IMPACT OF COLUMN STIFFNESS ON CURVED BRIDGE PERFORMANCE

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Abstract

When determining the optimal design of a bridge column, there are multiple column diameters and reinforcement ratios that could be selected. Increasing the column size attracts additional forces and shortens the period of the structure. To determine in detail the impact of column stiffness, this paper investigates the design of various column diameters with varying longitudinal reinforcement ratios following the requirements of AASHTO LRFD Bridge Design Specifications [2] and AASHTO Seismic Guide Specifications [3]. This parametric study is applied to a curved bridge that is going to be tested with the shake table system at the University of Nevada, Reno.

Introduction

To gain a better understanding of how curved steel plate girder bridges respond during seismic events, the University of Nevada, Reno (UNR) is conducting an experimental test program funded by the Federal Highway Administration. This research is focused on understanding how different system variables will change the response of curved bridges during seismic events. A 2/5ths-scale model of a three span, three-girder bridge is being constructed. A total of nine different configurations will be tested. Variables include column type (conventional, post-tensioned and rocking) support conditions (isolation, conventional, with/without shear keys) and abutment conditions (with/without pounding). Different column diameters will also be investigated to understand how column diameter and reinforcement affects system response. This paper will focus on the design of the substructure for the conventional columns and the impact of column diameters.

Bridge Properties and Design Parameters

The properties of the superstructure of the bridge were based on a prototype bridge, FHWA Seismic Design of Bridges Design Example No. 6 [1]. This prototype bridge was modified from a concrete box girder bridge to a steel plate girder bridge. The properties of the bridge are listed in Table 1. These properties were then scaled by 2/5ths for laboratory testing.

The design of the substructure followed the AASHTO LRFD Bridge Design Specifications [2] and the AASHTO Seismic Guide Specifications [3]. For the design of

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the prototype column system, the following load combinations were chosen: Strength I, Strength III, Strength V, Service I, Service II, and Extreme I, see Table 2. All uniform temperature loads had a load factor equal to 1.0. For loading case Extreme I, a load factor of 0.5 was used for the calculation of live load, braking, and vehicular centrifugal forces. To simplify the process of design, it was assumed that the wind would act on the outer girder of curvature to maximize the force effect in the substructure. To determine the earthquake loading, the Reno response spectrum was used. It was assumed that the site class of the bridge was a "Class B", a soil classified as "Rock with 2,500 ft/sec < vs < 5,000 ft/sec" [3], with 5% damping. This resulted in a response spectrum with a peak ground acceleration equal to 0.472g, S_s = 1.135g, and S_1 = 0.41g, Figure 1. The bridge was also assumed to have an importance category of "other" for the use of response modification factors.

Prototype Column Design

To design the columns for the prototype structure, a standard size column for the Reno area was selected. For the selection of standard column details, Nevada Department of Transportation (NDOT) was contacted to determine their typical starting range for the design of bridge columns. According to NDOT, a typical column design would have the following properties: diameter of 60-66 inches (152.4-167.64 cm), longitudinal reinforcement ratio of 1-2%, transverse reinforcement ratio of 0.8-1%, and an axial load between 0.1f'c and 0.2f'c. Two column diameters were selected for the prototype design, a 40-inch (101.6 cm) column with 2% longitudinal reinforcement ratio and a 60-inch (152.4 cm) column with 1% longitudinal reinforcement ratio. Both columns were designed to have a transverse reinforcement ratio of 1%. The inclusion of the 40-inch (101.6 cm) column is smaller than the typical column constructed in the Reno area by NDOT. It was originally selected to ensure that the scaled version of the column would be able to be successfully tested in the laboratory. Later analyses have shown that the lab facilities will be able to test larger columns prompting an increase in prototype column size back into the typical design range.

To determine if the two column types would be acceptable, the loads determined from the LRFD load combinations [2] were compared against the axial-moment interaction diagram for the section. This interaction diagram was modified by the AASHTO loading factors to ensure that the column would meet design requirements. The results for the 60-inch (152.4cm) column are shown in Figure 2. The closest demand load case is from Extreme I. The largest demand from the basic strength load combinations are from Strength I. The results of the 40-inch (101.6cm) column are shown in Figure 3. When examining this interaction diagram, three load combinations exceed the capacity of the system: Strength I, Strength V, and Extreme I. The temperature range for the column was based on the minimum and maximum extreme cases using method 2 of the AASHTO design which is 115°F (46°C). AASHTO LRFD Bridge Design Specifications describes this method for temperature loading as a calibrated procedure that is used for extreme bridge design temperatures [2]. This method was chosen due to the integration within SAP 2000. With this extreme range in temperatures, the temperature effects push the demand above the capacity of the section. If the temperature range is reduced to 85°F (29°C), the demand is within the capacity of the section. This value would be acceptable in the design since the bridge would not undergo a temperature change of 115°F (46.11°C) if the bridge initial temperature is controlled. The results of this updated diagram are shown in Figure 4. With this reduction in temperature, the 40-inch (101.6cm) columns are acceptable for all load combinations except Extreme I which is acceptable due to the expectation of column yielding due to the design level earthquake. From a strength and service view of the columns, a smaller size column appears to be preferable by reducing the moment demand in the system.

Scaled Column Design

After determining acceptable column designs for strength and service loading combinations, the focus of the design for the scaled columns was switched to a limit state design for the extreme events. To determine what the optimal design would be for the scaled system to be built in the laboratory, the AASHTO seismic design specifications were followed to ensure that the columns would meet standard expected performances.

