

APPENDIX A

SDOF Column Investigation Sample Calculations and Results

Investigation

Diversity of Hazard Effects

Performance-based seismic design (PBSD) as defined in the proposed AASHTO guidelines is a four-step process. The steps are as follows: Step 1, hazard levels and characteristics are defined; Step 2, analysis techniques are used to identify earthquake resisting systems performance (structural response), including soil–structure interaction; Step 3, earthquake resistance elements (ERE) are evaluated for damage (usually strains, displacements, etc.) and compared with prescribed performance limits; and, Step 4, performance is evaluated in consideration to loss (usually in time, resources, or human injury). Through each of these steps the designer and owner are working together to set the performance goals and reevaluate the impacts of the analysis and decisions to design outcomes. Each one of these steps is layered with options that can vary, depending on the needs of the public, owners, designers, or locational characteristics, etc. This study stems from Step 1 above and the question: “How is the PBSD process affected by seismic hazard and varying response spectrum characteristics throughout the country?”

To create a limited bridge design example to represent PBSD for all the users of this document, it was important to understand the effects of the various hazards throughout the country on the PBSD process. Ground shaking generates the measure of acceleration defined, which then gets adjusted by response coefficients based on site classification, and then is quantified as a site specific response spectrum. The defining characteristics of earthquakes, such as slip faults, subduction events, fault rupture, or modes of travel (P-waves or S-Waves), vary throughout the country along with the types of soil and profile through which the accelerations propagate. Before introducing the design example of a standard bridge (see Appendix A of the proposed *Guidelines for Performance-Based Seismic Bridge Design*), the team studied a set of single degree of freedom (SDOF) columns with simplified rigid boundary conditions to evaluate multiple hazard effects quickly and apply the basic PBSD approach to several typical circular column ERE configurations. This study was done to determine trends for this methodology regarding Step 1.

This appendix accompanying *NCHRP Research Report 949* outlines the process used, highlights specific findings, and adds discussion topics the team gleaned from this summary, with example calculations set for one condition as well as the results of all the hazard locations.

Single Degree of Freedom Element Used

The bridge assumed for this study is a long viaduct structure with single columns and very stiff foundations to eliminate boundary condition effects. The structure is free to move transversely and longitudinally at the pier under consideration without additional iterations being required to define the

amount of mass participation at this location or soil–structure interaction effects. The columns are assumed to be fixed at the base and connected at the top to a rigid continuous superstructure to generate double bending. Axial load is treated as a variable with values of 8%, 12%, and 16% of the gross section axial capacity. The seismic weight is set as 1.6 times the dead load axial force to account for variability in column length and span length along the bridge. Live load or other loading conditions are not included. The column diameter is 6 feet for all cases, based on a standard single column pier configuration.

Other variables of this example include longitudinal reinforcement ratios of approximately 1%, 1.5%, and 2% and transverse volumetric steel ratios of approximately 0.7% and 1%. The column aspect ratios varied from 4, 5, and 6. The matrix for all of these variables is shown later in the sample calculation. Material properties are based on A706 Grade 60 reinforcement as defined in the current *Guide Specifications*.

Hazard Inputs Used

Hazards were selected from areas throughout the country. It is important to recognize that there are areas in the country not captured by this selection that may produce effects that are not represented in this exercise. Owners and designers should verify with geotechnical engineers who are familiar with their project site seismic characteristics to verify how each location compares with the hazards identified and used herein.

Table A-1 provides the inputs for the 975-year return period (1000-year return period) and Table A-2 provides the inputs form the 100-year return period. These ground motion values were taken from the U.S. Geological Survey hazard website using the 2014 data, except for the latest maps for Juneau, which are based on 2007 data. The tables provide PGA, Ss, S1, and S2 for firm ground and then are adjusted for local site effects for a selection of cities from the East Coast to Juneau. Site Class D is used for the site conditions, assuming that somewhere in the area of the selected city there would be a condition that classifies as Site Class D.

Table A-1. Ground motion values for 975-year return period.

City	Location		B/C Firm-Ground Motions (g)				Site Factors			Site-Adjusted Ground Motions (g)			
	Latitude	Longitude	PGA	Ss	S1	S2	F _{pga}	F _a	F _v	PGAd	Sds	Sd1	Sd2
Charleston	34.8497	-79.9311	0.065	0.13	0.045	0.024	1.6	1.6	2.4	0.10	0.21	0.11	0.06
New York City	41.0332	-73.6727	0.081	0.14	0.028	0.013	1.6	1.6	2.4	0.13	0.22	0.07	0.03
Boston	42.3638	-71.0594	0.08	0.14	0.032	0.015	1.6	1.6	2.4	0.13	0.23	0.08	0.04
Knoxville	35.9631	-83.9188	0.23	0.35	0.066	0.032	1.36	1.52	2.4	0.31	0.53	0.16	0.08
Paducah	37.083	-88.6027	0.37	0.65	0.17	0.079	1.13	1.28	2.13	0.42	0.83	0.35	0.17
St Louis	38.6272	-90.1866	0.15	0.28	0.086	0.043	1.6	1.58	2.4	0.24	0.44	0.21	0.10
Helena	46.5894	-112.0391	0.13	0.30	0.08	0.036	1.58	1.56	2.4	0.21	0.47	0.19	0.09
Salt Lake City	40.7534	-111.8892	0.43	1.00	0.29	0.12	1.06	1.11	1.82	0.46	1.10	0.52	0.22
Los Angeles	34.0542	-118.2532	0.60	1.41	0.41	0.18	1	1	1.59	0.60	1.41	0.65	0.28
San Francisco	37.7733	-122.4173	0.56	1.28	0.42	0.21	1	1	1.58	0.56	1.28	0.66	0.32
Seattle	47.6083	-122.3316	0.43	0.97	0.28	0.13	1.07	1.11	1.83	0.46	1.08	0.52	0.25
Juneau	58.3066	-134.4191	0.14	0.35	0.20	0.12	1.51	1.52	2	0.22	0.52	0.40	0.24
New Madrid	36.5856	-89.5251	1.29	2.24	0.60	0.28	1	1	1.5	1.29	2.24	0.90	0.42

Table A-2. Ground motion values for 100-year return period.

City	Location		B/C Firm-Ground Motions (g)				Site Factors			Site-Adjusted Ground Motions (g)			
	Latitude	Longitude	PGA	S _s	S ₁	S ₂	F _{PGA}	F _a	F _v	P _{GAd}	S _{d3}	S _{d1}	S _{d2}
Charleston	34.8497	-79.9311	0.012	0.21	0.0081	0.0034	1.6	1.6	2.4	0.02	0.33	0.02	0.01
New York City	41.0332	-73.6727	0.011	0.022	0.0063	0.0025	1.6	1.6	2.4	0.02	0.04	0.02	0.01
Boston	42.3638	-71.0594	0.013	0.029	0.0072	0.0028	1.6	1.6	2.4	0.02	0.05	0.02	0.01
Knoxville	35.9631	-83.9188	0.035	0.065	0.014	0.0056	1.6	1.6	2.4	0.06	0.10	0.03	0.01
Paducah	37.083	-88.6027	0.047	0.088	0.016	0.0062	1.6	1.6	2.4	0.08	0.14	0.04	0.01
St Louis	38.6272	-90.1866	0.025	0.050	0.011	0.0043	1.6	1.6	2.4	0.04	0.08	0.03	0.01
Helena	46.5894	-112.0391	0.032	0.069	0.022	0.0099	1.6	1.6	2.4	0.05	0.11	0.05	0.02
Salt Lake City	40.7534	-111.8892	0.049	0.11	0.032	0.015	1.6	1.6	2.4	0.08	0.17	0.08	0.04
Los Angeles	34.0542	-118.2532	0.19	0.41	0.11	0.047	1.43	1.47	2.36	0.27	0.60	0.26	0.11
San Francisco	37.7733	-122.4173	0.202	0.43	0.12	0.056	1.41	1.45	2.31	0.28	0.63	0.28	0.13
Seattle	47.6083	-122.3316	0.13	0.27	0.068	0.028	1.57	1.58	2.4	0.20	0.43	0.16	0.07
Juneau	58.3066	-134.4191	0.048	0.10	0.051	0.028	1.6	1.6	2.4	0.08	0.16	0.12	0.07
New Madrid	36.5856	-89.5251	0.077	0.13	0.020	0.0071	1.6	1.6	2.4	0.12	0.21	0.05	0.02

Figure A-1 shows the relationship of the two largest earthquakes (New Madrid and San Francisco) in the upper level 1000-year return period hazard with the lower level 100-year return period hazard. The San Francisco location represents a condition where a high upper hazard can also produce a higher lower hazard, whereas the New Madrid location produces the highest upper level hazard in this study with one of the lowest lower level hazards.

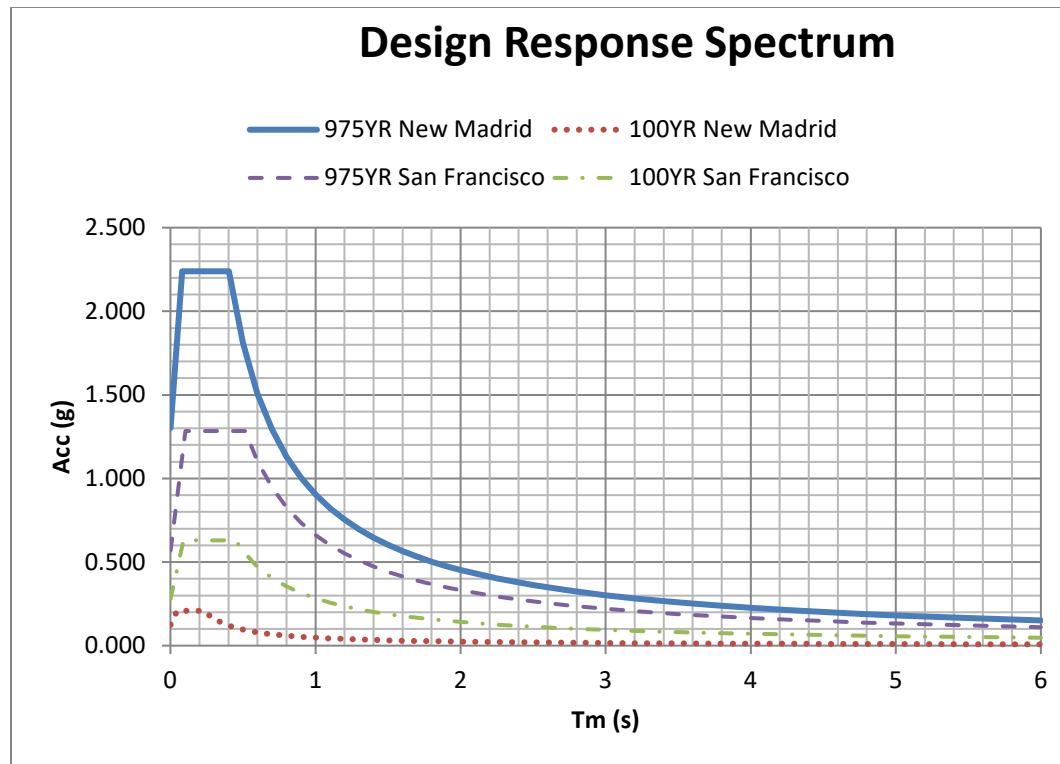


Figure A-1. New Madrid and San Francisco design response spectrum curves for both 975-year and 100-year hazards, constructed using 3-point method from the current *Guide Specifications*.

An important finding of this exercise is that the lower level hazards do not tend to produce displacements that would result in significant damage. In other words, it is highly likely that if the 100-year event is used as the lower level event in the performance criteria, that there is a high probability that minimal repair is required to allow daily traffic to use the facility (i.e., limited interruption of service) and that emergency

vehicles would have immediate access to the facility. Figure A-2 shows the ratio of displacement demand at the 100-year hazard to the controlling performance level displacement limit, as summarized from the tables found later in the Results section. This shows that only four of the columns in the parametric study would have had 100-year displacement demands exceeding the displacement capacity defined by PL3. Even for these four, the exceedance is minor. As discussed in these guidelines, owners and engineers can work together to define what level of hazard is appropriate for the level of performance expected at each specific bridge location. In the event that the 500-year hazard was selected for the lower level event, additional modifications to the ERE may be required to meet the owner specified performance criteria.

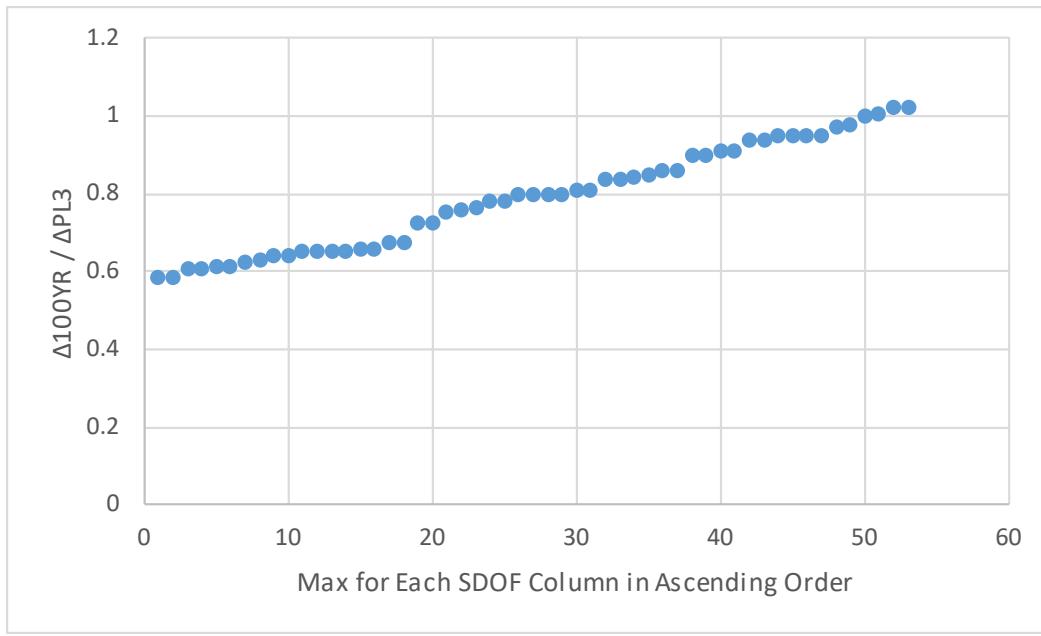


Figure A-2. 100-year displacement demand to minimum performance displacement level (PL3) in ascending order for each column in SDOF study.

The evaluation in the Results section included only the *Guide Specifications* calculations of the displacement demand. The direct displacement-based design (DDBD) displacement demand is not provided for this example, because it would be identical to the *Guide Specifications* approach. This is the result of the ductility being less than 1.5 (likely less than 1.0) and both approaches to determining displacement demands utilizing only elastic damping at this level of ductility. This will generally be the rule, as the expected nominal moment capacity is based on the Concrete strain limit set by PL3, unless defined otherwise by the owner.

Comparison of Performance Levels for SDOF Column

Table A-3, Table A-4, and Table A-5 are shown here as a representation of all the tables in the Results section. The color-coding used in the tables is for describing the following aspects of the tables. The light yellow section defines the designation of the column in question; C1.5,0.7,8,5 for Table A-3 represents the changing characteristics for this column run in the comparison study. The chart showing the entire study breakout of variables with the designation callouts is found in the front of the Results section.

The light green highlights identify the strain limits set for concrete or steel that are based on the performance level requirement from these guidelines. Note that PL1 concrete is not included. All of the studies showed that PL1 steel bar buckling strain limit will control well before PL1 Concrete crushing strain

limit in all cases and that most analytical software will stop running at the controlling PL1 limit state. Plotting the PL1 Concrete point would be an unused extension of the shear force to displacement curve that could be plotted as a dashed line.

The light orange highlights provide the calculated displacements associated with the strain limits set. A sample calculation of one of these runs is found in the subsequent Sample Calculations section. The location of these limiting displacements is shown on the plot as visual reference for a comparison to the displacement demands. Note that the PL3 concrete and steel limits are set to be near initial yielding of the column reinforcement, meeting the expectations at that performance limit state. As identified above, reaching the PL3 limit state is infrequent for the 100-year return period displacement demand. This set of three tables was selected to show a representative comparison of the study, but specifically due to the San Francisco lower bound displacement demand nearly reaching the PL3 concrete limit in Table A-5.

The light gray highlights show the design response spectral acceleration (S_a) and corresponding displacement demand for both the 100-year and 975-year return period spectral analysis for each hazard location identified. The column characteristics impact the displacement demand, specifically the column aspect ratio and corresponding stiffness and period of the pier. The displacement demands are plotted on the shear force to displacement curve with light blue-“+” for the 100-year return period and purple-“x” for the 975-year return period.

The light blue highlights provide the displacement demand generated using the DDBD method for the two highest displacement demand hazard locations based on the upper bound event, specifically San Francisco and New Madrid. These are plotted on the shear force to displacement curve as the black circle and the black square, respectively.

Table A-3. Column comparison of a 1.5% longitudinal steel ratio, 0.7% transverse volumetric steel ratio, 8% axial load ratio to expected concrete strength and gross column area, and a column aspect ratio of 5.

C1.5,0.7,8,5								
Ast/Ag	0.015		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4Ah/D's	0.007		San Francisco	5.3	in			
P/f'ceAg	0.08		New Madrid	8.7	in			
L/D	5							
				(g)	(in)	(g)	(in)	
Strain Limits		City	Sa 100YR	Δ 100YR	Sa 975YR	Δ 975YR		
ec.cc	-0.004	PL3 conc	Charleston	0.026	0.1	0.142	0.8	
es.rcw	0.01	PL3 steel	New York City	0.020	0.1	0.089	0.5	
es.bb	0.0346	PL1 steel	Boston	0.023	0.1	0.102	0.6	
ec.u	-0.0169	PL2 conc	Knoxville	0.045	0.3	0.209	1.2	
0.8*es.bb	0.0277	PL2 steel	Paducah	0.051	0.3	0.470	2.6	
			St Louis	0.035	0.2	0.273	1.5	
Limit State Disp. (in)		Helena	0.071	0.4	0.258	1.4		
Δ @ ec.cc	3.3	PL3 conc	Salt Lake City	0.103	0.6	0.693	3.9	
Δ @ es.rcw	4.2	PL3 steel	Los Angeles	0.344	1.9	0.864	4.8	
Δ @ es.bb	13.4	PL1 steel	San Francisco	0.376	2.1	0.878	4.9	
Δ @ ec.u	11.9	PL2 conc	Seattle	0.217	1.2	0.688	3.8	
Δ @ 0.8*es.bb	10.8	PL2 steel	Juneau	0.163	1.1	0.524	3.6	
			New Madrid	0.063	0.4	1.200	6.7	

— C1.5,0.7,8,5 □ PL3 conc ♦ PL3 steel ▲ PL2 steel ○ PL1 steel
 ✕ PL2 conc + 100YR ✕ 975YR ● DDBA SF □ DDBA NM

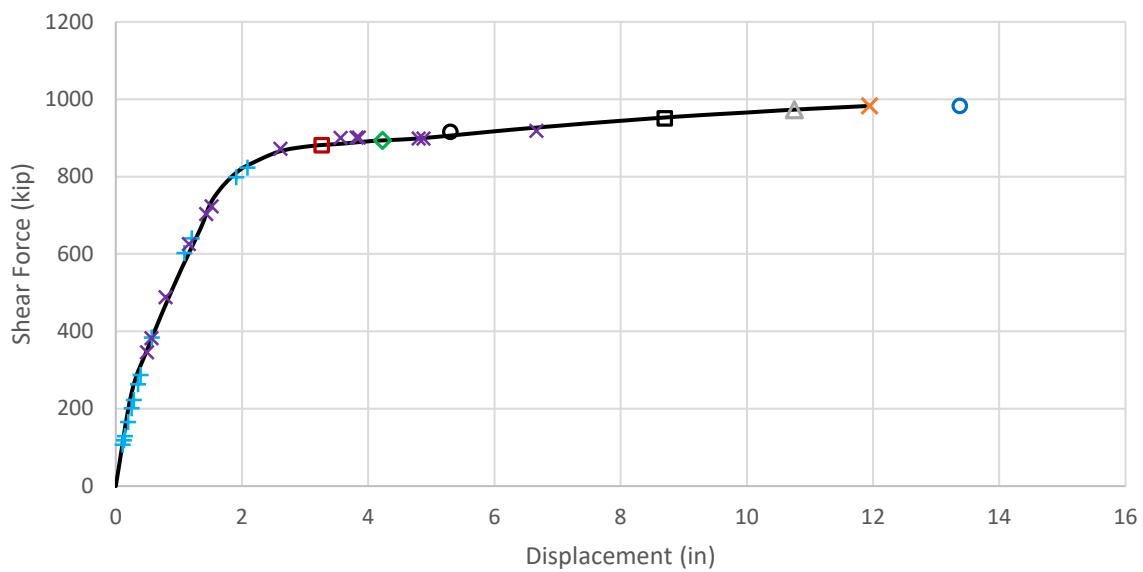


Figure A-3. Shear force to displacement curve corresponding to Table A-3.

Table A-4. Column comparison of a 1.5% longitudinal steel ratio, 0.7% transverse volumetric steel ratio, 12% axial load ratio to expected concrete strength and gross column area, and a column aspect ratio of 5.

C1.5,0.7,12,5								
Ast/Ag	0.015		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4Ah/D's	0.007		San Francisco	6.85	in			
P/f'ceAg	0.12		New Madrid	11.55	in			
L/D	5							
				(g)	(in)	(g)	(in)	
Strain Limits		City	Sa 100YR	Δ 100YR	Sa 975YR	Δ 975YR		
ec.cc	-0.004	PL3 conc	Charleston	0.022	0.2	0.120	0.9	
es.rcw	0.01	PL3 steel	New York City	0.017	0.1	0.076	0.6	
es.bb	0.0290	PL1 steel	Boston	0.019	0.2	0.086	0.7	
ec.u	-0.0169	PL2 conc	Knoxville	0.038	0.3	0.176	1.4	
0.8*es.bb	0.0232	PL2 steel	Paducah	0.043	0.3	0.397	3.1	
			St Louis	0.030	0.2	0.231	1.8	
Limit State Disp. (in)		Helena	0.060	0.5	0.218	1.7		
Δ @ ec.cc	3.1	PL3 conc	Salt Lake City	0.087	0.7	0.586	4.6	
Δ @ es.rcw	4.4	PL3 steel	Los Angeles	0.290	2.3	0.730	5.7	
Δ @ es.bb	11.7	PL1 steel	San Francisco	0.317	2.5	0.741	5.8	
Δ @ ec.u	11.0	PL2 conc	Seattle	0.183	1.4	0.581	4.5	
Δ @ 0.8*es.bb	9.5	PL2 steel	Juneau	0.137	1.1	0.447	3.7	
			New Madrid	0.054	0.4	1.014	7.9	

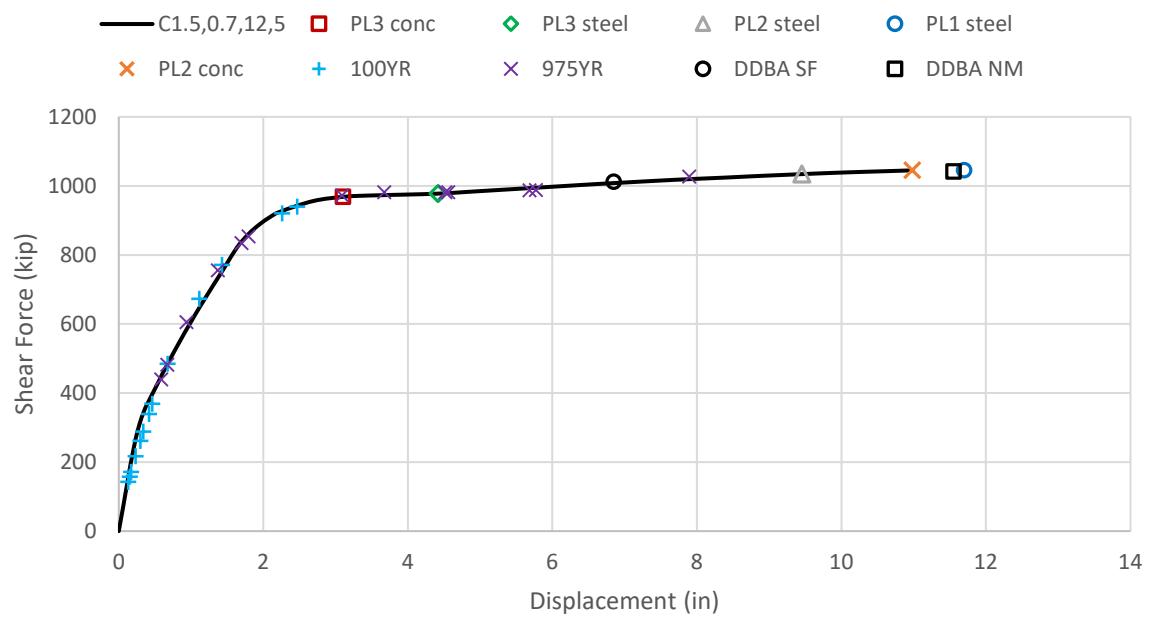


Figure A-4. Shear force to displacement curve corresponding to Table A-4.

Table A-5. Column comparison of a 1.5% longitudinal steel ratio, 0.7% transverse volumetric steel ratio, 16% axial load ratio to expected concrete strength and gross column area, and a column aspect ratio of 5.

C1.5,0.7,16,5								
Ast/Ag	0.015		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4Ah/D's	0.007		San Francisco	8.2	in			
P/f'ceAg	0.16		New Madrid	14.25	in			
L/D	5				(g)	(in)	(g)	(in)
Strain Limits			City	Sa 100YR	Δ 100YR	Sa 975YR	Δ 975YR	
ec.cc	-0.004	PL3 conc	Charleston	0.019	0.2	0.107	1.1	
es.rcw	0.01	PL3 steel	New York City	0.015	0.1	0.067	0.7	
es.bb	0.0234	PL1 steel	Boston	0.017	0.2	0.077	0.8	
ec.u	-0.0169	PL2 conc	Knoxville	0.034	0.3	0.157	1.5	
0.8*es.bb	0.0187	PL2 steel	Paducah	0.039	0.4	0.354	3.5	
			St Louis	0.027	0.3	0.206	2.0	
Limit State Disp. (in)			Helena	0.053	0.5	0.194	1.9	
Δ @ ec.cc	2.9	PL3 conc	Salt Lake City	0.077	0.8	0.522	5.1	
Δ @ es.rcw	4.6	PL3 steel	Los Angeles	0.259	2.5	0.650	6.4	
Δ @ es.bb	9.9	PL1 steel	San Francisco	0.282	2.8	0.660	6.5	
Δ @ ec.u	10.1	PL2 conc	Seattle	0.163	1.6	0.517	5.1	
Δ @ 0.8*es.bb	8.0	PL2 steel	Juneau	0.122	1.2	0.398	3.9	
			New Madrid	0.048	0.5	0.903	8.9	

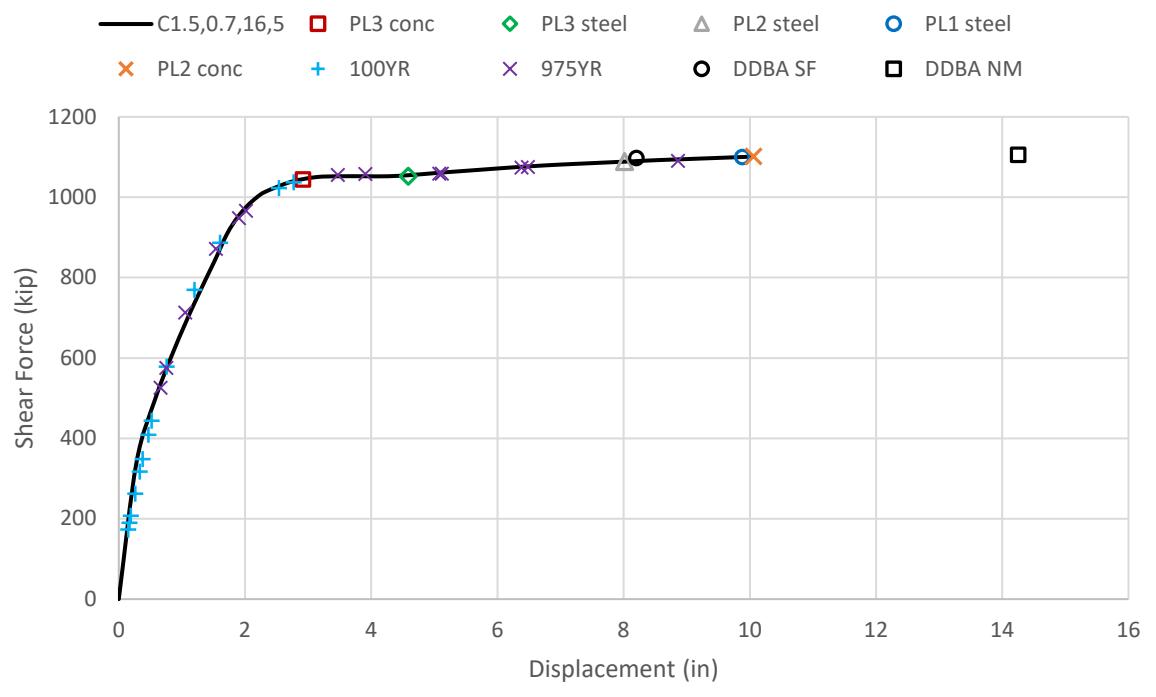


Figure A-5. Shear force to displacement curve corresponding to Table A-5.

The data shown in these three tables demonstrate several points of interest in using the PBSD methodology and in comparing the two design methods permitted in these guidelines, namely, the displacement design of the *Seismic Guide Specifications* and the DDBD methods as outlined in Appendix C of the *Guidelines for Performance-Based Seismic Bridge Design*, which will be published by AASHTO.

The first clarification is the diverging nature of the two design methods as the effect of structural damping increases. For the lower bound hazards, the results are similar, if not the same, as discussed above. However, with larger ductility demands the results of this study showed a consistent increase in the effect that damping has on the structure for generating the displacement demands. Using the New Madrid 975-year return period hazard as the example in these three tables, Table A-3 shows a difference of 2 in. with this method. Table A-4 and Table A-5 show a difference of 3.65 in. and 5.35 in., respectively, which prompted the discussion outlined in Appendix B of the proposed *Guidelines for Performance-Based Seismic Bridge Design* on seismic dampening, indicating that it may be less conservative to use the *Guide Specifications* approach in some situations.

Similarly, using the DDBD methodology for the design of the column parameters outlined in Table A-4 and Table A-5 for the New Madrid upper bound event, the PBSD approach would require a redesign of the column to meet the performance criteria. In Table A-4, the exceedance of the PL2 concrete limit is such that some minor modifications to the confinement reinforcement may be all that is required to get the design to work. However, the exceedance shown in Table A-5 is such that a significant redesign of the column would include increased reinforcement both longitudinally and for confinement, and a change in boundary conditions or adjacent EREs may be required to draw demand away from this pier.

The initial purpose of this exercise was to work through the design steps of PBSD with a simplified structure to see how variations of hazards might effect this process and then select a hazard to move forward with the design example shown in Appendix A of the *Guidelines for Performance-Based Seismic Bridge Design*, which will be published by AASHTO.

. The results obtained from working through the process helped to identify gaps in the development of these guidelines and is a resource for understanding both PBSD and DDBD as a comparison to the current methodology outlined in the *Guide Specifications*. The hazard selected for the design example in the guidelines is the New Madrid upper and lower bounds, except for some additional comparisons to the San Francisco hazards, when a larger displacement demand is beneficial from the lower bound hazard.

Sample Calculations.

The purpose of this study was to evaluate the proposed strain and plastic hinge length recommendations for a range of column details typical of bridge construction. For each column, displacement demands were evaluated with the Guide Spec procedure for the 100YR and 975YR hazards at thirteen sites around the country with varying seismicity. As a means of comparison, a Direct Displacement-Based Assessment (DDBA) procedure was employed to predict displacement demands in the largest 975YR hazards. The following section provides information on the column details used to build the parametric study matrix. Sample calculations for a single column within the matrix are presented with commentary to describe the Guide Spec and DDBA approaches.

Parameters used to Build the SDOF Column Matrix:

$D := 6\text{ft}$	circular column diameter
$A_g := \frac{\pi \cdot D^2}{4} = 28.2743 \cdot \text{ft}^2$	column gross area
$f_c := 4\text{ksi}$	concrete compressive strength
$f_{ce} := 1.3 \cdot f_c = 5.2\text{ksi}$	expected concrete compressive strength

Determine Dead Load Axial Based on Typical Axial Load Ratios of 8, 12, and 16%:

$$\begin{aligned} P_8 &:= 0.08 \cdot f_{ce} \cdot A_g = 1693.7457 \cdot \text{kip} & P_{\text{axial.8}} &:= 1700 \cdot \text{kip} \\ P_{12} &:= 0.12 \cdot f_{ce} \cdot A_g = 2540.6185 \cdot \text{kip} & P_{\text{axial.12}} &:= 2550 \cdot \text{kip} \\ P_{16} &:= 0.16 \cdot f_{ce} \cdot A_g = 3387.4914 \cdot \text{kip} & P_{\text{axial.16}} &:= 3400 \cdot \text{kip} \end{aligned}$$

Determine Column Lengths Based on Aspect Ratios 4, 5, and 6:

$$\begin{aligned} L_4 &:= 4 \cdot D = 24 \cdot \text{ft} \\ L_5 &:= 5 \cdot D = 30 \cdot \text{ft} \\ L_6 &:= 6 \cdot D = 36 \cdot \text{ft} \end{aligned}$$

Determine Longitudinal Reinforcement for Steel Ratios of 1, 1.5 and 2%:

$$\begin{aligned} A_b &:= 1.56 \cdot \text{in}^2 && \text{area of a number 11 bar} \\ d_b &:= 1.41 \cdot \text{in} && \text{diameter of a number 11 bar} \\ A_{st1} &:= 0.01 \cdot A_g = 40.715 \cdot \text{in}^2 \\ n_1 &:= \frac{A_{st1}}{A_b} = 26.0994 && \text{would require 26 bars at 1% steel} \end{aligned}$$

$$Ast_{1.5} := 0.015 \cdot Ag = 61.0726 \cdot in^2$$

$$n_{1.5} := \frac{Ast_{1.5}}{Ab} = 39.1491 \quad \text{would require 40 bars at 1.5% steel}$$

$$Ast_2 := 0.02 \cdot Ag = 0.5655 \cdot ft^2$$

$$n_2 := \frac{Ast_2}{Ab} = 52.1988 \quad \text{would require 52 bars at 2% steel}$$

Determine Hoop Detailing for 0.7 and 1% Transverse Volumetric Steel:

Compute hoop spacing required for 0.7% transverse volumetric steel

$$dh_6 := 0.75in \quad \text{assume butt welded #6 hoops}$$

$$cover := 2in \quad \text{concrete cover}$$

$$D'_6 := D - 2 \cdot cover - dh_6 = 5.6042 \cdot ft \quad \text{confined core diameter between hoop CLs}$$

$$Ah_6 := 0.44in^2 \quad \text{area of a single hoop}$$

$$s_{0.7} := \frac{4 \cdot Ah_6}{D'_6 \cdot 0.007} = 3.7387 \cdot in \quad \text{equivalent to a #6 hoop at 3.5in}$$

$$\text{spacing}_6 := 3.5in \quad \text{for single hoops}$$

Compute hoop spacing required for 1% transverse volumetric steel

$$dh_7 := 0.875in \quad \text{assume butt welded #7 hoops}$$

$$D'_7 := D - 2 \cdot cover - dh_7 = 5.5938 \cdot ft \quad \text{confined core diameter between hoop CLs}$$

$$Ah_7 := 0.6in^2 \quad \text{area of a single hoop}$$

$$s_{0.1} := \frac{4 \cdot Ah_7}{D'_7 \cdot 0.01} = 3.5754 \cdot in \quad \text{equivalent to back-to-back #7 hoops at 7in}$$

$$\text{spacing}_7 := 3.5in \quad \text{equivalent to a #7 hoop at 3.5in}$$

Actual Longitudinal Reinforcement Ratios:

$$\rho_{\text{long},1} := \frac{26 \cdot A_b}{A_g} = 0.9962\%$$

$$\rho_{\text{long},1.5} := \frac{40 \cdot A_b}{A_g} = 1.5326\%$$

$$\rho_{\text{long},2} := \frac{26.2 \cdot A_b}{A_g} = 1.9924\%$$

Actual Transverse Volumetric Steel Ratios:

$$\rho_{\text{trans},0.7} := \frac{4 \cdot A_{h6}}{D'_6 \cdot \text{spacing}_6} = 0.7477\%$$

$$\rho_{\text{trans},1} := \frac{4 \cdot A_{h7}}{D'_7 \cdot \text{spacing}_7} = 1.0215\%$$

Actual Column Dead Load Axial Load Ratios:

$$\frac{P_{\text{axial},8}}{f_{ce} \cdot A_g} = 8.0295\% \quad P_{\text{axial},8} = 1700\text{-kip}$$

$$\frac{P_{\text{axial},12}}{f_{ce} \cdot A_g} = 12.0443\% \quad P_{\text{axial},12} = 2550\text{-kip}$$

$$\frac{P_{\text{axial},16}}{f_{ce} \cdot A_g} = 16.0591\% \quad P_{\text{axial},16} = 3400\text{-kip}$$

Compute the Guide Spec Displacement Demands for Each SDOF Column:

- Conduct Moment-Curvature
- Define the Elastic Stiffness
- Compute the Elastic Period
- Determine the Seismic Demand
- Compare to the Capacity as Defined by the Strain Limits

For each column, the tributary seismic weight was taken as 1.6x the dead load axial force for the specific column axial load ratios of interest. The 1.6 factor on the tributary seismic weight is representative of a two span bridge with free abutments and a monolithic cap capable of placing the column in double bending under longitudinal response.

SF := 1.6 scale factor applied to the column dead load to obtain the tributary seismic mass

Diameter	Long. Steel	Ast/Ag	Trans. Steel	4Ah/D's	Axial Load	P/f'ceAg	L/D	Designation
72 in	26 - #11	0.01	#6 @ 3.5 in	0.007	1700 kip	0.08	4	C1,0,7,8,4
							5	C1,0,7,8,5
							6	C1,0,7,8,6
					2550 kip	0.12	4	C1,0,7,12,4
							5	C1,0,7,12,5
							6	C1,0,7,12,6
		0.01	#7 @ 3.5 in	0.01	3400 kip	0.16	4	C1,0,7,16,4
							5	C1,0,7,16,5
							6	C1,0,7,16,6
		40 - #11	0.015	#6 @ 3.5 in	1700 kip	0.08	4	C1,1,8,4
							5	C1,1,8,5
							6	C1,1,8,6
					2550 kip	0.12	4	C1,1,12,4
							5	C1,1,12,5
							6	C1,1,12,6
			0.01	#7 @ 3.5 in	3400 kip	0.16	4	C1,1,16,4
							5	C1,1,16,5
							6	C1,1,16,6
		52 - #11	0.02	#6 @ 3.5 in	1700 kip	0.08	4	C1.5,0,7,8,4
							5	C1.5,0,7,8,5
							6	C1.5,0,7,8,6
					2550 kip	0.12	4	C1.5,0,7,12,4
							5	C1.5,0,7,12,5
							6	C1.5,0,7,12,6
			0.01	#7 @ 3.5 in	3400 kip	0.16	4	C1.5,0,7,16,4
							5	C1.5,0,7,16,5
							6	C1.5,0,7,16,6
					1700 kip	0.08	4	C2,0,7,8,4
							5	C2,0,7,8,5
							6	C2,0,7,8,6
			0.01	#7 @ 3.5 in	2550 kip	0.12	4	C2,0,7,12,4
							5	C2,0,7,12,5
							6	C2,0,7,12,6
					3400 kip	0.16	4	C2,0,7,16,4
							5	C2,0,7,16,5
							6	C2,0,7,16,6
			0.02	#6 @ 3.5 in	1700 kip	0.08	4	C2,1,8,4
							5	C2,1,8,5
							6	C2,1,8,6
					2550 kip	0.12	4	C2,1,12,4
							5	C2,1,12,5
							6	C2,1,12,6
			0.01	#7 @ 3.5 in	3400 kip	0.16	4	C2,1,16,4
							5	C2,1,16,5
							6	C2,1,16,6

Sample Calculations for C1.5,0.7,12,5

- 1.5% longitudinal reinforcement ratio
- 0.7% transverse volumetric steel ratio
- 12% axial load ratio
- aspect ratio L/D of 5
- column is in double bending
- seismic weight scale factor of 1.6 applied to the dead load axial

Material Properties from Table 8.4.2-1 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, all Reinforcement ASTM A706 Gr60

$\epsilon_{uh} := 0.12$	ultimate tensile strain for the transverse reinf, #6 for this section
$f_{yhe} := 68\text{-ksi}$	expected yield stress of trans reinf
$\epsilon_{su} := 0.09$	ultimate tensile strain for the longitudinal reinf, #11 for this section
$\epsilon_{su_r} := 0.06$	reduced ultimate tensile strain for the longitudinal reinf
$\epsilon_{sh} := 0.0115$	onset of strain hardening for the longitudinal reinf
$f_{ye} := 68\text{ksi}$	expected yield stress long reinf
$f_{ue} := 95\text{ksi}$	expected tensile/ultimate stress long/trans reinf
$\epsilon_{ye} := 0.0023$	expected yield stain long/trans reinf
$E_s := 29000\text{ksi}$	steel modulus of elasticity
$f_c = 4\text{-ksi}$	compressive strength of concrete
$f_{ce} = 5.2\text{-ksi}$	expected compressive strength of concrete from Eq. 8.4.4-1
$E_{ce} := 1820 \cdot \sqrt{\frac{f_{ce}}{\text{ksi}}} \cdot \text{ksi} = 4150.2385\text{-ksi}$	expected concrete modulus of elasticity

Section Details:

$D = 6\text{-ft}$	circular section diameter
$A_g = 28.2743\text{-ft}^2$	circular section gross area
$n := 40$	number of longitudinal bars within the section
$d_{b1} := d_b = 1.41\text{-in}$	longitudinal bar diameter for #11
$A_{b1} := A_b = 1.56\text{-in}^2$	long bar area

$A_{st} := n \cdot A_{bl} = 62.4 \cdot \text{in}^2$	total longitudinal steel area within the section
$dh := dh_6 = 0.75 \cdot \text{in}$	transverse bar diameter for #6 hoop
$A_h := Ah_6 = 0.44 \cdot \text{in}^2$	trans bar area
$sh := \text{spacing}_6 = 3.5 \cdot \text{in}$	centerline spacing of single hoops
$\text{cover} = 2 \cdot \text{in}$	concrete cover measured to the outside of the transverse steel
$P_{DL} := P_{\text{axial},12} = 2550 \cdot \text{kip}$	dead load axial force
$f_{ce} = 5.2 \cdot \text{ksi}$	expected concrete compressive strength
$E_{ce} = 4150.2385 \cdot \text{ksi}$	concrete modulus with expected material properties
$L_{col} := 5 \cdot D = 30 \cdot \text{ft}$	total column height, note the column is in double bending
$D' := D - 2 \cdot \text{cover} - dh = 67.25 \cdot \text{in}$	confined core diameter measured between hoop centerlines

Longitudinal Parameters:

$ALR_{DL} := \frac{P_{DL}}{f_{ce} \cdot A_g} = 0.1204$	axial load ratio
$\rho_1 := \frac{A_{st}}{A_g} = 0.0153$	longitudinal steel ratio
$\rho_s := \frac{4 \cdot A_h}{D' \cdot sh} = 0.0075$	transverse volumetric steel ratio

Material Strain Limits:

All calculations utilize expected material properties.

