For this example, the modal analysis was carried out for 30 modes. The following steps outline the process required to complete the LDA analysis.

Step 1: Extract the vectors of transverse components of the mode shapes (U_y). The extracted data is imported into an Excel spreadsheet. The first 10 modes are shown in Table 20.22.8.4-1.

Step 2: Compute the fault rupture effective modal masses, $m_{eff,I}$, as shown in Equation 20.22.6.2.2-1.

Matrix operations within the Excel spreadsheet are used to compute $m_{eff,i}$. The fault rupture mass participation factor (*MR*) for each mode is computed as:

$$MR_i = m_{eff,i} / m_{tot} \quad (20.22.8.3\text{-}2)$$

where m_{tot} is the total superstructure mass, or the sum of the superstructure masses (*m*) shown in Table 20.22.8.3-1.

The modes are then sorted from highest to lowest based on *MR*. Cumulative effective modal mass ratios (CMR) are computed by adding the *MR* of each mode to the CMR of all previous modes. In this example, six modes are required to reach a CMR larger than 90%. These modes are used for the LDA procedure. Results are shown in Table 20.22.8.4-2.

Table 20.22.8.4-1 Sample mode shapes

Location/ Mode #	Period (sec)	Abut 1	Bent 2	Bent 3	Bent 4	Bent 5	Bent 6	Abut 7
1	1.66	1.093	0.811	0.399	0.147	-0.149	-0.628	-0.966
2	1.57	-0.482	-0.490	-0.516	-0.540	-0.584	-0.685	-0.775
3	1.11	0.269	0.144	-0.018	-0.087	-0.121	-0.090	-0.046
4	0.90	-1.186	-0.464	0.439	0.684	0.554	-0.372	-1.179
5	0.48	-0.044	-0.006	0.028	0.024	-0.013	-0.013	-0.004
6	0.38	-0.028	0.003	0.061	0.002	-0.047	-0.004	0.080
7	0.37	1.176	0.035	-0.727	-0.174	0.661	0.106	-1.193
8	0.27	0.007	-0.001	-0.004	0.007	-0.005	-0.005	-0.009
9	0.24	0.030	0.003	0.001	-0.013	0.009	-0.007	0.018
10	0.22	-0.035	0.040	0.060	0.029	-0.020	-0.010	0.056

Table 20.22.8.4-2 Mode shapes with highest fault rupture modal mass ratio

Mode #/ Variable	1	3	20	8	19	25
Period (sec)	1.66	1.11	0.16	0.27	0.17	0.15
MR	34.5	19.1	15.6	8.3	8.2	4.7
CMR	34.5	53.7	69.3	77.6	85.8	90.5

Step 3: Compute fault rupture modal participation factors, Γ_i, as shown in Equation 20.22.6.2.2-3:

$$\phi_1^T m\Delta_N = 1.093 \times 17.42 \times 1.000 + 0.811 \times 45.62 \times 0.773 + 0.399 \times 43.79 \times 0.411 + \ldots = 94.01$$

$$\phi_1^T m\phi_1 = 1.093^2 \times 17.42 + 0.811^2 \times 45.62 + 0.399^2 \times 43.79 + \ldots = 96.56$$

$$\Gamma_1 = \frac{\phi_1^T m\Delta_N}{\phi_1^T m\phi_1} = \frac{94.01}{96.56} = 0.97$$

Table 20.22.8.4-3 Fault rupture modal participation factors

Mode #/ Variable	1	3	20	8	19	25
$\phi_i^T m\Delta_N$	94.01	13.73	-16.29	0.4036	-8.44	1.758
$\phi_i^T m\phi_i$	96.56	3.72	6.41	0.0074	3.28	0.248
Γ_n	0.97	3.69	-2.54	54.65	-2.57	7.09

Step 4: Compute modal load vectors, F_i as shown in Equation 20.22.6.2.2-2.

Computed load vectors are shown in Table 20.22.8.4-4.