Mander et al. (1988) Ultimate Concrete Compressive Strain, PL2 Conc:

$f_l := 0.5 \cdot \rho_s \cdot f_{yhe} = 0.2542 \cdot \text{ksi}$	lateral confining stress
$A_c := \frac{\pi \cdot D'^2}{4} = 3552.0123 \cdot \text{in}^2$	area of the confined core

$$\rho_{cc} := \frac{A_{st}}{A_c} = 0.0176 \quad \text{longitudinal steel ratio for the confined core}$$

$$s' := sh - dh = 2.75 \text{-in} \quad \text{clear spacing between circular hoops}$$

$$Ke := \frac{\left(1 - \frac{s'}{2D'}\right)^2}{(1 - \rho_{cc})} = 0.9767 \quad \text{confinement effectiveness coefficient for circular hoops}$$

$$f_1 := Ke \cdot f_y = 0.2483 \cdot \text{ksi} \quad \text{effective lateral confining stress}$$

$$f_{cce} := f_{ce} \cdot \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 \cdot f_1}{f_{ce}}} - 2 \cdot \frac{f_1}{f_{ce}} \right) = 6.7471 \cdot \text{ksi} \quad \text{maximum confined concrete compressive strength}$$

$$\epsilon_{uh} = 0.12 \quad \text{ultimate tensile strain for the transverse reinf}$$

$$\epsilon_{cu} := 0.004 + \frac{1.4 \cdot \rho_s \cdot f_y h_e \cdot \epsilon_{uh}}{f_{cce}} = 0.0167 \quad \text{ultimate concrete compressive strain}$$

ϵ_{cu} matches value computed within the moment-curvature software

140% Mander et al. (1988) Ultimate Concrete Compressive Strain, PL1 Conc:

$$\epsilon_{cu,PL1} := 1.4 \cdot \epsilon_{cu} = 0.0233$$

Plot the Original and Modified Concrete Compressive Stress vs. Strain Curves:

$$\epsilon_{co} := 0.002 \quad \text{unconfined concrete strain}$$

$$\epsilon_{cc} := \epsilon_{co} \left[1 + 5 \left(\frac{f_{cce}}{f_{ce}} - 1 \right) \right] = 0.005 \quad \text{strain at maximum confined concrete stress}$$

$$E_{sec} := \frac{f_{cce}}{\epsilon_{cc}} = 1356.1377 \cdot \text{ksi} \quad \text{secant stiffness to the maximum confined concrete stress}$$

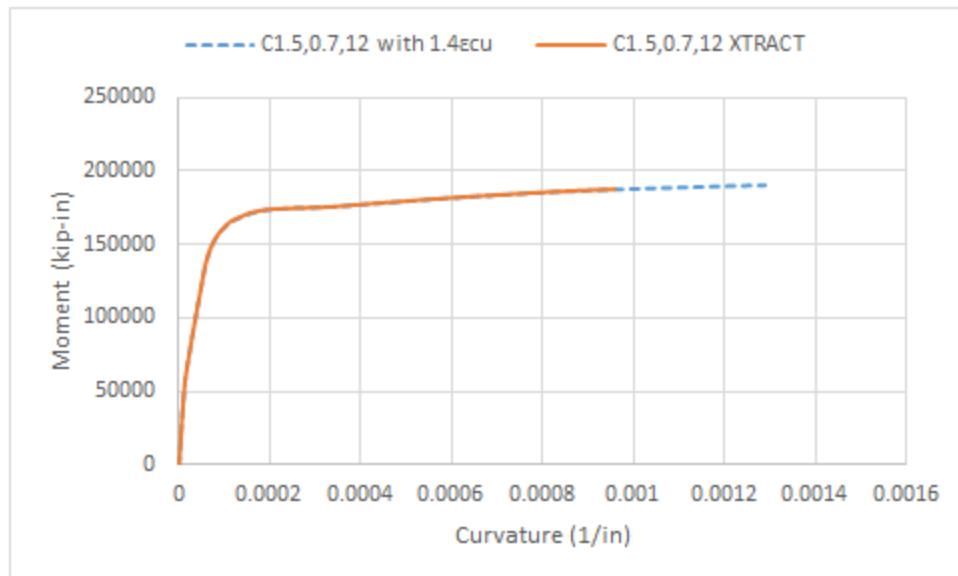
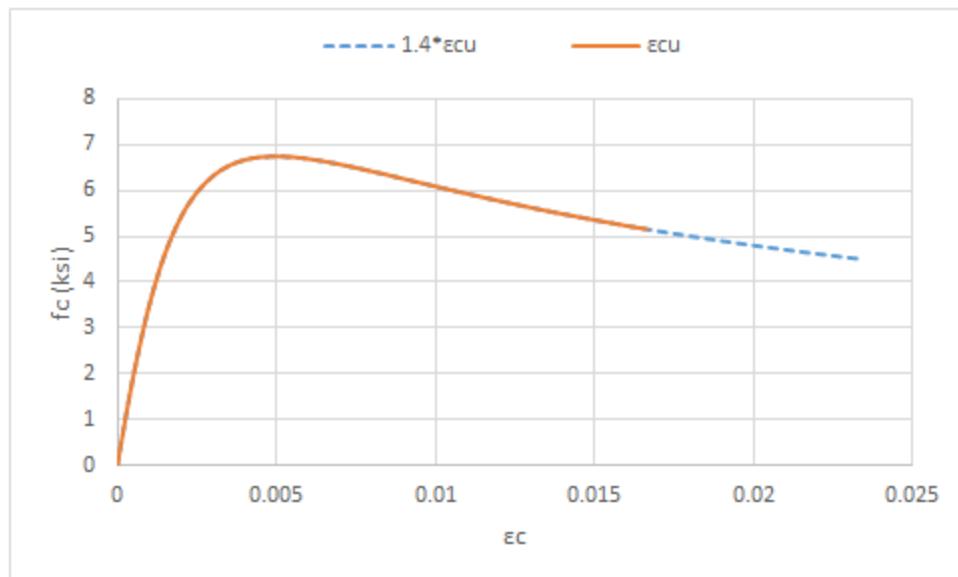
$$E_{ce} = 4150.2385 \cdot \text{ksi} \quad \text{concrete modulus}$$

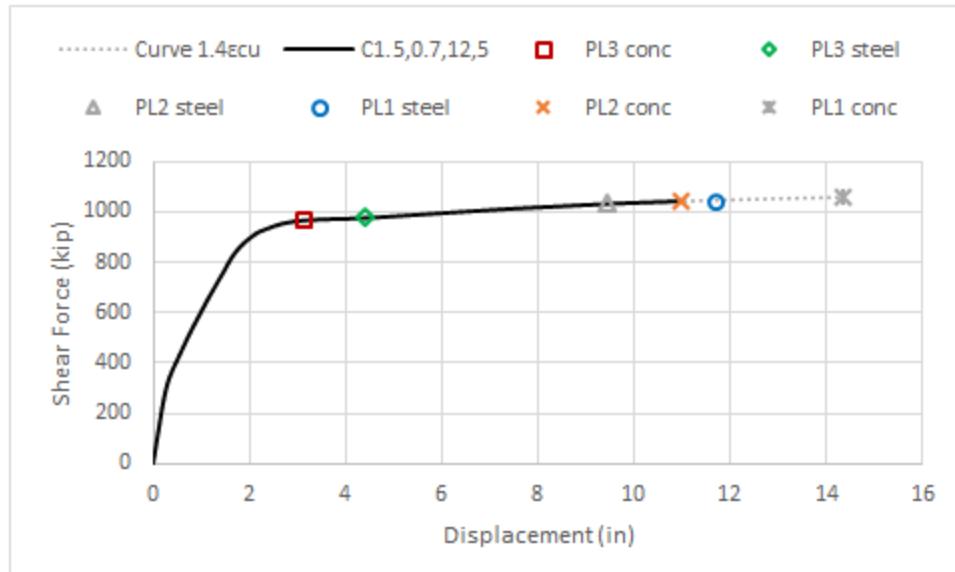
$$r := \frac{E_{ce}}{E_{ce} - E_{sec}} = 1.4854$$

Plot the Original and Extended Mander et al. (1988) Confined Concrete Stress-Strain Curve:

$$f_c = \frac{f_{cc} \cdot x \cdot r}{r - 1 + x}$$

where $x = \frac{\varepsilon_c}{\varepsilon_{cc}}$





Setting PL1 Concrete to $1.4\epsilon_{cu}$ provides a continuation of the moment curvature curve to higher rotations. As defined, the longitudinal bar buckling strain limit PL1 Steel is very likely to control, since detailing to increase the bar buckling strain limit will also increase confinement.

Longitudinal Bar Buckling Steel Tensile Strain Limit $\epsilon_{s,bb}$, PL1 Steel:

$$\epsilon_{s,bb} := 0.032 + 790 \cdot \rho_s \cdot \frac{f_y h_e}{E_s} - 0.14 \cdot ALR_{DL} = 0.029 \quad \text{bar buckling tensile strain limit}$$

Reduced Probability of Long. Bar Buckling Steel Tensile Strain Limit $\epsilon_{s,rbb}$, PL2 Steel:

$$\epsilon_{s,rbb} := 0.8 \cdot \epsilon_{s,bb} = 0.0232 \quad \text{reduced probability of bar buckling tensile strain limit}$$

Cover Concrete Crushing $\epsilon_{c,cc}$, PL3 Conc:

$$\epsilon_{c,cc} := 0.004 \quad \text{compressive strain at initial cover concrete crushing}$$

Residual Crack Widths $\epsilon_{c,rcw}$, PL3 Steel:

$$\epsilon_{s,rcw} := 0.01 \quad \text{residual crack widths expected to exceed threshold for repair}$$

Plastic Hinge Lengths:

$$k := \min\left[0.2 \cdot \left(\frac{f_{ue}}{f_{ye}} - 1\right), 0.08\right] = 0.0794 \quad \text{moment gradient coefficient}$$

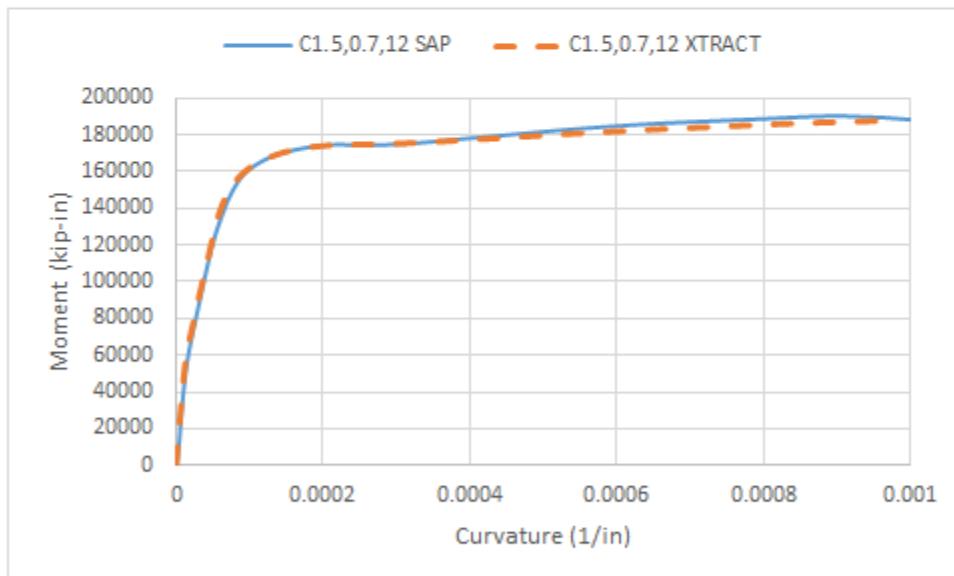
$$L_c := 0.5 \cdot L_{col} = 15 \cdot \text{ft} \quad \text{length from the point of contraflexure to max moment}$$

$$L_{sp} := 0.15 \cdot \frac{f_{ye}}{\text{ksi}} \cdot \text{dbl} = 14.382 \cdot \text{in} \quad \text{equivalent strain penetration length}$$

$$L_{pc} := \max(k \cdot L_c + L_{sp}, 2 \cdot L_{sp}) = 28.764 \cdot \text{in} \quad \text{compressive hinge length, equiv to Guide Spec } L_p$$

$$L_{pt} := L_{pc} + 0.4 \cdot D = 57.564 \cdot \text{in} \quad \text{tensile hinge length}$$

$$L_{eff} := L_{col} + 2 \cdot L_{sp} = 32.397 \cdot \text{ft} \quad \text{effective column length for double bending}$$

Obtain Limit State Curvatures from the Moment-Curvature Response for the Section:

For each section, the moment-curvature response predicted by two commercial software were compared to validate the input dimensions, constitutive models, and analysis results.

$$\phi y' := 0.00005761 \cdot \frac{1}{\text{in}} \quad \text{curvature at first yield of extreme fiber longitudinal reinf}$$

$$M_{y'} := 136400 \cdot \text{kip-in} \quad \text{moment at first yield of extreme fiber longitudinal reinf}$$

Find the Curvature for PL3 Conc and Steel:

$$\varepsilon_{c_{cc}} = 0.004 \quad \text{compressive strain at initial cover concrete crushing, PL3 conc}$$

$$\phi_{\varepsilon_{c,cc}} := 0.0001968 \cdot \frac{1}{in} \quad \text{limit state curvature at cover crushing, PL3 conc}$$

$$M_{\varepsilon_{c,cc}} := 174233.19 \cdot \text{kip} \cdot \text{in} \quad \text{moment at PL3 conc}$$

$$\varepsilon_{s_{rcw}} = 0.01 \quad \text{residual crack widths expected to exceed threshold for repair}$$

$$\phi_{\varepsilon_{s,rcw}} := 0.0002095 \cdot \frac{1}{in} \quad \text{limit state curvature for residual crack widths, PL3 steel}$$

The initial cover crushing strain limit is reached first, controlling the definition of PL3.

Find the Curvature for PL2 Conc and Steel:

$$\varepsilon_{cu} = 0.0167 \quad \text{ultimate concrete compressive strain, PL2 conc}$$

$$\phi_{\varepsilon_{cu}} := 0.0009616 \cdot \frac{1}{in} \quad \text{limit state curvature at the ult conc comp strain, PL2 conc}$$

$$M_{\varepsilon_{cu}} := 188100 \cdot \text{kip} \cdot \text{in} \quad \text{moment at the ult conc comp strain, PL2 conc}$$

$$\varepsilon_{s_{rbb}} = 0.0232 \quad \text{reduced probability of bar buckling tensile strain limit, PL2 steel}$$

$$\phi_{\varepsilon_{s,rbb}} := 0.00047513 \cdot \frac{1}{in} \quad \text{limit state curvature at PL2 steel}$$

Within the approach, the compressive and tensile strain limits are translated to lateral displacements with different plastic hinge lengths, thus the comparison for which occurs first cannot be made solely on the basis of limit state curvature. The relationship between force and displacement is always defined by the standard compressive plastic hinge length. Computed displacements at tensile strain limits evaluated with the tensile plastic hinge length are simply overlain on top of the force-displacement relationship defined by the compressive hinge length.

Find the Curvature for PL1 Steel:

$$\varepsilon_{s_{bb}} = 0.029 \quad \text{longitudinal bar buckling tensile strain limit, PL1 steel}$$

$$\phi_{\varepsilon_{s,bb}} := 0.00059358 \cdot \frac{1}{in} \quad \text{limit state curvature at bar buckling, PL1 steel}$$

By inspection, the PL1 Concrete strain limit corresponding the $1.4 * \varepsilon_{cu}$ does not control. This will be discussed in additional detail later.

There are two potential definitions for the bi-linear approximation, which impact the definition of the plastic curvature and thus plastic rotation utilized in deformation estimates using the plastic hinge method:

- Guide Spec Approach to determine ϕ_i and M_p with zero second slope stiffness
- Direct Displacement-Based (Priestley et al. 2007) Approach based on the computed equivalent yield curvature and nominal moment, with a positive second slope stiffness to the ultimate curvature.

Following the DDBD (Priestley et al. 2007) Approach:

$$M_n := 174233.19 \text{-kip-in} \quad \text{nominal moment corresponding to } \varepsilon_{c_{cc}} \text{ or } \varepsilon_{s_{rcw}} \text{ whichever occurs first}$$

$$\phi_y := \phi_y' \cdot \frac{M_n}{M_y'} = 7.3589 \times 10^{-5} \cdot \frac{1}{\text{in}} \quad \text{equivalent yield curvature}$$

$$\phi_u := \phi_{\varepsilon_{cu}} = 9.616 \times 10^{-4} \cdot \frac{1}{\text{in}} \quad \text{ultimate curvature defined by } \varepsilon_{cu}$$

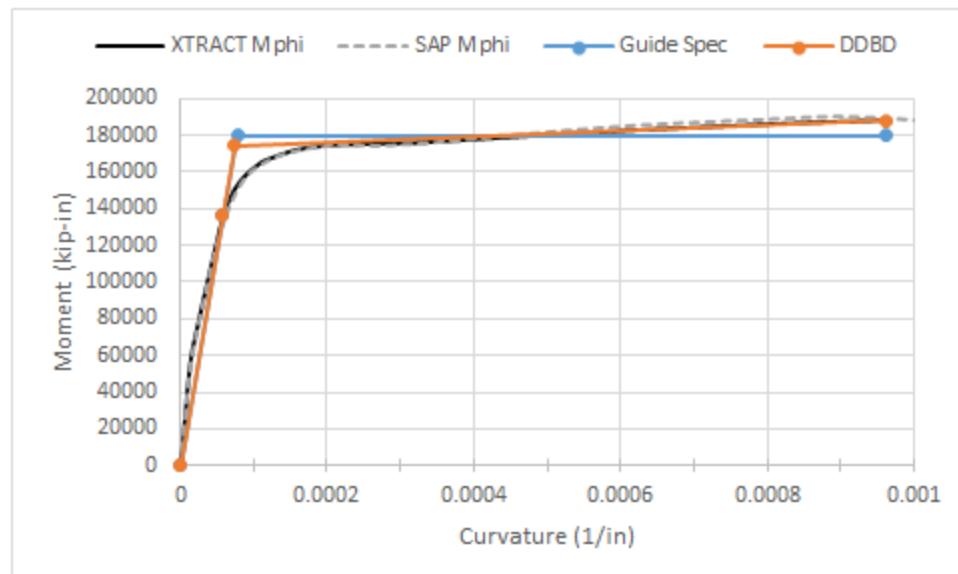
$$M_u := M_{\varepsilon_{cu}} = 1.881 \times 10^5 \text{-kip-in} \quad \text{ultimate moment defined by } \varepsilon_{cu}$$

Following the Guide Spec Approach:

$$\phi_{yi} := 7.872 \times 10^{-5} \cdot \frac{1}{\text{in}} \quad \text{Idealized yield curvature and moment obtained from matching equal areas above and below the second slope of the bi-linear approximation to the moment-curvature response.}$$

$$M_p := 179724 \text{-kip-in}$$

Comparison of Bi-Linear Approximation Methods:



For the purpose of this exercise, the bi-linear approximation is not used. The only impact on the values presented is the definition of the yield curvature (equivalent with DDBD, idealized with Guide Spec) which impacts the computed plastic curvature at the limit state of interest. The DDBD (Priestley et al. 2007) approach for defining equivalent yield was employed in this study.

$$\phi_{yi} = 7.872 \times 10^{-5} \cdot \frac{1}{in} \quad \text{idealized yield curvature with the Guide Spec Approach}$$

$$\phi_y = 7.3589 \times 10^{-5} \cdot \frac{1}{in} \quad \text{equivalent yield curvature from DDBD (Priestley et al. 2007)}$$

$$\text{Ratio} := \frac{\phi_{yi}}{\phi_y} = 1.0697 \quad \text{the two values are within general agreement}$$

The computed crack section moment of inertia is the same among the two approaches.

$$I_{cr} := \frac{My^4}{E_{ce} \cdot \phi_y} = 27.5118 \cdot ft^4 \quad \text{through first yield}$$

$$\frac{Mn}{E_{ce} \cdot \phi_y} = 27.5118 \cdot ft^4 \quad \text{through equivalent yield}$$

Compute the Limit State Displacements:

Displacement at PL3 Conc:

$$\phi_{\varepsilon_{c,cc}} = 1.968 \times 10^{-4} \cdot \frac{1}{in} \quad \text{limit state curvature at cover crushing, PL3 conc}$$

$$\phi p_{\varepsilon_{c,cc}} := \phi_{\varepsilon_{c,cc}} - \phi_y = 1.2321 \times 10^{-4} \cdot \frac{1}{in} \quad \text{plastic curvature at PL3 conc}$$

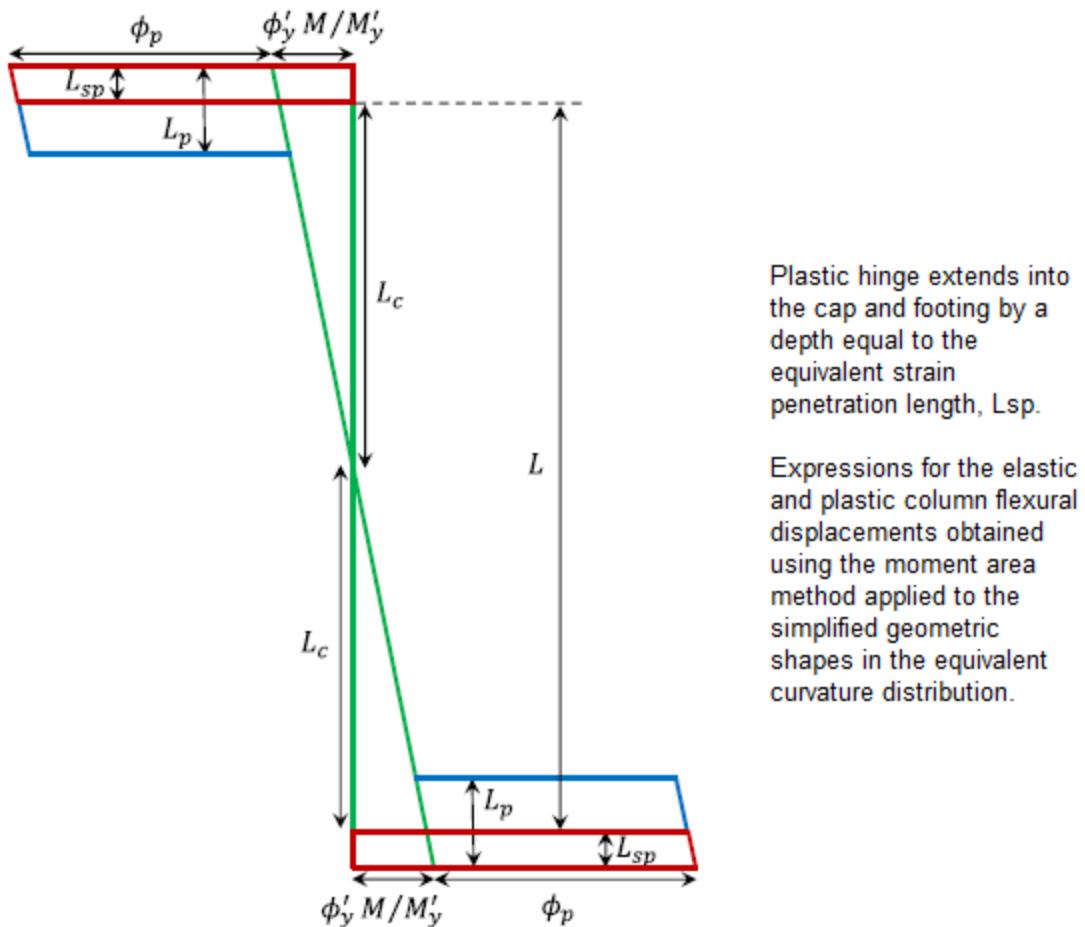
$$\Delta y := \frac{\phi_y \cdot L_{eff}^2}{6} = 1.8537 \cdot in \quad \text{elastic displacement for a column in double bending}$$

$$\theta_{p_{\varepsilon_{c,cc}}} := \phi p_{\varepsilon_{c,cc}} \cdot L_{pc} = 0.0035 \cdot rad \quad \text{plastic rotation at PL3 conc}$$

$$\Delta p_{\varepsilon_{c,cc}} := \phi p_{\varepsilon_{c,cc}} \cdot L_{pc} \cdot L_{col} = 1.2759 \cdot in \quad \text{plastic displacement at PL3 conc}$$

$$\Delta_{\varepsilon_{c,cc}} := \Delta y + \Delta p_{\varepsilon_{c,cc}} = 3.1295 \cdot in \quad \text{displacement at PL3 conc}$$

Compare to the more accurate double bending equivalent curvature distribution:



$$\Delta y' := \frac{\phi y' \cdot L_{eff}^2}{6} = 1.4512\text{-in} \quad \text{first yield displacement}$$

$$\Delta_{elastic_{\epsilon c.cc}} := \Delta y' \cdot \frac{M_{\epsilon c.cc}}{My'} = 1.8537\text{-in} \quad \text{elastic displacement}$$

$$\Delta_{plastic_{\epsilon c.cc}} := \left(\phi_{\epsilon c.cc} - \phi y' \cdot \frac{M_{\epsilon c.cc}}{My'} \right) \cdot L_{pc} \cdot (L_{col} + 2 \cdot L_{sp} - L_{pc}) = 1.2759\text{-in} \quad \text{plastic disp}$$

$$\Delta_{total_{\epsilon c.cc}} := \Delta_{elastic_{\epsilon c.cc}} + \Delta_{plastic_{\epsilon c.cc}} = 3.1295\text{-in} \quad \text{total flexural disp}$$

At cover crushing, the more accurate equivalent curvature distribution doesn't significantly impact displacement prediction when compared to the simplified approach. Note that all displacement estimates for the SDOF Hazard study came from the more accurate definition of the equivalent curvature distribution.

Displacement at PL2 Conc:

$$\phi_{\varepsilon_{cu}} = 9.616 \times 10^{-4} \cdot \frac{1}{in} \quad \text{limit state curvature at the ult conc comp strain, PL2 conc}$$

$$M_{\varepsilon_{cu}} = 1.881 \times 10^5 \cdot \text{kip}\cdot\text{in} \quad \text{moment at PL2 conc}$$

$$\phi p_{\varepsilon_{cu}} := \phi_{\varepsilon_{cu}} - \phi_y = 8.8801 \times 10^{-4} \cdot \frac{1}{in} \quad \text{plastic curvature at PL2 conc}$$

$$\theta p_{\varepsilon_{cu}} := \phi p_{\varepsilon_{cu}} \cdot L_{p_c} = 0.0255 \cdot \text{rad} \quad \text{plastic rotation at PL2 conc}$$

$$\Delta p_{\varepsilon_{cu}} := \phi p_{\varepsilon_{cu}} \cdot L_{p_c} \cdot L_{col} = 9.1954 \cdot \text{in} \quad \text{plastic displacement at PL2 conc}$$

$$\Delta_{\varepsilon_{cu}} := \Delta_y + \Delta p_{\varepsilon_{cu}} = 11.0491 \cdot \text{in} \quad \text{displacement at PL2 conc}$$

Compare to the more accurate double bending equivalent curvature distribution:

$$\Delta_{elastic_{\varepsilon_{cu}}} := \Delta_y' \cdot \frac{M_{\varepsilon_{cu}}}{My'} = 2.0012 \cdot \text{in} \quad \text{elastic disp}$$

$$\Delta_{plastic_{\varepsilon_{cu}}} := \left(\phi_{\varepsilon_{cu}} - \phi_y' \cdot \frac{M_{\varepsilon_{cu}}}{My'} \right) \cdot L_{p_c} \cdot (L_{col} + 2 \cdot L_{sp} - L_{p_c}) = 9.1347 \cdot \text{in} \quad \text{plastic disp}$$

$$\Delta_{total_{\varepsilon_{cu}}} := \Delta_{elastic_{\varepsilon_{cu}}} + \Delta_{plastic_{\varepsilon_{cu}}} = 11.136 \cdot \text{in} \quad \text{displacement at PL2 conc}$$

At the ultimate concrete compressive strain, the more accurate equivalent curvature distribution doesn't significantly impact displacement prediction when compared to the simplified approach. Note that all displacement estimates for the SDOF Hazard study came from the more accurate definition of the equivalent curvature distribution.

Displacement at PL1 Steel:

$$\phi_{\varepsilon_{s,rcw}} = 2.095 \times 10^{-4} \cdot \frac{1}{in} \quad \text{limit state curvature for residual crack widths, PL3 steel}$$

$$\phi p_{\varepsilon_{s,rcw}} := \phi_{\varepsilon_{s,rcw}} - \phi_y = 1.3591 \times 10^{-4} \cdot \frac{1}{in} \quad \text{plastic curvature at PL3 steel}$$

$$\theta p_{\varepsilon_{s,rcw}} := \phi p_{\varepsilon_{s,rcw}} \cdot L_{p_t} = 0.0078 \cdot \text{rad} \quad \text{plastic rotation at PL3 steel with } L_{p,t}$$

$$\phi p_{\varepsilon_{s,rcw}} \cdot L_{p_c} = 0.0039 \cdot \text{rad} \quad \text{plastic rotation at PL3 steel with } L_{p,c}$$

The increase from the compressive hinge length to the tensile hinge length is related to the impact of inclined flexural shear cracking on equilibrium within the disturbed region of the column plastic hinge. There is a spread in the extent of plastic curvatures as the inclined crack distribution is forming, where L_{pt} was fit to the total spread in plasticity. For the residual crack width limit state, the displacement ductility is generally low, thus a lower value between L_{pc} and L_{pt} may be more appropriate. L_{pt} is applicable without adjustment at the bar buckling strain limits for PL1 and PL2. For this exercise L_{pt} was applied even at the residual crack width limit state, which may overestimate its displacement.

$$\Delta p_{\varepsilon s,rcw} := \phi p_{\varepsilon s,rcw} \cdot L_{pt} \cdot L_{col} = 2.8165 \cdot \text{in} \quad \text{plastic displacement at PL3 steel}$$

$$\Delta_{\varepsilon s,rcw} := \Delta y + \Delta p_{\varepsilon s,rcw} = 4.6702 \cdot \text{in} \quad \text{displacement at PL3 steel}$$

$$\Delta y + \phi p_{\varepsilon s,rcw} \left(\frac{L_{pt} + L_{pc}}{2} \right) \cdot L_{col} = 3.9656 \cdot \text{in} \quad \text{for comparison, if a value midway between } L_{pc} \text{ and } L_{pt} \text{ were employed at the residual crack width limit state}$$

$$\begin{aligned} \Delta y + \phi p_{\varepsilon s,rcw} \cdot (L_{pc}) \cdot L_{col} &= 3.261 \cdot \text{in} \\ \Delta_{\varepsilon c,cc} &= 3.1295 \cdot \text{in} \end{aligned} \quad \text{even if the compressive hinge length were used, cover concrete crushing would control, which is often the case for bridge columns}$$

Displacement at PL2 Steel:

$$\phi_{\varepsilon s,rbb} = 4.7513 \times 10^{-4} \cdot \frac{1}{\text{in}} \quad \text{limit state curvature for reduced probability of initial bar buckling, PL2 steel}$$

$$\phi p_{\varepsilon s,rbb} := \phi_{\varepsilon s,rbb} - \phi_y = 4.0154 \times 10^{-4} \cdot \frac{1}{\text{in}} \quad \text{plastic curvature at PL2 steel}$$

$$\theta p_{\varepsilon s,rbb} := \phi p_{\varepsilon s,rbb} \cdot L_{pt} = 0.0231 \cdot \text{rad} \quad \text{plastic rotation at PL2 steel with } L_{pt}$$

$$\Delta p_{\varepsilon s,rbb} := \phi p_{\varepsilon s,rbb} \cdot L_{pt} \cdot L_{col} = 8.3211 \cdot \text{in} \quad \text{plastic displacement at PL2 steel}$$

$$\Delta_{\varepsilon s,rbb} := \Delta y + \Delta p_{\varepsilon s,rbb} = 10.1748 \cdot \text{in} \quad \text{displacement at PL2 steel}$$

Displacement at PL1 Steel:

$$\phi_{\varepsilon s,bb} = 5.9358 \times 10^{-4} \cdot \frac{1}{\text{in}} \quad \text{limit state curvature for initial longitudinal bar buckling, PL1 steel}$$

$$\phi p_{\varepsilon s,bb} := \phi_{\varepsilon s,bb} - \phi_y = 5.1999 \times 10^{-4} \cdot \frac{1}{\text{in}} \quad \text{plastic curvature at PL1 steel}$$

$$\theta_{p_{\varepsilon s, bb}} := \phi p_{\varepsilon s, bb} \cdot L_{p_t} = 0.0299 \text{-rad}$$

plastic rotation at PL1 steel with L_{p.t}

$$\Delta p_{\varepsilon s, bb} := \phi p_{\varepsilon s, bb} \cdot L_{p_t} \cdot L_{col} = 10.7758 \text{-in}$$

plastic displacement at PL1 steel

$$\Delta_{\varepsilon s, bb} := \Delta y + \Delta p_{\varepsilon s, bb} = 12.6295 \text{-in}$$

displacement at PL1 steel

Displacement Demands Computed from the Guide Spec Approach:

For the demonstration column, compute the elastic stiffness.

$$Fy' := \frac{2 \cdot M_y'}{L_{col}} = 757.7778 \text{-kip}$$

lateral force at first yield of the longitudinal reinf

$$\Delta y' = 1.4512 \text{-in}$$

displacement at first yield of the longitudinal reinf

$$k_{elastic} := \frac{Fy'}{\Delta y'} = 522.1835 \cdot \frac{\text{kip}}{\text{in}}$$

elastic flexural stiffness from the pushover curve

$$\frac{12 \cdot E_{ce} \cdot I_{cr}}{L_{eff}^3} = 483.548 \cdot \frac{\text{kip}}{\text{in}}$$

would place yield moment at L_{sp} into the foundation, where it should be at the interface of the adjoining member

$$\frac{12 \cdot E_{ce} \cdot I_{cr}}{L_{col}^3} = 608.9621 \cdot \frac{\text{kip}}{\text{in}}$$

would not include the additional flexibility due to strain penetration into the adjoining member

$$P_{DL} = 2550 \text{-kip}$$

column dead load axial force

For each column, the tributary seismic weight was taken as 1.6x the dead load axial force for the specific column axial load ratios of interest. The 1.6 factor on the tributary seismic weight is representative of a two span bridge with free abutments and a monolithic cap capable of placing the column in double bending under longitudinal response.

$$SF = 1.6$$

scale factor applied to the column dead load to obtain the tributary seismic mass

$$W_{DL} := SF \cdot P_{DL} = 4080 \text{-kip}$$

total seismic weight attributable to the column

$$M_{DL} := \frac{W_{DL}}{g} = 10.5675 \cdot \frac{\text{kip} \cdot \text{s}^2}{\text{in}}$$

total seismic mass attributable to the column

$$T_{elastic} := 2 \cdot \pi \cdot \sqrt{\frac{M_{DL}}{k_{elastic}}} = 0.8938 \text{ s}$$

elastic period

For all thirteen sites around the country, the displacement demands were computed in the 100YR and 975YR seismic hazards. For this demonstration, the New Madrid 975YR hazard was selected.

Hazard Definition: (New Madrid Seismic Region) 975YR Earthquake

$$A_s := 1.29g$$

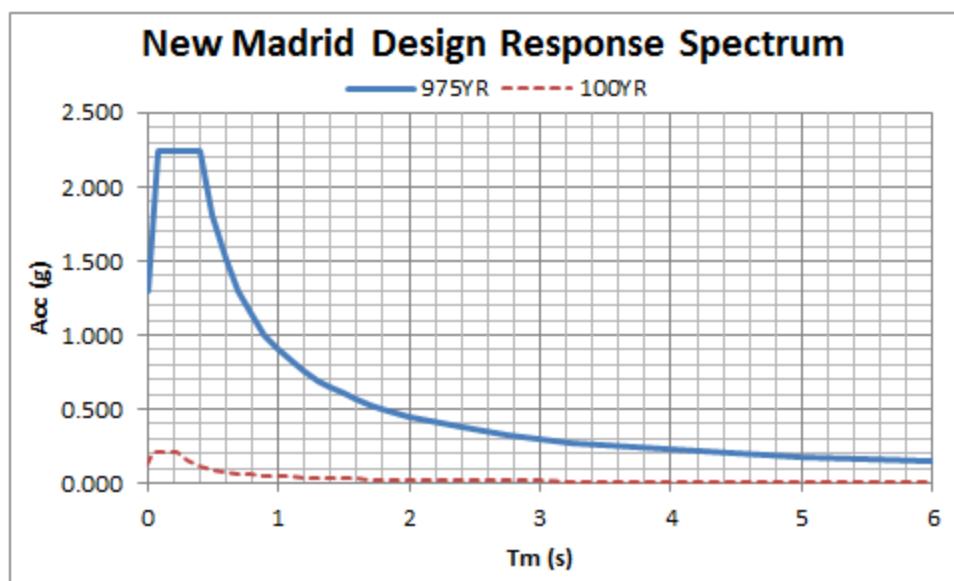
$$S_{ds} := 2.24g$$

$$S_{d1} := 0.90g$$

$$T_s := \frac{S_{d1}}{S_{ds}} \cdot \text{sec} = 0.4018 \text{ s}$$

$$T_o := 0.2 \cdot T_s = 0.0804 \text{ s}$$

$$T_{star} := 1.25 \cdot T_s = 0.5022 \text{ s}$$



$$S_a := \frac{S_{d1}}{\frac{T_{elastic}}{\text{sec}}} = 1.0069 \cdot g \quad \text{spectral acceleration at the elastic period}$$

$$V_{elastic} := S_a \cdot M_{DL} = 4108.1641 \cdot \text{kip} \quad \text{elastic shear force}$$

$$\Delta_{elastic} := \frac{V_{elastic}}{k_{elastic}} = 7.8673 \cdot \text{in} \quad \text{elastic displacement demand}$$

$$\frac{T_{\text{star}}}{T_{\text{elastic}}} = 0.5619 \quad \text{which is less than 1, therefore displacement magnification is not required}$$

$$R_d := 1.0 \quad \text{longitudinal displacement magnification factor}$$

$$\Delta_D := \Delta_{\text{elastic}} \cdot R_d = 7.8673 \text{-in} \quad \text{displacement demand in the selected hazard}$$

Displacement Demands from the Displacement-Based Assessment Approach:

This section follows the Displacement-Based Assessment Approach in which the demand displacement is iterated upon until the base shear and deformation align with the pushover curve. At this point, the displacement ductility demand and computed equivalent viscous damping are consistent with the reduction in spectral ordinates used to compute the demand displacement. The governing principles for Direct Displacement-Based Design are described in Priestley et al. (2007).

$$\Delta_d := 11.55 \text{in} \quad \text{demand displacement, iterated until convergence}$$

$$\Delta_y = 1.8537 \text{-in}$$

$$\mu_\Delta := \frac{\Delta_d}{\Delta_y} = 6.2308 \quad \text{displacement ductility}$$

$$\xi_{\text{eff}} := 0.05 + 0.444 \cdot \left(\frac{\mu_\Delta - 1}{\mu_\Delta \cdot \pi} \right) = 0.1686 \quad \text{equivalent viscous damping with tangent stiffness proportional elastic damping}$$

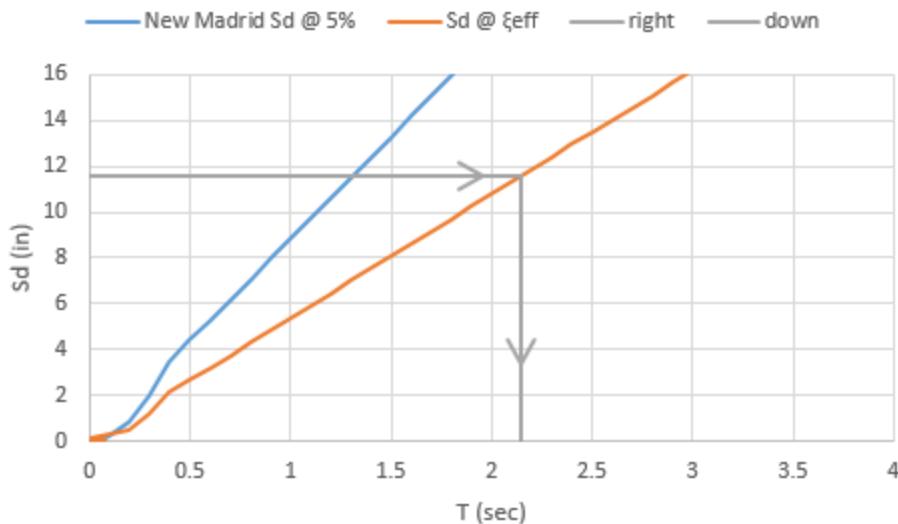
$$M_{\text{DL}} = 10.5675 \cdot \frac{\text{kip} \cdot \text{s}^2}{\text{in}} \quad \text{tributary seismic mass}$$

First the acceleration spectra is converted to a displacement spectra.

$$S_d = \frac{S_a}{\left(\frac{4 \cdot \pi^2}{T^2} \right)} \quad \text{resulting in the 5% damped displacement spectra}$$

Determine the effective period at the demand displacement corresponding to the damped displacement spectra at the computed value of equivalent viscous damping.

$$R_\xi := \left(\frac{0.07}{0.02 + \xi_{\text{eff}}} \right)^{0.5} = 0.6091 \quad \text{damping modifier for spectral ordinates employed in DDBD}$$



Enter with the demand displacement, read across to the damped displacement spectra and down to the effective period. The effective period is obtained as...

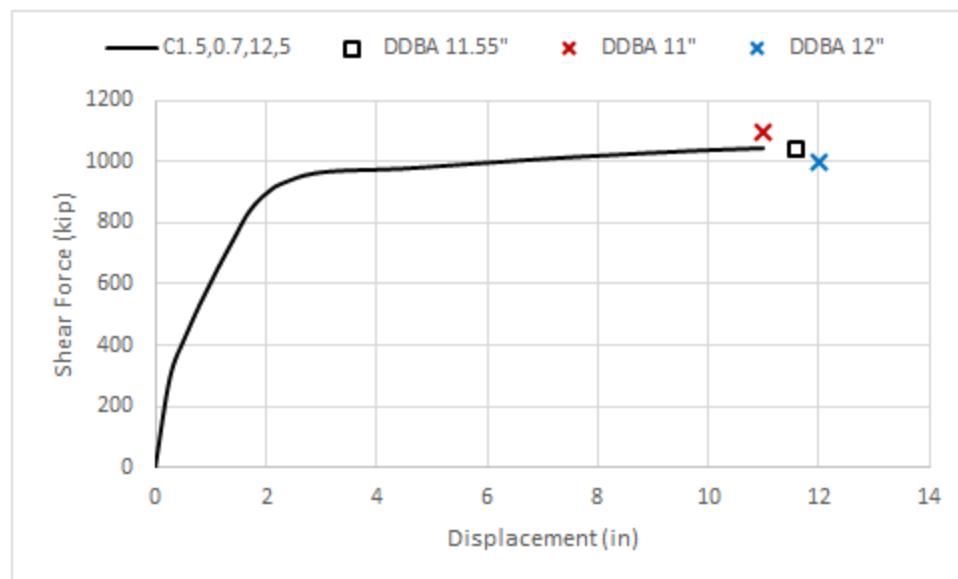
$T_{eff} := 2.15\text{sec}$ effective period from the damped displacement spectra at the demand disp

$$K_{eff} := \frac{4 \cdot \pi^2 \cdot M_{DL}}{T_{eff}^2} = 90.2518 \cdot \frac{\text{kip}}{\text{in}}$$

secant stiffness to the demand displacement

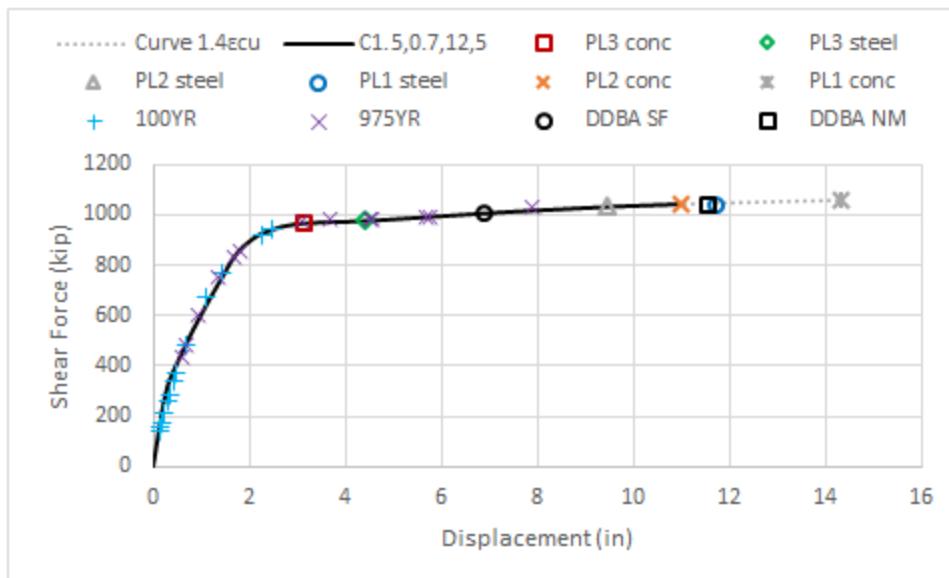
$V_{col} := K_{eff} \cdot \Delta d = 1042.4086\text{-kip}$ column shear computed from the secant stiffness

Plot the column shear and demand displacement along the pushover curve for the column.



The impact of iteration in the demand displacement is shown above. A demand displacement of 11" was above the curve while a demand displacement of 12" fell below the curve. The demand displacement of 11.55" is beyond the pushover which had ended at the ultimate concrete compressive strain, but at a similar force indicating it is close to an extension of the curve.

For each column the 100YR and 975Year Displacement Demands from the Guide Spec are shown on the column pushover curve with DDBA Demands for the San Francisco and New Madrid 975YR hazards. Limit state displacements are shown on the column pushover results to provide an indication of the expected damage. Use of DDBA with tangent stiffness proportional elastic damping produces displacement demands where longitudinal bar buckling would be expected. For this column, the largest 100YR displacement demands computed with the Guide Spec wouldn't exceed the strain limits defined under PL1, indicating that repairs would not likely be required.



$$\Delta_{\text{DDBA}} := \Delta d = 11.55\text{-in}$$

Impact of tangent stiffness proportional elastic damping included in the DDBA significant.

$$\Delta_{\text{GuideSpec}} := \Delta D = 7.8673\text{-in}$$

Damping Investigation using DDBA:

Use of initial stiffness proportional elastic damping is not recommended within DDBD, however initial stiffness proportional elastic damping is evaluated in the following section to understand the impact of damping model selection on demand displacements. The following figure demonstrates the impact of initial versus tangent stiffness proportional elastic damping. Specifically, the Thin Takeda (TT) curve corresponds to hysteretic rules typical of reinforced concrete bridge columns with degrading stiffness.

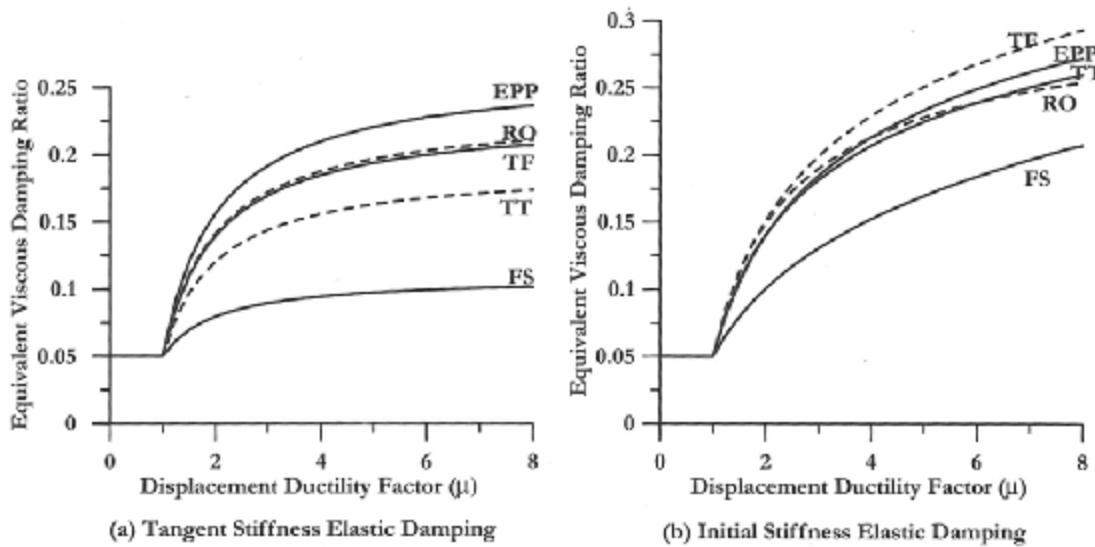


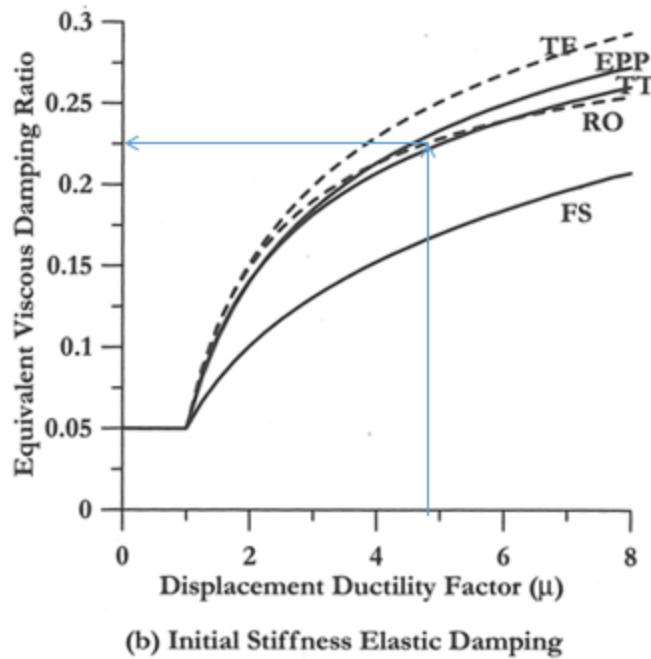
Fig.3.14 Design Equivalent Viscous Damping Ratios for 5% Elastic Damping

Repeating the DDBA for the demonstration column with initial stiffness proportional elastic damping. Note that tangent stiffness proportional elastic damping is recommended.

$$\Delta d_{IS} := 8.9\text{in} \quad \text{demand displacement, iterated until convergence}$$

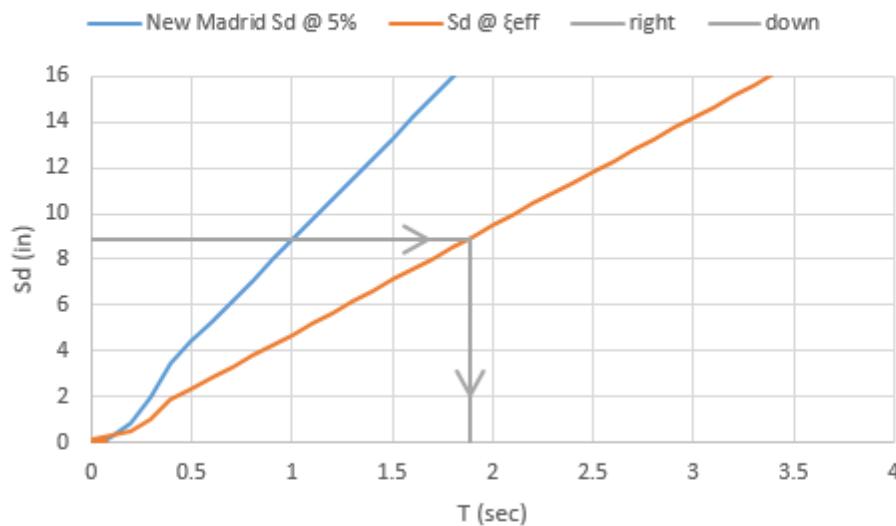
$$\Delta y = 1.8537\text{-in}$$

$$\mu_{\Delta IS} = \frac{\Delta d_{IS}}{\Delta y} = 4.8013 \quad \text{displacement ductility}$$



$\xi_{\text{eff,IS}} := 0.225$ equivalent viscous damping with initial stiffness proportional elastic damping

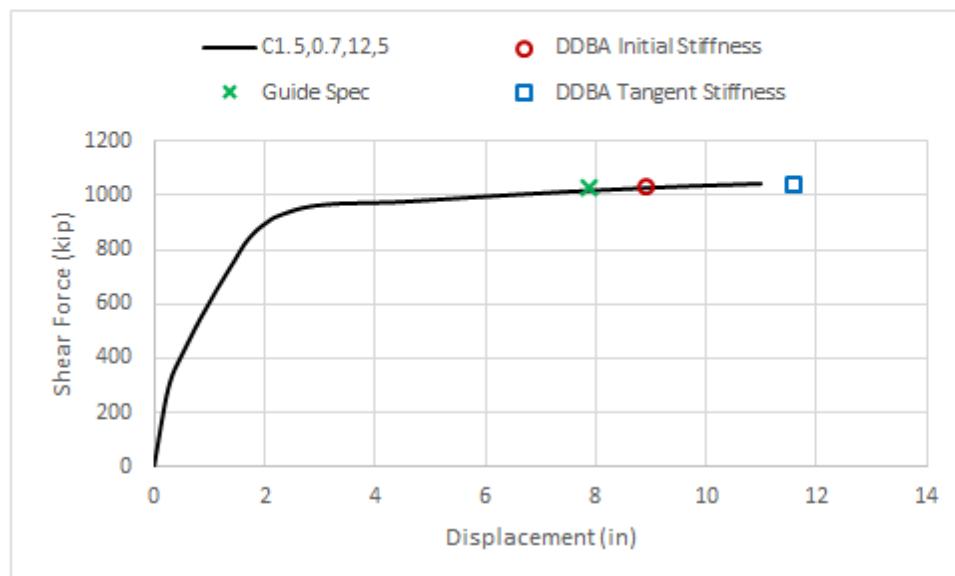
$$R_{\xi, \text{IS}} := \left(\frac{0.07}{0.02 + \xi_{\text{eff,IS}}} \right)^{0.5} = 0.5345 \quad \text{damping modifier for spectral ordinates employed in DDBD}$$



$T_{\text{eff,IS}} := 1.89 \text{ sec}$ effective period from the damped displacement spectra at the demand disp

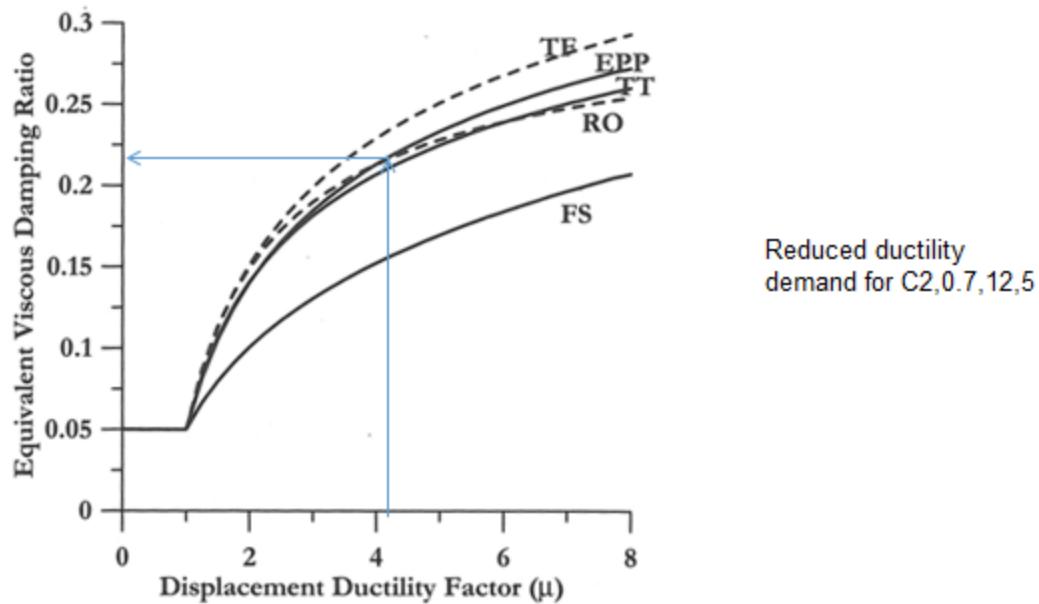
$$K_{\text{eff,IS}} := \frac{4 \cdot \pi^2 \cdot M_{\text{DL}}}{T_{\text{eff,IS}}^2} = 116.791 \frac{\text{kip}}{\text{in}} \quad \text{secant stiffness to the demand displacement}$$

$V_{\text{col,IS}} := K_{\text{eff,IS}} \cdot \Delta d_{\text{IS}} = 1039.4398 \text{ kip}$ column shear computed from the secant stiffness

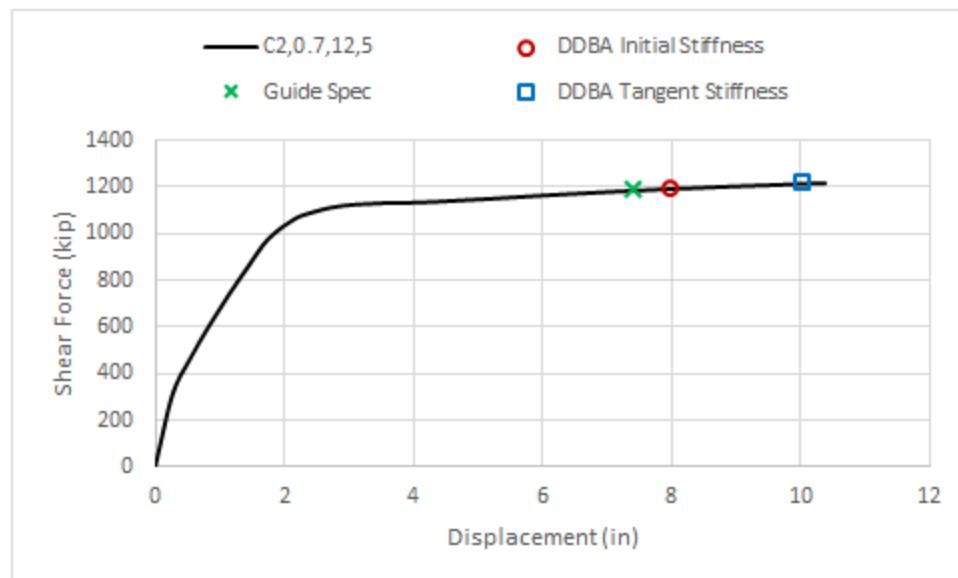


Use of initial stiffness proportional damping would bring the demand displacement closer to the Guide Spec. Evaluate several more columns to show general impact. The error in the equal displacement approximation employed by the Guide Spec (even after including short period modification R_d) appears to be greater under higher ductility demands.

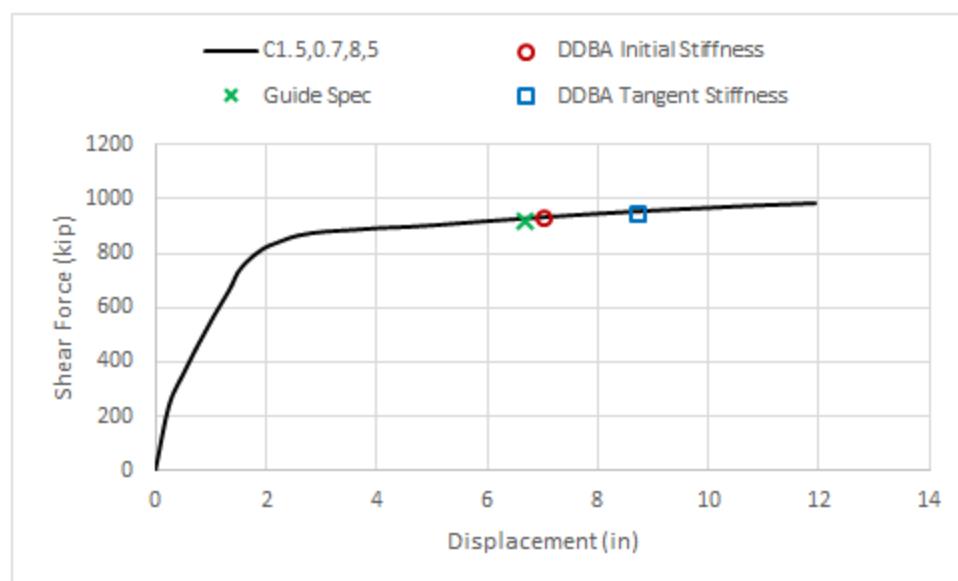
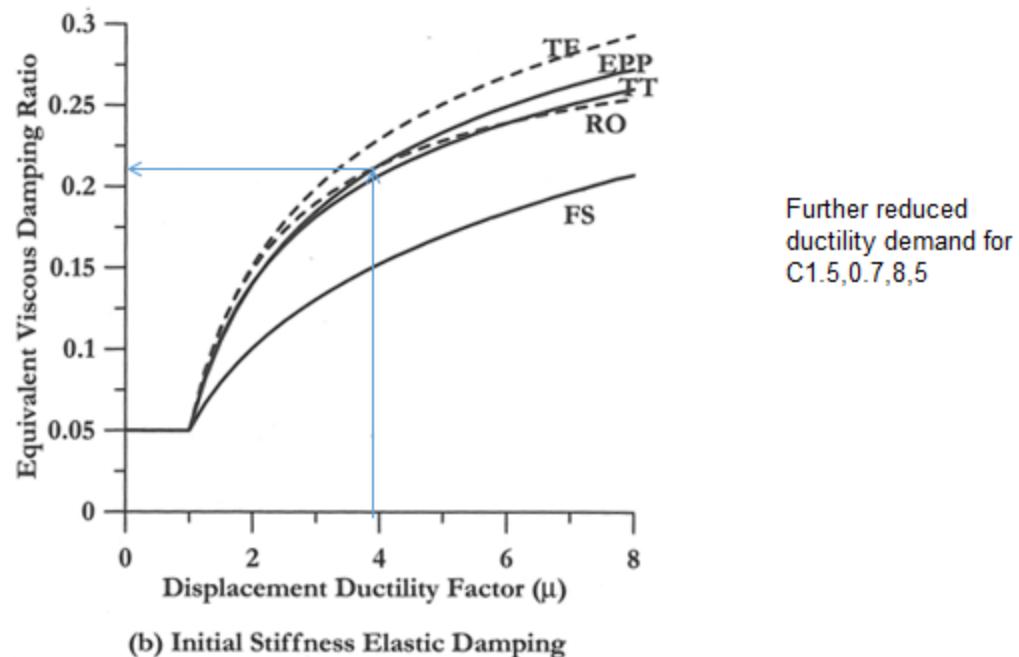
For the original column, but with 2% longitudinal steel, C2,0.7,12,5



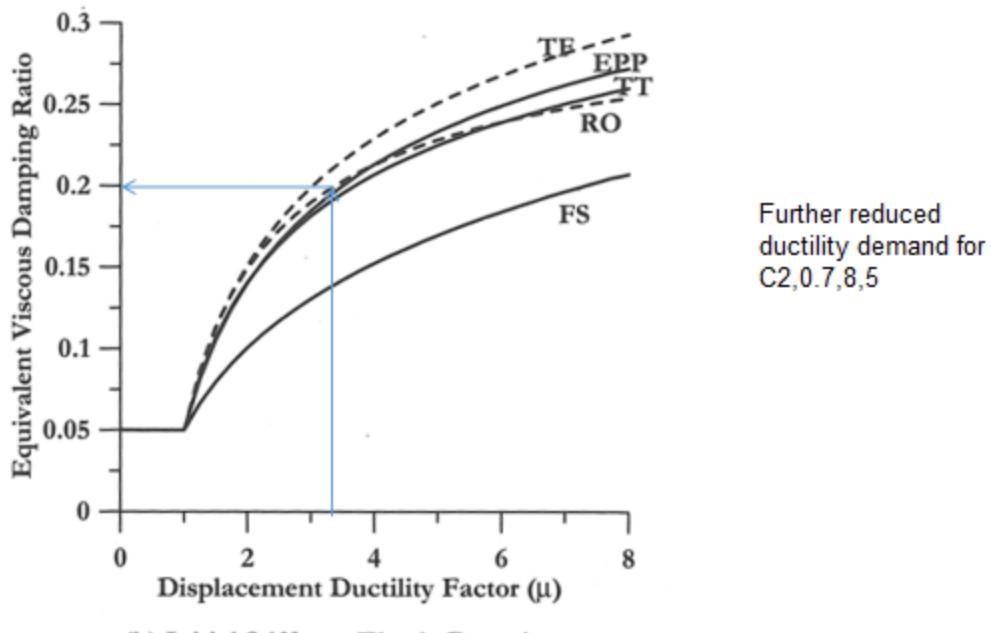
(b) Initial Stiffness Elastic Damping



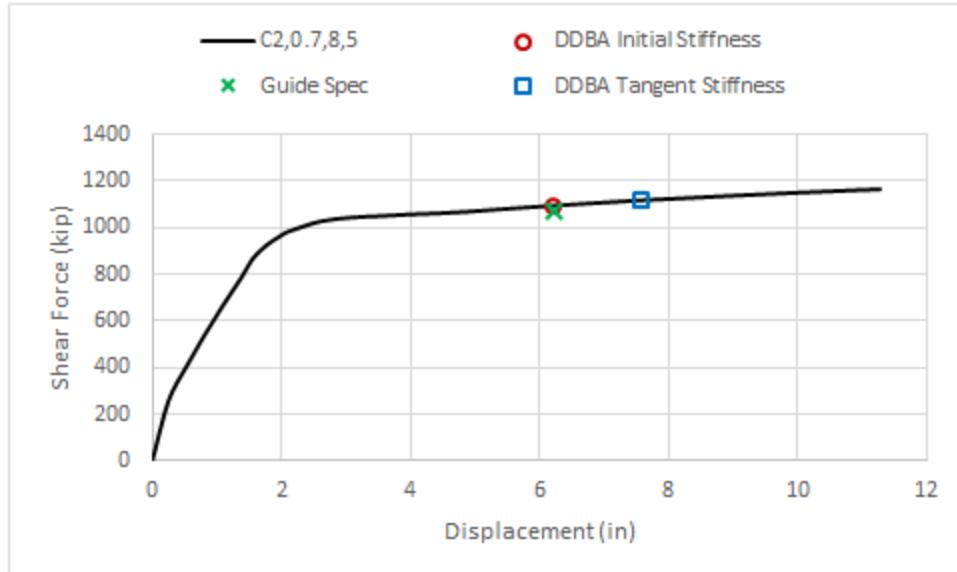
For the original column, but with 8% axial load, C1.5,0.7,8,5



Same as the previous column, but now with 2% longitudinal steel, C2,0.7,8,5

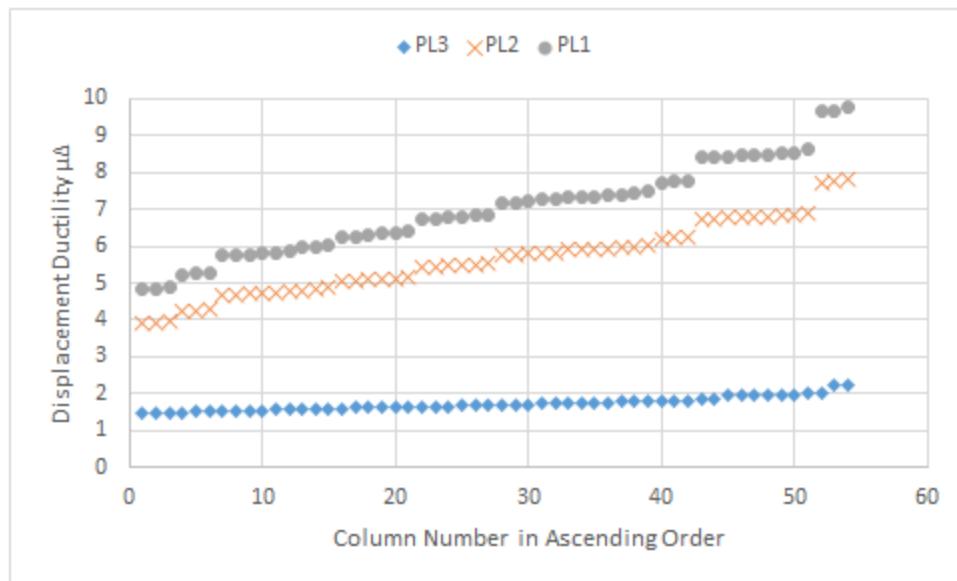


(b) Initial Stiffness Elastic Damping



Investigation of Displacement Ductility at the Governing Strain Limits at PL1, PL2, & PL3

For each column within the study matrix, the governing strain limits (concrete or steel) at each performance level were converted to limit state displacements. These displacements were then divided by the equivalent yield displacement to obtain the governing displacement ductility at each performance level. Observations were then sorted into ascending order and plotted to gain an understanding of the range in displacement ductility at strain limits which define each performance level.

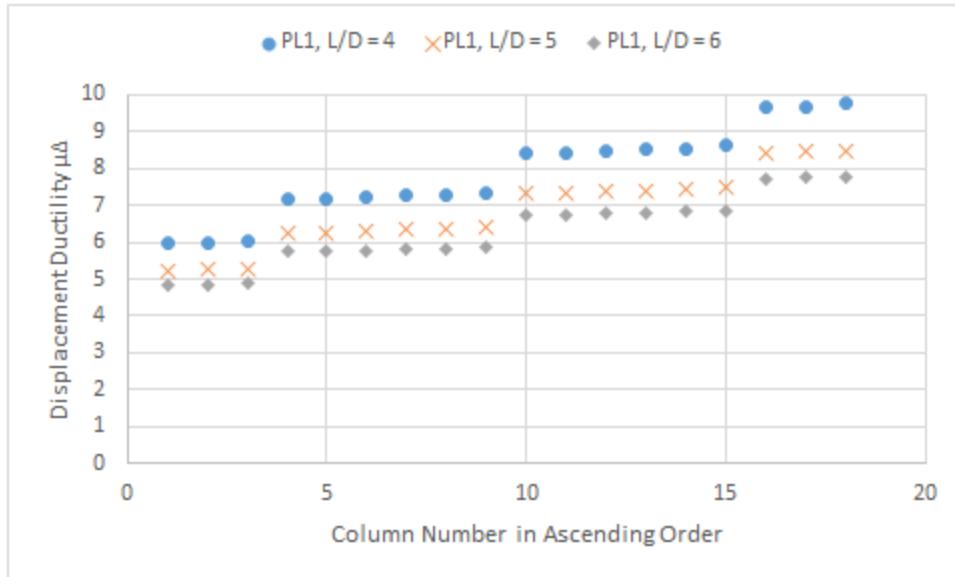


In general, PL3 was governed by the unconfined concrete compression strain limit of 0.004 where cover concrete crushing is expected to begin. The steel tensile strain limit of 0.01 related to residual crack widths may control in a multi-column bent under transverse demands due to the decrease in axial force on the column with seismic uplift.

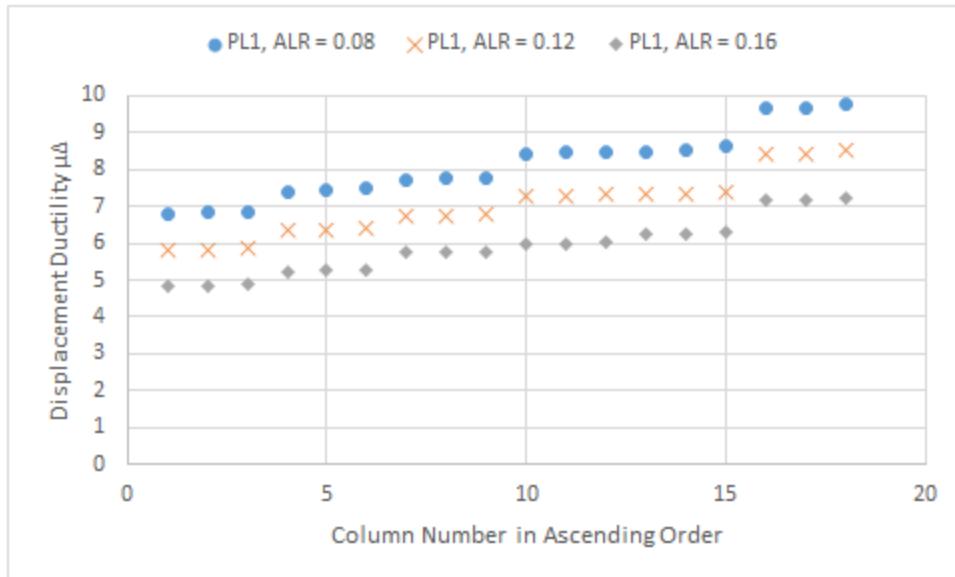
PL2 was generally governed by the steel tensile strain limit of $0.8 \times \epsilon_{s,bb}$, which represents a reduced probability of initial longitudinal bar buckling. Similarly, PL1 was governed by the longitudinal bar buckling steel tensile strain limit. For PL2 and PL1, similar trends in the computed limit state displacement ductility were observed. When defined by the longitudinal bar buckling strain limit, large variations in the computed displacement ductility were observed for columns in the study matrix. This is expected given the range in strength, detailing, and loading for columns in the study matrix. A constant limit state ductility would have implied that displacement ductility, rather than material strains was an accurate indicator of damage, which is not generally the case.

The variations in ductility defined by the bar buckling strain limits was investigated on a variable by variable basis to gain and understand for trends implied by the equation and to compare them to known behavior.

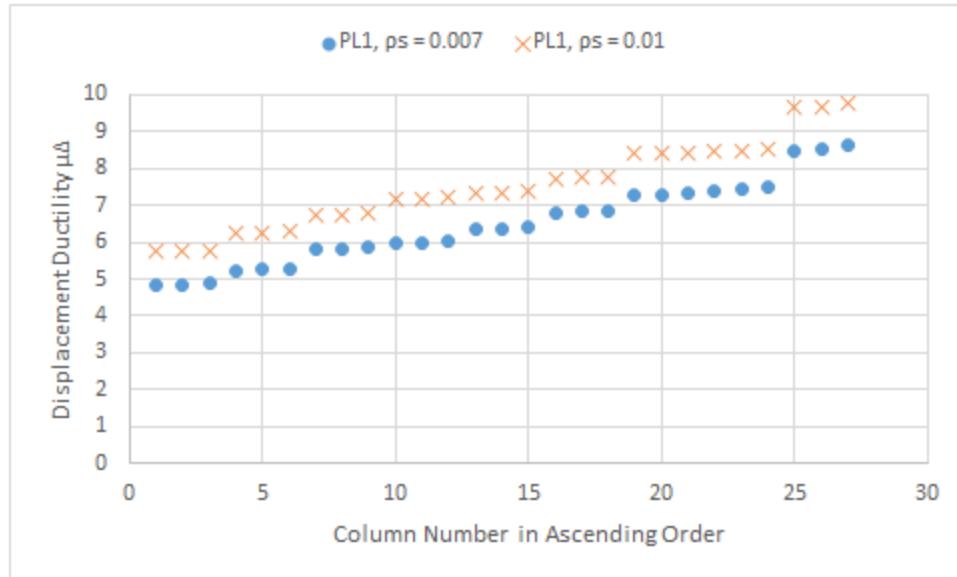
For the longitudinal bar buckling steel tensile PL1 strain limit, columns were separated by aspect ratio to compare its impact on the computed limit state displacement ductilities. As expected, the shorter columns had higher ductility demands due to their reduced equivalent yield displacement.



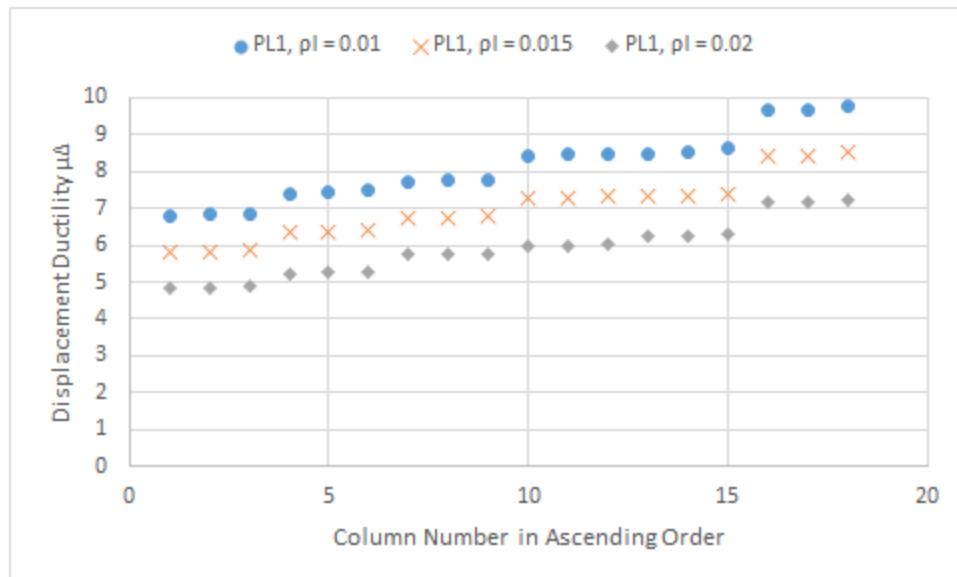
The impact of axial load ratio was then evaluated within the data set. As expected, columns with higher axial load ratios had lower displacement ductility capacities when defined by bar buckling.



The impact of transverse volumetric steel ratio was evaluated, with higher ratios providing additional restraint for the longitudinal bars, delaying bar buckling to higher displacement ductility levels.



Finally, the impact off longitudinal steel ratio on the computed bar buckling displacement ductility was evaluated, with higher steel contents reducing the ductility capacity as expected.



Use of a constant displacement ductility limit as a means of predicting damage would not provide the same relationships among the variables of interest and column behavior.

Results

The purpose of this study was to evaluate the proposed strain and plastic hinge length recommendations for a range of column details typical of bridge construction. The study column had a 6-ft diameter circular cross section as shown in Figure A-6. Longitudinal steel ratios, transverse volumetric steel ratios, axial load ratios, and column aspect ratios were varied in the study. Additional information regarding the selected detailing can be found in the study matrix of Table A-6. All of the columns were considered to act in double bending (double curvature). For each column, the tributary seismic weight was taken as 1.6 times the dead load axial force for the specific column axial load ratios of interest. The 1.6 factor on the tributary seismic weight is representative of a two-span bridge with free abutments and a monolithic cap capable of placing the column in double bending under longitudinal response.

For each column, moment-curvature analysis was conducted, and pushover curves were formed following the plastic hinge method recommended in these guidelines. Column displacements at key damage states defined by the recommended concrete and steel strain limits were plotted along the pushover curves. Displacement demands were evaluated with the *Guide Specifications* procedure for the 100YR and 975YR hazards at thirteen sites around the country with varying seismicity. Information used to construct the design response spectra for the 100YR and 975YR hazards can be found in Table A-7 and Table A-8. As a means of comparison, a direct displacement-based assessment (DDBA) procedure was employed to predict displacement demands in the San Francisco and New Madrid 975YR hazards. Sample calculations for a single column within the matrix were presented with commentary to describe the *Guide Specifications* and DDBA approaches in the preceding Sample Calculations section.

Results for individual columns within the study matrix are presented on Pages A-44 through A-97 according to the column designation defined within Table A-6. For example, the designation C1.5,0.7,12,5 represents a column with a longitudinal steel ratio of 1.5%, a transverse volumetric steel ratio of 0.7%, a dead load axial load ratio of 12%, and an aspect ratio of 5. The moment-curvature relation was run until the core concrete strain reached the ultimate concrete compressive strain (ϵ_{cu}) for the respective detailing. For this study, the moment-curvature relationship and computed pushover curves were not continued until the PL1 concrete limit defined by $1.4x \epsilon_{cu}$, since it was apparent that the PL1 steel limit defined by longitudinal bar buckling would control. The process for extending the pushover curve to the PL1 concrete limit is in the preceding Sample Calculations.



Figure A-6. Column cross sections.

Table A-6. SDOF double bending column matrix.

Diameter	Long. Steel	A _{st} /A _g	Trans. Steel	4Ah/D's	Axial Load	P/f' _{ce} A _g	L/D	Designation
26 - #11	0.01	#6 @ 3.5 in	0.007	1700 kip	0.08	4	C1,0,7,8,4	
						5	C1,0,7,8,5	
						6	C1,0,7,8,6	
				2550 kip	0.12	4	C1,0,7,12,4	
						5	C1,0,7,12,5	
						6	C1,0,7,12,6	
		#7 @ 3.5 in	0.01	3400 kip	0.16	4	C1,0,7,16,4	
						5	C1,0,7,16,5	
						6	C1,0,7,16,6	
				1700 kip	0.08	4	C1,1,8,4	
						5	C1,1,8,5	
						6	C1,1,8,6	
		#7 @ 3.5 in	0.01	2550 kip	0.12	4	C1,1,12,4	
						5	C1,1,12,5	
						6	C1,1,12,6	
				3400 kip	0.16	4	C1,1,16,4	
						5	C1,1,16,5	
						6	C1,1,16,6	
72 in	0.015	#6 @ 3.5 in	0.007	1700 kip	0.08	4	C1.5,0,7,8,4	
						5	C1.5,0,7,8,5	
						6	C1.5,0,7,8,6	
				2550 kip	0.12	4	C1.5,0,7,12,4	
						5	C1.5,0,7,12,5	
						6	C1.5,0,7,12,6	
		#7 @ 3.5 in	0.01	3400 kip	0.16	4	C1.5,0,7,16,4	
						5	C1.5,0,7,16,5	
						6	C1.5,0,7,16,6	
				1700 kip	0.08	4	C1.5,1,8,4	
						5	C1.5,1,8,5	
						6	C1.5,1,8,6	
		#7 @ 3.5 in	0.01	2550 kip	0.12	4	C1.5,1,12,4	
						5	C1.5,1,12,5	
						6	C1.5,1,12,6	
				3400 kip	0.16	4	C1.5,1,16,4	
						5	C1.5,1,16,5	
						6	C1.5,1,16,6	
52 - #11	0.02	#6 @ 3.5 in	0.007	1700 kip	0.08	4	C2,0,7,8,4	
						5	C2,0,7,8,5	
						6	C2,0,7,8,6	
				2550 kip	0.12	4	C2,0,7,12,4	
						5	C2,0,7,12,5	
						6	C2,0,7,12,6	
		#7 @ 3.5 in	0.01	3400 kip	0.16	4	C2,0,7,16,4	
						5	C2,0,7,16,5	
						6	C2,0,7,16,6	
				1700 kip	0.08	4	C2,1,8,4	
						5	C2,1,8,5	
						6	C2,1,8,6	
		#7 @ 3.5 in	0.01	2550 kip	0.12	4	C2,1,12,4	
						5	C2,1,12,5	
						6	C2,1,12,6	
				3400 kip	0.16	4	C2,1,16,4	
						5	C2,1,16,5	
						6	C2,1,16,6	

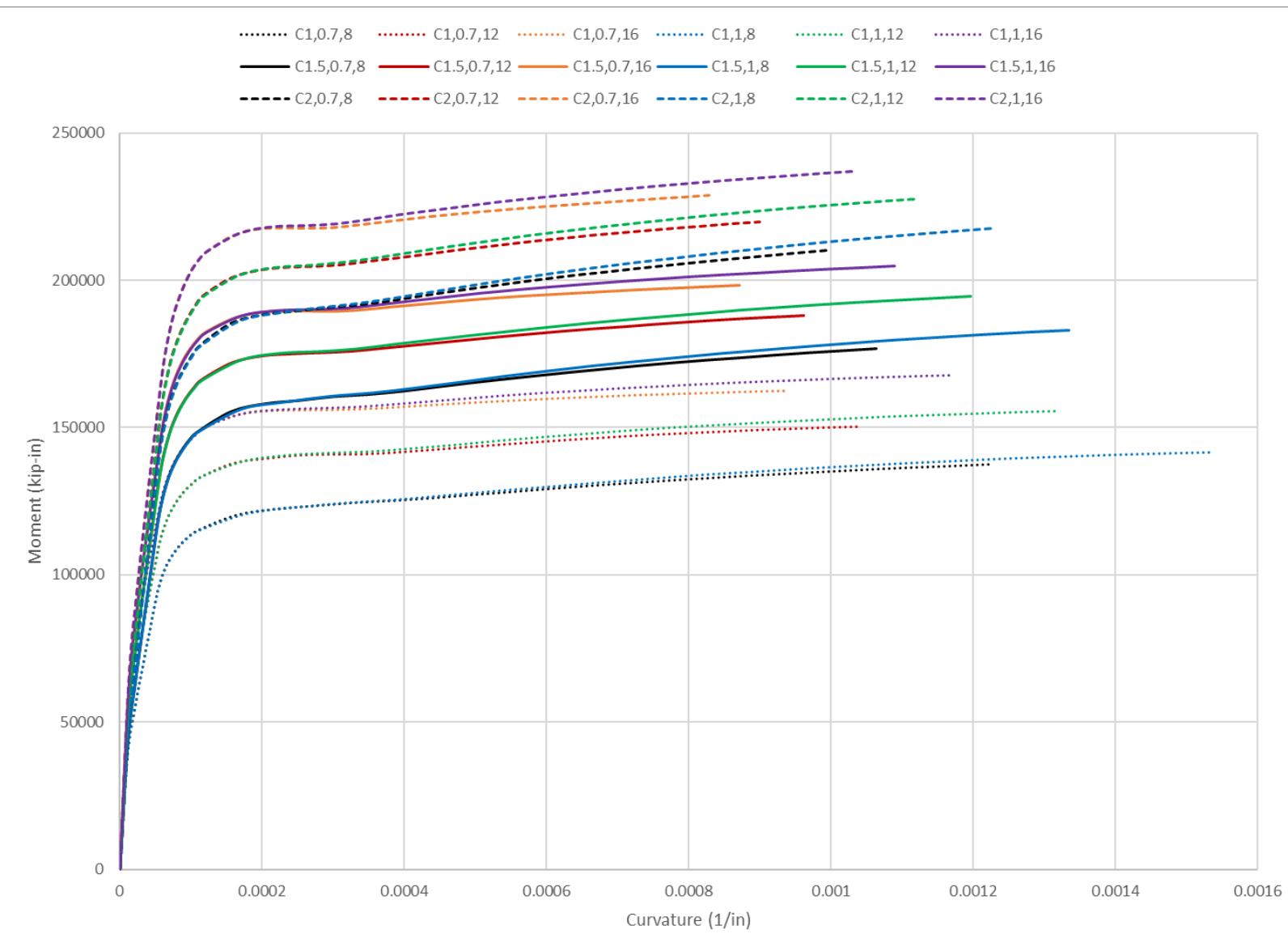


Figure A-7. Sectional moment-curvature relationships (plotted to ϵ_{cu}).

Table A-7. Ground motions at selected cities (100-year return period).

City	Location		B/C Firm-Ground Motions (g)				Site Factors			Site-Adjusted Ground Motions (g)			
	Latitude	Longitude	PGA	S _s	S ₁	S ₂	F _{pga}	F _a	F _v	PGA _d	S _{ds}	S _{d1}	S _{d2}
Charleston	34.8497	-79.9311	0.012	0.21	0.0081	0.0034	1.6	1.6	2.4	0.02	0.33	0.02	0.01
New York City	41.0332	-73.6727	0.011	0.022	0.0063	0.0025	1.6	1.6	2.4	0.02	0.04	0.02	0.01
Boston	42.3638	-71.0594	0.013	0.029	0.0072	0.0028	1.6	1.6	2.4	0.02	0.05	0.02	0.01
Knoxville	35.9631	-83.9188	0.035	0.065	0.014	0.0056	1.6	1.6	2.4	0.06	0.10	0.03	0.01
Paducah	37.083	-88.6027	0.047	0.088	0.016	0.0062	1.6	1.6	2.4	0.08	0.14	0.04	0.01
St Louis	38.6272	-90.1866	0.025	0.050	0.011	0.0043	1.6	1.6	2.4	0.04	0.08	0.03	0.01
Helena	46.5894	-112.0391	0.032	0.069	0.022	0.0099	1.6	1.6	2.4	0.05	0.11	0.05	0.02
Salt Lake City	40.7534	-111.8892	0.049	0.11	0.032	0.015	1.6	1.6	2.4	0.08	0.17	0.08	0.04
Los Angeles	34.0542	-118.2532	0.19	0.41	0.11	0.047	1.43	1.47	2.36	0.27	0.60	0.26	0.11
San Francisco	37.7733	-122.4173	0.202	0.43	0.12	0.056	1.41	1.45	2.31	0.28	0.63	0.28	0.13
Seattle	47.6083	-122.3316	0.13	0.27	0.068	0.028	1.57	1.58	2.4	0.20	0.43	0.16	0.07
Juneau	58.3066	-134.4191	0.048	0.10	0.051	0.028	1.6	1.6	2.4	0.08	0.16	0.12	0.07
New Madrid	36.5856	-89.5251	0.077	0.13	0.020	0.0071	1.6	1.6	2.4	0.12	0.21	0.05	0.02

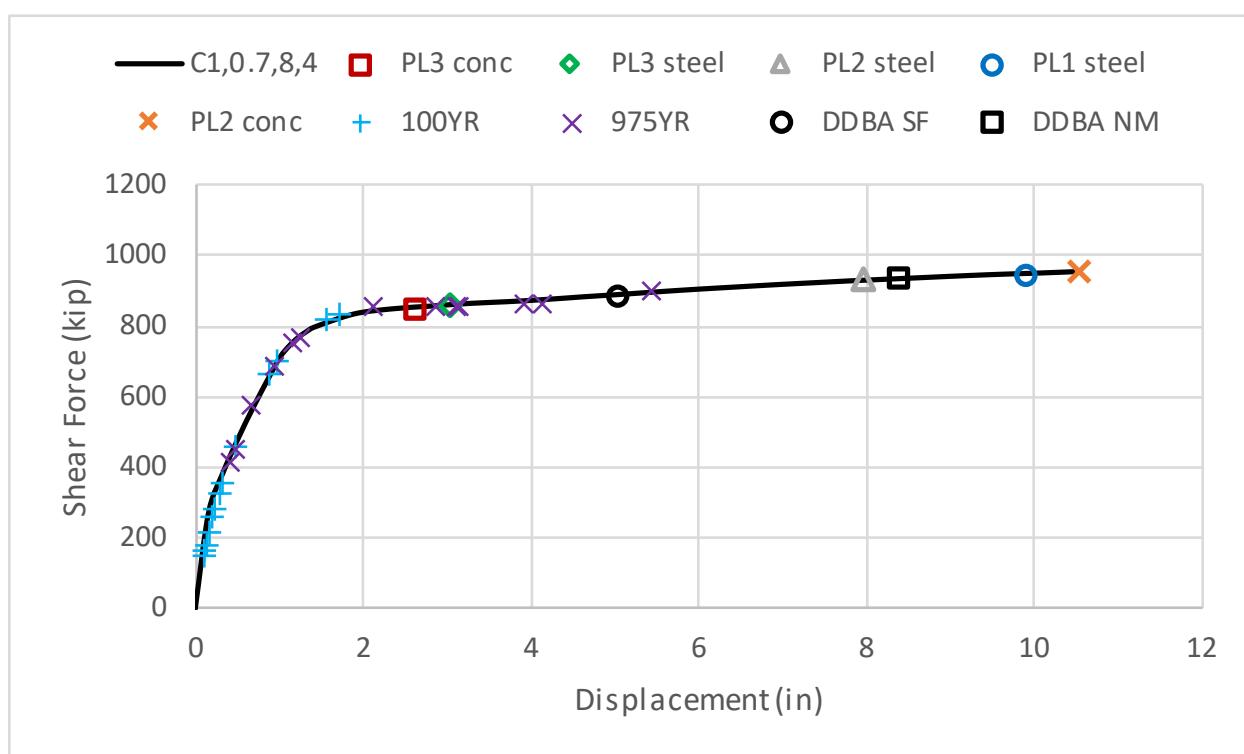
A42

Table A-8. Ground motions at selected cities (975-year return period).