Table 20.22.8.4-4 Modal load vectors

Location/ Mode #	A_i (g's)	Abut 1	Bent 2	Bent 3	Bent 4	Bent 5	Bent 6	Abut 7
1	0.589	598	1160	548	170	-243	-1008	-539
3	0.922	557	783	-93	-381	-751	-547	-98
20	1.636	556	-38	499	260	-631	-93	-418
8	1.533	207	-92	-276	473	-498	-430	-270
19	1.635	156	351	378	-373	-577	255	-258
25	1.637	276	-191	198	150	-326	-394	146

Step 5: Apply the modal forces from Table 20.22.8.4-4 to the bridge model (Figure 20.22.8.4-1), one load case for each mode, to determine the dynamic displacement demands, $\Delta_{u,i}$, for the six modes selected. Use SRSS to combine modal displacement demands. Results are presented in Table 20.22.8.4-5.

Table 20.22.8.4-5 Modal displacement $\Delta_{u,i}$ (in.), and LDA displacement demands

Location/ Mode #	Abut 1	Bent 2	Bent 3	Bent 4	Bent 5	Bent 6	Abut 7
1	14.33	10.41	4.92	1.73	-1.95	-8.09	-12.55
3	14.65	9.12	1.68	-1.88	-4.91	-8.05	-9.96
20	18.00	12.55	5.62	1.52	-3.27	-10.72	-16.48
8	5.40	3.13	0.15	-1.64	-4.32	-9.31	-13.19
19	10.97	7.25	2.00	-0.87	-3.25	-5.43	-7.23
25	5.76	3.58	0.97	-0.51	-2.11	-3.65	-4.14
$\Delta_{u,LDA}$	30.4	20.6	8.0	3.5	8.5	19.4	27.8

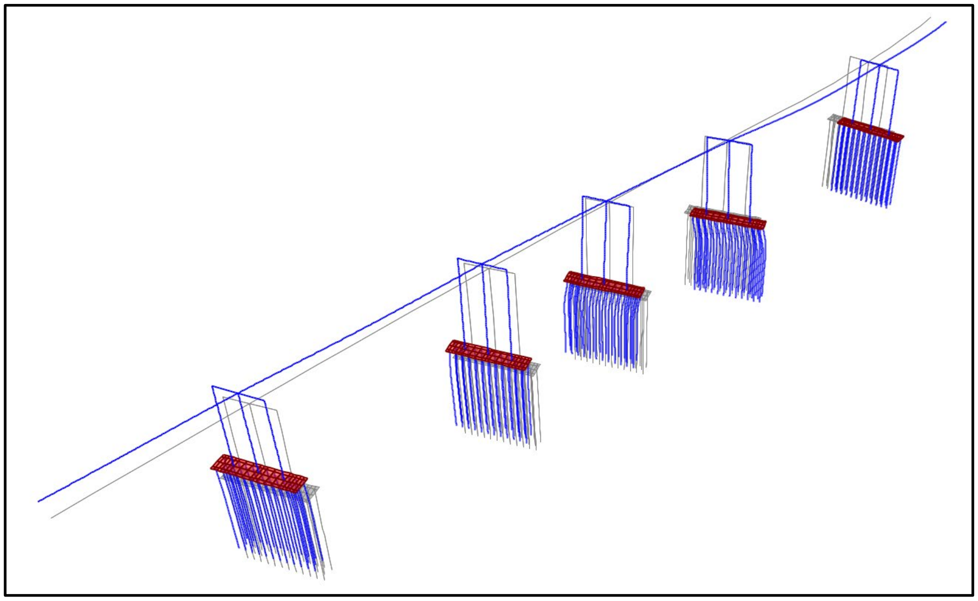


Figure 20.22.8.4-1 Final deformed shape resulting from fault rupture offset

20.22.8.5 Displacement Demands Due to Static Fault Offset

The design fault offset is 19.7 in. in the transverse direction. Half of this offset is assumed to occur on one side of the fault, while the other half occurs on the other side of the fault, in the opposite direction. As stated earlier, we will only consider the case of fault rupture occurring between Bents 4 and 5. A nonlinear static pushover load case is created in CSiBridge. Static fault offset displacements, Δ_{FR} *(1)*, are applied to the model within this load case. A transverse displacement of +9.85 inches is applied to all ground nodes of the soil springs at Abutment 1, Bents 2, 3, and 4. A transverse displacement load of -9.85 inches is applied to all ground nodes of the soil springs at Bents 5, 6, and Abutment 7. The final step of the pushover case is shown in Figure 20.22.8.4.1. The superstructure displacements at the support locations, Δ_{SUPER} *(2)*, are presented in Table 20.22.8.5.1. These are the span displacements due to the applied fault offset, Δ_{FR} *(1)*. Net superstructure displacement at a support location, $\Delta_{u,FR}$, is the difference between the ground displacement, Δ_{FR} *(1)*, and the resulting span displacements, Δ_{SUPER} *(2)*. Total displacement demand at each superstructure support location, Δ_u, is computed as:

$$\Delta_u = \Delta_{u,FR} + \left(\Delta_{u,LSA} \text{ or } \Delta_{u,LDA}\right) \quad (20.22.8.5.1)$$