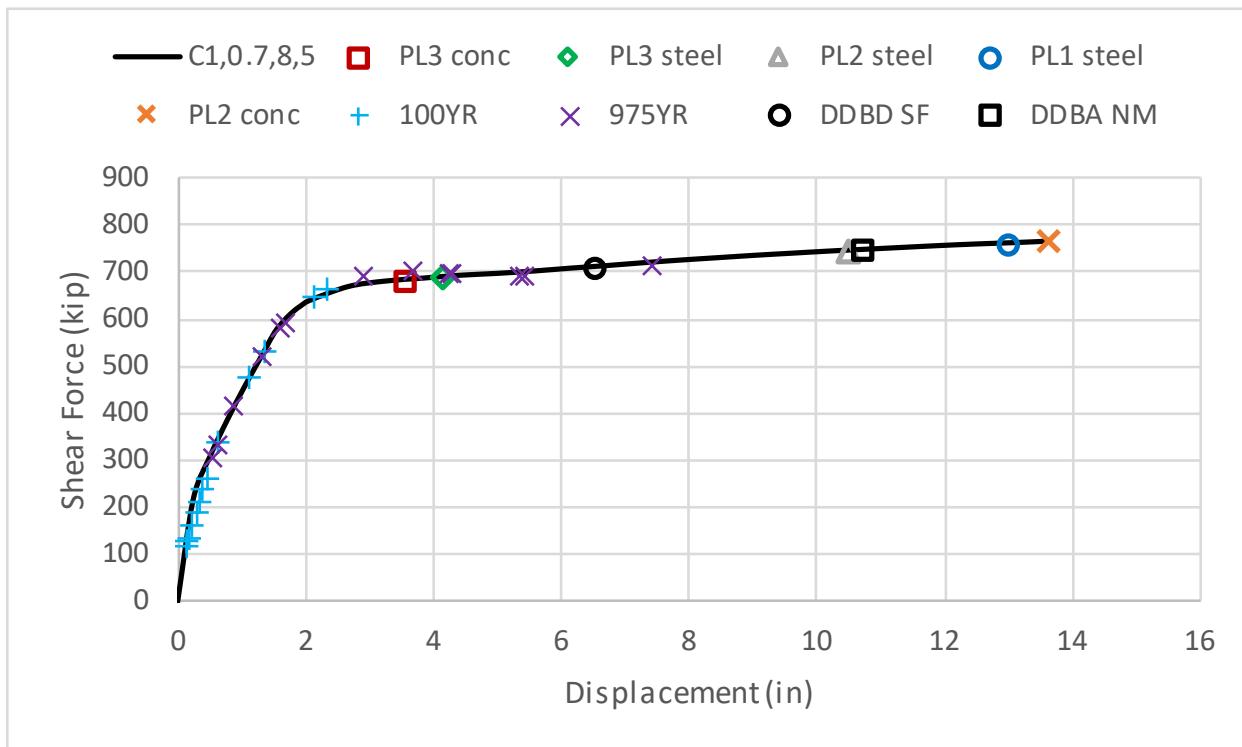
City	Location		B/C Firm-Ground Motions (g)				Site Factors			Site-Adjusted Ground Motions (g)			
	Latitude	Longitude	PGA	S _s	S ₁	S ₂	F _{pga}	F _a	F _v	PGA _d	S _{ds}	S _{d1}	S _{d2}
Charleston	34.8497	-79.9311	0.065	0.13	0.045	0.024	1.6	1.6	2.4	0.10	0.21	0.11	0.06
New York City	41.0332	-73.6727	0.081	0.14	0.028	0.013	1.6	1.6	2.4	0.13	0.22	0.07	0.03
Boston	42.3638	-71.0594	0.08	0.14	0.032	0.015	1.6	1.6	2.4	0.13	0.23	0.08	0.04
Knoxville	35.9631	-83.9188	0.23	0.35	0.066	0.032	1.36	1.52	2.4	0.31	0.53	0.16	0.08
Paducah	37.083	-88.6027	0.37	0.65	0.17	0.079	1.13	1.28	2.13	0.42	0.83	0.35	0.17
St Louis	38.6272	-90.1866	0.15	0.28	0.086	0.043	1.6	1.58	2.4	0.24	0.44	0.21	0.10
Helena	46.5894	-112.0391	0.13	0.30	0.08	0.036	1.58	1.56	2.4	0.21	0.47	0.19	0.09
Salt Lake City	40.7534	-111.8892	0.43	1.00	0.29	0.12	1.06	1.11	1.82	0.46	1.10	0.52	0.22
Los Angeles	34.0542	-118.2532	0.60	1.41	0.41	0.18	1	1	1.59	0.60	1.41	0.65	0.28
San Francisco	37.7733	-122.4173	0.56	1.28	0.42	0.21	1	1	1.58	0.56	1.28	0.66	0.32
Seattle	47.6083	-122.3316	0.43	0.97	0.28	0.13	1.07	1.11	1.83	0.46	1.08	0.52	0.25
Juneau	58.3066	-134.4191	0.14	0.35	0.20	0.12	1.51	1.52	2	0.22	0.52	0.40	0.24
New Madrid	36.5856	-89.5251	1.29	2.24	0.60	0.28	1	1	1.5	1.29	2.24	0.90	0.42

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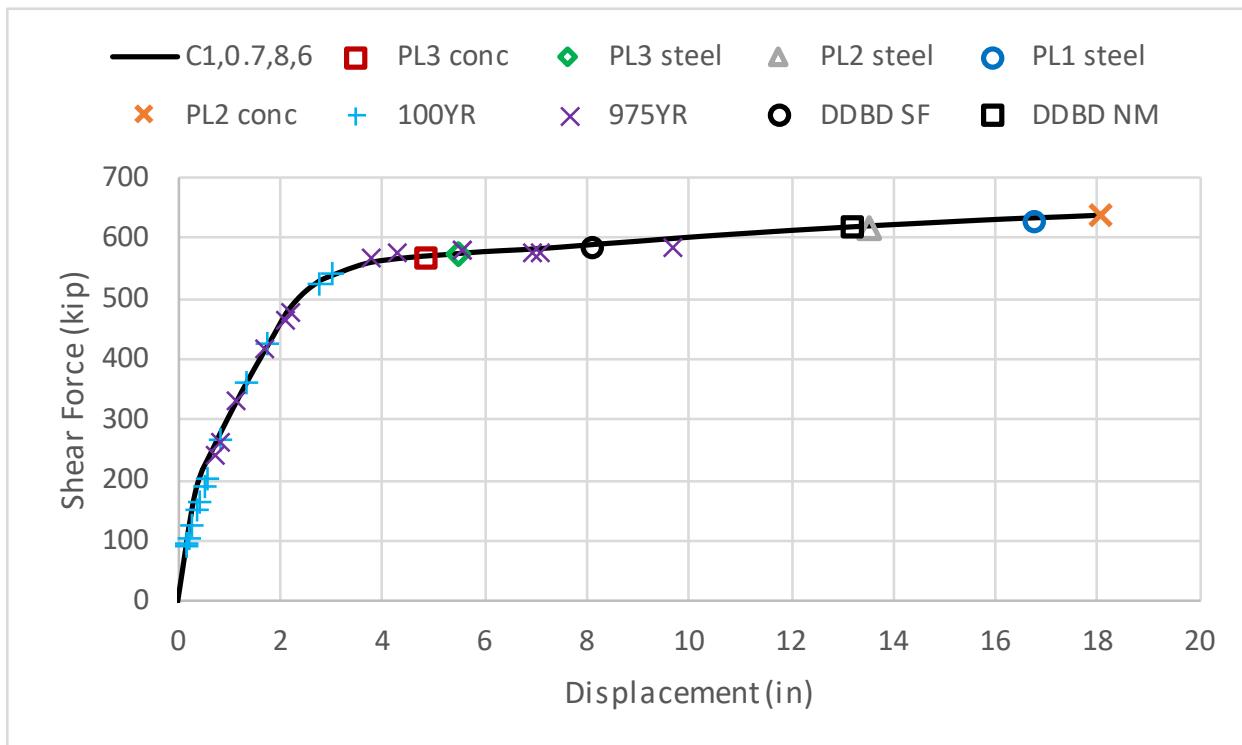
C1,0.7,8,4							
A _{st} /A _g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 5 in					
P/f' _{ce} A _g	0.08	New Madrid 8.35 in					
L/D	4						
Strain Limits				(g)	(in)	(g)	(in)
				S _a		S _a	
		City		100YR	△ 100YR	975YR	△ 975YR
ε _{c.cc}	-0.004	PL3 conc	Charleston	0.032	0.1	0.175	0.7
ε _{s.rcw}	0.01	PL3 steel	New York City	0.025	0.1	0.110	0.4
ε _{s.bb}	0.0346	PL1 steel	Boston	0.028	0.1	0.125	0.5
ε _{c.u}	-0.0166	PL2 conc	Knoxville	0.056	0.2	0.257	0.9
0.8*ε _{s.bb}	0.0277	PL2 steel	Paducah	0.063	0.2	0.578	2.1
			St Louis	0.043	0.2	0.336	1.2
Limit State Disp. (in)				Helena	0.087	0.3	0.317
Δ @ ε _{c.cc}	2.6	PL3 conc	Salt Lake City	0.126	0.5	0.853	3.1
Δ @ ε _{s.rcw}	3.0	PL3 steel	Los Angeles	0.423	1.6	1.064	3.9
Δ @ ε _{s.bb}	9.9	PL1 steel	San Francisco	0.462	1.7	1.080	4.1
Δ @ ε _{c.u}	10.5	PL2 conc	Seattle	0.267	1.0	0.846	3.1
Δ @ 0.8*ε _{s.bb}	8.0	PL2 steel	Juneau	0.164	0.9	0.524	2.9
			New Madrid	0.078	0.3	1.477	5.4



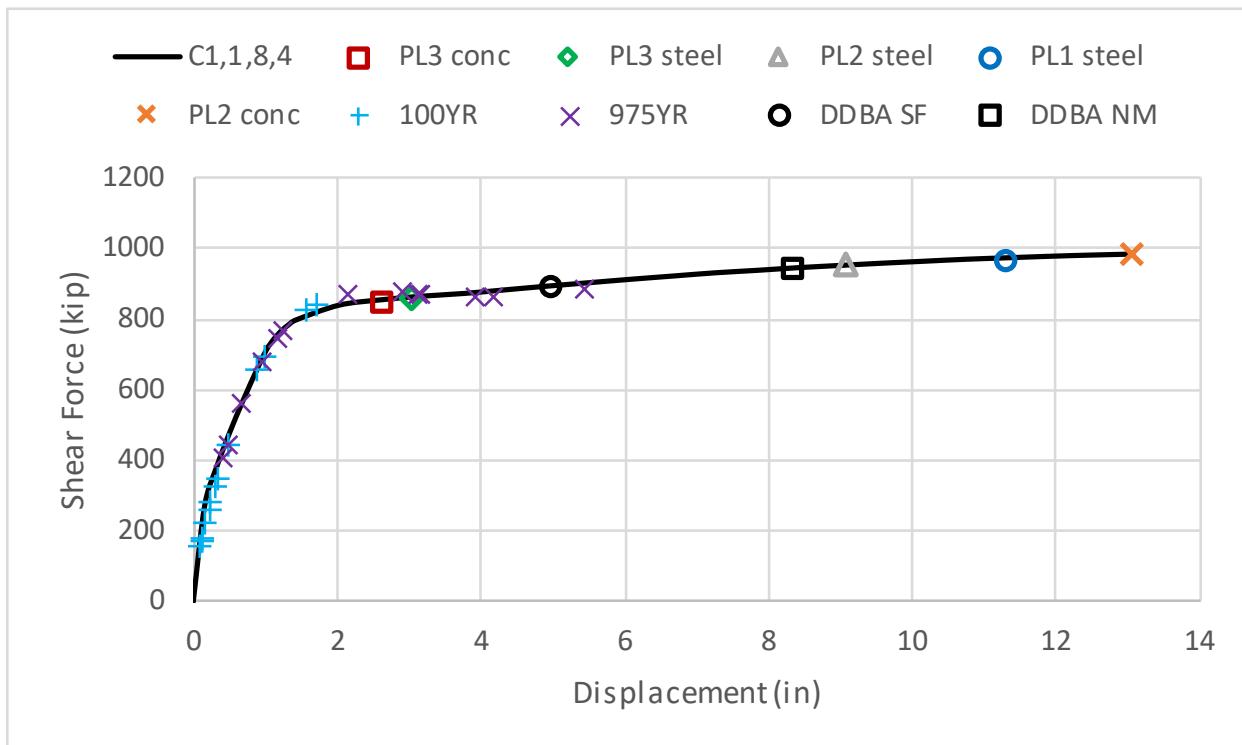
C1,0.7,8,5								
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ						
$4A_h/D's$	0.007	San Francisco 6.5 in						
$P/f'_{ce}A_g$	0.08	New Madrid 10.7 in						
L/D	5							
Strain Limits			City	(g)	(in)	(g)	(in)	
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.023	0.2	0.128	0.9	
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.018	0.1	0.080	0.6	
$\epsilon_{s,bb}$	0.0346	PL1 steel	Boston	0.021	0.1	0.091	0.6	
$\epsilon_{c,u}$	-0.0166	PL2 conc	Knoxville	0.041	0.3	0.187	1.3	
$0.8^*\epsilon_{s,bb}$	0.0277	PL2 steel	Paducah	0.046	0.3	0.421	2.9	
			St Louis	0.032	0.2	0.245	1.7	
Limit State Disp. (in)				Helena	0.063	0.4	0.231	1.6
$\Delta @ \epsilon_{c,cc}$	3.5	PL3 conc	Salt Lake City	0.092	0.6	0.622	4.3	
$\Delta @ \epsilon_{s,rcw}$	4.1	PL3 steel	Los Angeles	0.308	2.1	0.775	5.4	
$\Delta @ \epsilon_{s,bb}$	13.0	PL1 steel	San Francisco	0.337	2.3	0.787	5.4	
$\Delta @ \epsilon_{c,u}$	13.6	PL2 conc	Seattle	0.195	1.3	0.616	4.3	
$\Delta @ 0.8^*\epsilon_{s,bb}$	10.5	PL2 steel	Juneau	0.146	1.1	0.475	3.7	
			New Madrid	0.057	0.4	1.076	7.4	



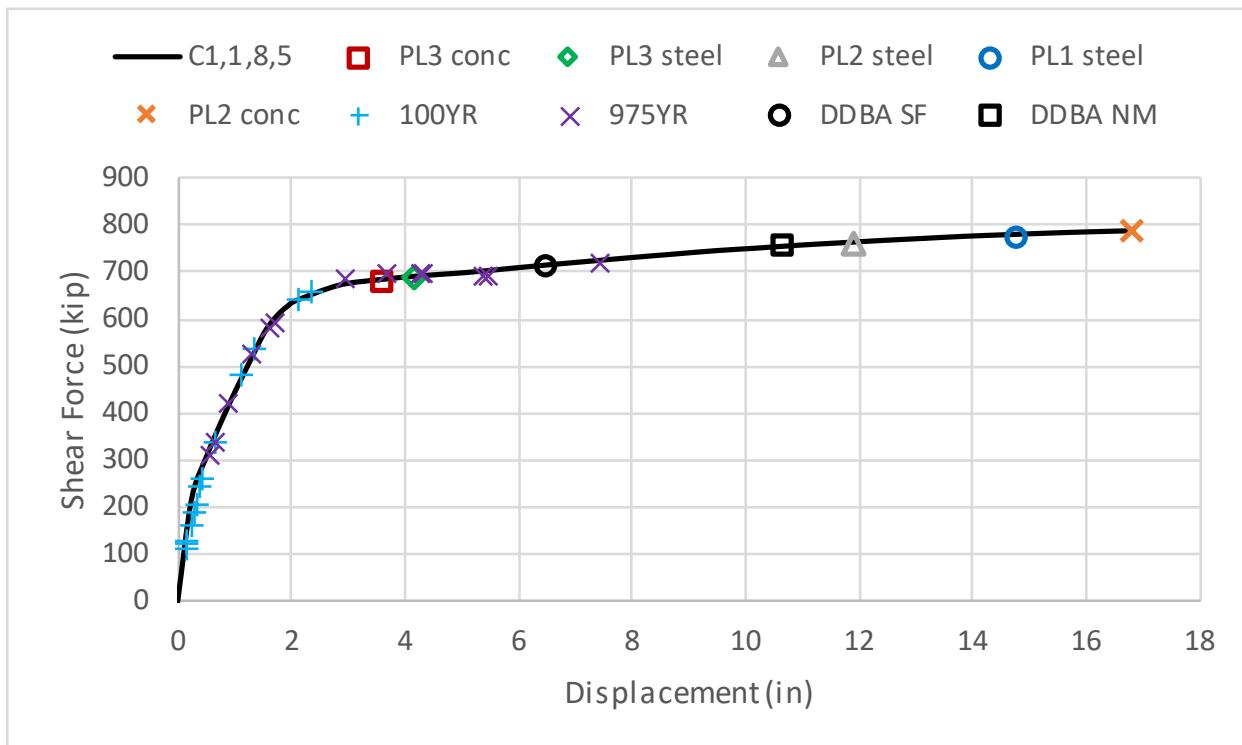
C1,0.7,8,6							
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.007	San Francisco 8.1 in					
$P/f'_{ce}A_g$	0.08	New Madrid 13.15 in					
L/D	6						
Strain Limits				(g)	(in)	(g)	(in)
		City		S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.018	0.2	0.098	1.1
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.014	0.2	0.062	0.7
$\epsilon_{s,bb}$	0.0346	PL1 steel	Boston	0.016	0.2	0.070	0.8
$\epsilon_{c,u}$	-0.0166	PL2 conc	Knoxville	0.031	0.4	0.144	1.7
$0.8^*\epsilon_{s,bb}$	0.0277	PL2 steel	Paducah	0.035	0.4	0.324	3.8
			St Louis	0.024	0.3	0.189	2.2
Limit State Disp. (in)			Helena	0.049	0.6	0.178	2.1
$\Delta @ \epsilon_{c,cc}$	4.8	PL3 conc	Salt Lake City	0.071	0.8	0.479	5.6
$\Delta @ \epsilon_{s,rcw}$	5.5	PL3 steel	Los Angeles	0.237	2.8	0.597	7.0
$\Delta @ \epsilon_{s,bb}$	16.7	PL1 steel	San Francisco	0.259	3.0	0.606	7.1
$\Delta @ \epsilon_{c,u}$	18.0	PL2 conc	Seattle	0.150	1.7	0.474	5.5
$\Delta @ 0.8^*\epsilon_{s,bb}$	13.5	PL2 steel	Juneau	0.112	1.3	0.365	4.3
			New Madrid	0.044	0.5	0.828	9.7



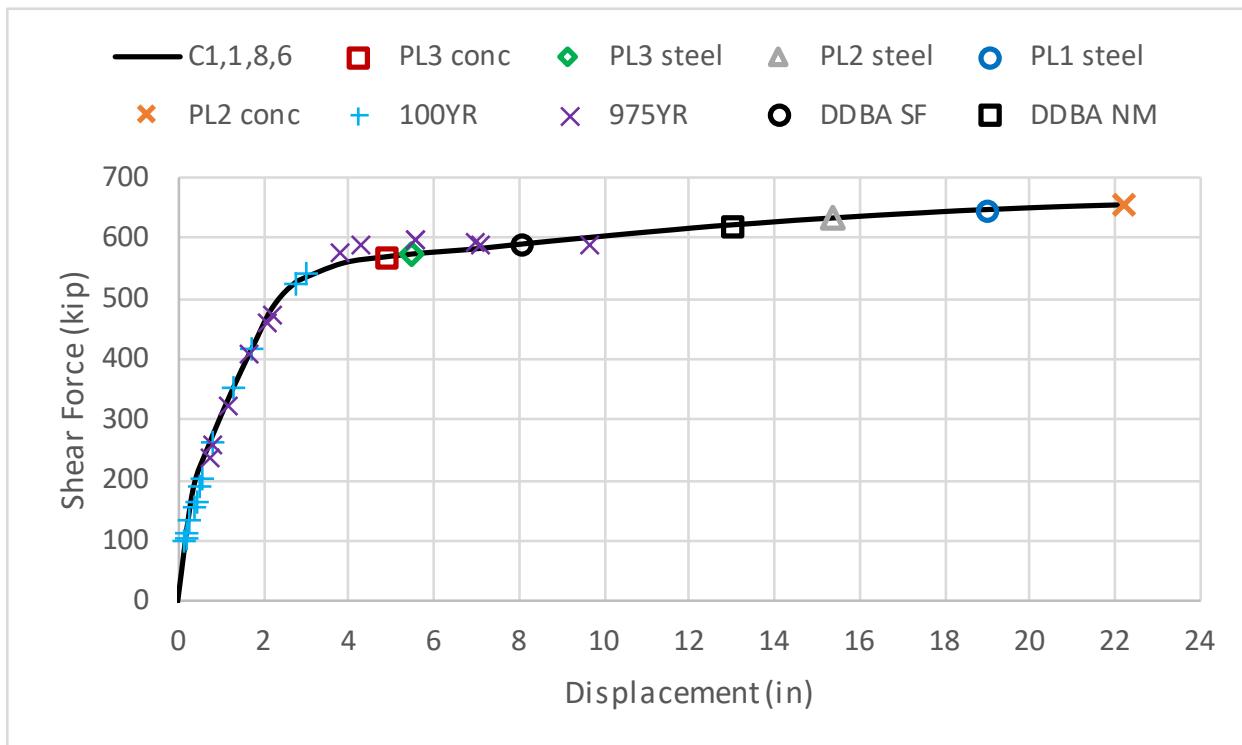
C1,1,8,4		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.01	San Francisco	4.95	in			
4A _h /D's	0.01	New Madrid	8.3	in			
P/f' _{ce} A _g	0.08						
L/D	4						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.032	0.1	0.175	0.7
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.025	0.1	0.110	0.4
$\varepsilon_{s,bb}$	0.0397	PL1 steel	Boston	0.028	0.1	0.125	0.5
$\varepsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.056	0.2	0.256	0.9
0.8* $\varepsilon_{s,bb}$	0.0317	PL2 steel	Paducah	0.063	0.2	0.577	2.1
			St Louis	0.043	0.2	0.336	1.2
Limit State Disp. (in)		Helena	0.087	0.3	0.317	1.2	
$\Delta @ \varepsilon_{c,cc}$	2.6	PL3 conc	Salt Lake City	0.126	0.5	0.851	3.1
$\Delta @ \varepsilon_{s,rcw}$	3.0	PL3 steel	Los Angeles	0.422	1.6	1.061	3.9
$\Delta @ \varepsilon_{s,bb}$	11.3	PL1 steel	San Francisco	0.461	1.7	1.078	4.1
$\Delta @ \varepsilon_{c,u}$	13.1	PL2 conc	Seattle	0.267	1.0	0.844	3.1
$\Delta @ 0.8*\varepsilon_{s,bb}$	9.1	PL2 steel	Juneau	0.164	0.9	0.524	2.9
			New Madrid	0.078	0.3	1.474	5.4



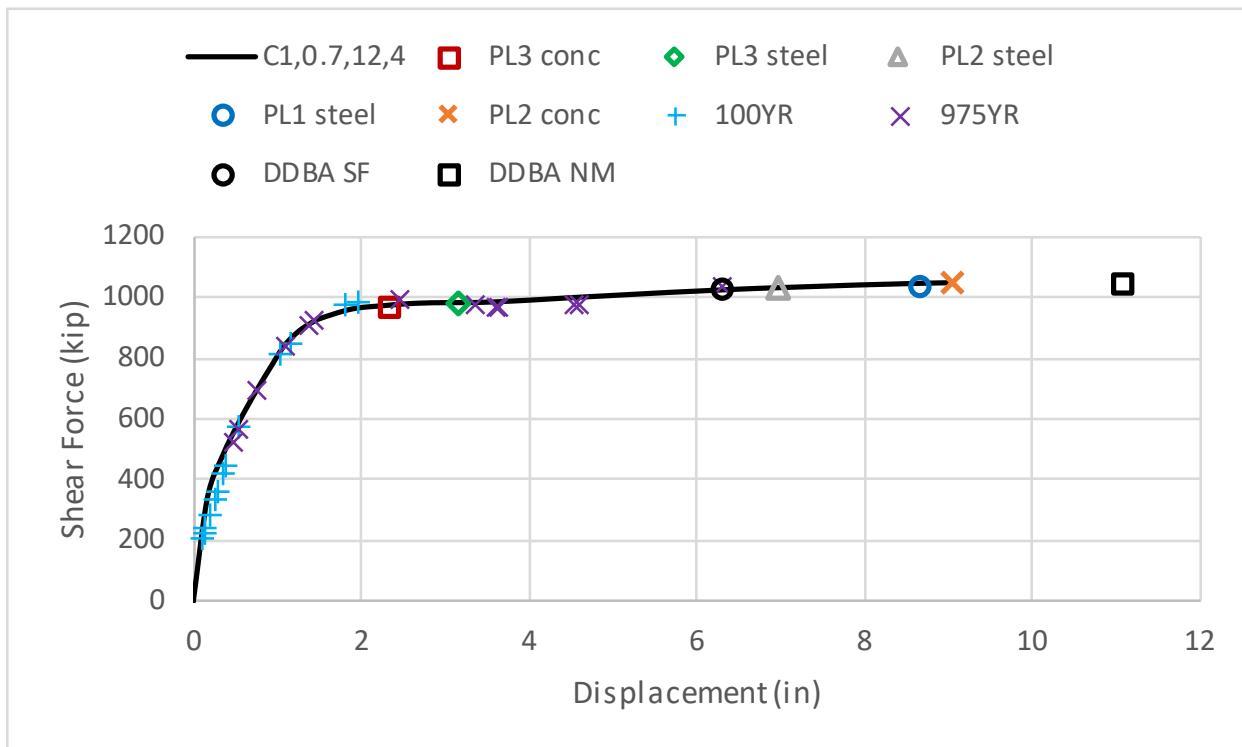
C1,1,8,5		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.01	San Francisco	6.45	in			
4A _h /D's	0.01	New Madrid	10.6	in			
P/f' _{ce} A _g	0.08						
L/D	5						
Strain Limits		City	S _a 100YR	(g)	(in)	S _a 975YR	(g)
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.023	0.2	0.127	0.9
ε _{s,rcw}	0.01	PL3 steel	New York City	0.018	0.1	0.080	0.6
ε _{s,bb}	0.0397	PL1 steel	Boston	0.021	0.1	0.091	0.6
ε _{c,u}	-0.0201	PL2 conc	Knoxville	0.040	0.3	0.187	1.3
0.8*ε _{s,bb}	0.0317	PL2 steel	Paducah	0.046	0.3	0.420	2.9
			St Louis	0.032	0.2	0.244	1.7
Limit State Disp. (in)		Helena	0.063	0.4	0.231	1.6	
Δ @ ε _{c,cc}	3.6	PL3 conc	Salt Lake City	0.092	0.6	0.620	4.3
Δ @ ε _{s,rcw}	4.2	PL3 steel	Los Angeles	0.308	2.1	0.773	5.4
Δ @ ε _{s,bb}	14.8	PL1 steel	San Francisco	0.336	2.3	0.785	5.5
Δ @ ε _{c,u}	16.8	PL2 conc	Seattle	0.194	1.3	0.615	4.3
Δ @ 0.8*ε _{s,bb}	11.9	PL2 steel	Juneau	0.146	1.1	0.474	3.7
			New Madrid	0.057	0.4	1.074	7.5



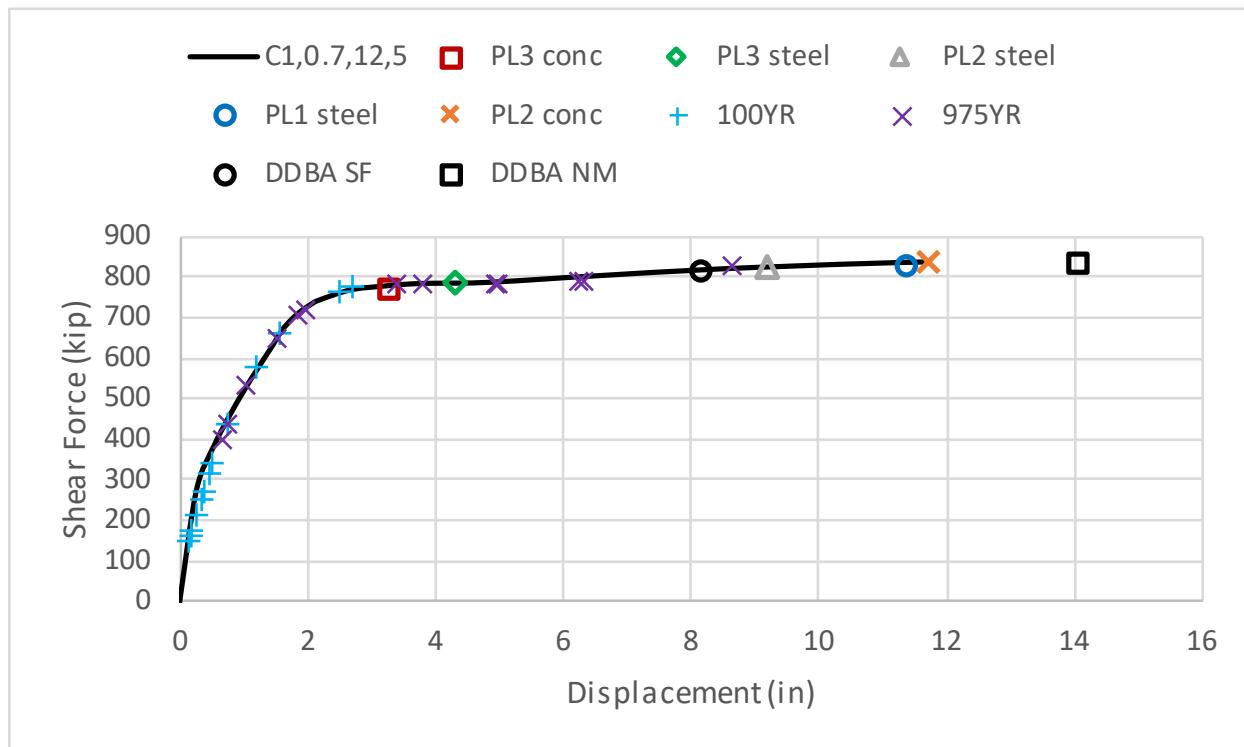
C1,1,8,6		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.01	San Francisco		8.05	in		
4A _h /D's	0.01	New Madrid		13	in		
P/f' _{ce} A _g	0.08						
L/D	6						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.018	0.2	0.098	1.1
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.014	0.2	0.062	0.7
$\varepsilon_{s,bb}$	0.0397	PL1 steel	Boston	0.016	0.2	0.070	0.8
$\varepsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.031	0.4	0.144	1.7
$0.8^*\varepsilon_{s,bb}$	0.0317	PL2 steel	Paducah	0.035	0.4	0.324	3.8
			St Louis	0.024	0.3	0.188	2.2
Limit State Disp. (in)			Helena	0.049	0.6	0.178	2.1
$\Delta @ \varepsilon_{c,cc}$	4.8	PL3 conc	Salt Lake City	0.071	0.8	0.478	5.6
$\Delta @ \varepsilon_{s,rcw}$	5.5	PL3 steel	Los Angeles	0.237	2.8	0.595	7.0
$\Delta @ \varepsilon_{s,bb}$	19.0	PL1 steel	San Francisco	0.259	3.0	0.605	7.1
$\Delta @ \varepsilon_{c,u}$	22.2	PL2 conc	Seattle	0.150	1.8	0.474	5.5
$\Delta @ 0.8^*\varepsilon_{s,bb}$	15.4	PL2 steel	Juneau	0.112	1.3	0.365	4.3
			New Madrid	0.044	0.5	0.827	9.7



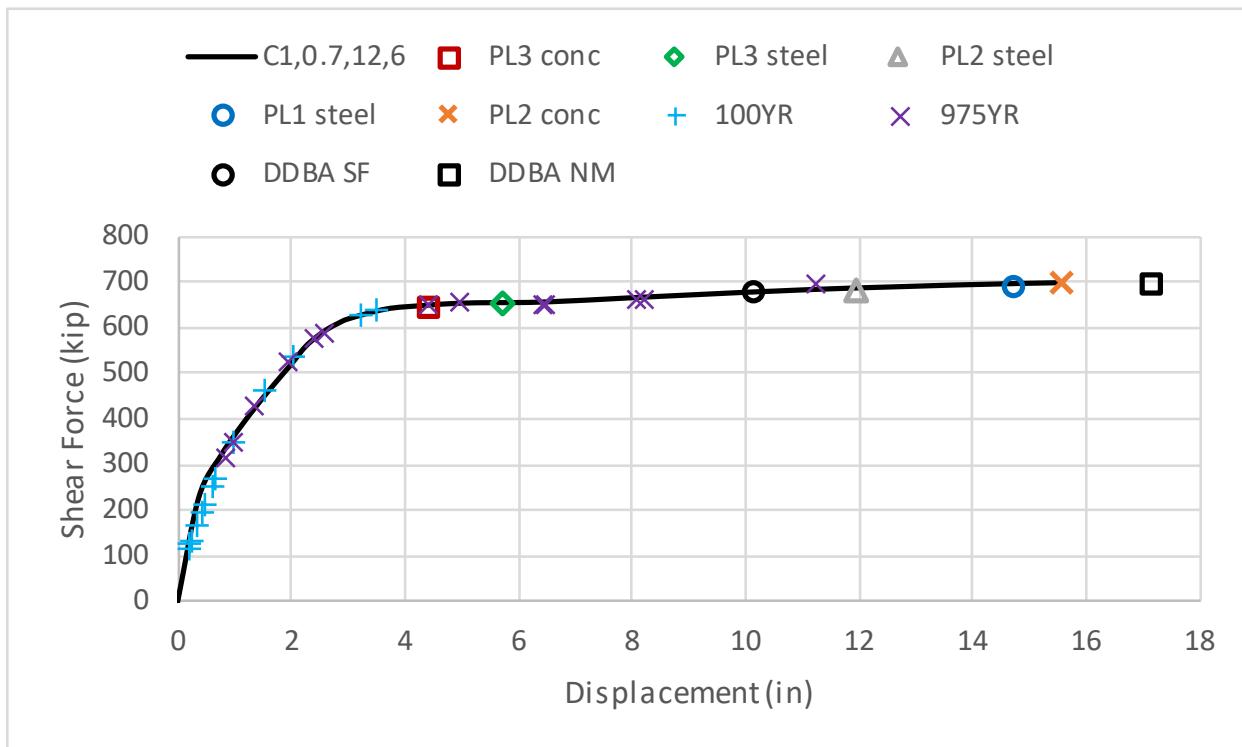
C1,0.7,12,4							
A _{st} /A _g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 6.3 in					
P/f' _{ce} A _g	0.12	New Madrid 11.05 in					
L/D	4						
Strain Limits		City		(g)	(in)	(g)	(in)
$\varepsilon_{c,cc}$		Charleston		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
$\varepsilon_{s,rcw}$		New York City		0.027	0.1	0.151	0.7
$\varepsilon_{s,bb}$		Boston		0.021	0.1	0.095	0.5
$\varepsilon_{c,u}$		Knoxville		0.024	0.1	0.108	0.5
-0.0166		Paducah		0.048	0.2	0.221	1.1
0.8* $\varepsilon_{s,bb}$		St Louis		0.054	0.3	0.498	2.5
		Helena		0.037	0.2	0.289	1.4
Limit State Disp. (in)		Salt Lake City		0.075	0.4	0.273	1.4
$\Delta @ \varepsilon_{c,cc}$		Los Angeles		0.109	0.5	0.734	3.6
$\Delta @ \varepsilon_{s,rcw}$		San Francisco		0.364	1.8	0.915	4.5
$\Delta @ \varepsilon_{s,bb}$		Seattle		0.398	2.0	0.930	4.6
$\Delta @ \varepsilon_{c,u}$		Juneau		0.230	1.1	0.728	3.6
$\Delta @ 0.8 * \varepsilon_{s,bb}$		New Madrid		0.164	1.0	0.524	3.4
				0.067	0.3	1.271	6.3



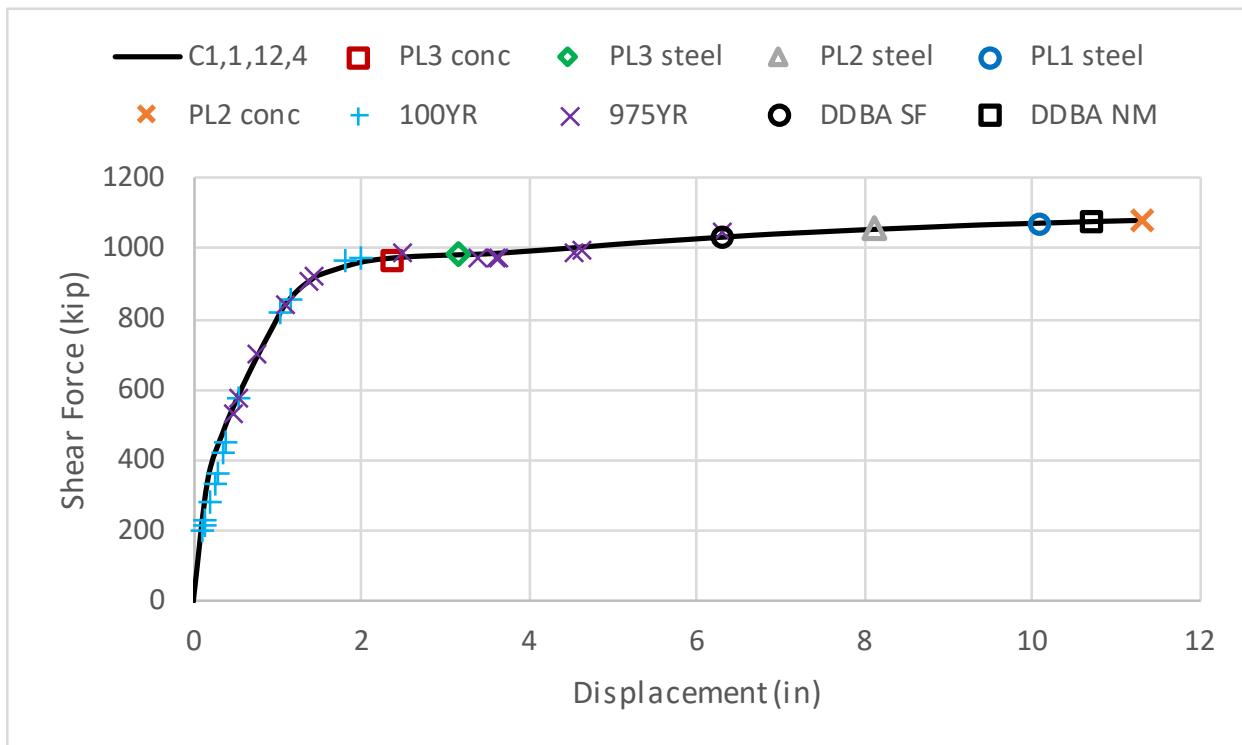
C1,0.7,12,5		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.01	San Francisco	8.15	in			
4A _h /D's	0.007	New Madrid	14.05	in			
P/f' _{ce} A _g	0.12						
L/D	5						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.020	0.2	0.110	1.0
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.015	0.1	0.069	0.6
$\epsilon_{s,bb}$	0.0290	PL1 steel	Boston	0.018	0.2	0.079	0.7
$\epsilon_{c,u}$	-0.0166	PL2 conc	Knoxville	0.035	0.3	0.161	1.5
0.8* $\epsilon_{s,bb}$	0.0232	PL2 steel	Paducah	0.040	0.4	0.363	3.4
			St Louis	0.027	0.3	0.211	2.0
Limit State Disp. (in)		Helena	0.055	0.5	0.199	1.9	
$\Delta @ \epsilon_{c,cc}$	3.2	PL3 conc	Salt Lake City	0.079	0.7	0.535	5.0
$\Delta @ \epsilon_{s,rcw}$	4.3	PL3 steel	Los Angeles	0.265	2.5	0.667	6.2
$\Delta @ \epsilon_{s,bb}$	11.4	PL1 steel	San Francisco	0.290	2.7	0.677	6.3
$\Delta @ \epsilon_{c,u}$	11.7	PL2 conc	Seattle	0.168	1.6	0.530	4.9
$\Delta @ 0.8*\epsilon_{s,bb}$	9.2	PL2 steel	Juneau	0.126	1.2	0.408	3.8
			New Madrid	0.049	0.5	0.926	8.6



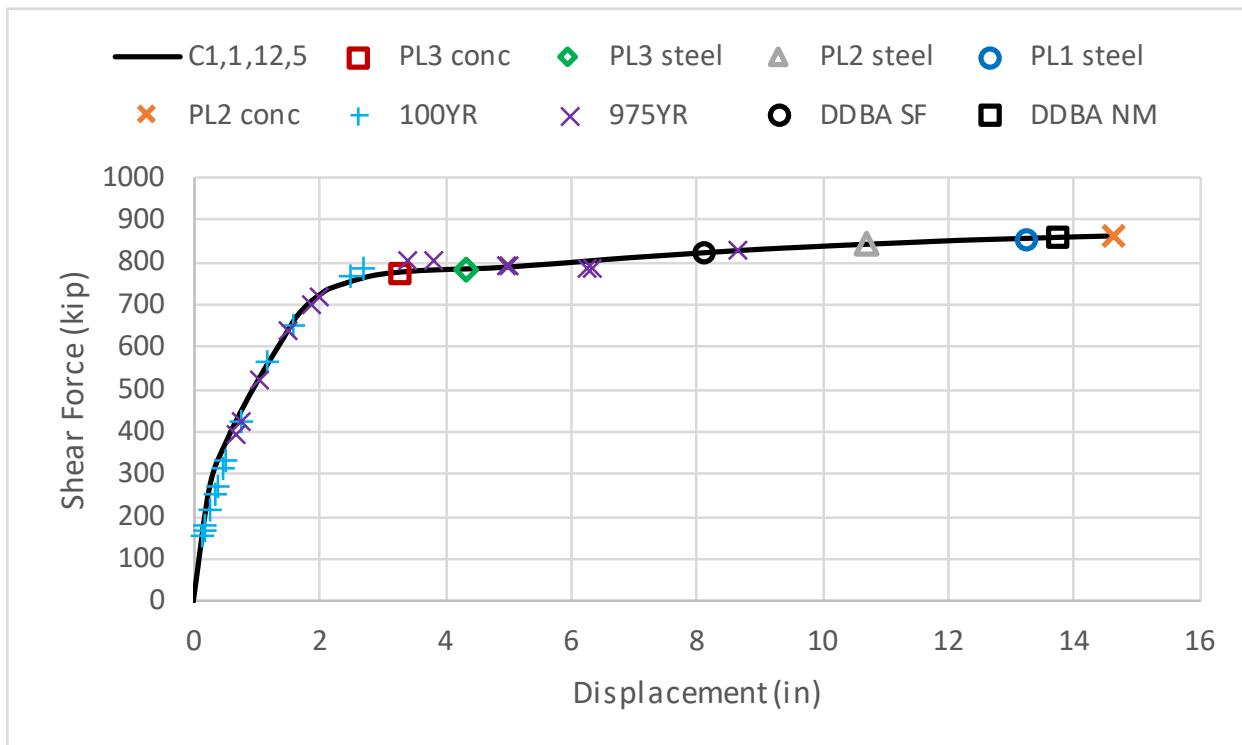
C1,0.7,12,6		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.01	San Francisco	10.1	in			
4A _h /D's	0.007	New Madrid	17.1	in			
P/f' _{ce} A _g	0.12						
L/D	6						
Strain Limits		City	S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR	
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.015	0.2	0.085	1.3
ε _{s,rcw}	0.01	PL3 steel	New York City	0.012	0.2	0.053	0.8
ε _{s,bb}	0.0290	PL1 steel	Boston	0.014	0.2	0.061	1.0
ε _{c,u}	-0.0166	PL2 conc	Knoxville	0.027	0.4	0.124	2.0
0.8*ε _{s,bb}	0.0232	PL2 steel	Paducah	0.030	0.5	0.279	4.4
			St Louis	0.021	0.3	0.162	2.6
Limit State Disp. (in)		Helena	0.042	0.7	0.153	2.4	
Δ @ ε _{c,cc}	4.4	PL3 conc	Salt Lake City	0.061	1.0	0.412	6.5
Δ @ ε _{s,rcw}	5.7	PL3 steel	Los Angeles	0.204	3.2	0.513	8.1
Δ @ ε _{s,bb}	14.7	PL1 steel	San Francisco	0.223	3.5	0.521	8.2
Δ @ ε _{c,u}	15.5	PL2 conc	Seattle	0.129	2.0	0.408	6.4
Δ @ 0.8*ε _{s,bb}	11.9	PL2 steel	Juneau	0.097	1.5	0.314	5.0
			New Madrid	0.038	0.6	0.713	11.2



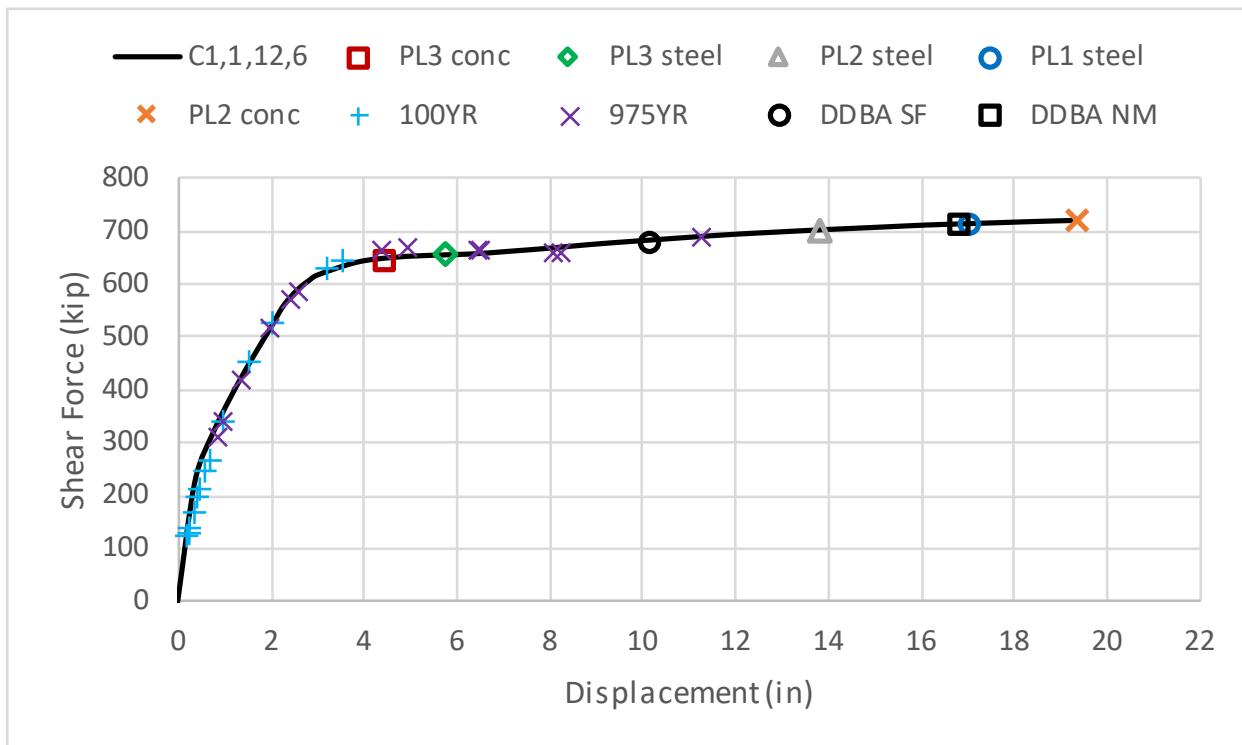
C1,1,12,4							
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 6.27 in					
$P/f'_{ce}A_g$	0.12	New Madrid 10.7 in					
L/D	4						
Strain Limits				City	(g)	(in)	(g)
					S_a 100YR	Δ 100YR	S_a 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.027	0.1	0.150	0.7
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.021	0.1	0.095	0.5
$\epsilon_{s,bb}$	0.0341	PL1 steel	Boston	0.024	0.1	0.108	0.5
$\epsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.048	0.2	0.220	1.1
$0.8^*\epsilon_{s,bb}$	0.0272	PL2 steel	Paducah	0.054	0.3	0.497	2.5
			St Louis	0.037	0.2	0.289	1.4
Limit State Disp. (in)				Helena	0.075	0.4	0.273
$\Delta @ \epsilon_{c,cc}$	2.3	PL3 conc	Salt Lake City	0.108	0.5	0.733	3.6
$\Delta @ \epsilon_{s,rcw}$	3.2	PL3 steel	Los Angeles	0.363	1.8	0.914	4.5
$\Delta @ \epsilon_{s,bb}$	10.1	PL1 steel	San Francisco	0.397	2.0	0.928	4.6
$\Delta @ \epsilon_{c,u}$	11.3	PL2 conc	Seattle	0.230	1.1	0.727	3.6
$\Delta @ 0.8^*\epsilon_{s,bb}$	8.1	PL2 steel	Juneau	0.164	1.0	0.524	3.4
			New Madrid	0.067	0.3	1.268	6.3



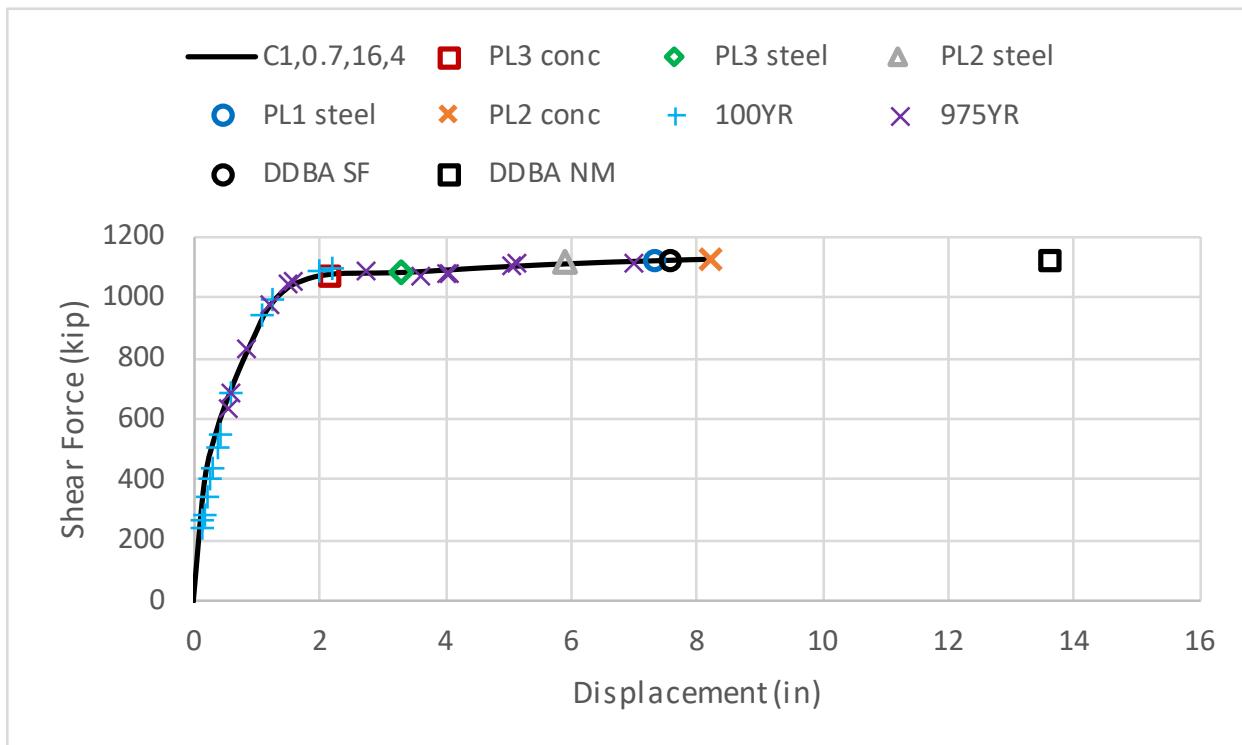
C1,1,12,5							
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 8.1 in					
$P/f'_{ce}A_g$	0.12	New Madrid 13.7 in					
L/D	5						
Strain Limits				City	(g)	(in)	(g)
					S_a 100YR	Δ 100YR	S_a 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.020	0.2	0.110	1.0
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.015	0.1	0.069	0.6
$\epsilon_{s,bb}$	0.0341	PL1 steel	Boston	0.018	0.2	0.078	0.7
$\epsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.035	0.3	0.161	1.5
$0.8^*\epsilon_{s,bb}$	0.0272	PL2 steel	Paducah	0.039	0.4	0.362	3.4
			St Louis	0.027	0.3	0.210	2.0
Limit State Disp. (in)				Helena	0.054	0.5	0.199
$\Delta @ \epsilon_{c,cc}$	3.2	PL3 conc	Salt Lake City	0.079	0.7	0.534	5.0
$\Delta @ \epsilon_{s,rcw}$	4.3	PL3 steel	Los Angeles	0.265	2.5	0.665	6.2
$\Delta @ \epsilon_{s,bb}$	13.2	PL1 steel	San Francisco	0.289	2.7	0.676	6.3
$\Delta @ \epsilon_{c,u}$	14.6	PL2 conc	Seattle	0.167	1.6	0.529	5.0
$\Delta @ 0.8^*\epsilon_{s,bb}$	10.7	PL2 steel	Juneau	0.125	1.2	0.408	3.8
			New Madrid	0.049	0.5	0.924	8.7



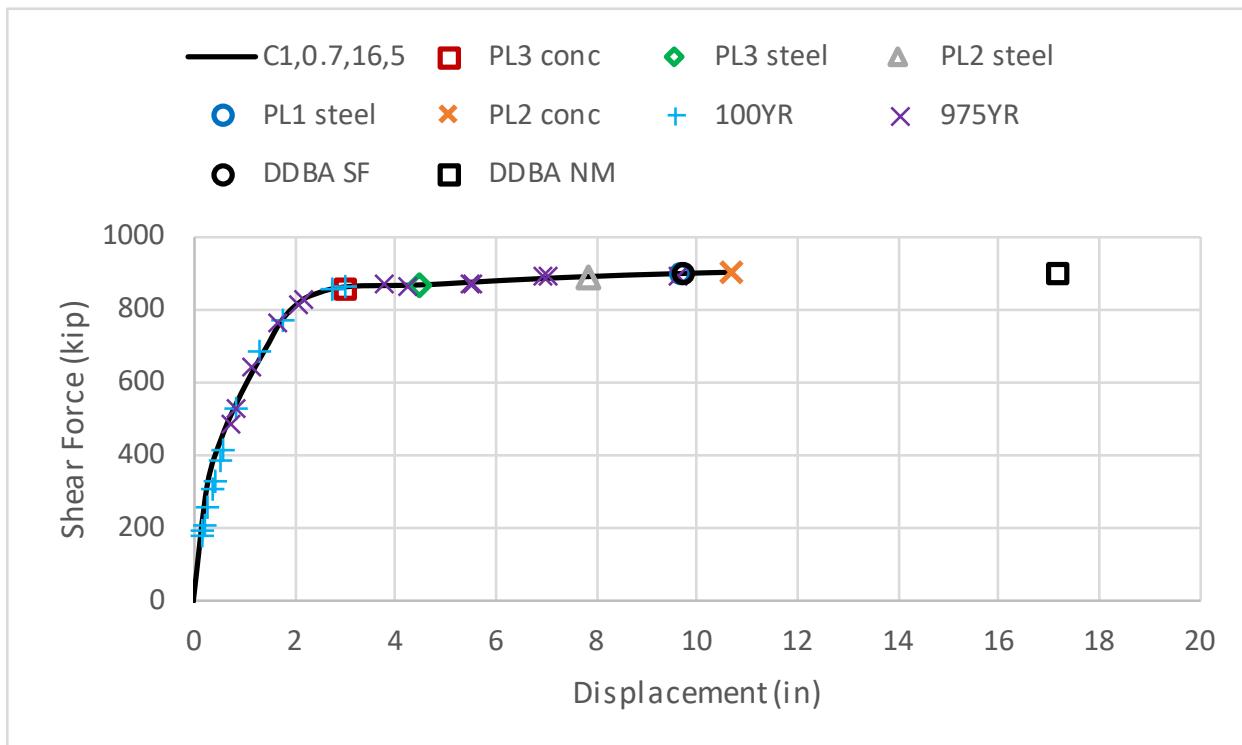
C1,1,12,6							
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 10.1 in					
$P/f'_{ce}A_g$	0.12	New Madrid 16.8 in					
L/D	6						
Strain Limits				(g)	(in)	(g)	(in)
		City		S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.015	0.2	0.084	1.3
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.012	0.2	0.053	0.8
$\epsilon_{s,bb}$	0.0341	PL1 steel	Boston	0.014	0.2	0.060	1.0
$\epsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.027	0.4	0.124	2.0
$0.8^*\epsilon_{s,bb}$	0.0272	PL2 steel	Paducah	0.030	0.5	0.279	4.4
			St Louis	0.021	0.3	0.162	2.6
Limit State Disp. (in)			Helena	0.042	0.7	0.153	2.4
$\Delta @ \epsilon_{c,cc}$	4.4	PL3 conc	Salt Lake City	0.061	1.0	0.411	6.5
$\Delta @ \epsilon_{s,rcw}$	5.7	PL3 steel	Los Angeles	0.204	3.2	0.512	8.1
$\Delta @ \epsilon_{s,bb}$	17.0	PL1 steel	San Francisco	0.223	3.5	0.520	8.2
$\Delta @ \epsilon_{c,u}$	19.4	PL2 conc	Seattle	0.129	2.0	0.408	6.4
$\Delta @ 0.8^*\epsilon_{s,bb}$	13.8	PL2 steel	Juneau	0.096	1.5	0.314	5.0
			New Madrid	0.038	0.6	0.711	11.2



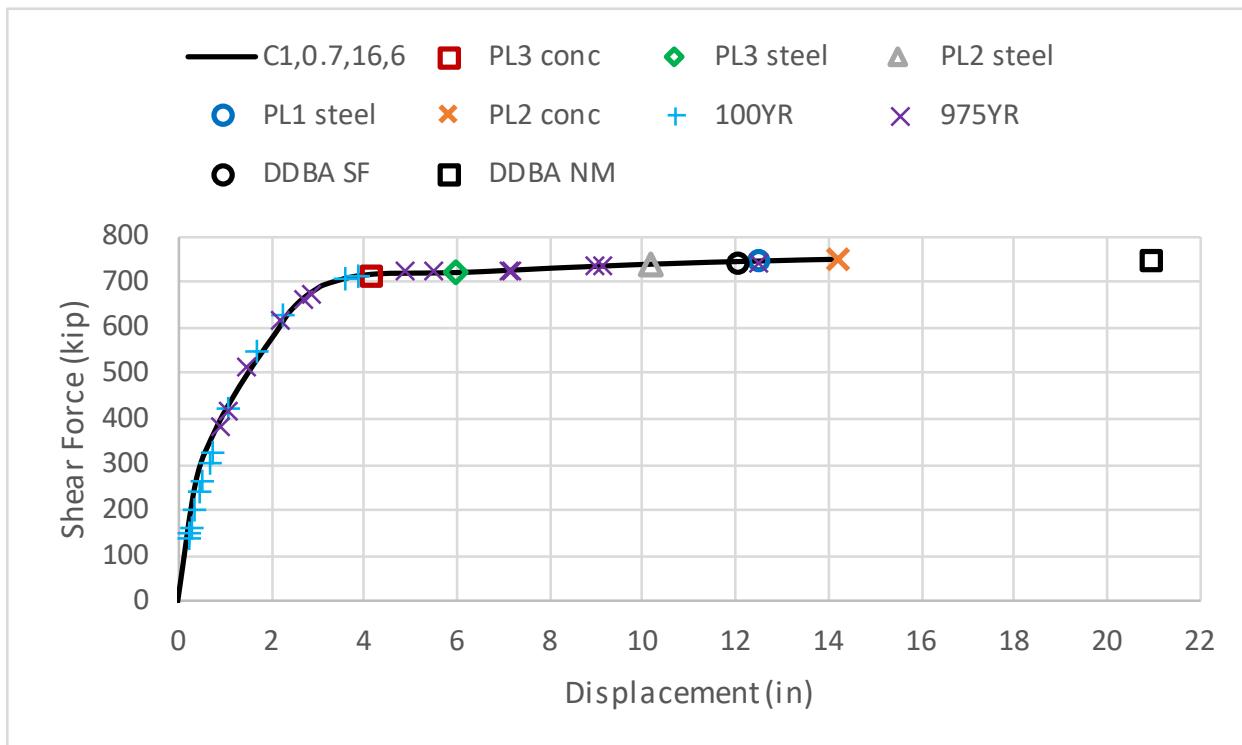
C1,0.7,16,4							
A _{st} /A _g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 7.57 in					
P/f' _{ce} A _g	0.16	New Madrid 13.6 in					
L/D	4						
Strain Limits				City	(g)	(in)	(g)
					S _a 100YR	Δ 100YR	S _a 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.025	0.2	0.136	0.8
ε _{s,rcw}	0.01	PL3 steel	New York City	0.019	0.1	0.085	0.5
ε _{s,bb}	0.0234	PL1 steel	Boston	0.022	0.1	0.097	0.6
ε _{c,u}	-0.0166	PL2 conc	Knoxville	0.043	0.3	0.199	1.2
0.8*ε _{s,bb}	0.0187	PL2 steel	Paducah	0.049	0.3	0.448	2.7
			St Louis	0.034	0.2	0.260	1.6
Limit State Disp. (in)				Helena	0.067	0.4	0.246
Δ @ ε _{c,cc}	2.1	PL3 conc	Salt Lake City	0.098	0.6	0.661	4.0
Δ @ ε _{s,rcw}	3.3	PL3 steel	Los Angeles	0.328	2.0	0.824	5.0
Δ @ ε _{s,bb}	7.3	PL1 steel	San Francisco	0.358	2.2	0.837	5.1
Δ @ ε _{c,u}	8.2	PL2 conc	Seattle	0.207	1.3	0.655	4.0
Δ @ 0.8*ε _{s,bb}	5.9	PL2 steel	Juneau	0.155	1.1	0.505	3.6
			New Madrid	0.060	0.4	1.144	7.0



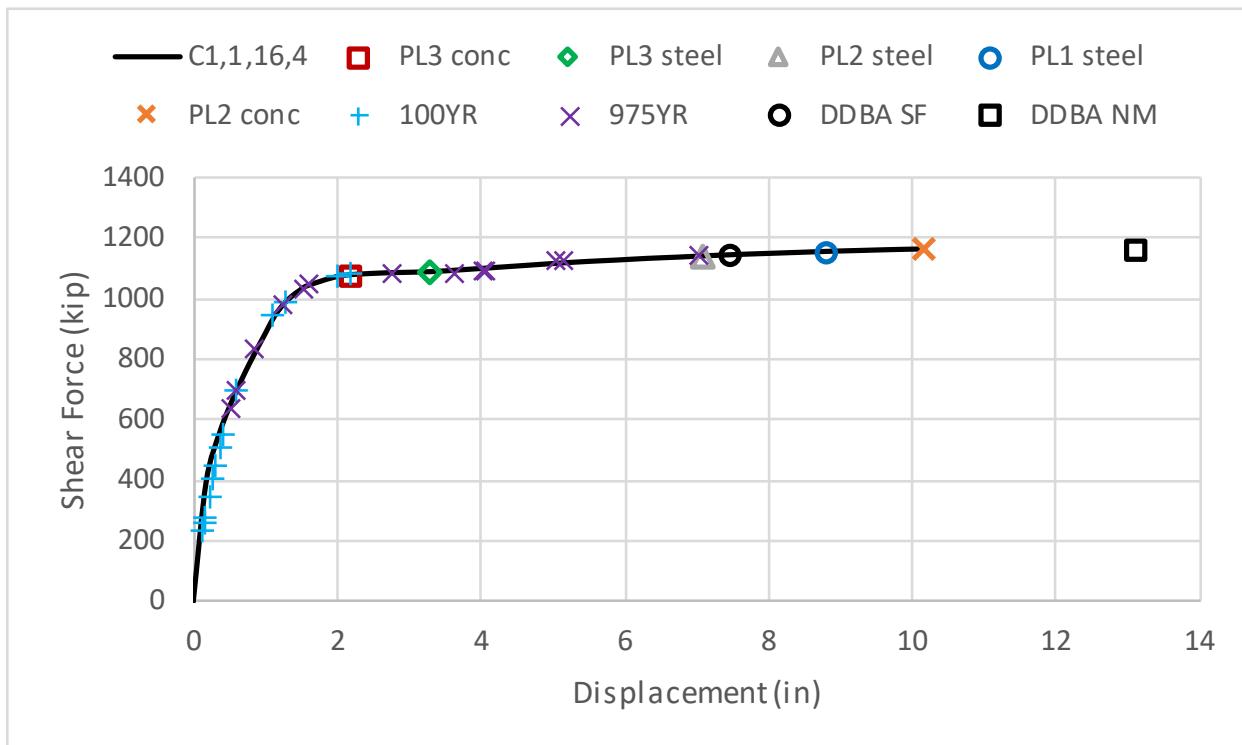
C1,0.7,16,5		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.01	4A _h /D's	0.007	San Francisco	9.7	in	
P/f' _{ce} A _g	0.16	New Madrid	17.15	in			
L/D	5						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.018	0.2	0.099	1.1
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.014	0.2	0.062	0.7
$\varepsilon_{s,bb}$	0.0234	PL1 steel	Boston	0.016	0.2	0.071	0.8
$\varepsilon_{c,u}$	-0.0166	PL2 conc	Knoxville	0.031	0.4	0.145	1.7
$0.8^*\varepsilon_{s,bb}$	0.0187	PL2 steel	Paducah	0.036	0.4	0.326	3.8
			St Louis	0.025	0.3	0.190	2.2
Limit State Disp. (in)		Helena	0.049	0.6	0.179	2.1	
$\Delta @ \varepsilon_{c,cc}$	3.0	PL3 conc	Salt Lake City	0.071	0.8	0.481	5.5
$\Delta @ \varepsilon_{s,rcw}$	4.5	PL3 steel	Los Angeles	0.239	2.8	0.600	6.9
$\Delta @ \varepsilon_{s,bb}$	9.6	PL1 steel	San Francisco	0.261	3.0	0.609	7.0
$\Delta @ \varepsilon_{c,u}$	10.7	PL2 conc	Seattle	0.151	1.7	0.477	5.5
$\Delta @ 0.8^*\varepsilon_{s,bb}$	7.8	PL2 steel	Juneau	0.113	1.3	0.368	4.2
			New Madrid	0.044	0.5	0.833	9.6



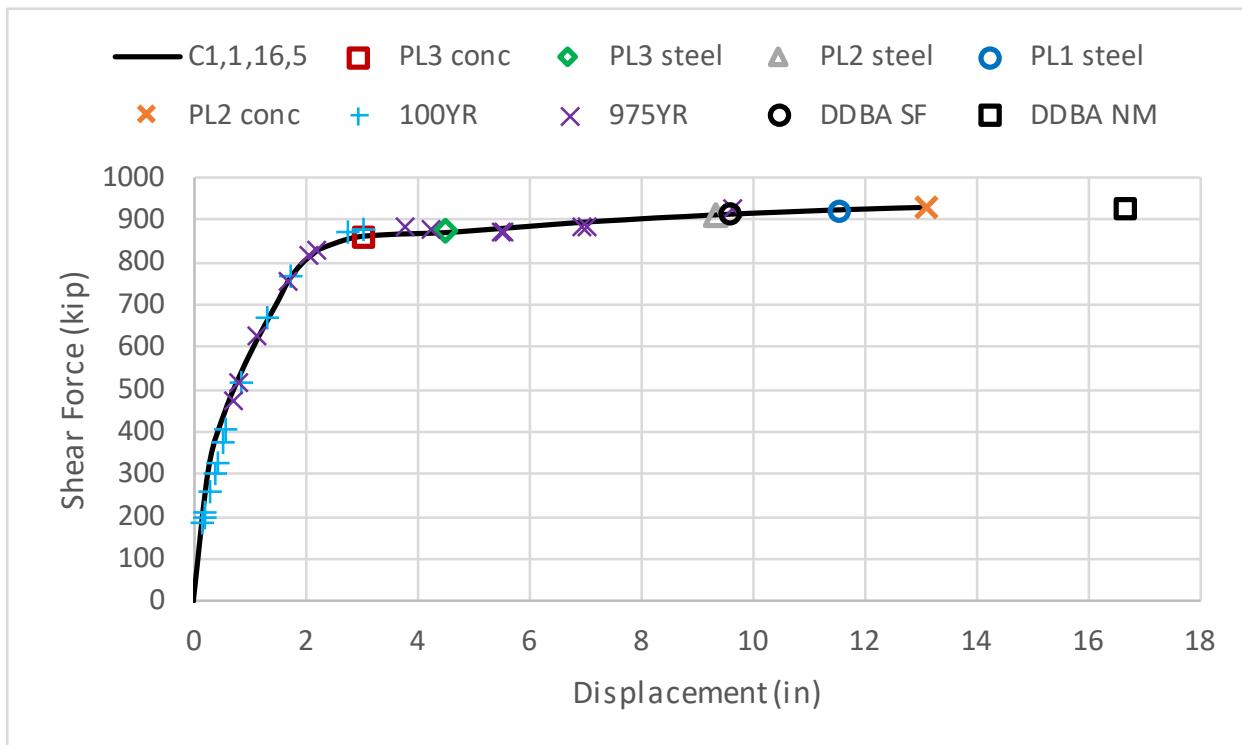
C1,0.7,16,6							
A _{st} /A _g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 12 in					
P/f' _{ce} A _g	0.16	New Madrid 20.9 in					
L/D	6						
Strain Limits				(g)	(in)	(g)	(in)
		City		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.014	0.3	0.076	1.5
ε _{s,rcw}	0.01	PL3 steel	New York City	0.011	0.2	0.048	0.9
ε _{s,bb}	0.0234	PL1 steel	Boston	0.012	0.2	0.054	1.1
ε _{c,u}	-0.0166	PL2 conc	Knoxville	0.024	0.5	0.112	2.2
0.8*ε _{s,bb}	0.0187	PL2 steel	Paducah	0.027	0.5	0.251	4.9
			St Louis	0.019	0.4	0.146	2.8
Limit State Disp. (in)			Helena	0.038	0.7	0.138	2.7
Δ @ ε _{c,cc}	4.1	PL3 conc	Salt Lake City	0.055	1.1	0.371	7.2
Δ @ ε _{s,rcw}	6.0	PL3 steel	Los Angeles	0.184	3.6	0.462	9.0
Δ @ ε _{s,bb}	12.5	PL1 steel	San Francisco	0.201	3.9	0.469	9.1
Δ @ ε _{c,u}	14.2	PL2 conc	Seattle	0.116	2.3	0.368	7.1
Δ @ 0.8*ε _{s,bb}	10.2	PL2 steel	Juneau	0.087	1.7	0.283	5.5
			New Madrid	0.034	0.7	0.642	12.5



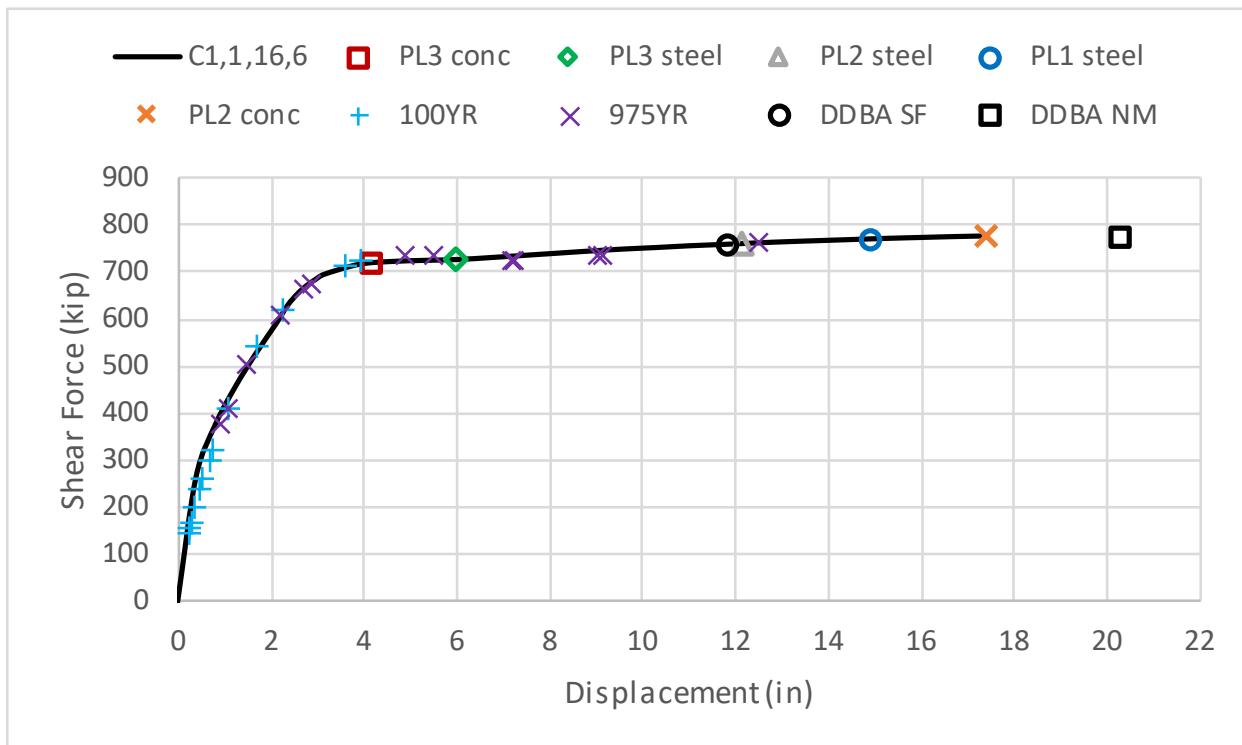
C1,1,16,4							
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 7.45 in					
$P/f'_{ce}A_g$	0.16	New Madrid 13.1 in					
L/D	4						
Strain Limits				(g)	(in)	(g)	(in)
		City		S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.025	0.2	0.135	0.8
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.019	0.1	0.085	0.5
$\epsilon_{s,bb}$	0.0284	PL1 steel	Boston	0.022	0.1	0.097	0.6
$\epsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.043	0.3	0.198	1.2
$0.8^*\epsilon_{s,bb}$	0.0228	PL2 steel	Paducah	0.049	0.3	0.447	2.7
			St Louis	0.034	0.2	0.260	1.6
Limit State Disp. (in)			Helena	0.067	0.4	0.245	1.5
$\Delta @ \epsilon_{c,cc}$	2.2	PL3 conc	Salt Lake City	0.098	0.6	0.660	4.0
$\Delta @ \epsilon_{s,rcw}$	3.3	PL3 steel	Los Angeles	0.327	2.0	0.822	5.0
$\Delta @ \epsilon_{s,bb}$	8.8	PL1 steel	San Francisco	0.357	2.2	0.835	5.1
$\Delta @ \epsilon_{c,u}$	10.1	PL2 conc	Seattle	0.207	1.3	0.654	4.0
$\Delta @ 0.8^*\epsilon_{s,bb}$	7.1	PL2 steel	Juneau	0.155	1.1	0.504	3.6
			New Madrid	0.060	0.4	1.142	7.0



C1,1,16,5							
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 9.55 in					
$P/f'_{ce}A_g$	0.16	New Madrid 16.65 in					
L/D	5						
Strain Limits				(g)	(in)	(g)	(in)
		City		S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.018	0.2	0.099	1.1
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.014	0.2	0.062	0.7
$\epsilon_{s,bb}$	0.0284	PL1 steel	Boston	0.016	0.2	0.071	0.8
$\epsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.031	0.4	0.145	1.7
$0.8^*\epsilon_{s,bb}$	0.0228	PL2 steel	Paducah	0.036	0.4	0.326	3.8
			St Louis	0.024	0.3	0.189	2.2
Limit State Disp. (in)			Helena	0.049	0.6	0.179	2.1
$\Delta @ \epsilon_{c,cc}$	3.0	PL3 conc	Salt Lake City	0.071	0.8	0.480	5.6
$\Delta @ \epsilon_{s,rcw}$	4.5	PL3 steel	Los Angeles	0.238	2.8	0.599	6.9
$\Delta @ \epsilon_{s,bb}$	11.5	PL1 steel	San Francisco	0.260	3.0	0.608	7.0
$\Delta @ \epsilon_{c,u}$	13.1	PL2 conc	Seattle	0.151	1.7	0.476	5.5
$\Delta @ 0.8^*\epsilon_{s,bb}$	9.3	PL2 steel	Juneau	0.113	1.3	0.367	4.2
			New Madrid	0.044	0.5	0.832	9.6

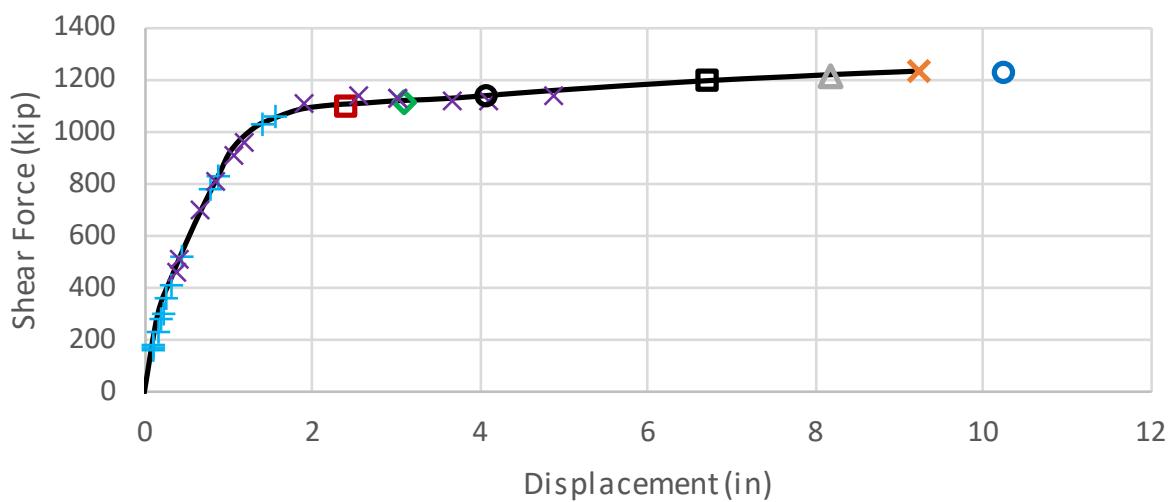
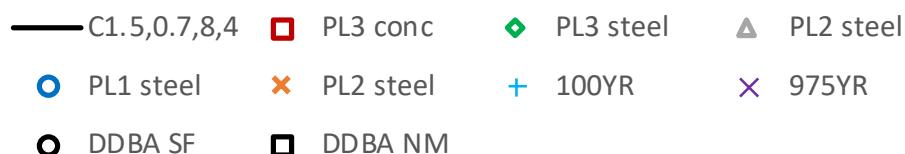


C1,1,16,6							
A_{st}/A_g	0.01	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 11.8 in					
$P/f'_{ce}A_g$	0.16	New Madrid 20.25 in					
L/D	6						
Strain Limits				(g)	(in)	(g)	(in)
		City		S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.014	0.3	0.076	1.5
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.011	0.2	0.048	0.9
$\epsilon_{s,bb}$	0.0284	PL1 steel	Boston	0.012	0.2	0.054	1.1
$\epsilon_{c,u}$	-0.0201	PL2 conc	Knoxville	0.024	0.5	0.111	2.2
$0.8^*\epsilon_{s,bb}$	0.0228	PL2 steel	Paducah	0.027	0.5	0.251	4.9
			St Louis	0.019	0.4	0.146	2.8
Limit State Disp. (in)			Helena	0.038	0.7	0.138	2.7
$\Delta @ \epsilon_{c,cc}$	4.1	PL3 conc	Salt Lake City	0.055	1.1	0.370	7.2
$\Delta @ \epsilon_{s,rcw}$	6.0	PL3 steel	Los Angeles	0.183	3.6	0.461	9.0
$\Delta @ \epsilon_{s,bb}$	14.9	PL1 steel	San Francisco	0.200	3.9	0.468	9.1
$\Delta @ \epsilon_{c,u}$	17.4	PL2 conc	Seattle	0.116	2.3	0.367	7.2
$\Delta @ 0.8^*\epsilon_{s,bb}$	12.1	PL2 steel	Juneau	0.087	1.7	0.283	5.5
			New Madrid	0.034	0.7	0.640	12.5



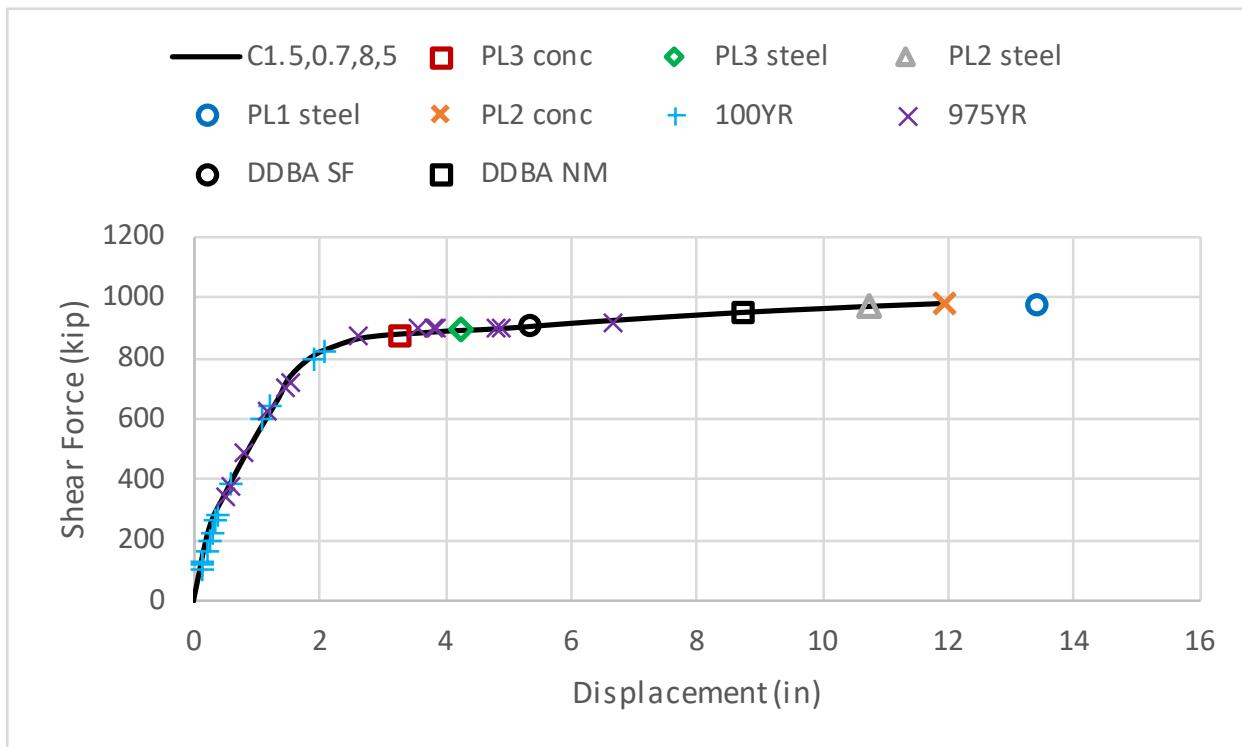
C1.5,0.7,8,4

A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ						
$4A_h/D'$	0.007	San Francisco 4.05 in						
$P/f'_{ce}A_g$	0.08	New Madrid 6.7 in						
L/D	4							
Strain Limits			City	S_a 100YR	(g)	(in)	(g)	
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.035	0.1	0.195	0.7	
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.027	0.1	0.122	0.4	
$\epsilon_{s,bb}$	0.0346	PL1 steel	Boston	0.031	0.1	0.139	0.4	
$\epsilon_{c,u}$	-0.0169	PL2 conc	Knoxville	0.062	0.2	0.286	0.8	
$0.8^*\epsilon_{s,bb}$	0.0277	PL2 steel	Paducah	0.070	0.2	0.643	1.9	
			St Louis	0.048	0.1	0.374	1.2	
Limit State Disp. (in)			Helena	0.097	(in)	0.3	0.353	1.0
$\Delta @ \epsilon_{c,cc}$	2.4	PL3 conc	Salt Lake City	0.140	0.4	0.949	3.0	
$\Delta @ \epsilon_{s,rcw}$	3.1	PL3 steel	Los Angeles	0.471	1.4	1.183	3.7	
$\Delta @ \epsilon_{s,bb}$	10.2	PL1 steel	San Francisco	0.514	1.6	1.202	4.1	
$\Delta @ \epsilon_{c,u}$	9.2	PL2 conc	Seattle	0.297	0.9	0.941	3.0	
$\Delta @ 0.8^*\epsilon_{s,bb}$	8.2	PL2 steel	Juneau	0.164	0.8	0.524	2.5	
			New Madrid	0.087	0.3	1.643	4.9	

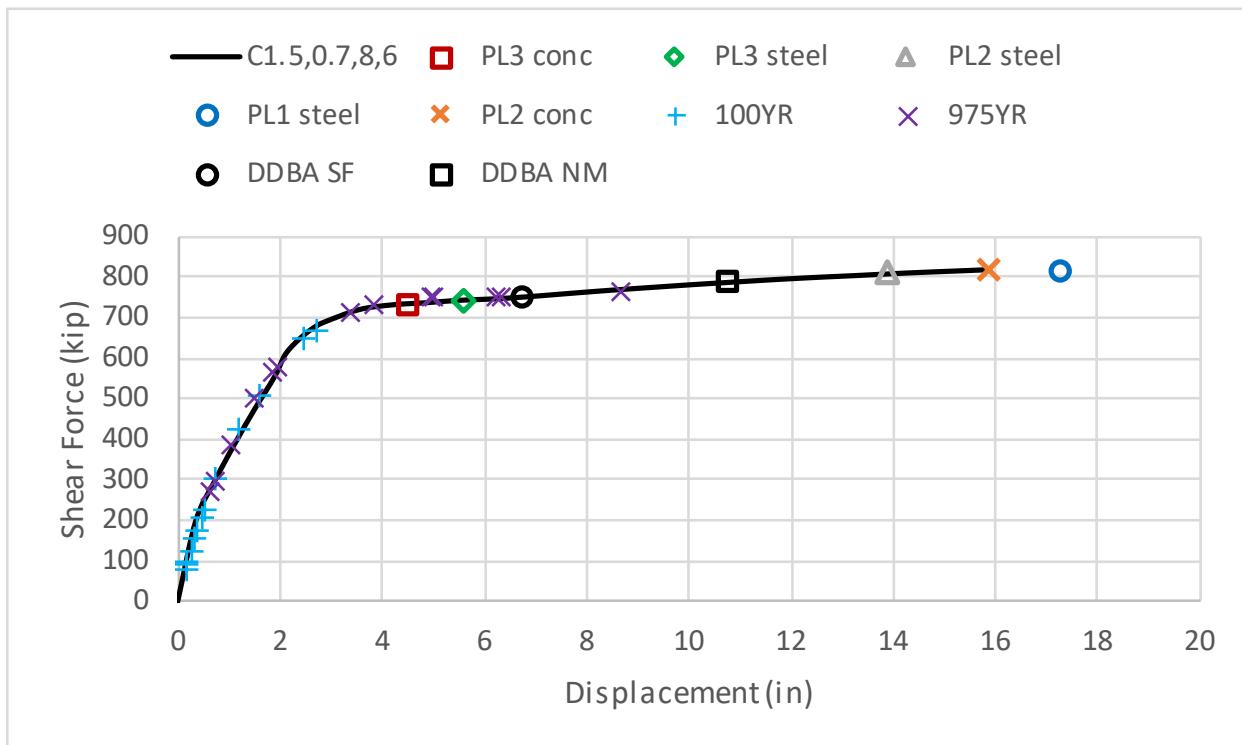


C1.5,0.7,8,5

A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.007	San Francisco 5.3 in					
$P/f'_{ce}A_g$	0.08	New Madrid 8.7 in					
L/D	5						
Strain Limits			City	(g)	(in)	(g)	(in)
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.026	0.1	0.142	0.8
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.020	0.1	0.089	0.5
$\epsilon_{s,bb}$	0.0346	PL1 steel	Boston	0.023	0.1	0.102	0.6
$\epsilon_{c,u}$	-0.0169	PL2 conc	Knoxville	0.045	0.3	0.209	1.2
$0.8^*\epsilon_{s,bb}$	0.0277	PL2 steel	Paducah	0.051	0.3	0.470	2.6
			St Louis	0.035	0.2	0.273	1.5
Limit State Disp. (in)			Helena	0.071	0.4	0.258	1.4
$\Delta @ \epsilon_{c,cc}$	3.3	PL3 conc	Salt Lake City	0.103	0.6	0.693	3.9
$\Delta @ \epsilon_{s,rcw}$	4.2	PL3 steel	Los Angeles	0.344	1.9	0.864	4.8
$\Delta @ \epsilon_{s,bb}$	13.4	PL1 steel	San Francisco	0.376	2.1	0.878	4.9
$\Delta @ \epsilon_{c,u}$	11.9	PL2 conc	Seattle	0.217	1.2	0.688	3.8
$\Delta @ 0.8^*\epsilon_{s,bb}$	10.8	PL2 steel	Juneau	0.163	1.1	0.524	3.6
			New Madrid	0.063	0.4	1.200	6.7



C1.5,0.7,8,6							
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 6.7 in					
P/f' _{ce} A _g	0.08	New Madrid 10.7 in					
L/D	6						
Strain Limits		City		(g)	(in)	(g)	(in)
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.020	0.2	0.110	1.0
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.015	0.1	0.069	0.6
$\varepsilon_{s,bb}$	0.0346	PL1 steel	Boston	0.018	0.2	0.079	0.7
$\varepsilon_{c,u}$	-0.0169	PL2 conc	Knoxville	0.035	0.3	0.161	1.5
0.8* $\varepsilon_{s,bb}$	0.0277	PL2 steel	Paducah	0.040	0.4	0.362	3.4
			St Louis	0.027	0.3	0.211	2.0
Limit State Disp. (in)			Helena	0.055	0.5	0.199	1.9
$\Delta @ \varepsilon_{c,cc}$	4.4	PL3 conc	Salt Lake City	0.079	0.7	0.534	5.0
$\Delta @ \varepsilon_{s,rcw}$	5.6	PL3 steel	Los Angeles	0.265	2.5	0.666	6.2
$\Delta @ \varepsilon_{s,bb}$	17.2	PL1 steel	San Francisco	0.289	2.7	0.677	6.3
$\Delta @ \varepsilon_{c,u}$	15.9	PL2 conc	Seattle	0.167	1.6	0.530	5.0
$\Delta @ 0.8*\varepsilon_{s,bb}$	13.9	PL2 steel	Juneau	0.125	1.2	0.408	3.8
			New Madrid	0.049	0.5	0.925	8.6



C1.5,1,8,4

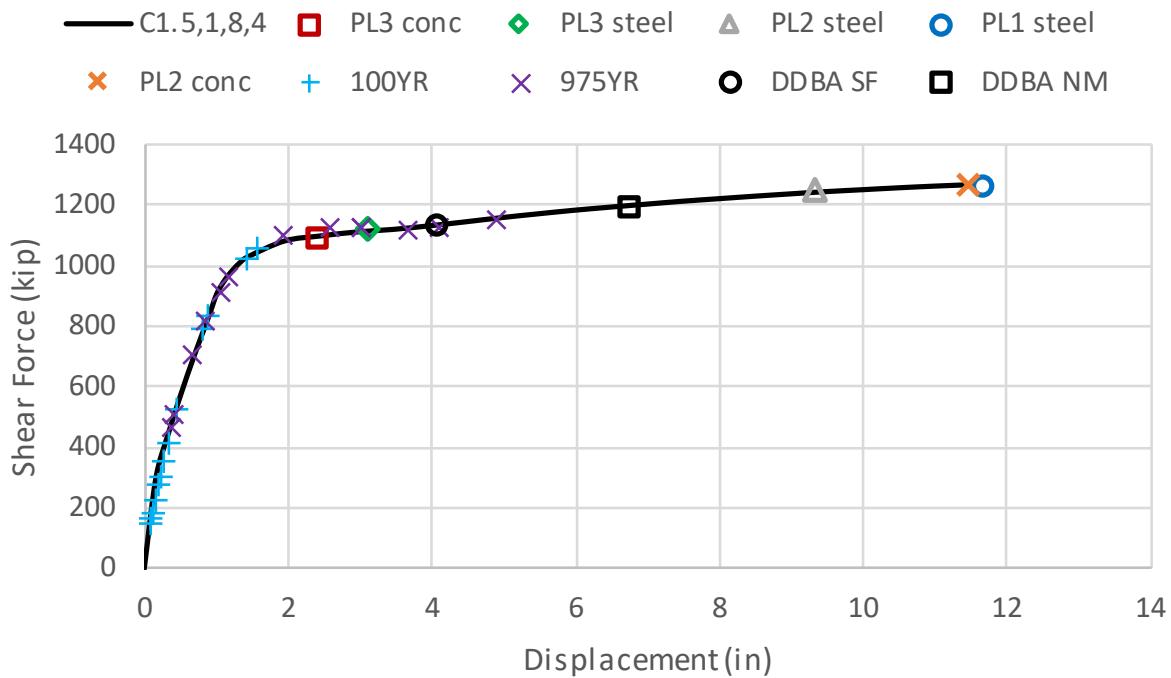
A_{st}/A_g	0.015
$4A_h/D'$	0.01
$P/f'_{ce}A_g$	0.08
L/D	4