Note that in this example, the displacement demands from the Linear Static Analysis method can be much larger than those from the Linear Dynamic Analysis method. This is to be expected since LSA is based on the peak of the response spectrum acceleration.

Table 20.22.8.5-1 Total displacement demands (in.)

Location/ Case	Abut 1	Bent 2	Bent 3	Bent 4	Bent 5	Bent 6	Abut 7
Δ_{FR} *(1)*	9.85	9.85	9.85	9.85	-9.85	-9.85	-9.85
Δ_{SUPER} *(2)*	15.95	12.24	6.36	2.32	-2.67	-10.42	-15.39
$\Delta_{u,FR}$ *(|2-1|)*	6.1	2.4	3.5	7.5	7.2	0.6	5.5
$\|\Delta_{u,LSA}\|$	39.0	28.4	13.5	4.7	5.7	23.0	35.7
$\Delta_{u,LDA}$	30.4	20.6	8.0	3.5	8.5	19.4	27.8
$\Delta_{u,FR}+\|\Delta_{u,LSA}\|$	**45.1**	**30.8**	**17.0**	**12.2**	**12.9**	**23.6**	**41.2**
$\Delta_{u,FR}+\Delta_{u,LDA}$	**36.5**	**23.0**	**11.5**	**11.0**	**15.7**	**20.0**	**33.3**

20.22.8.6 Displacement Capacities

The same bridge model is used to perform transverse pushover analysis to establish bent displacement capacities at the superstructure. Columns have P-M hinges at the column tops just below the superstructure soffit. Displacement-controlled pushover load cases are used to push each bent transversely until one or more columns in the bent reach full curvature capacity, and the displacement capacity is recorded, Δ_c. Except for the plastic hinge length, which is computed using SDC section 5.3.4 and manually input into the analysis model, all other hinge properties are automatically generated by the program.

Displacement demands and capacities are compared in Table 20.22.8.6-1. P-Δ moment should not exceed 25% of the column's plastic moment, M_p. A maximum displacement capacity, $\Delta_{cP\Delta}$, based on the P-Δ limit, is computed as:

$$\Delta_{cP\Delta} = 0.25Mp/P_{dl} \quad (20.22.8.6\text{-}1)$$

Computed $\Delta_{cP\Delta}$ are also included in Table 20.22.8.6-1. P-Δ does not control displacement capacity at any column support location. Displacement capacity at each bent is controlled by column curvature capacity. At all bent locations, the as-designed bridge has adequate displacement capacity.

At the abutments, the displacement demand is about three feet. The superstructure is a box girder section with 4.25 ft. overhangs and 0.67 ft. girder offset (sloped exterior girder). This means that the superstructure would have to move more than 4.92 ft. for the exterior edge of the superstructure to project beyond the edge of the abutment. Even using LSA displacement demands, the superstructure has ample support length at the abutments.

Table 20.22.8.6-1 Displacement demand vs. capacity at bent locations

Location/ Case	Abut 1	Bent 2	Bent 3	Bent 4	Bent 5	Bent 6	Abut 7
Δ_u by LSA (in.)	45.1	30.8	17.0	12.2	12.9	23.6	41.2
Δ_u by LDA (in.)	36.5	23.0	11.5	11.0	15.7	20.0	33.3
Δ_c by PUSH (in.)	**—**	**35.6**	**52.3**	**52.7**	**41.6**	**35.2**	**—**
P_{dl} (kip)	—	627	581	420	657	695	—
M_P (kip-ft.)	—	12,416	12,370	12,190	12,455	12,491	—
$\Delta_{cP\Delta}$ (in.)	—	59.4	63.9	87.1	56.9	53.9	—