DDBA (Tangent Stiffness) Comparison for the 975YR EQ

San Francisco 4.05 in

New Madrid 6.7 in

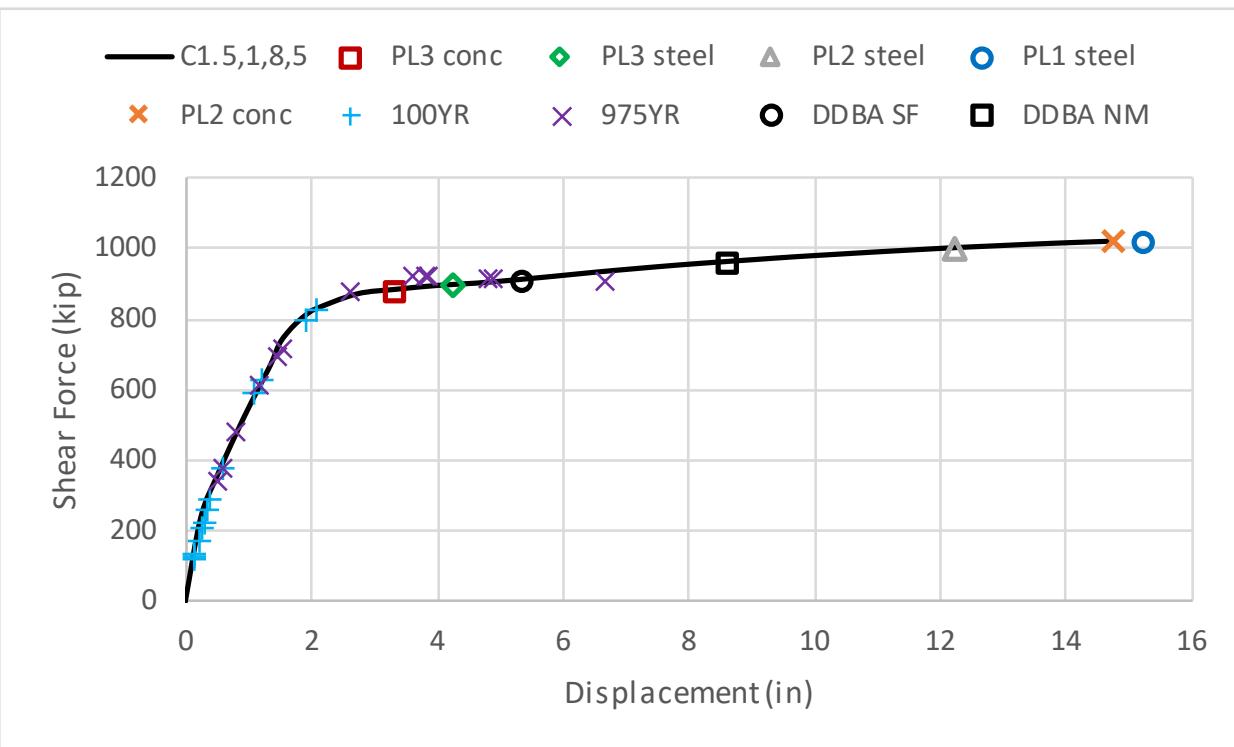
Strain Limits			City	(g)	(in)	(g)	(in)
				S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.035	0.1	0.195	0.7
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.027	0.1	0.122	0.4
$\epsilon_{s,bb}$	0.0397	PL1 steel	Boston	0.031	0.1	0.139	0.4
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.062	0.2	0.285	0.8
$0.8^*\epsilon_{s,bb}$	0.0317	PL2 steel	Paducah	0.070	0.2	0.642	1.9
			St Louis	0.048	0.1	0.373	1.2
Limit State Disp. (in)			Helena	0.097	0.3	0.352	1.0
$\Delta @ \epsilon_{c,cc}$	2.4	PL3 conc	Salt Lake City	0.140	0.4	0.947	3.0
$\Delta @ \epsilon_{s,rcw}$	3.1	PL3 steel	Los Angeles	0.470	1.4	1.181	3.7
$\Delta @ \epsilon_{s,bb}$	11.7	PL1 steel	San Francisco	0.513	1.6	1.199	4.1
$\Delta @ \epsilon_{c,u}$	11.5	PL2 conc	Seattle	0.297	0.9	0.939	3.0
$\Delta @ 0.8^*\epsilon_{s,bb}$	9.3	PL2 steel	Juneau	0.164	0.8	0.524	2.6
			New Madrid	0.087	0.3	1.640	4.9



C1.5,1,8,5

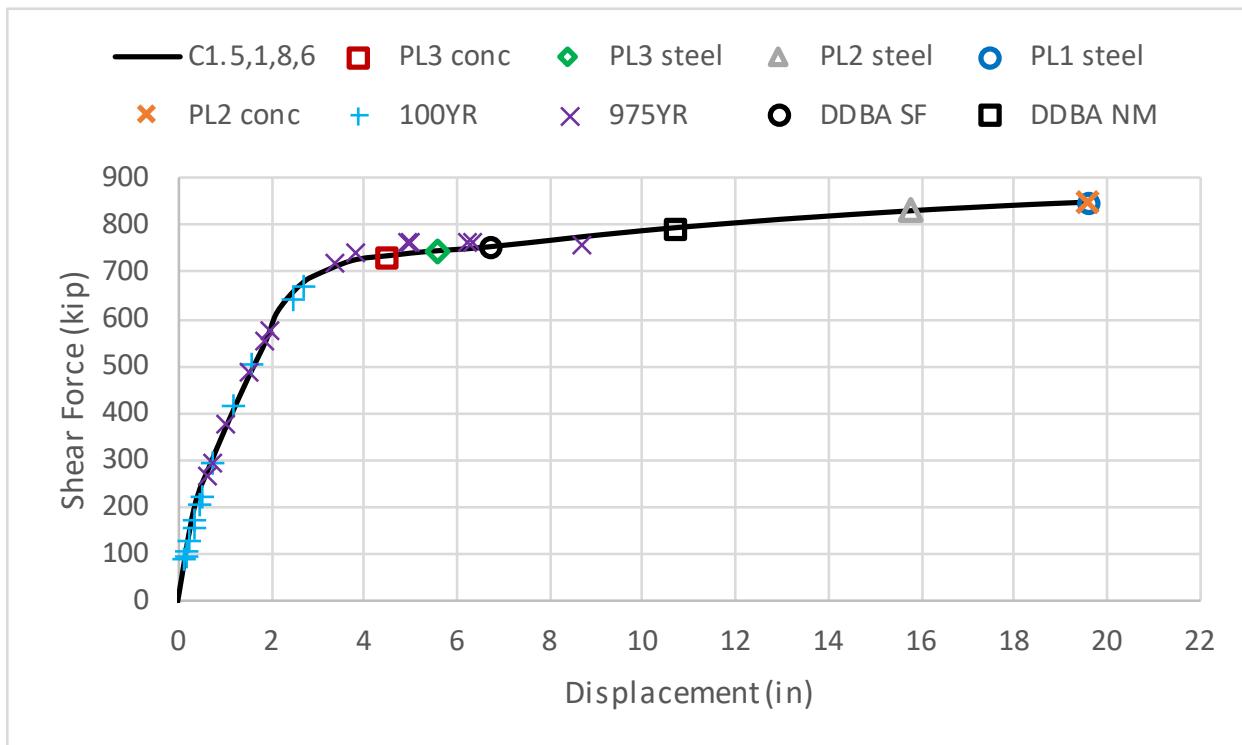
A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ			
$4A_h/D'$	0.01	San Francisco	5.32	in	
$P/f'_{ce}A_g$	0.08	New Madrid	8.6	in	
L/D	5				

Strain Limits	$\varepsilon_{c,cc}$	$\varepsilon_{s,rcw}$	$\varepsilon_{s,bb}$	$\varepsilon_{c,u}$	$0.8^*\varepsilon_{s,bb}$	(g)		(in)		(g)		(in)	
						City	S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR			
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.026	0.1	0.142	0.8						
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.020	0.1	0.089	0.5						
$\varepsilon_{s,bb}$	0.0397	PL1 steel	Boston	0.023	0.1	0.102	0.6						
$\varepsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.045	0.3	0.208	1.2						
$0.8^*\varepsilon_{s,bb}$	0.0317	PL2 steel	Paducah	0.051	0.3	0.469	2.6						
			St Louis	0.035	0.2	0.273	1.5						
Limit State Disp. (in)						Helena	0.071	0.4	0.258	1.4			
$\Delta @ \varepsilon_{c,cc}$	3.3	PL3 conc	Salt Lake City	0.102	0.6	0.692	3.9						
$\Delta @ \varepsilon_{s,rcw}$	4.2	PL3 steel	Los Angeles	0.343	1.9	0.863	4.8						
$\Delta @ \varepsilon_{s,bb}$	15.2	PL1 steel	San Francisco	0.375	2.1	0.876	4.9						
$\Delta @ \varepsilon_{c,u}$	14.8	PL2 conc	Seattle	0.217	1.2	0.686	3.8						
$\Delta @ 0.8^*\varepsilon_{s,bb}$	12.2	PL2 steel	Juneau	0.162	1.1	0.524	3.6						
			New Madrid	0.063	0.4	1.198	6.7						

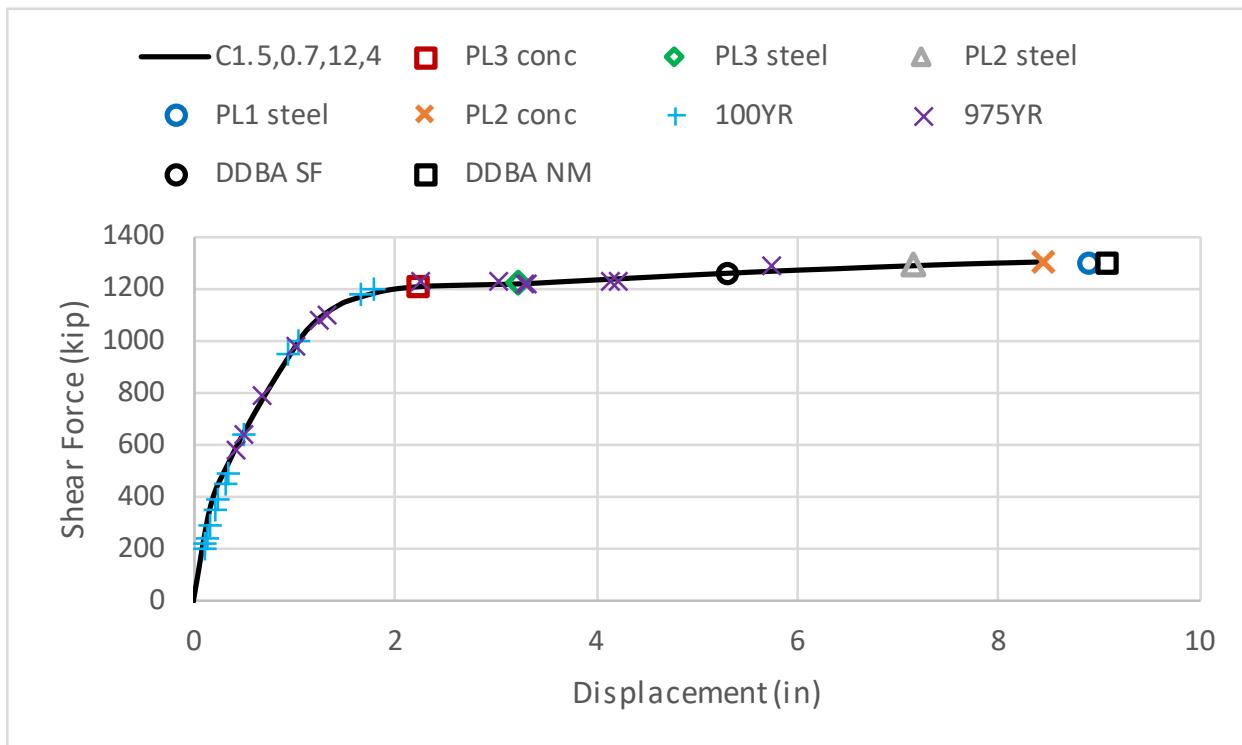


C1.5,1,8,6

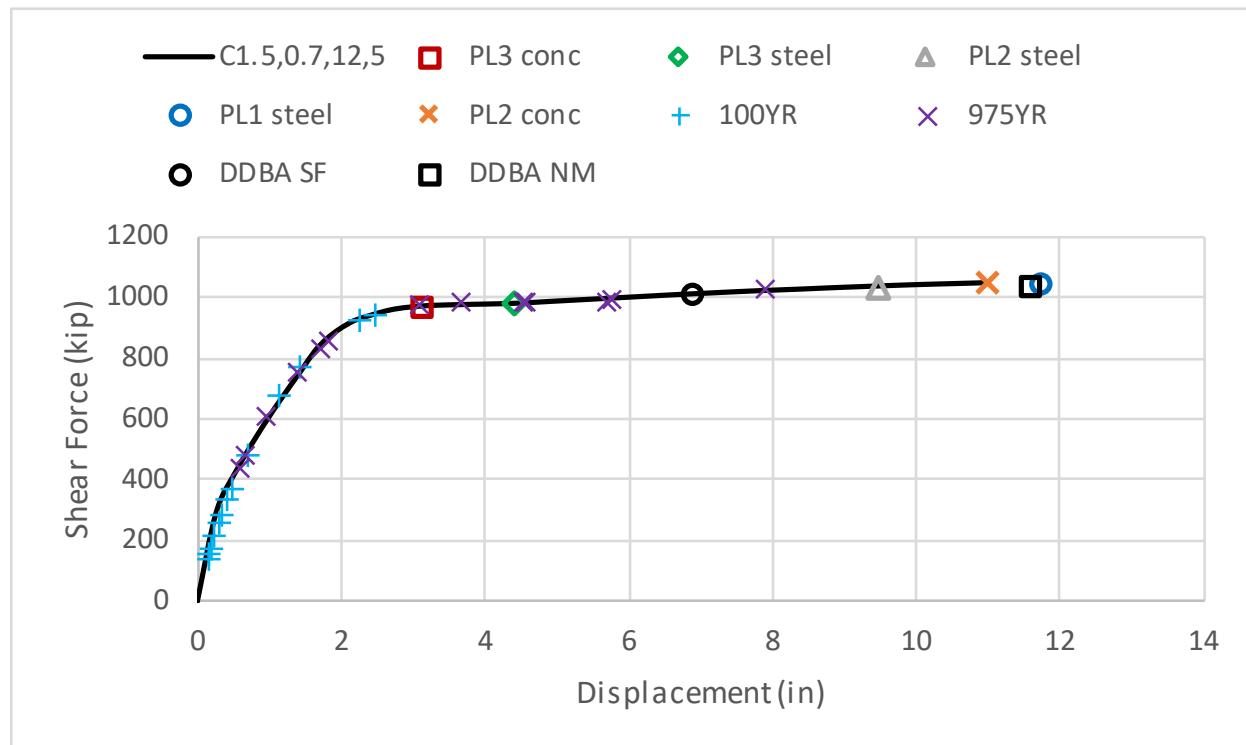
A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco	6.7	in			
$P/f'_{ce}A_g$	0.08	New Madrid	10.7	in			
L/D	6						
Strain Limits			(g)	(in)	(g)	(in)	
		City	S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR	
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.020	0.2	0.110	1.0
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.015	0.1	0.069	0.6
$\epsilon_{s,bb}$	0.0397	PL1 steel	Boston	0.018	0.2	0.078	0.7
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.035	0.3	0.161	1.5
$0.8^*\epsilon_{s,bb}$	0.0317	PL2 steel	Paducah	0.039	0.4	0.362	3.4
			St Louis	0.027	0.3	0.210	2.0
Limit State Disp. (in)			Helena	0.054	0.5	0.198	1.9
$\Delta @ \epsilon_{c,cc}$	4.5	PL3 conc	Salt Lake City	0.079	0.7	0.534	5.0
$\Delta @ \epsilon_{s,rcw}$	5.6	PL3 steel	Los Angeles	0.265	2.5	0.665	6.2
$\Delta @ \epsilon_{s,bb}$	19.6	PL1 steel	San Francisco	0.289	2.7	0.675	6.3
$\Delta @ \epsilon_{c,u}$	19.6	PL2 conc	Seattle	0.167	1.6	0.529	5.0
$\Delta @ 0.8^*\epsilon_{s,bb}$	15.8	PL2 steel	Juneau	0.125	1.2	0.407	3.8
			New Madrid	0.049	0.5	0.924	8.7



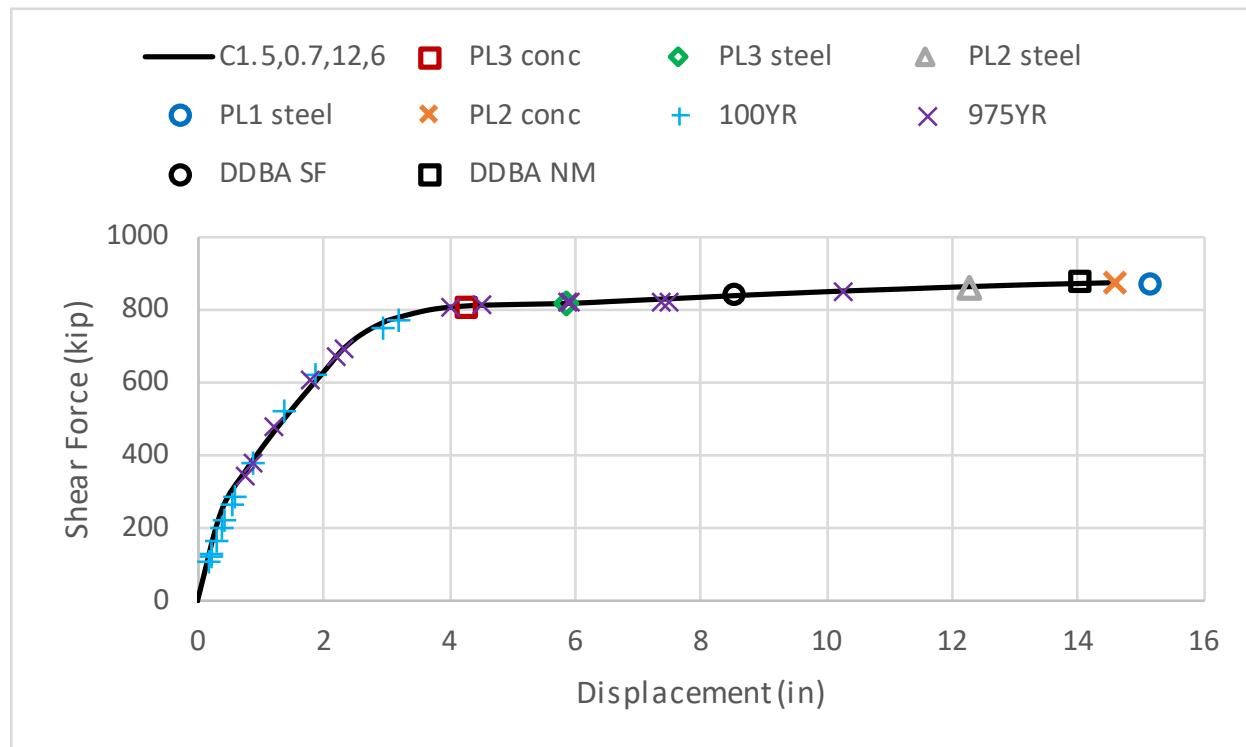
C1.5,0.7,12,4							
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 5.3 in					
P/f' _{ce} A _g	0.12	New Madrid 9.05 in					
L/D	4						
Strain Limits		City		(g)	(in)	(g)	(in)
$\varepsilon_{c,cc}$		Charleston		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
$\varepsilon_{s,rcw}$	0.01	New York City		0.023	0.1	0.104	0.4
$\varepsilon_{s,bb}$	0.0290	Boston		0.027	0.1	0.118	0.5
$\varepsilon_{c,u}$	-0.0169	Knoxville		0.052	0.2	0.242	1.0
0.8* $\varepsilon_{s,bb}$	0.0232	Paducah		0.059	0.2	0.545	2.3
		St Louis		0.041	0.2	0.317	1.3
Limit State Disp. (in)			Helena		0.082	0.3	0.299
$\Delta @ \varepsilon_{c,cc}$	2.2	Salt Lake City		0.119	0.5	0.804	3.3
$\Delta @ \varepsilon_{s,rcw}$	3.2	Los Angeles		0.399	1.6	1.002	4.1
$\Delta @ \varepsilon_{s,bb}$	8.9	San Francisco		0.435	1.8	1.018	4.2
$\Delta @ \varepsilon_{c,u}$	8.4	Seattle		0.252	1.0	0.797	3.3
$\Delta @ 0.8*\varepsilon_{s,bb}$	7.1	Juneau		0.164	0.9	0.524	3.0
		New Madrid		0.073	0.3	1.392	5.7



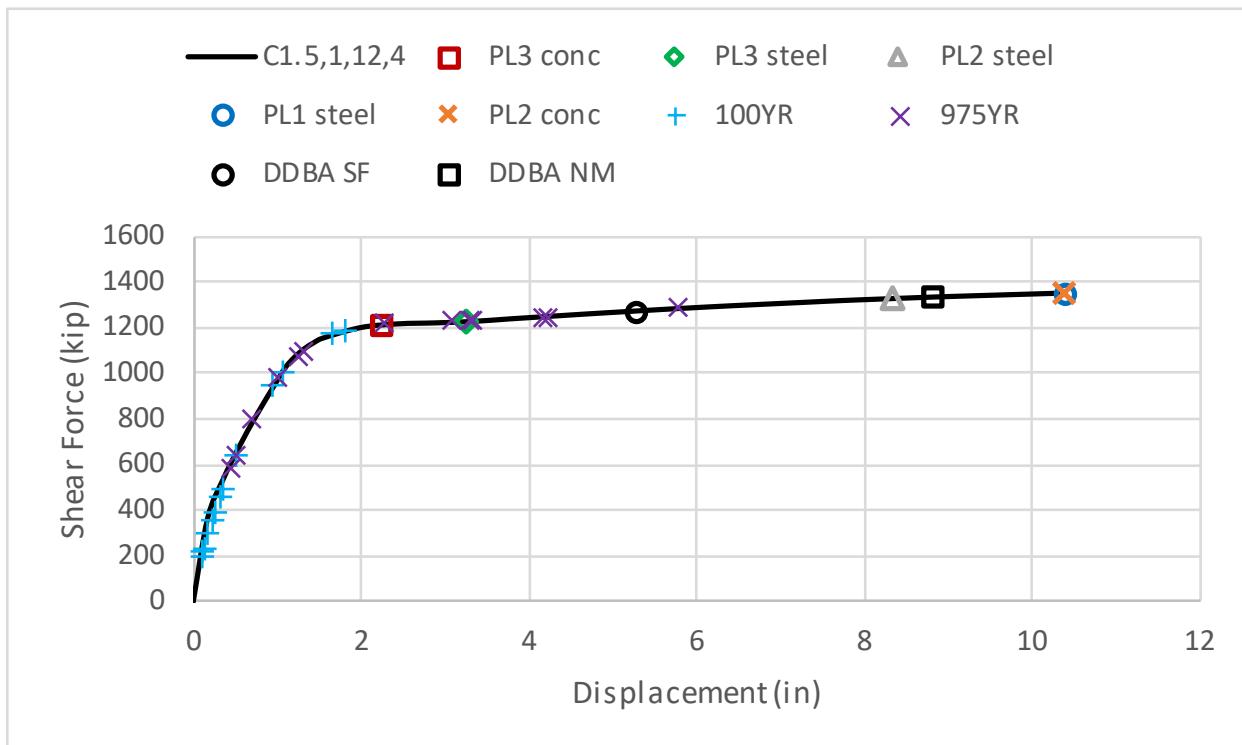
C1.5,0.7,12,5							
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 6.85 in					
P/f' _{ce} A _g	0.12	New Madrid 11.55 in					
L/D	5						
Strain Limits		City		(g)	(in)	(g)	(in)
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.022	0.2	0.120	0.9
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.017	0.1	0.076	0.6
$\epsilon_{s,bb}$	0.0290	PL1 steel	Boston	0.019	0.2	0.086	0.7
$\epsilon_{c,u}$	-0.0169	PL2 conc	Knoxville	0.038	0.3	0.176	1.4
0.8* $\epsilon_{s,bb}$	0.0232	PL2 steel	Paducah	0.043	0.3	0.397	3.1
			St Louis	0.030	0.2	0.231	1.8
Limit State Disp. (in)			Helena	0.060	0.5	0.218	1.7
$\Delta @ \epsilon_{c,cc}$	3.1	PL3 conc	Salt Lake City	0.087	0.7	0.586	4.6
$\Delta @ \epsilon_{s,rcw}$	4.4	PL3 steel	Los Angeles	0.290	2.3	0.730	5.7
$\Delta @ \epsilon_{s,bb}$	11.7	PL1 steel	San Francisco	0.317	2.5	0.741	5.8
$\Delta @ \epsilon_{c,u}$	11.0	PL2 conc	Seattle	0.183	1.4	0.581	4.5
$\Delta @ 0.8*\epsilon_{s,bb}$	9.5	PL2 steel	Juneau	0.137	1.1	0.447	3.7
			New Madrid	0.054	0.4	1.014	7.9



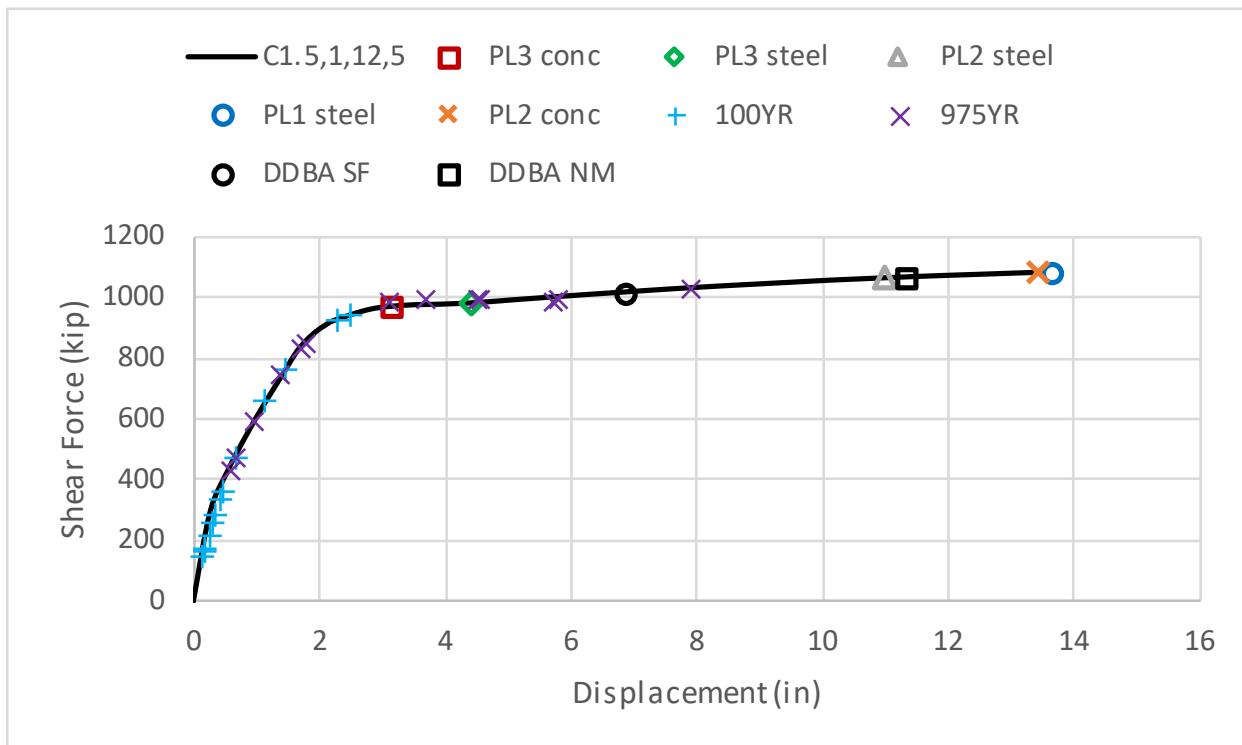
C1.5,0.7,12,6							
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 8.5 in					
P/f' _{ce} A _g	0.12	New Madrid 14 in					
L/D	6						
Strain Limits				City	(g)	(in)	(g)
					S _a 100YR	Δ 100YR	S _a 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.017	0.2	0.093	1.2
ε _{s,rcw}	0.01	PL3 steel	New York City	0.013	0.2	0.058	0.8
ε _{s,bb}	0.0290	PL1 steel	Boston	0.015	0.2	0.066	0.9
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.029	0.4	0.136	1.8
0.8*ε _{s,bb}	0.0232	PL2 steel	Paducah	0.033	0.4	0.306	4.0
			St Louis	0.023	0.3	0.178	2.3
Limit State Disp. (in)			Helena	0.046	0.6	0.168	2.2
Δ @ ε _{c,cc}	4.2	PL3 conc	Salt Lake City	0.067	0.9	0.451	5.9
Δ @ ε _{s,rcw}	5.9	PL3 steel	Los Angeles	0.224	2.9	0.562	7.4
Δ @ ε _{s,bb}	15.1	PL1 steel	San Francisco	0.244	3.2	0.571	7.5
Δ @ ε _{c,u}	14.6	PL2 conc	Seattle	0.141	1.9	0.447	5.9
Δ @ 0.8*ε _{s,bb}	12.3	PL2 steel	Juneau	0.106	1.4	0.344	4.5
			New Madrid	0.041	0.5	0.781	10.2



C1.5,1,12,4								
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ						
4A _h /D's	0.01	San Francisco 5.25 in						
P/f' _{ce} A _g	0.12	New Madrid 8.8 in						
L/D	4							
Strain Limits		City	(g)	(in)	(g)	(in)		
			S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR		
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.030	0.1	0.165	0.7	
ε _{s,rcw}	0.01	PL3 steel	New York City	0.023	0.1	0.104	0.4	
ε _{s,bb}	0.0341	PL1 steel	Boston	0.027	0.1	0.118	0.5	
ε _{c,u}	-0.0206	PL2 conc	Knoxville	0.052	0.2	0.241	1.0	
0.8*ε _{s,bb}	0.0272	PL2 steel	Paducah	0.059	0.2	0.544	2.3	
			St Louis	0.041	0.2	0.316	1.3	
Limit State Disp. (in)			Helena	0.082	0.3	0.298	1.2	
Δ @ ε _{c,cc}	2.2	PL3 conc	Salt Lake City	0.119	0.5	0.802	3.3	
Δ @ ε _{s,rcw}	3.2	PL3 steel	Los Angeles	0.398	1.7	1.000	4.2	
Δ @ ε _{s,bb}	10.4	PL1 steel	San Francisco	0.434	1.8	1.015	4.2	
Δ @ ε _{c,u}	10.4	PL2 conc	Seattle	0.251	1.0	0.795	3.3	
Δ @ 0.8*ε _{s,bb}	8.3	PL2 steel	Juneau	0.164	0.9	0.524	3.1	
			New Madrid	0.073	0.3	1.388	5.8	

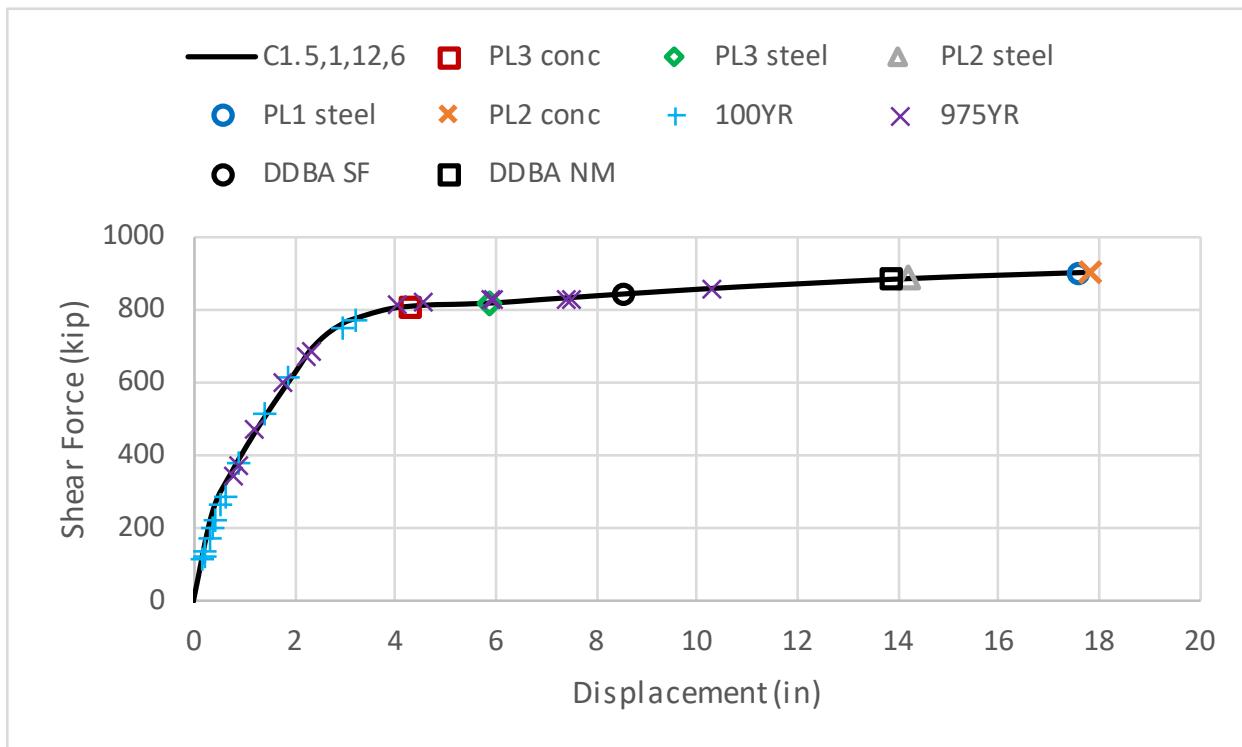


C1.5,1,12,5							
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.01	San Francisco 6.85 in					
P/f' _{ce} A _g	0.12	New Madrid 11.3 in					
L/D	5						
Strain Limits				(g)	(in)	(g)	(in)
		City		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.022	0.2	0.120	0.9
ε _{s,rcw}	0.01	PL3 steel	New York City	0.017	0.1	0.075	0.6
ε _{s,bb}	0.0341	PL1 steel	Boston	0.019	0.2	0.086	0.7
ε _{c,u}	-0.0206	PL2 conc	Knoxville	0.038	0.3	0.176	1.4
0.8*ε _{s,bb}	0.0272	PL2 steel	Paducah	0.043	0.3	0.396	3.1
			St Louis	0.030	0.2	0.230	1.8
Limit State Disp. (in)			Helena	0.060	0.5	0.217	1.7
Δ @ ε _{c,cc}	3.1	PL3 conc	Salt Lake City	0.086	0.7	0.584	4.6
Δ @ ε _{s,rcw}	4.4	PL3 steel	Los Angeles	0.290	2.3	0.728	5.7
Δ @ ε _{s,bb}	13.6	PL1 steel	San Francisco	0.316	2.5	0.740	5.8
Δ @ ε _{c,u}	13.4	PL2 conc	Seattle	0.183	1.4	0.579	4.5
Δ @ 0.8*ε _{s,bb}	11.0	PL2 steel	Juneau	0.137	1.1	0.446	3.7
			New Madrid	0.053	0.4	1.011	7.9

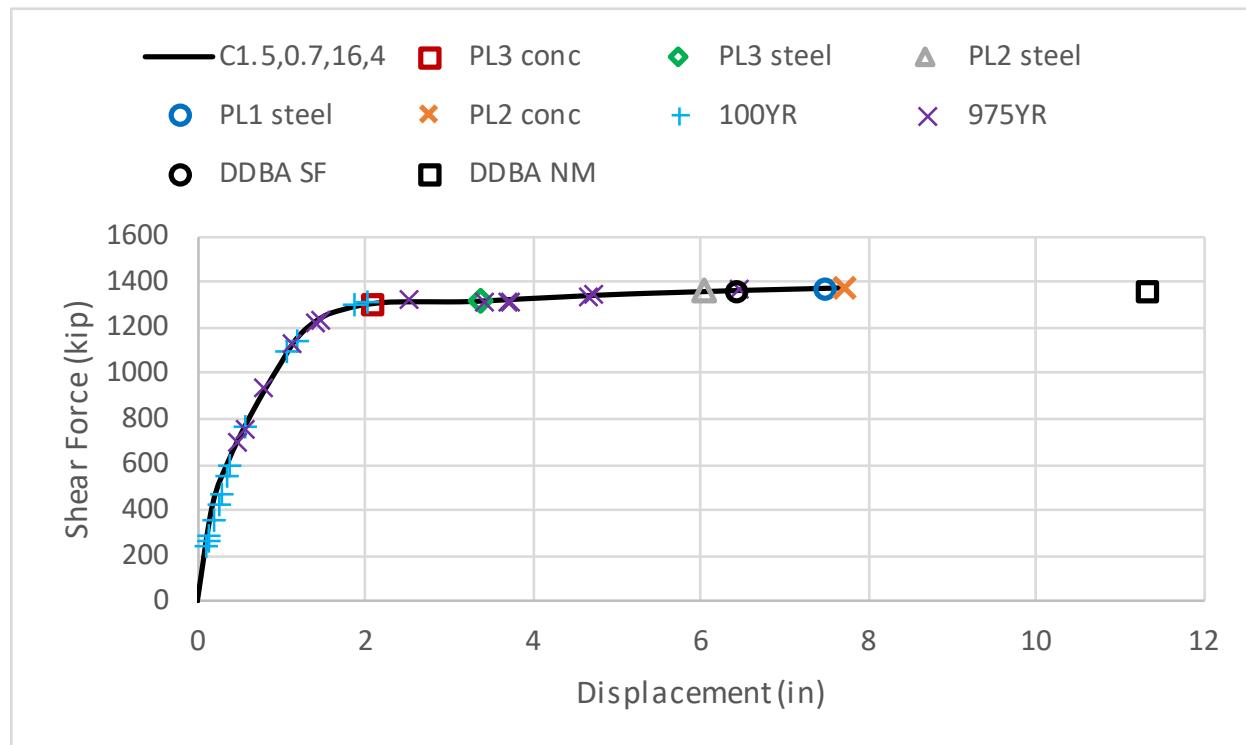


C1.5,1,12,6

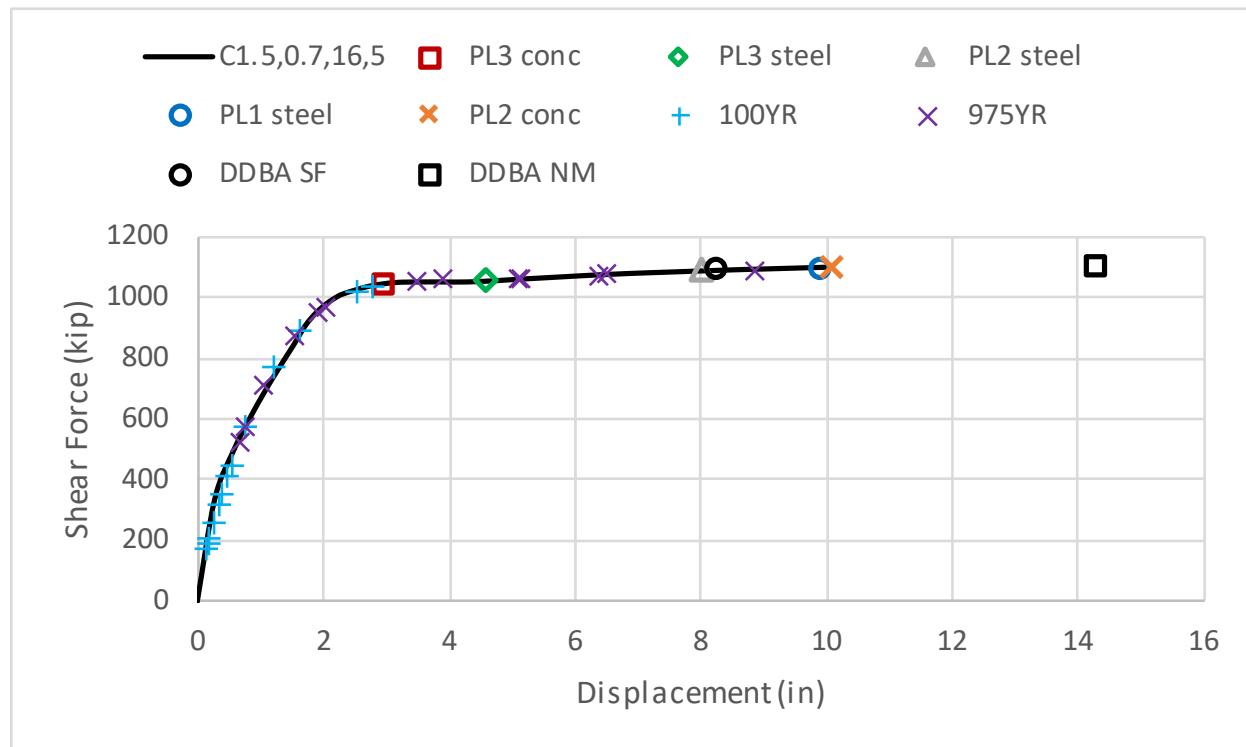
A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$ s	0.01	San Francisco	8.5	in			
$P/f'_{ce}A_g$	0.12	New Madrid	13.85	in			
L/D	6						
Strain Limits				(g)	(in)	(g)	(in)
			City	S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.017	0.2	0.092	1.2
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.013	0.2	0.058	0.8
$\epsilon_{s,bb}$	0.0341	PL1 steel	Boston	0.015	0.2	0.066	0.9
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.029	0.4	0.135	1.8
$0.8^*\epsilon_{s,bb}$	0.0272	PL2 steel	Paducah	0.033	0.4	0.305	4.0
			St Louis	0.023	0.3	0.177	2.3
Limit State Disp. (in)			Helena	0.046	0.6	0.167	2.2
$\Delta @ \epsilon_{c,cc}$	4.3	PL3 conc	Salt Lake City	0.067	0.9	0.450	5.9
$\Delta @ \epsilon_{s,rcw}$	5.9	PL3 steel	Los Angeles	0.223	2.9	0.561	7.4
$\Delta @ \epsilon_{s,bb}$	17.6	PL1 steel	San Francisco	0.244	3.2	0.570	7.5
$\Delta @ \epsilon_{c,u}$	17.8	PL2 conc	Seattle	0.141	1.9	0.446	5.9
$\Delta @ 0.8^*\epsilon_{s,bb}$	14.2	PL2 steel	Juneau	0.106	1.4	0.344	4.5
			New Madrid	0.041	0.5	0.779	10.3



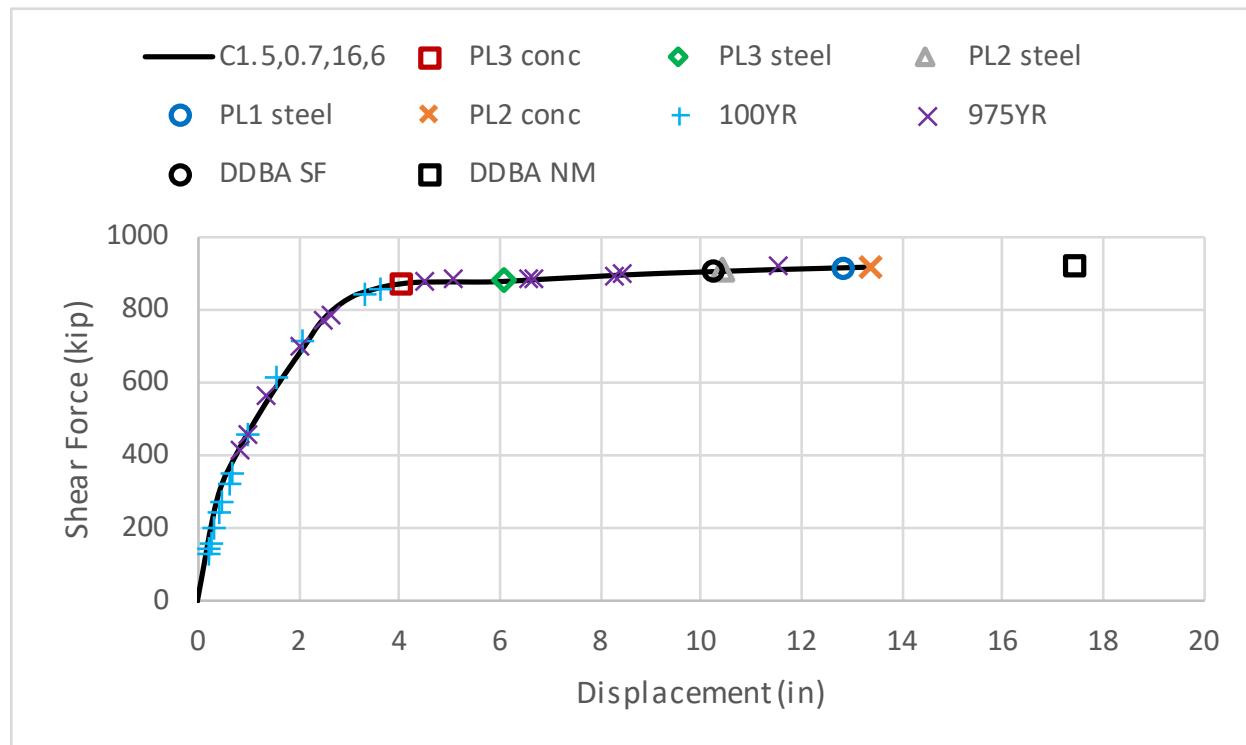
C1.5,0.7,16,4							
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 6.4 in					
P/f' _{ce} A _g	0.16	New Madrid 11.3 in					
L/D	4						
Strain Limits				City	(g)	(in)	(g)
					S _a 100YR	Δ 100YR	S _a 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.027	0.1	0.147	0.8
ε _{s,rcw}	0.01	PL3 steel	New York City	0.021	0.1	0.092	0.5
ε _{s,bb}	0.0234	PL1 steel	Boston	0.024	0.1	0.105	0.5
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.047	0.2	0.215	1.1
0.8*ε _{s,bb}	0.0187	PL2 steel	Paducah	0.053	0.3	0.485	2.5
			St Louis	0.037	0.2	0.282	1.5
Limit State Disp. (in)			Helena	0.073	0.4	0.266	1.4
Δ @ ε _{c,cc}	2.1	PL3 conc	Salt Lake City	0.106	0.6	0.716	3.7
Δ @ ε _{s,rcw}	3.4	PL3 steel	Los Angeles	0.355	1.8	0.893	4.6
Δ @ ε _{s,bb}	7.5	PL1 steel	San Francisco	0.388	2.0	0.906	4.7
Δ @ ε _{c,u}	7.7	PL2 conc	Seattle	0.224	1.2	0.710	3.7
Δ @ 0.8*ε _{s,bb}	6.0	PL2 steel	Juneau	0.164	1.1	0.524	3.4
			New Madrid	0.065	0.3	1.239	6.5



C1.5,0.7,16,5								
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ						
4A _h /D's	0.007	San Francisco 8.2 in						
P/f' _{ce} A _g	0.16	New Madrid 14.25 in						
L/D	5							
Strain Limits		City		(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$		Charleston		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR	
$\varepsilon_{s,rcw}$	0.01	PL3 steel		New York City	0.015	0.1	0.067	0.7
$\varepsilon_{s,bb}$	0.0234	Boston			0.017	0.2	0.077	0.8
$\varepsilon_{c,u}$	-0.0169	Knoxville			0.034	0.3	0.157	1.5
0.8* $\varepsilon_{s,bb}$	0.0187	Paducah			0.039	0.4	0.354	3.5
		St Louis			0.027	0.3	0.206	2.0
Limit State Disp. (in)			Helena		0.053	0.5	0.194	1.9
$\Delta @ \varepsilon_{c,cc}$	2.9	PL3 conc		Salt Lake City	0.077	0.8	0.522	5.1
$\Delta @ \varepsilon_{s,rcw}$	4.6	PL3 steel		Los Angeles	0.259	2.5	0.650	6.4
$\Delta @ \varepsilon_{s,bb}$	9.9	PL1 steel		San Francisco	0.282	2.8	0.660	6.5
$\Delta @ \varepsilon_{c,u}$	10.1	PL2 conc		Seattle	0.163	1.6	0.517	5.1
$\Delta @ 0.8 * \varepsilon_{s,bb}$	8.0	PL2 steel		Juneau	0.122	1.2	0.398	3.9
		New Madrid			0.048	0.5	0.903	8.9

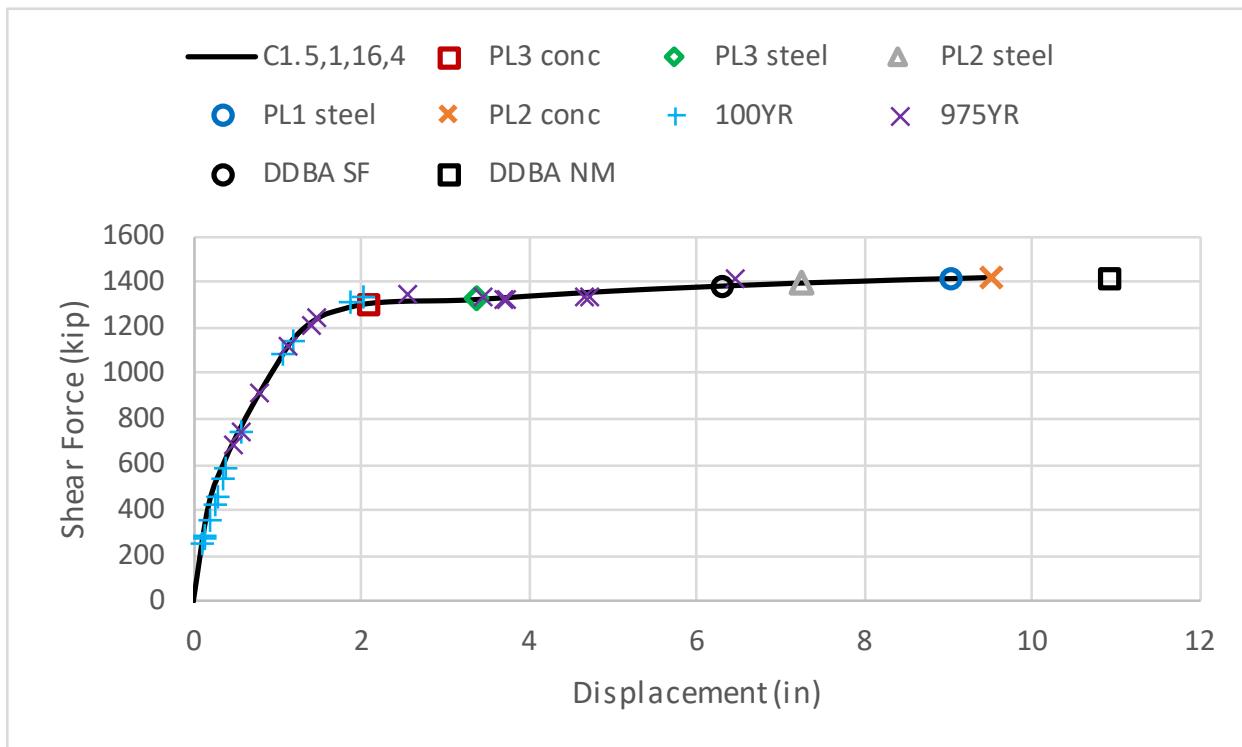


C1.5,0.7,16,6							
A _{st} /A _g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 10.2 in					
P/f' _{ce} A _g	0.16	New Madrid 17.4 in					
L/D	6						
Strain Limits				City	S _a 100YR	Δ 100YR	S _a 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.015	0.2	0.082	1.4
ε _{s,rcw}	0.01	PL3 steel	New York City	0.012	0.2	0.052	0.9
ε _{s,bb}	0.0234	PL1 steel	Boston	0.013	0.2	0.059	1.0
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.026	0.4	0.121	2.0
0.8*ε _{s,bb}	0.0187	PL2 steel	Paducah	0.030	0.5	0.272	4.5
			St Louis	0.020	0.3	0.158	2.6
Limit State Disp. (in)			Helena	0.041	0.7	0.149	2.5
Δ @ ε _{c,cc}	4.0	PL3 conc	Salt Lake City	0.059	1.0	0.402	6.6
Δ @ ε _{s,rcw}	6.1	PL3 steel	Los Angeles	0.199	3.3	0.501	8.3
Δ @ ε _{s,bb}	12.8	PL1 steel	San Francisco	0.217	3.6	0.508	8.4
Δ @ ε _{c,u}	13.4	PL2 conc	Seattle	0.126	2.1	0.398	6.6
Δ @ 0.8*ε _{s,bb}	10.4	PL2 steel	Juneau	0.094	1.6	0.307	5.1
			New Madrid	0.037	0.6	0.695	11.5



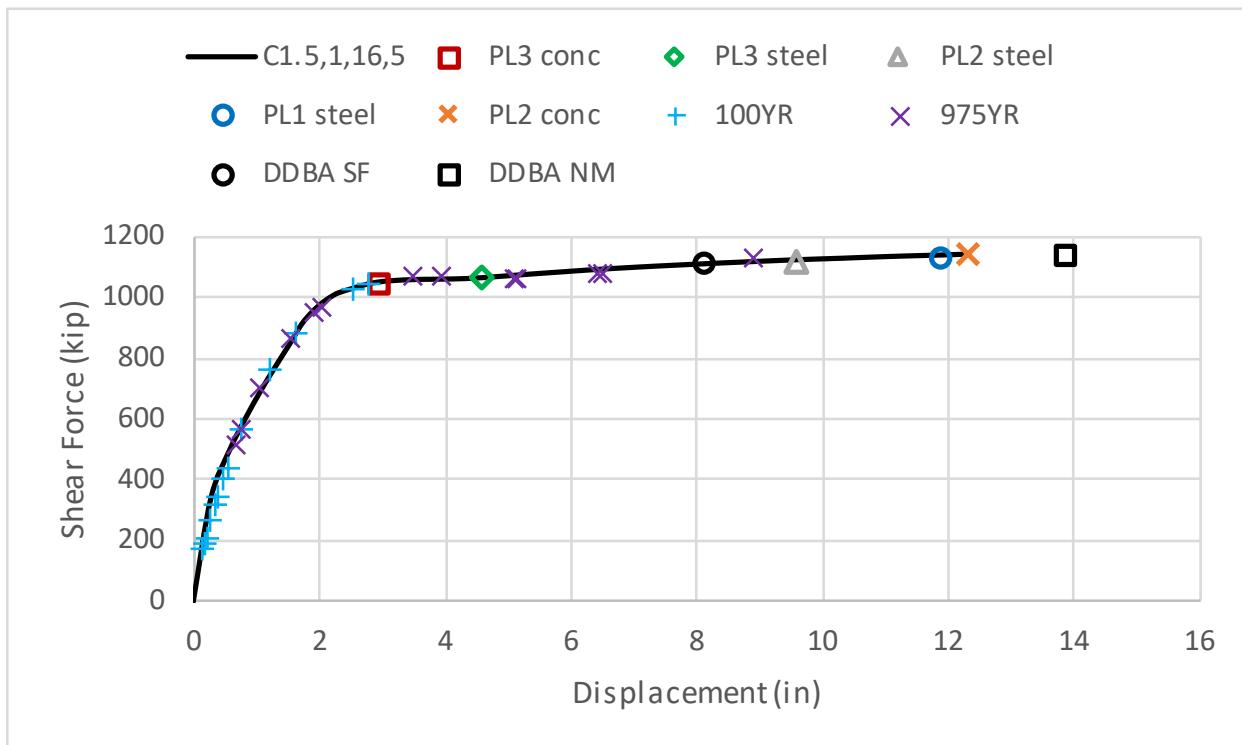
C1.5,1,16,4

A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco	6.3	in			
$P/f'_{ce}A_g$	0.16	New Madrid	10.9	in			
L/D	4						
Strain Limits			(g)	(in)	(g)	(in)	
		City	S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR	
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.027	0.1	0.147	0.8
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.021	0.1	0.092	0.5
$\epsilon_{s,bb}$	0.0284	PL1 steel	Boston	0.024	0.1	0.105	0.5
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.047	0.2	0.215	1.1
$0.8^*\epsilon_{s,bb}$	0.0228	PL2 steel	Paducah	0.053	0.3	0.484	2.5
			St Louis	0.036	0.2	0.282	1.5
Limit State Disp. (in)			Helena	0.073	0.4	0.266	1.4
$\Delta @ \epsilon_{c,cc}$	2.1	PL3 conc	Salt Lake City	0.106	0.6	0.715	3.7
$\Delta @ \epsilon_{s,rcw}$	3.4	PL3 steel	Los Angeles	0.354	1.9	0.891	4.7
$\Delta @ \epsilon_{s,bb}$	9.0	PL1 steel	San Francisco	0.387	2.0	0.905	4.7
$\Delta @ \epsilon_{c,u}$	9.5	PL2 conc	Seattle	0.224	1.2	0.708	3.7
$\Delta @ 0.8^*\epsilon_{s,bb}$	7.3	PL2 steel	Juneau	0.164	1.1	0.524	3.5
			New Madrid	0.065	0.3	1.237	6.5



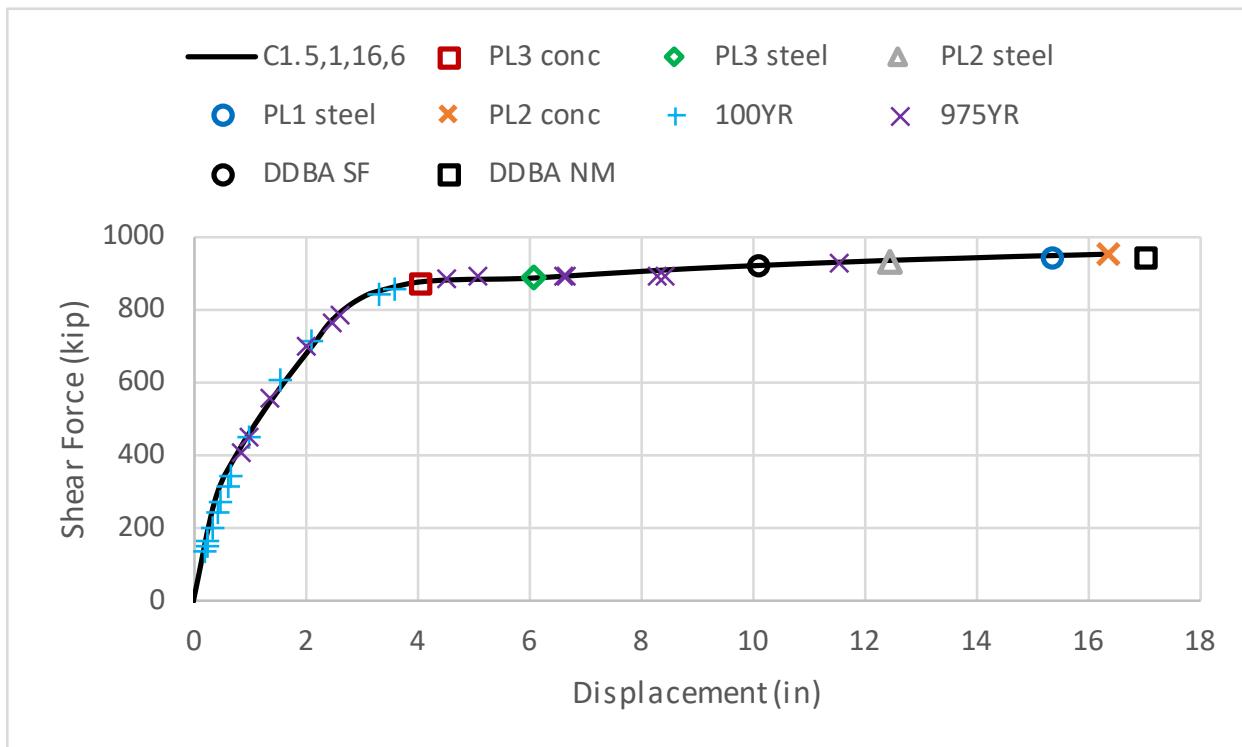
C1.5,1,16,5

A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco	8.1	in			
$P/f'_{ce}A_g$	0.16	New Madrid	13.85	in			
L/D	5						
Strain Limits			(g)	(in)	(g)	(in)	
		City	S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR	
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.019	0.2	0.107	1.1
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.015	0.1	0.067	0.7
$\epsilon_{s,bb}$	0.0284	PL1 steel	Boston	0.017	0.2	0.077	0.8
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.034	0.3	0.157	1.5
$0.8^*\epsilon_{s,bb}$	0.0228	PL2 steel	Paducah	0.038	0.4	0.353	3.5
			St Louis	0.027	0.3	0.205	2.0
Limit State Disp. (in)			Helena	0.053	0.5	0.194	1.9
$\Delta @ \epsilon_{c,cc}$	2.9	PL3 conc	Salt Lake City	0.077	0.8	0.520	5.1
$\Delta @ \epsilon_{s,rcw}$	4.6	PL3 steel	Los Angeles	0.258	2.5	0.649	6.4
$\Delta @ \epsilon_{s,bb}$	11.8	PL1 steel	San Francisco	0.282	2.8	0.659	6.5
$\Delta @ \epsilon_{c,u}$	12.3	PL2 conc	Seattle	0.163	1.6	0.516	5.1
$\Delta @ 0.8^*\epsilon_{s,bb}$	9.6	PL2 steel	Juneau	0.122	1.2	0.397	3.9
			New Madrid	0.048	0.5	0.901	8.9

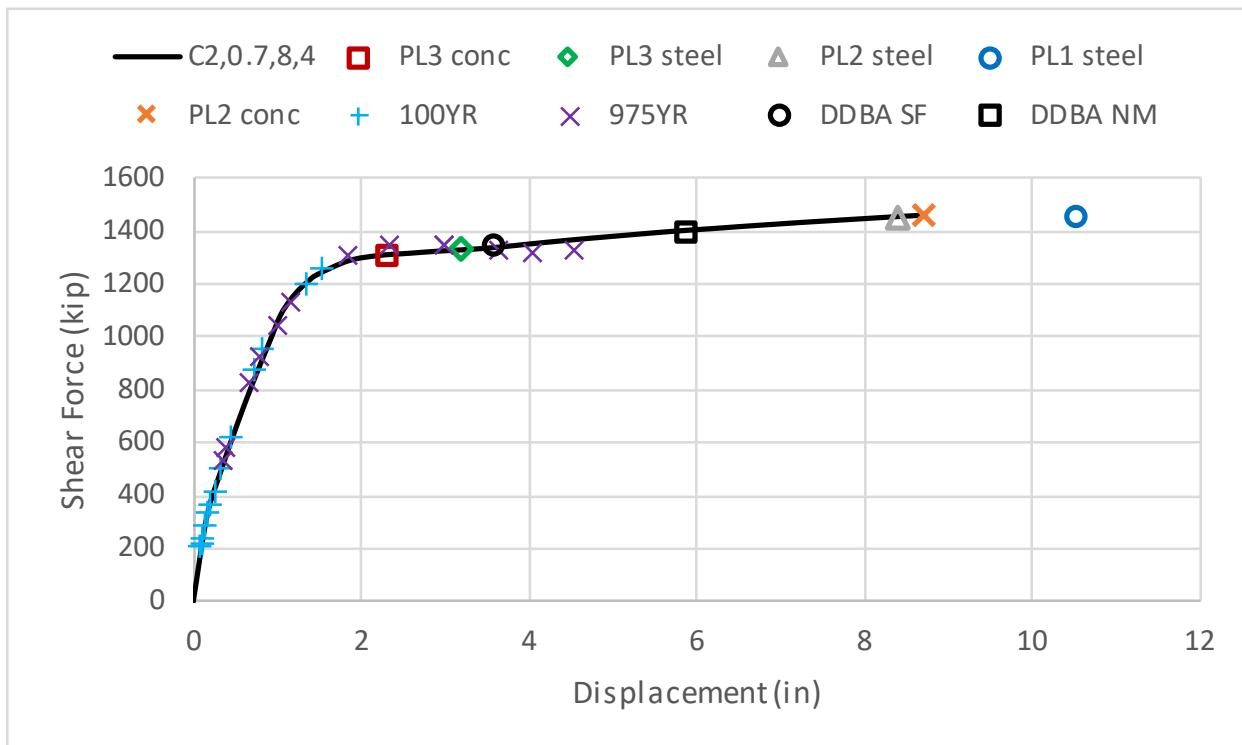


C1.5,1,16,6

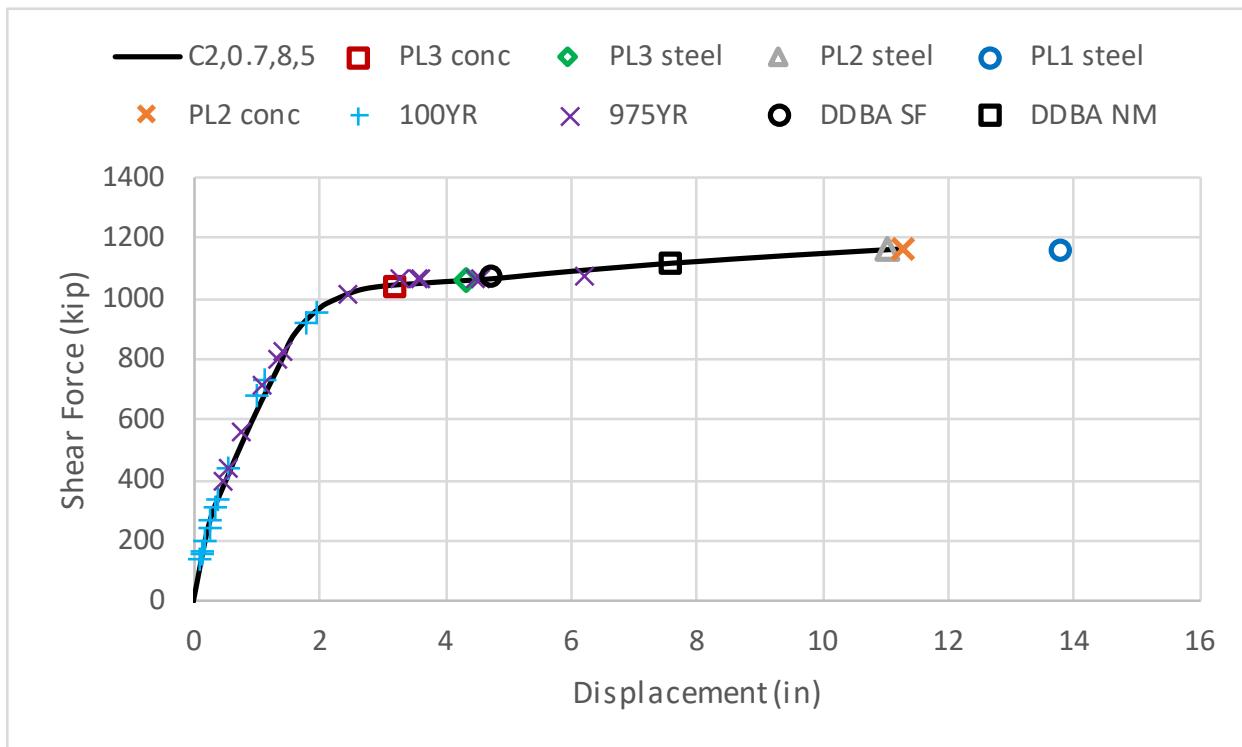
A_{st}/A_g	0.015	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco			10.1	in	
$P/f'_{ce}A_g$	0.16	New Madrid			17	in	
L/D	6						
Strain Limits			City	S_a 100YR	(g)	(in)	(g)
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.015	0.2	0.082	1.4
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.012	0.2	0.052	0.9
$\varepsilon_{s,bb}$	0.0284	PL1 steel	Boston	0.013	0.2	0.059	1.0
$\varepsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.026	0.4	0.121	2.0
$0.8^*\varepsilon_{s,bb}$	0.0228	PL2 steel	Paducah	0.030	0.5	0.272	4.5
			St Louis	0.020	0.3	0.158	2.6
Limit State Disp. (in)			Helena	0.041	0.7	0.149	2.5
$\Delta @ \varepsilon_{c,cc}$	4.0	PL3 conc	Salt Lake City	0.059	1.0	0.401	6.7
$\Delta @ \varepsilon_{s,rcw}$	6.1	PL3 steel	Los Angeles	0.199	3.3	0.500	8.3
$\Delta @ \varepsilon_{s,bb}$	15.3	PL1 steel	San Francisco	0.217	3.6	0.507	8.4
$\Delta @ \varepsilon_{c,u}$	16.4	PL2 conc	Seattle	0.126	2.1	0.397	6.6
$\Delta @ 0.8^*\varepsilon_{s,bb}$	12.4	PL2 steel	Juneau	0.094	1.6	0.306	5.1
			New Madrid	0.037	0.6	0.694	11.5



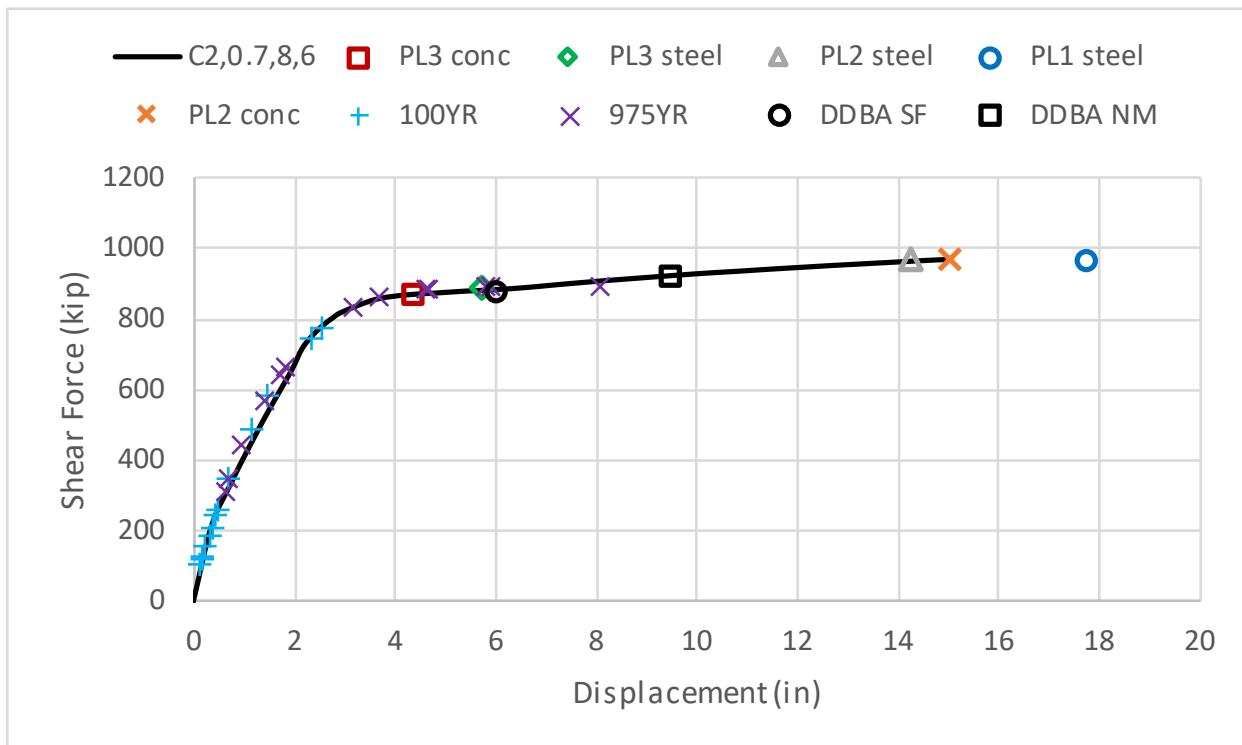
C2,0.7,8,4							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 3.55 in					
P/f' _{ce} A _g	0.08	New Madrid 5.85 in					
L/D	4						
Strain Limits				City	(g)	(in)	(g)
					S _a 100YR	Δ 100YR	S _a 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.038	0.1	0.209	0.7
ε _{s,rcw}	0.01	PL3 steel	New York City	0.030	0.1	0.132	0.3
ε _{s,bb}	0.0346	PL1 steel	Boston	0.034	0.1	0.150	0.4
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.067	0.2	0.307	0.8
0.8*ε _{s,bb}	0.0277	PL2 steel	Paducah	0.075	0.2	0.691	1.8
			St Louis	0.052	0.1	0.402	1.2
Limit State Disp. (in)				Helena	0.104	0.3	0.379
Δ @ ε _{c,cc}	2.3	PL3 conc	Salt Lake City	0.151	0.4	1.020	3.0
Δ @ ε _{s,rcw}	3.2	PL3 steel	Los Angeles	0.506	1.4	1.271	3.6
Δ @ ε _{s,bb}	10.5	PL1 steel	San Francisco	0.552	1.5	1.284	4.0
Δ @ ε _{c,u}	8.7	PL2 conc	Seattle	0.319	0.8	1.011	3.0
Δ @ 0.8*ε _{s,bb}	8.4	PL2 steel	Juneau	0.164	0.7	0.524	2.3
			New Madrid	0.093	0.2	1.765	4.5



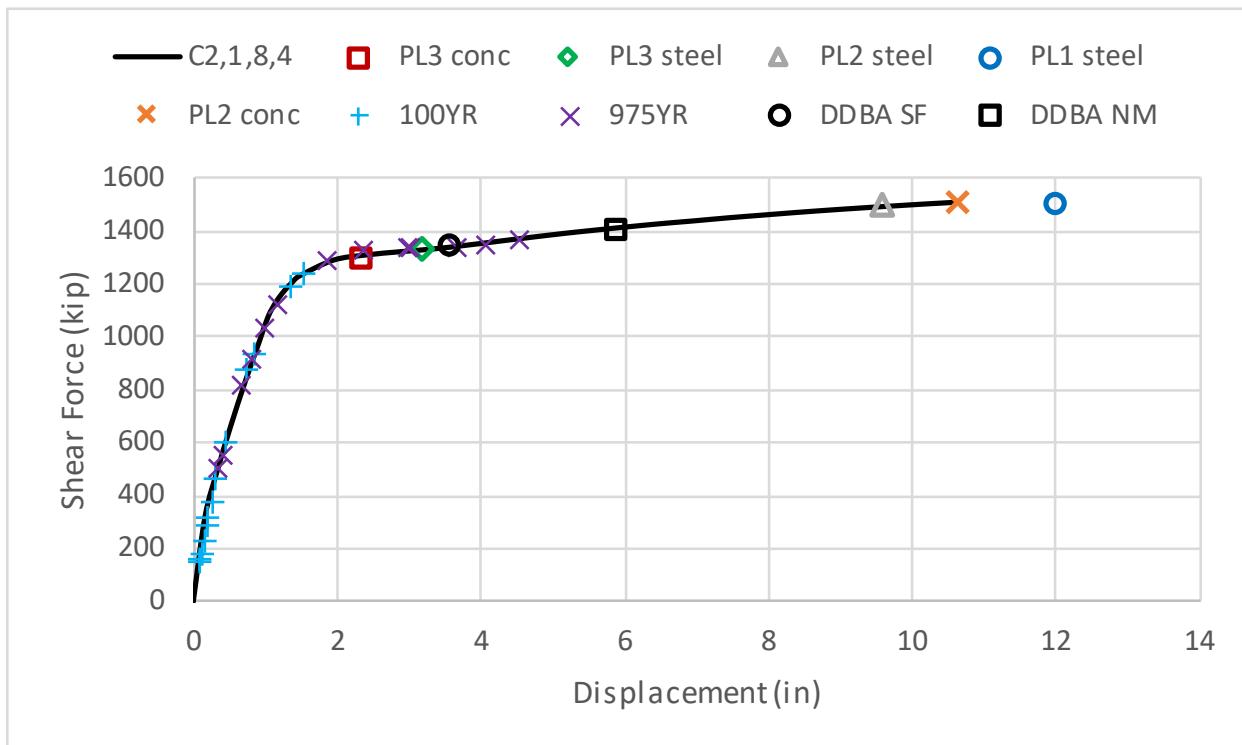
C2,0.7,8,5		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.02	San Francisco		4.7	in		
4A _h /D's	0.007	New Madrid		7.55	in		
P/f' _{ce} A _g	0.08						
L/D	5						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.028	0.1	0.153	0.7
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.022	0.1	0.096	0.5
$\varepsilon_{s,bb}$	0.0346	PL1 steel	Boston	0.025	0.1	0.109	0.5
$\varepsilon_{c,u}$	-0.0169	PL2 conc	Knoxville	0.049	0.2	0.224	1.1
0.8* $\varepsilon_{s,bb}$	0.0277	PL2 steel	Paducah	0.055	0.3	0.505	2.4
			St Louis	0.038	0.2	0.293	1.4
Limit State Disp. (in)		Helena	0.076	0.4	0.277	1.3	
$\Delta @ \varepsilon_{c,cc}$	3.2	PL3 conc	Salt Lake City	0.110	0.5	0.744	3.6
$\Delta @ \varepsilon_{s,rcw}$	4.3	PL3 steel	Los Angeles	0.369	1.8	0.928	4.5
$\Delta @ \varepsilon_{s,bb}$	13.7	PL1 steel	San Francisco	0.403	1.9	0.942	4.5
$\Delta @ \varepsilon_{c,u}$	11.3	PL2 conc	Seattle	0.233	1.1	0.738	3.6
$\Delta @ 0.8*\varepsilon_{s,bb}$	11.0	PL2 steel	Juneau	0.164	1.0	0.524	3.3
			New Madrid	0.068	0.3	1.288	6.2



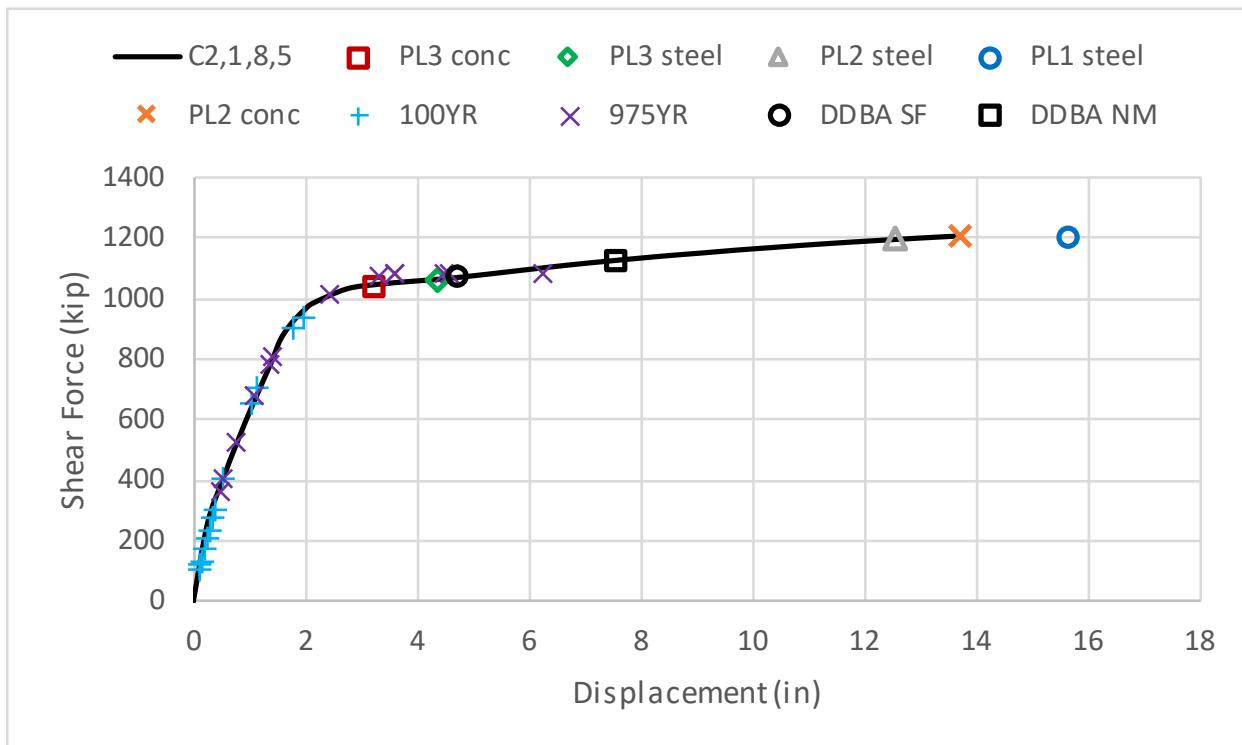
C2,0.7,8,6								
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ						
4A _h /D's	0.007	San Francisco 6 in						
P/f' _{ce} A _g	0.08	New Madrid 9.45 in						
L/D	6							
Strain Limits		City		(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$		Charleston		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR	
$\varepsilon_{s,rcw}$	0.01	PL3 steel		New York City	0.017	0.1	0.074	0.6
$\varepsilon_{s,bb}$	0.0346	Boston			0.019	0.2	0.084	0.7
$\varepsilon_{c,u}$	-0.0169	Knoxville			0.037	0.3	0.173	1.4
0.8* $\varepsilon_{s,bb}$	0.0277	Paducah			0.042	0.3	0.389	3.2
		St Louis			0.029	0.2	0.226	1.8
Limit State Disp. (in)				Helena	0.058	0.5	0.213	1.7
$\Delta @ \varepsilon_{c,cc}$	4.3	Salt Lake City			0.085	0.7	0.573	4.7
$\Delta @ \varepsilon_{s,rcw}$	5.7	Los Angeles			0.284	2.3	0.715	5.8
$\Delta @ \varepsilon_{s,bb}$	17.7	San Francisco			0.311	2.5	0.726	5.9
$\Delta @ \varepsilon_{c,u}$	15.0	Seattle			0.180	1.5	0.569	4.6
$\Delta @ 0.8*\varepsilon_{s,bb}$	14.3	Juneau			0.135	1.1	0.438	3.7
		New Madrid			0.052	0.4	0.993	8.1



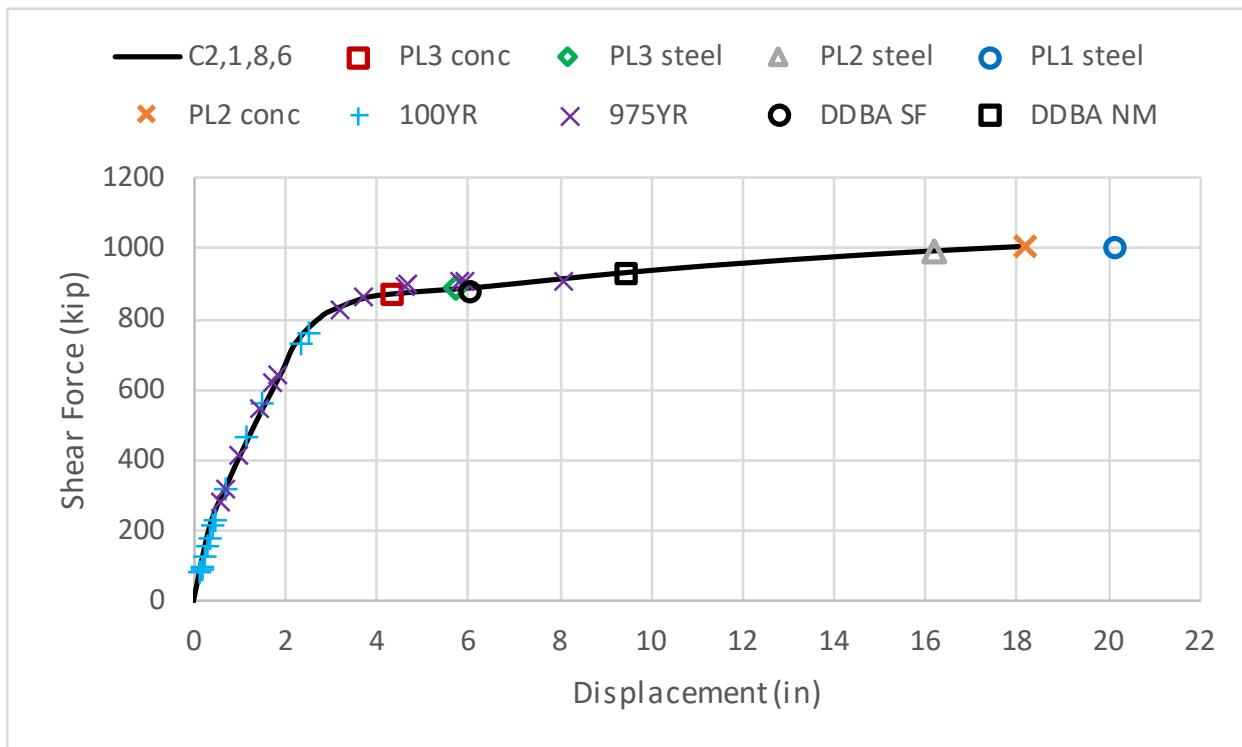
C2,1,8,4		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.02	San Francisco		3.55	in		
4A _h /D's	0.01	New Madrid		5.85	in		
P/f' _{ce} A _g	0.08						
L/D	4						
Strain Limits		City	S _a 100YR	(g)	(in)	S _a 975YR	(g)
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.038	0.1	0.209	0.7
ε _{s,rcw}	0.01	PL3 steel	New York City	0.029	0.1	0.131	0.3
ε _{s,bb}	0.0397	PL1 steel	Boston	0.034	0.1	0.150	0.4
ε _{c,u}	-0.0206	PL2 conc	Knoxville	0.066	0.2	0.306	0.8
0.8*ε _{s,bb}	0.0317	PL2 steel	Paducah	0.075	0.2	0.690	1.8
			St Louis	0.052	0.1	0.401	1.2
Limit State Disp. (in)		Helena	0.104	0.3	0.379	1.0	
Δ @ ε _{c,cc}	2.3	PL3 conc	Salt Lake City	0.150	0.4	1.017	3.0
Δ @ ε _{s,rcw}	3.2	PL3 steel	Los Angeles	0.505	1.4	1.268	3.6
Δ @ ε _{s,bb}	12.0	PL1 steel	San Francisco	0.551	1.5	1.284	4.1
Δ @ ε _{c,u}	10.6	PL2 conc	Seattle	0.319	0.8	1.009	3.0
Δ @ 0.8*ε _{s,bb}	9.6	PL2 steel	Juneau	0.164	0.7	0.524	2.4
			New Madrid	0.093	0.2	1.761	4.5



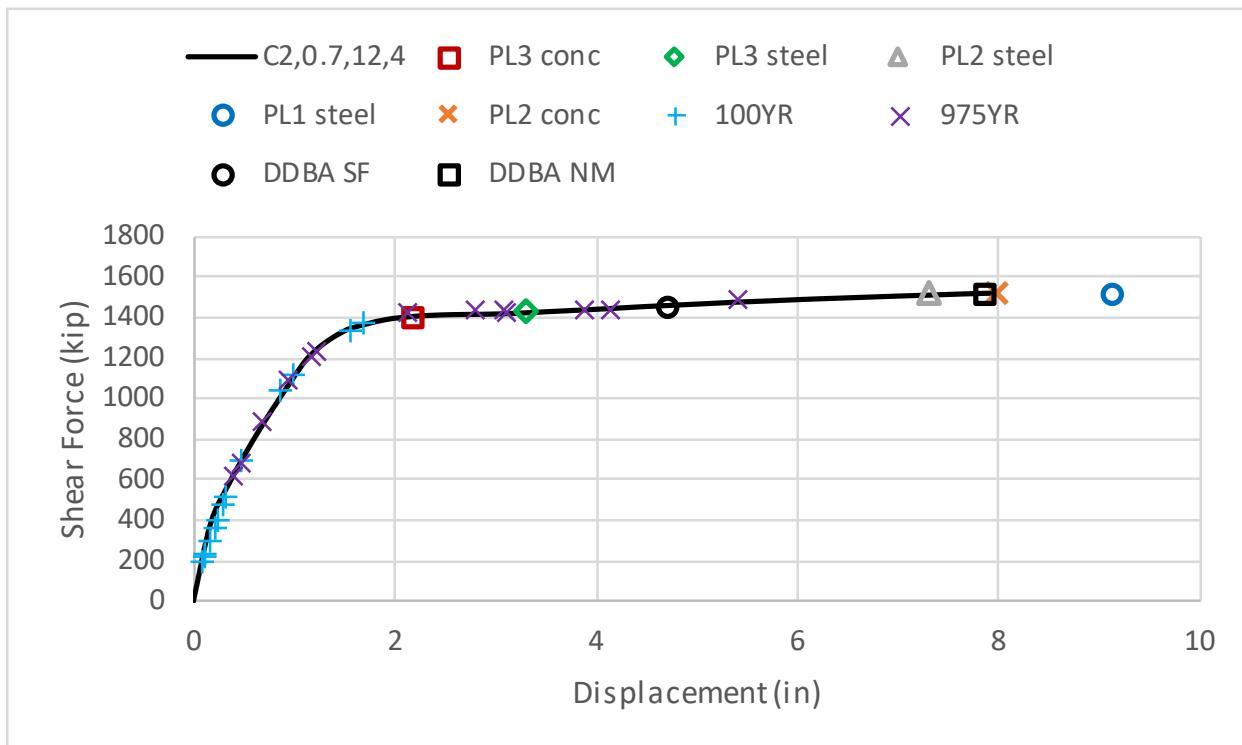
C2,1,8,5		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.02	San Francisco		4.7	in		
4A _h /D's	0.01	New Madrid		7.5	in		
P/f' _{ce} A _g	0.08						
L/D	5						
Strain Limits		City	S _a 100YR	(g)	(in)	S _a 975YR	(g)
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.028	0.1	0.152	0.7
ε _{s,rcw}	0.01	PL3 steel	New York City	0.021	0.1	0.096	0.5
ε _{s,bb}	0.0397	PL1 steel	Boston	0.025	0.1	0.109	0.5
ε _{c,u}	-0.0206	PL2 conc	Knoxville	0.048	0.2	0.223	1.1
0.8*ε _{s,bb}	0.0317	PL2 steel	Paducah	0.055	0.3	0.503	2.4
			St Louis	0.038	0.2	0.293	1.4
Limit State Disp. (in)		Helena	0.076	0.4	0.276	1.3	
Δ @ ε _{c,cc}	3.2	PL3 conc	Salt Lake City	0.110	0.5	0.743	3.6
Δ @ ε _{s,rcw}	4.3	PL3 steel	Los Angeles	0.368	1.8	0.926	4.5
Δ @ ε _{s,bb}	15.6	PL1 steel	San Francisco	0.402	1.9	0.940	4.6
Δ @ ε _{c,u}	13.7	PL2 conc	Seattle	0.233	1.1	0.736	3.6
Δ @ 0.8*ε _{s,bb}	12.6	PL2 steel	Juneau	0.164	1.0	0.524	3.3
		New Madrid	0.068	0.3	1.285	6.2	



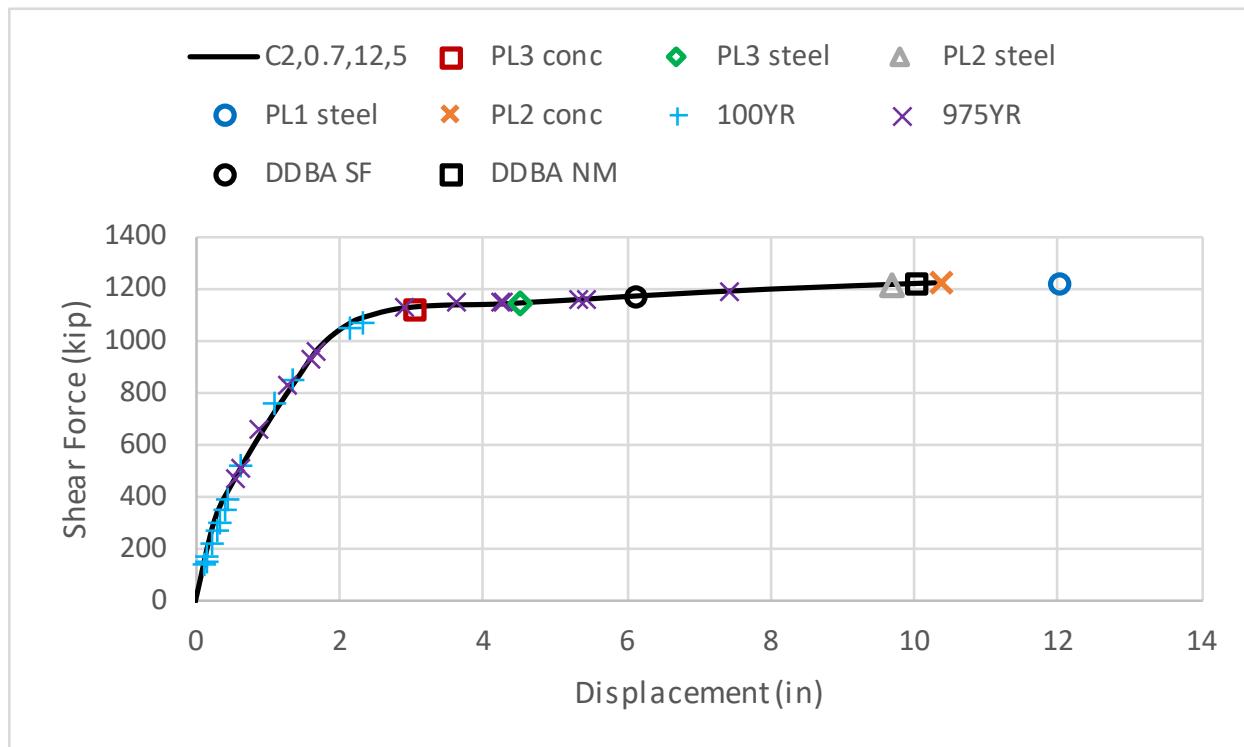
C2,1,8,6		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.02	San Francisco	6	in			
4A _h /D's	0.01	New Madrid	9.4	in			
P/f' _{ce} A _g	0.08						
L/D	6						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.021	0.2	0.117	1.0
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.017	0.1	0.074	0.6
$\epsilon_{s,bb}$	0.0397	PL1 steel	Boston	0.019	0.2	0.084	0.7
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.037	0.3	0.172	1.4
0.8* $\epsilon_{s,bb}$	0.0317	PL2 steel	Paducah	0.042	0.3	0.388	3.2
			St Louis	0.029	0.2	0.225	1.8
Limit State Disp. (in)		Helena	0.058	0.5	0.213	1.7	
$\Delta @ \epsilon_{c,cc}$	4.3	PL3 conc	Salt Lake City	0.085	0.7	0.572	4.7
$\Delta @ \epsilon_{s,rcw}$	5.7	PL3 steel	Los Angeles	0.284	2.3	0.713	5.8
$\Delta @ \epsilon_{s,bb}$	20.1	PL1 steel	San Francisco	0.310	2.5	0.724	5.9
$\Delta @ \epsilon_{c,u}$	18.2	PL2 conc	Seattle	0.179	1.5	0.567	4.6
$\Delta @ 0.8*\epsilon_{s,bb}$	16.2	PL2 steel	Juneau	0.134	1.1	0.437	3.7
			New Madrid	0.052	0.4	0.990	8.1



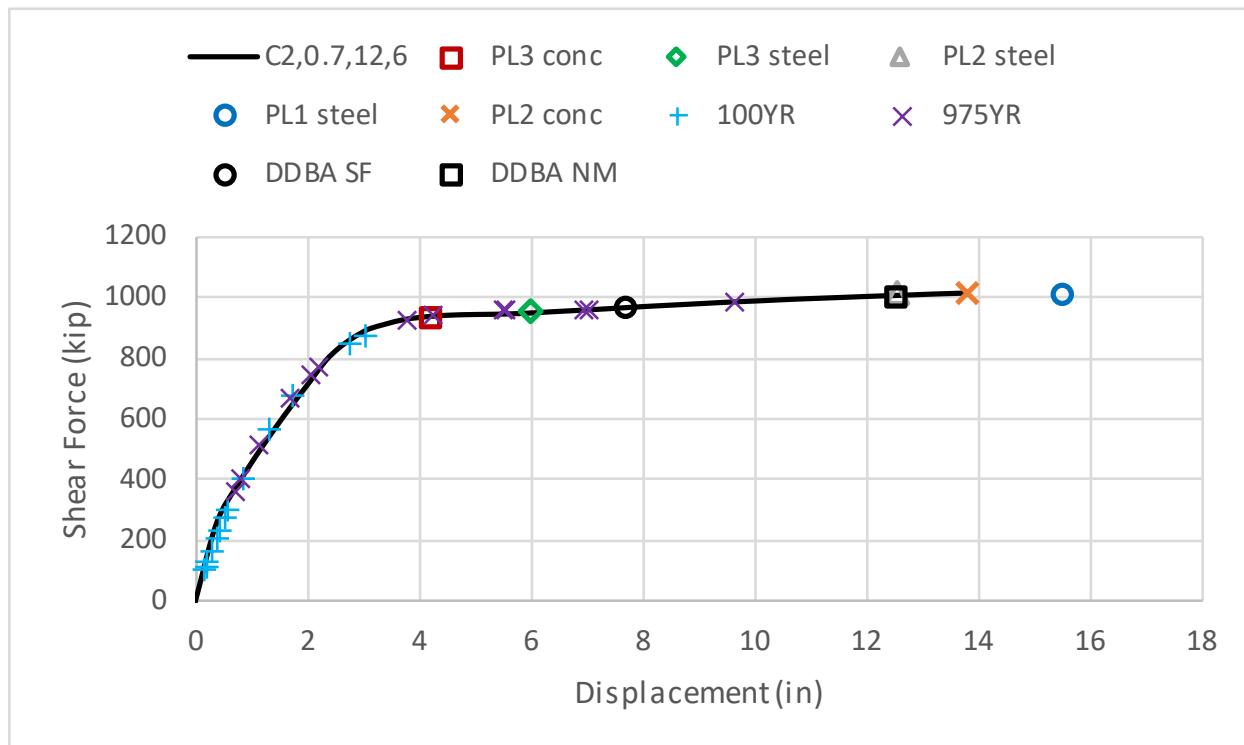
C2,0.7,12,4							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 4.7 in					
P/f' _{ce} A _g	0.12	New Madrid 7.85 in					
L/D	4						
Strain Limits				(g)	(in)	(g)	(in)
		City		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.032	0.1	0.176	0.7
ε _{s,rcw}	0.01	PL3 steel	New York City	0.025	0.1	0.111	0.4
ε _{s,bb}	0.0290	PL1 steel	Boston	0.028	0.1	0.126	0.5
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.056	0.2	0.258	0.9
0.8*ε _{s,bb}	0.0232	PL2 steel	Paducah	0.063	0.2	0.581	2.1
			St Louis	0.044	0.2	0.338	1.2
Limit State Disp. (in)			Helena	0.087	0.3	0.319	1.2
Δ @ ε _{c,cc}	2.2	PL3 conc	Salt Lake City	0.127	0.5	0.857	3.1
Δ @ ε _{s,rcw}	3.3	PL3 steel	Los Angeles	0.425	1.5	1.068	3.9
Δ @ ε _{s,bb}	9.1	PL1 steel	San Francisco	0.464	1.7	1.085	4.1
Δ @ ε _{c,u}	8.0	PL2 conc	Seattle	0.268	1.0	0.850	3.1
Δ @ 0.8*ε _{s,bb}	7.3	PL2 steel	Juneau	0.164	0.9	0.524	2.8
			New Madrid	0.078	0.3	1.483	5.4



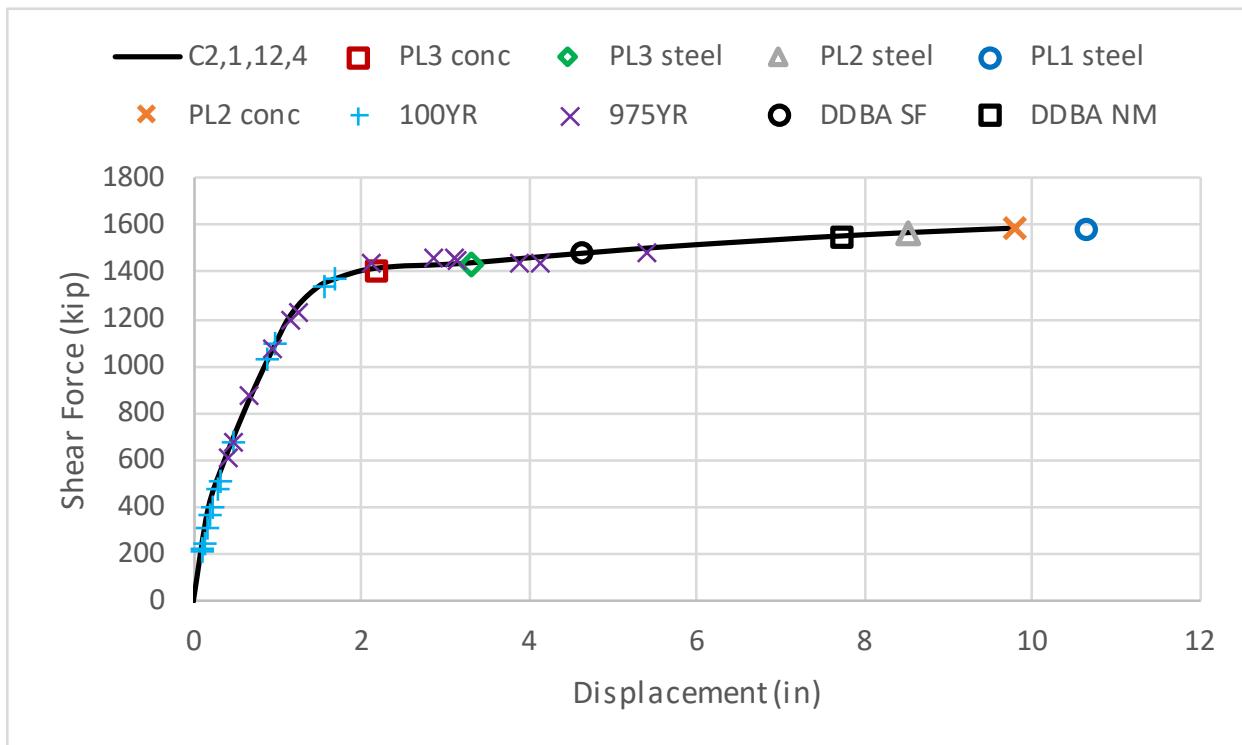
C2,0.7,12,5							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 6.1 in					
P/f' _{ce} A _g	0.12	New Madrid 10 in					
L/D	5						
Strain Limits				City	(g)	(in)	(g)
					S _a 100YR	Δ 100YR	S _a 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.023	0.2	0.128	0.9
ε _{s,rcw}	0.01	PL3 steel	New York City	0.018	0.1	0.081	0.6
ε _{s,bb}	0.0290	PL1 steel	Boston	0.021	0.1	0.092	0.6
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.041	0.3	0.188	1.3
0.8*ε _{s,bb}	0.0232	PL2 steel	Paducah	0.046	0.3	0.423	2.9
			St Louis	0.032	0.2	0.246	1.7
Limit State Disp. (in)				Helena	0.064	0.4	0.232
Δ @ ε _{c,cc}	3.0	PL3 conc	Salt Lake City	0.092	0.6	0.624	4.3
Δ @ ε _{s,rcw}	4.5	PL3 steel	Los Angeles	0.310	2.1	0.778	5.3
Δ @ ε _{s,bb}	12.0	PL1 steel	San Francisco	0.338	2.3	0.790	5.4
Δ @ ε _{c,u}	10.4	PL2 conc	Seattle	0.196	1.3	0.619	4.2
Δ @ 0.8*ε _{s,bb}	9.7	PL2 steel	Juneau	0.147	1.1	0.477	3.6
			New Madrid	0.057	0.4	1.081	7.4



C2,0.7,12,6							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco	7.65	in			
P/f' _{ce} A _g	0.12	New Madrid	12.5	in			
L/D	6						
Strain Limits		City		(g)	(in)	(g)	(in)
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.018	0.2	0.099	1.1
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.014	0.2	0.062	0.7
$\varepsilon_{s,bb}$	0.0290	PL1 steel	Boston	0.016	0.2	0.071	0.8
$\varepsilon_{c,u}$	-0.0169	PL2 conc	Knoxville	0.031	0.4	0.145	1.7
0.8* $\varepsilon_{s,bb}$	0.0232	PL2 steel	Paducah	0.036	0.4	0.326	3.8
			St Louis	0.025	0.3	0.189	2.2
Limit State Disp. (in)			Helena	0.049	0.6	0.179	2.1
$\Delta @ \varepsilon_{c,cc}$	4.1	PL3 conc	Salt Lake City	0.071	0.8	0.481	5.6
$\Delta @ \varepsilon_{s,rcw}$	6.0	PL3 steel	Los Angeles	0.238	2.8	0.599	6.9
$\Delta @ \varepsilon_{s,bb}$	15.5	PL1 steel	San Francisco	0.260	3.0	0.609	7.0
$\Delta @ \varepsilon_{c,u}$	13.8	PL2 conc	Seattle	0.151	1.7	0.477	5.5
$\Delta @ 0.8*\varepsilon_{s,bb}$	12.5	PL2 steel	Juneau	0.113	1.3	0.367	4.2
			New Madrid	0.044	0.5	0.832	9.6

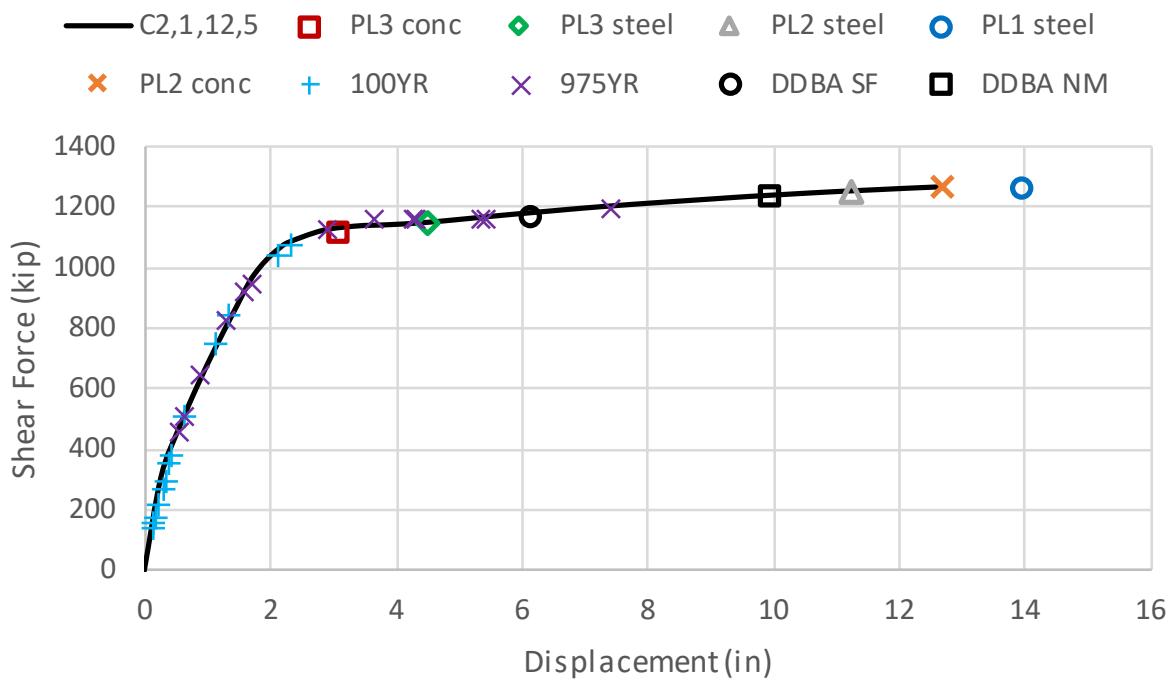


C2,1,12,4							
A_{st}/A_g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 4.62 in					
$P/f'_{ce}A_g$	0.12	New Madrid 7.7 in					
L/D	4						
Strain Limits				(g)	(in)	(g)	(in)
		City		S_a 100YR	Δ 100YR	S_a 975YR	Δ 975YR
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.032	0.1	0.176	0.7
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.025	0.1	0.110	0.4
$\epsilon_{s,bb}$	0.0341	PL1 steel	Boston	0.028	0.1	0.126	0.5
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.056	0.2	0.257	0.9
$0.8^*\epsilon_{s,bb}$	0.0272	PL2 steel	Paducah	0.063	0.2	0.580	2.1
			St Louis	0.044	0.2	0.337	1.2
Limit State Disp. (in)			Helena	0.087	0.3	0.318	1.2
$\Delta @ \epsilon_{c,cc}$	2.2	PL3 conc	Salt Lake City	0.126	0.5	0.855	3.1
$\Delta @ \epsilon_{s,rcw}$	3.3	PL3 steel	Los Angeles	0.424	1.5	1.066	3.9
$\Delta @ \epsilon_{s,bb}$	10.6	PL1 steel	San Francisco	0.463	1.7	1.082	4.1
$\Delta @ \epsilon_{c,u}$	9.8	PL2 conc	Seattle	0.268	1.0	0.848	3.1
$\Delta @ 0.8^*\epsilon_{s,bb}$	8.5	PL2 steel	Juneau	0.164	0.9	0.524	2.8
			New Madrid	0.078	0.3	1.480	5.4

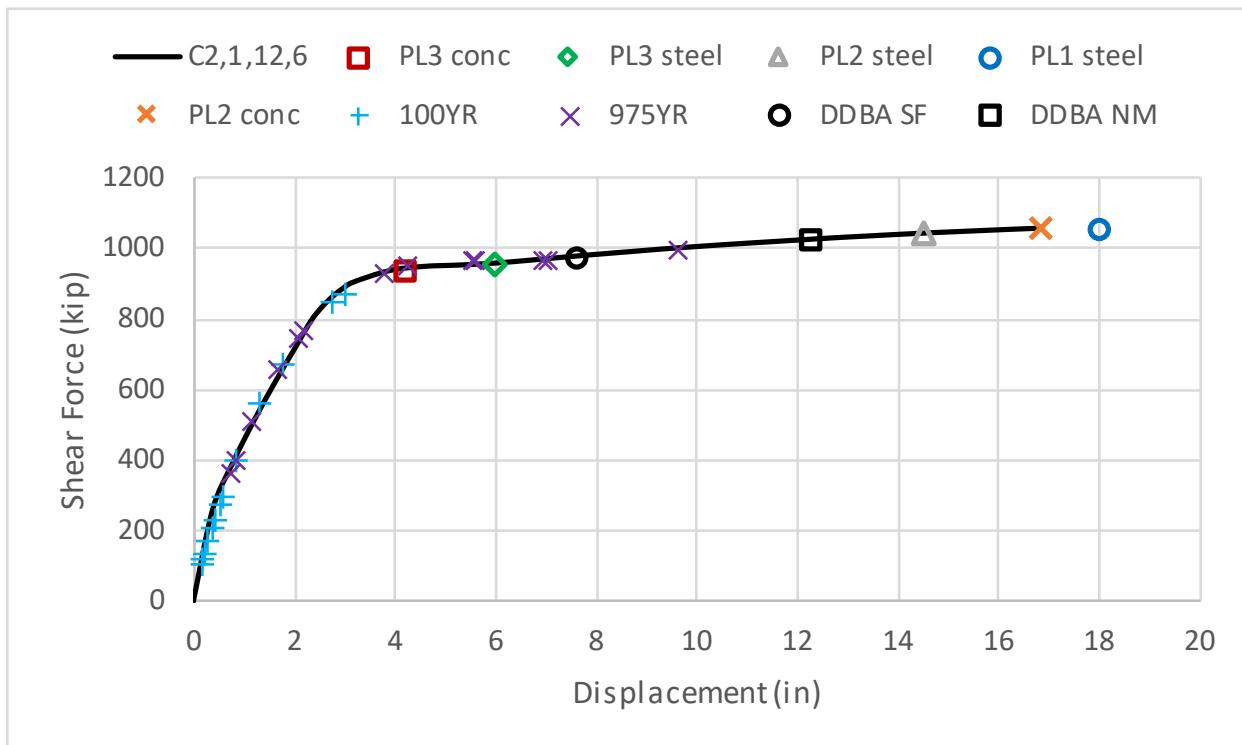


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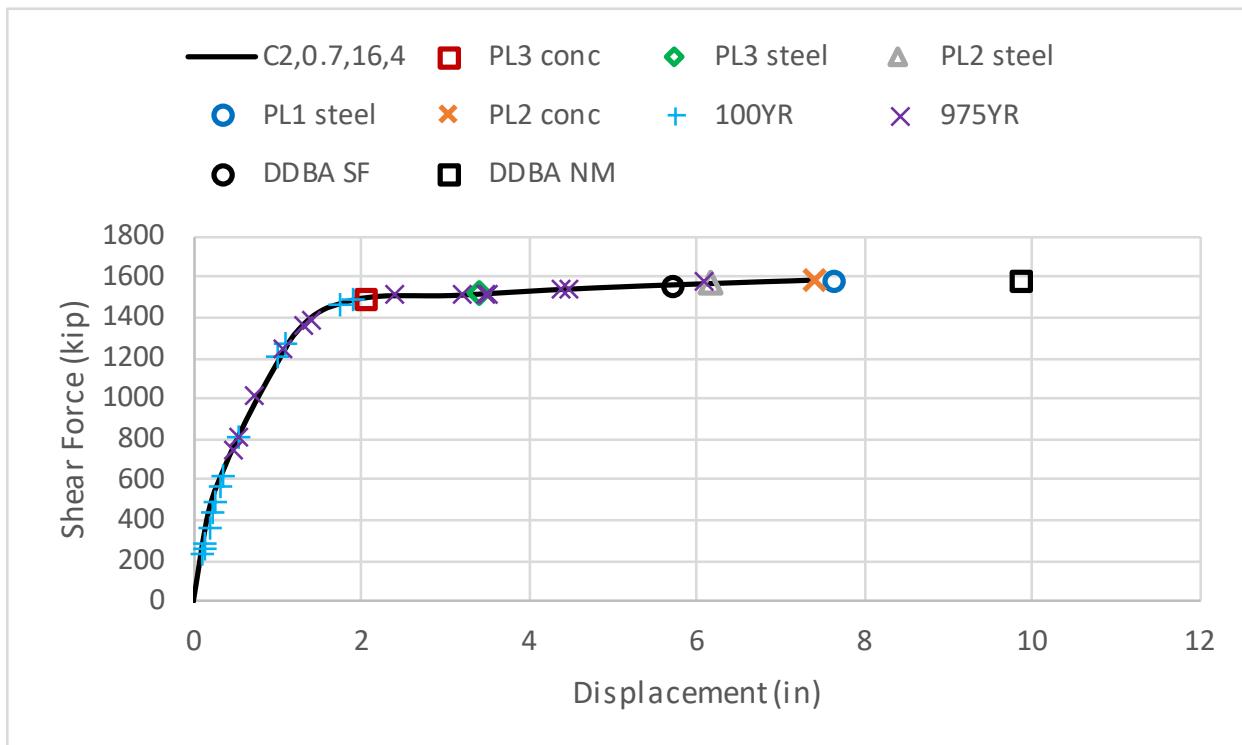
A_{st}/A_g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco	6.1	in			
$P/f'_{ce}A_g$	0.12	New Madrid	9.9	in			
L/D	5						
Strain Limits			City	(g)	(in)	(g)	(in)
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.023	0.2	0.128	0.9
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.018	0.1	0.080	0.6
$\epsilon_{s,bb}$	0.0341	PL1 steel	Boston	0.021	0.1	0.092	0.6
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.041	0.3	0.187	1.3
$0.8^*\epsilon_{s,bb}$	0.0272	PL2 steel	Paducah	0.046	0.3	0.422	2.9
			St Louis	0.032	0.2	0.245	1.7
Limit State Disp. (in)			Helena	0.064	0.4	0.232	1.6
$\Delta @ \epsilon_{c,cc}$	3.0	PL3 conc	Salt Lake City	0.092	0.6	0.623	4.3
$\Delta @ \epsilon_{s,rcw}$	4.5	PL3 steel	Los Angeles	0.309	2.1	0.776	5.3
$\Delta @ \epsilon_{s,bb}$	13.9	PL1 steel	San Francisco	0.337	2.3	0.788	5.4
$\Delta @ \epsilon_{c,u}$	12.7	PL2 conc	Seattle	0.195	1.3	0.617	4.3
$\Delta @ 0.8^*\epsilon_{s,bb}$	11.2	PL2 steel	Juneau	0.146	1.1	0.476	3.6
			New Madrid	0.057	0.4	1.078	7.4



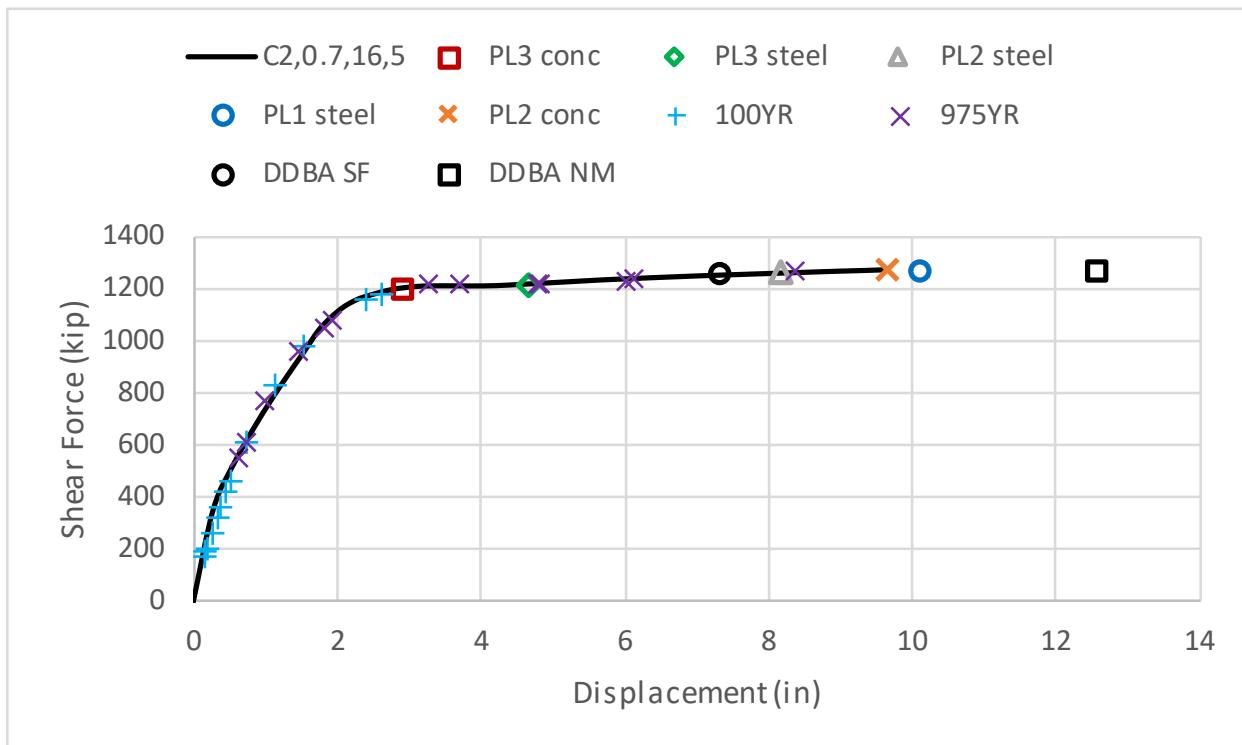
C2,1,12,6		DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
A _{st} /A _g	0.02	San Francisco	7.6	in			
4A _h /D's	0.01	New Madrid	12.25	in			
P/f' _{ce} A _g	0.12						
L/D	6						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.018	0.2	0.098	1.1
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.014	0.2	0.062	0.7
$\varepsilon_{s,bb}$	0.0341	PL1 steel	Boston	0.016	0.2	0.070	0.8
$\varepsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.031	0.4	0.144	1.7
0.8* $\varepsilon_{s,bb}$	0.0272	PL2 steel	Paducah	0.035	0.4	0.325	3.8
			St Louis	0.024	0.3	0.189	2.2
Limit State Disp. (in)		Helena	0.049	0.6	0.178	2.1	
$\Delta @ \varepsilon_{c,cc}$	4.2	PL3 conc	Salt Lake City	0.071	0.8	0.480	5.6
$\Delta @ \varepsilon_{s,rcw}$	6.0	PL3 steel	Los Angeles	0.238	2.8	0.598	6.9
$\Delta @ \varepsilon_{s,bb}$	18.0	PL1 steel	San Francisco	0.260	3.0	0.607	7.0
$\Delta @ \varepsilon_{c,u}$	16.8	PL2 conc	Seattle	0.150	1.7	0.475	5.5
$\Delta @ 0.8*\varepsilon_{s,bb}$	14.5	PL2 steel	Juneau	0.113	1.3	0.366	4.3
			New Madrid	0.044	0.5	0.830	9.6



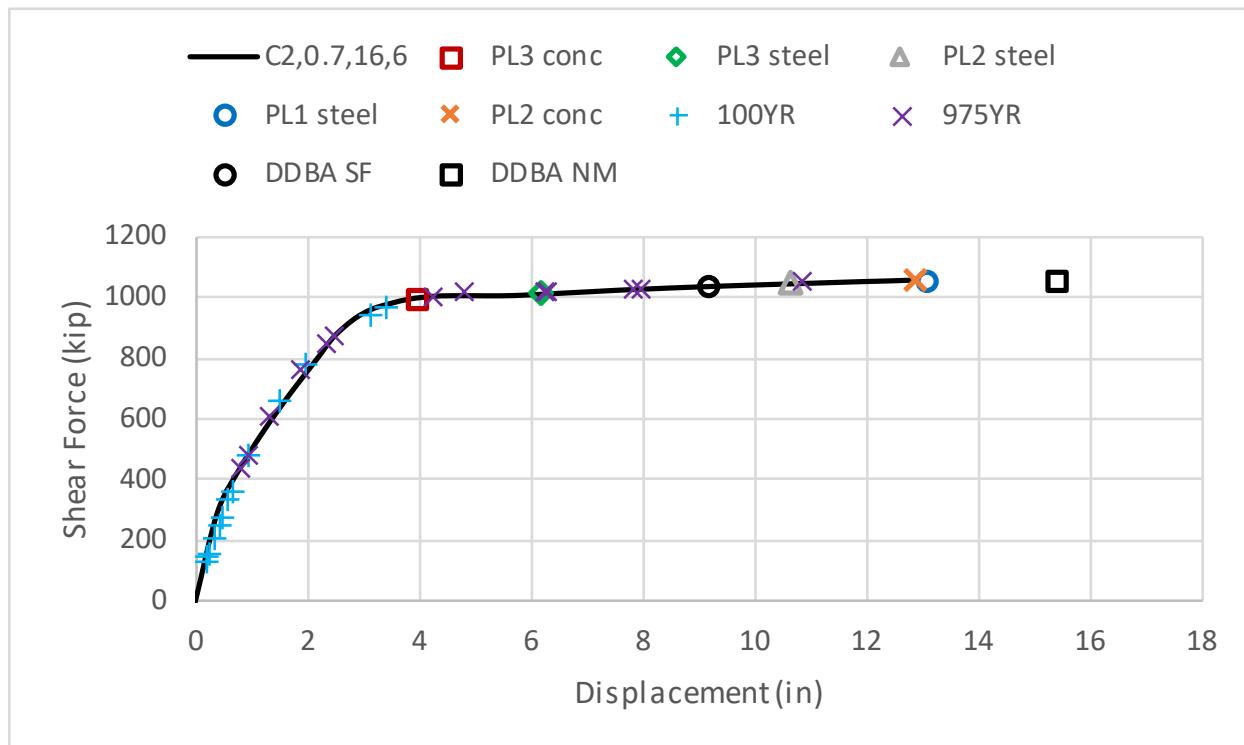
C2,0.7,16,4							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco	5.7	in			
P/f' _{ce} A _g	0.16	New Madrid	9.85	in			
L/D	4						
Strain Limits		City	(g)	(in)	(g)	(in)	
$\varepsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.028	0.1	0.156	0.7
$\varepsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.022	0.1	0.098	0.5
$\varepsilon_{s,bb}$	0.0234	PL1 steel	Boston	0.025	0.1	0.111	0.5
$\varepsilon_{c,u}$	-0.0169	PL2 conc	Knoxville	0.049	0.2	0.228	1.1
0.8* $\varepsilon_{s,bb}$	0.0187	PL2 steel	Paducah	0.056	0.3	0.514	2.4
			St Louis	0.039	0.2	0.299	1.4
Limit State Disp. (in)			Helena	0.077	0.4	0.282	1.3
$\Delta @ \varepsilon_{c,cc}$	2.0	PL3 conc	Salt Lake City	0.112	0.5	0.758	3.5
$\Delta @ \varepsilon_{s,rcw}$	3.4	PL3 steel	Los Angeles	0.376	1.7	0.945	4.4
$\Delta @ \varepsilon_{s,bb}$	7.6	PL1 steel	San Francisco	0.411	1.9	0.960	4.5
$\Delta @ \varepsilon_{c,u}$	7.4	PL2 conc	Seattle	0.238	1.1	0.752	3.5
$\Delta @ 0.8*\varepsilon_{s,bb}$	6.1	PL2 steel	Juneau	0.164	1.0	0.524	3.2
			New Madrid	0.069	0.3	1.313	6.1



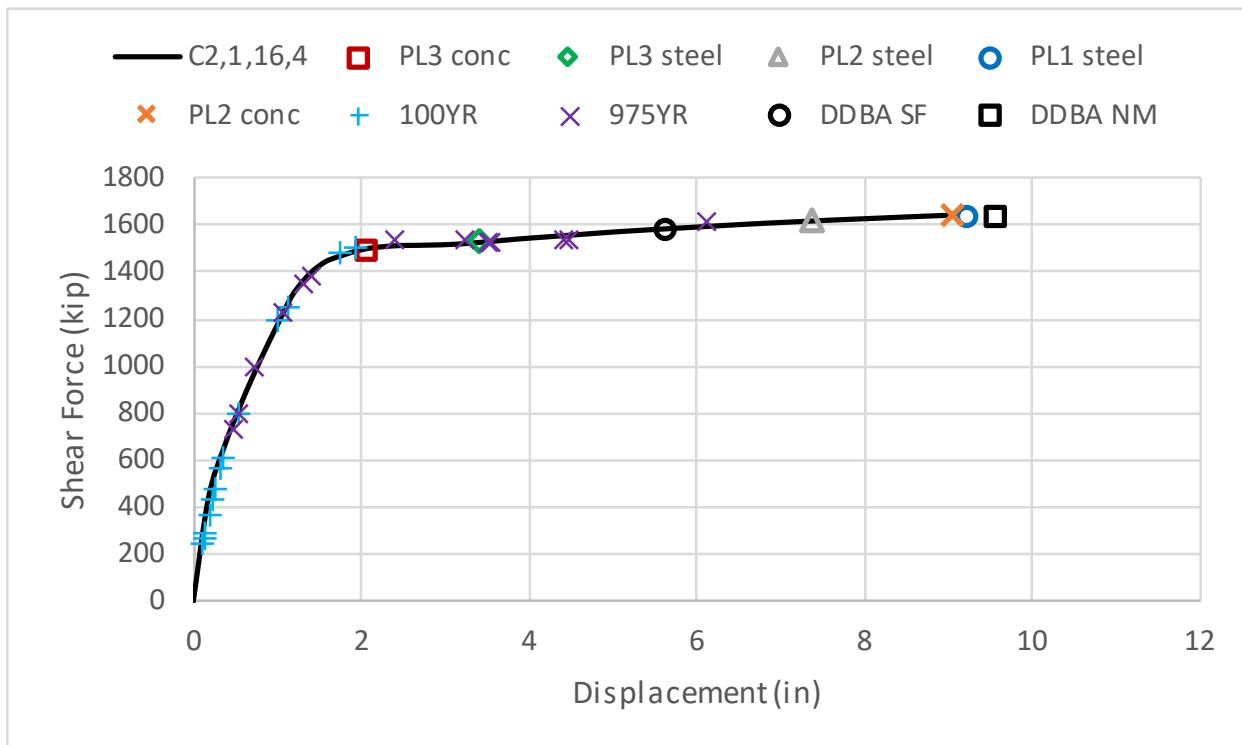
C2,0.7,16,5							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 7.3 in					
P/f' _{ce} A _g	0.16	New Madrid 12.55 in					
L/D	5						
Strain Limits				(g)	(in)	(g)	(in)
		City		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.021	0.2	0.113	1.0
ε _{s,rcw}	0.01	PL3 steel	New York City	0.016	0.1	0.071	0.6
ε _{s,bb}	0.0234	PL1 steel	Boston	0.018	0.2	0.081	0.7
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.036	0.3	0.166	1.5
0.8*ε _{s,bb}	0.0187	PL2 steel	Paducah	0.041	0.4	0.375	3.3
			St Louis	0.028	0.2	0.218	1.9
Limit State Disp. (in)			Helena	0.056	0.5	0.206	1.8
Δ @ ε _{c,cc}	2.9	PL3 conc	Salt Lake City	0.082	0.7	0.552	4.8
Δ @ ε _{s,rcw}	4.7	PL3 steel	Los Angeles	0.274	2.4	0.689	6.0
Δ @ ε _{s,bb}	10.1	PL1 steel	San Francisco	0.299	2.6	0.699	6.1
Δ @ ε _{c,u}	9.7	PL2 conc	Seattle	0.173	1.5	0.548	4.8
Δ @ 0.8*ε _{s,bb}	8.2	PL2 steel	Juneau	0.130	1.1	0.422	3.7
			New Madrid	0.050	0.4	0.956	8.4



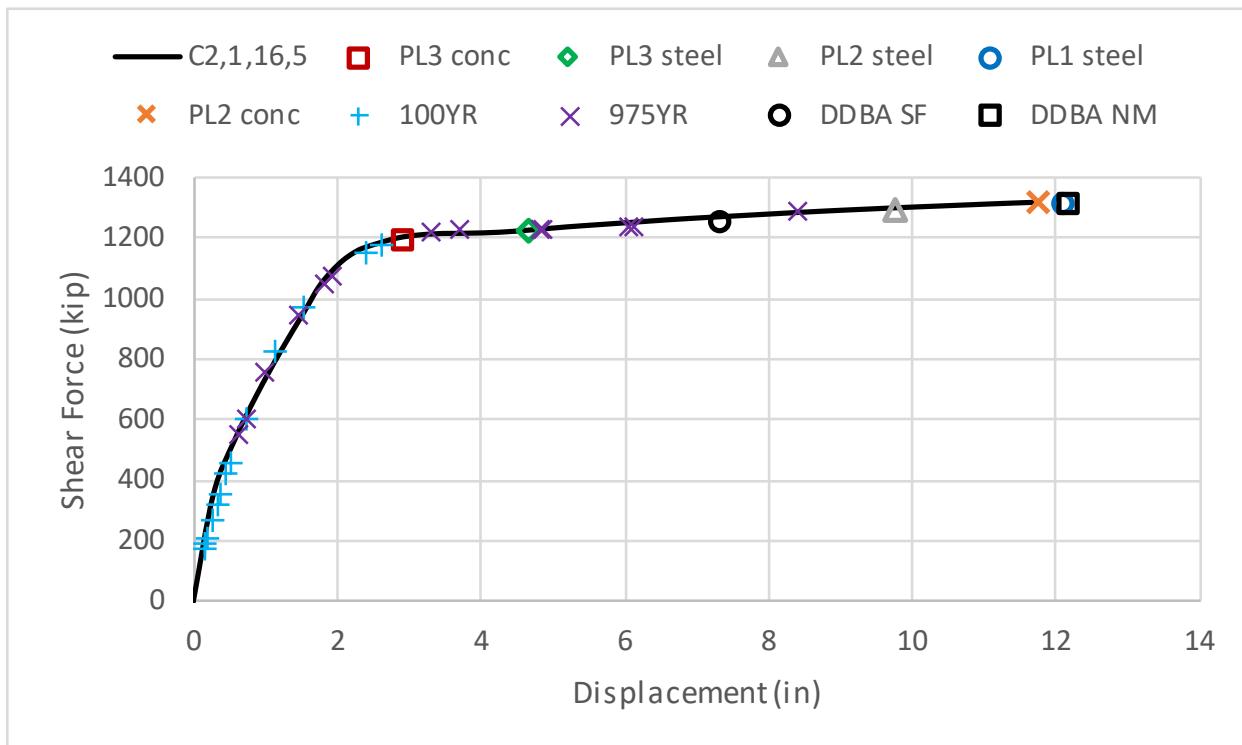
C2,0.7,16,6							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.007	San Francisco 9.15 in					
P/f' _{ce} A _g	0.16	New Madrid 15.4 in					
L/D	6						
Strain Limits				(g)	(in)	(g)	(in)
		City		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.016	0.2	0.087	1.3
ε _{s,rcw}	0.01	PL3 steel	New York City	0.012	0.2	0.055	0.8
ε _{s,bb}	0.0234	PL1 steel	Boston	0.014	0.2	0.063	0.9
ε _{c,u}	-0.0169	PL2 conc	Knoxville	0.028	0.4	0.128	1.9
0.8*ε _{s,bb}	0.0187	PL2 steel	Paducah	0.031	0.5	0.288	4.3
			St Louis	0.022	0.3	0.168	2.5
Limit State Disp. (in)			Helena	0.043	0.6	0.158	2.3
Δ @ ε _{c,cc}	4.0	PL3 conc	Salt Lake City	0.063	0.9	0.425	6.3
Δ @ ε _{s,rcw}	6.2	PL3 steel	Los Angeles	0.211	3.1	0.530	7.8
Δ @ ε _{s,bb}	13.1	PL1 steel	San Francisco	0.230	3.4	0.539	7.9
Δ @ ε _{c,u}	12.9	PL2 conc	Seattle	0.133	2.0	0.422	6.2
Δ @ 0.8*ε _{s,bb}	10.6	PL2 steel	Juneau	0.100	1.5	0.325	4.8
			New Madrid	0.039	0.6	0.736	10.9



C2,1,16,4							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.01	San Francisco 5.6 in					
P/f' _{ce} A _g	0.16	New Madrid 9.55 in					
L/D	4						
Strain Limits				City	(g)	(in)	(g)
					S _a 100YR	Δ 100YR	S _a 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.028	0.1	0.155	0.7
ε _{s,rcw}	0.01	PL3 steel	New York City	0.022	0.1	0.098	0.5
ε _{s,bb}	0.0284	PL1 steel	Boston	0.025	0.1	0.111	0.5
ε _{c,u}	-0.0206	PL2 conc	Knoxville	0.049	0.2	0.228	1.1
0.8*ε _{s,bb}	0.0228	PL2 steel	Paducah	0.056	0.3	0.513	2.4
			St Louis	0.039	0.2	0.298	1.4
Limit State Disp. (in)				Helena	0.077	0.4	0.282
Δ @ ε _{c,cc}	2.0	PL3 conc	Salt Lake City	0.112	0.5	0.757	3.5
Δ @ ε _{s,rcw}	3.4	PL3 steel	Los Angeles	0.375	1.7	0.943	4.4
Δ @ ε _{s,bb}	9.2	PL1 steel	San Francisco	0.410	1.9	0.958	4.5
Δ @ ε _{c,u}	9.0	PL2 conc	Seattle	0.237	1.1	0.750	3.5
Δ @ 0.8*ε _{s,bb}	7.4	PL2 steel	Juneau	0.164	1.0	0.524	3.2
			New Madrid	0.069	0.3	1.310	6.1



C2,1,16,5							
A _{st} /A _g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
4A _h /D's	0.01	San Francisco 7.3 in					
P/f' _{ce} A _g	0.16	New Madrid 12.15 in					
L/D	5						
Strain Limits				(g)	(in)	(g)	(in)
		City		S _a 100YR	Δ 100YR	S _a 975YR	Δ 975YR
ε _{c,cc}	-0.004	PL3 conc	Charleston	0.021	0.2	0.113	1.0
ε _{s,rcw}	0.01	PL3 steel	New York City	0.016	0.1	0.071	0.6
ε _{s,bb}	0.0284	PL1 steel	Boston	0.018	0.2	0.081	0.7
ε _{c,u}	-0.0206	PL2 conc	Knoxville	0.036	0.3	0.166	1.5
0.8*ε _{s,bb}	0.0228	PL2 steel	Paducah	0.041	0.4	0.374	3.3
			St Louis	0.028	0.2	0.217	1.9
Limit State Disp. (in)			Helena	0.056	0.5	0.205	1.8
Δ @ ε _{c,cc}	2.9	PL3 conc	Salt Lake City	0.082	0.7	0.551	4.8
Δ @ ε _{s,rcw}	4.7	PL3 steel	Los Angeles	0.273	2.4	0.687	6.0
Δ @ ε _{s,bb}	12.1	PL1 steel	San Francisco	0.299	2.6	0.698	6.1
Δ @ ε _{c,u}	11.8	PL2 conc	Seattle	0.173	1.5	0.547	4.8
Δ @ 0.8*ε _{s,bb}	9.8	PL2 steel	Juneau	0.129	1.1	0.421	3.7
			New Madrid	0.050	0.4	0.954	8.4



C2,1,16,6							
A_{st}/A_g	0.02	DDBA (Tangent Stiffness) Comparison for the 975YR EQ					
$4A_h/D'$	0.01	San Francisco 9.05 in					
$P/f'_{ce}A_g$	0.16	New Madrid 14.95 in					
L/D	6						
Strain Limits		City		(g)	(in)	(g)	(in)
$\epsilon_{c,cc}$	-0.004	PL3 conc	Charleston	0.016	0.2	0.087	1.3
$\epsilon_{s,rcw}$	0.01	PL3 steel	New York City	0.012	0.2	0.055	0.8
$\epsilon_{s,bb}$	0.0284	PL1 steel	Boston	0.014	0.2	0.062	0.9
$\epsilon_{c,u}$	-0.0206	PL2 conc	Knoxville	0.028	0.4	0.128	1.9
$0.8^*\epsilon_{s,bb}$	0.0228	PL2 steel	Paducah	0.031	0.5	0.288	4.3
			St Louis	0.022	0.3	0.167	2.5
Limit State Disp. (in)			Helena	0.043	0.6	0.158	2.3
$\Delta @ \epsilon_{c,cc}$	4.0	PL3 conc	Salt Lake City	0.063	0.9	0.424	6.3
$\Delta @ \epsilon_{s,rcw}$	6.2	PL3 steel	Los Angeles	0.210	3.1	0.529	7.8
$\Delta @ \epsilon_{s,bb}$	15.6	PL1 steel	San Francisco	0.230	3.4	0.537	8.0
$\Delta @ \epsilon_{c,u}$	15.6	PL2 conc	Seattle	0.133	2.0	0.421	6.2
$\Delta @ 0.8^*\epsilon_{s,bb}$	12.7	PL2 steel	Juneau	0.100	1.5	0.324	4.8
			New Madrid	0.039	0.6	0.735	10.9