The selected designs were scaled down from the prototype column to diameters of 16-inch (40.64cm) and 24-inch (60.96cm). With these two "extreme" designs, another column size was chosen for the analysis that would fall between the design ranges of the previous columns. For this system, a 20-inch (50.8cm) column was chosen. With the column sizes selected, the next parameter that was chosen was the longitudinal reinforcement ratio. By modifying the longitudinal reinforcement ratio (1%, 1.5% and 2%), the same response period in each column could be maintained while decreasing or increasing the plastic moment capacity of the column. The final parameter of the different column systems was the bent cap size. To ensure that the bent cap would be able to sufficiently develop the joint shear requirements provided by the seismic design specifications, each bent cap size was selected based on column ratios. The width of the bent cap was determined by taking the column diameter and adding 12 inches (30.48cm). The depth of the bent cap was determined by taking the column diameter and multiplying it by 1.25. These values are typically on the conservative side but will ensure that the bent cap will be able to develop the capacities required to keep the cap elastic during testing.

To determine if the various column sizes and reinforcement ratios were acceptable under the seismic design specifications, XTRACT [4] and SAP 2000 [5] were used to determine column properties and system responses. XTRACT was used to determine the plastic moment capacity, the yield displacement for single and double curvature, and the plastic displacement capacity of the column in single and double curvature. The values in XTRACT were determined by using Mander's Model for confined concrete and the required reinforcement values from the seismic design specifications. Once the model was created, each section had an axial load applied based on the dead load applied to the section and then the moment was incremented about a principle axis. This increasing moment was continued until one of the rebar elements or confined core elements failed. Since the XTRACT model is based on the beginning of core failure, the actual columns will have extra capacity in the system. To determine the results of the system, SAP 2000 was used to run a modal analysis and an elastic response spectrum analysis. These results were used to determine the period of the system and to determine the demands from the Reno response spectrum.

Scaled Column Results

A summary of the analysis results are shown in Table 3. There is a significant range of periods for each system. The first mode varies from 1.43 seconds to 0.681 seconds. The 16-inch (40.64cm) column has a long period for the structure that is typically outside of a conventional period for a bridge. It would be expected that this system would not benefit from applying isolation to reduce the loading into the substructure of the system. After examining the period of the structure, the plastic moment capacity of the column should be examined. This value is extremely important for the seismic guide specification as it is used to determine the plastic shear demand in the system. As expected, as the column diameter increases, the plastic moment capacity of the system dramatically increases. However, the plastic moment capacity decreases with a reduction of longitudinal reinforcement. With this reduction in plastic moment capacity, the required shear capacity of the system will decrease allowing a decreased demand/capacity ratio. Finally, when examining the axial load ratios of each column type, the 20-inch (50.8cm) and 24-inch (60.96cm) diameter columns are lower than the 16-in (40.64 cm) diameter column.

With the properties of the system determined, the next focus is on the demands due to the elastic response spectrum analysis. These column demands are shown in Table 4**.** The first demand on the system that was examined was the displacement and ductility demands on the column. The displacement and ductility calculations were based on displacements at the top of the column. With these values calculated, the ductility demand was calculated by taking the resultant displacement and dividing by the yield displacement of the column in single curvature. For seismic design category C, the design category requirements based on the one second spectral acceleration in the AASHTO Seismic Design Specifications [3], the ductility demand should be less than four. Since all types of columns are well under four, the lowest ductility demand would typically be chosen to reduce the amount of damage that the column experiences during the design earthquake. For the comparison, that column is the 24-inch (60.96cm) with 1% longitudinal reinforcement, however; it is only slightly smaller in comparison to the other columns. Next, the moment and shear demand due to the response spectrum is examined for the system. These results were then modified by an R factor as required for response spectrum analysis. These results showed an increase in demand with larger systems, which was expected due to the stiffer sections that will attract larger forces. These values should then be compared to the capacities of the section along with the ductility demands to determine the optimal section. These results are shown in Table 5**.** Focusing on the shear/demand capacity ratios, the largest demand/capacity ratio is the 24-inch (60.96cm) column with 1% longitudinal reinforcement. This demand/capacity is calculated from the maximum of the required plastic shear and the shear from the response spectrum analysis. After investigating the demand/capacity ratios, displacement yield, capacity, and demands were investigated. For this system, it is expected that the column will exhibit single curvature in the radial direction and double curvature in the tangential direction. With the true curvature of the system existing between both extremes, it is important to see the lower and upper boundaries of these two variables. By comparing the displacement demand to the yield and capacity displacements, the expected damage in the column can be estimated. For example, the displacement demand in the 16 inch (40.64cm) and 24 inch (60.96) column with 1% longitudinal reinforcement is 3.48 inches (8.84cm) and 1.57 inches (3.99cm) respectively. When comparing these to the yield displacement of the columns, each column has exceeded the yield displacement of the column and cracking would be expected. The columns however would not expect to see the beginning of crushing of the core since neither column has reached the displacement capacity of the column in double curvature.

By having the smallest ductility demand and the largest acceptable demand/capacity ratio, the 24-inch (60.96cm) column would be the optimal design choice. However, this section also has the lowest yield displacement so it would be possible that cracking in the column could occur at a lower earthquake level than that of a smaller diameter column. While this section provides the most optimal design with the lowest ductility, other column choices would also be acceptable designs and could possibly be preferred due to the higher yield displacements.

Time History Analysis

After determining that all of the investigated column sizes would work, time history analyses were applied to the system to determine the displacement and damage. To replicate the design level earthquake for the system, the Sylmar station response from the 1994 Northridge earthquake was scaled to match the design response spectrum for acceleration at 1 second. This resulted in a scaling factor of 0.475 for the design earthquake. To simulate testing in the laboratory, this ground motion was then followed by another earthquake at 1.5 times the design level earthquake. These displacements could then be compared to determine the peak displacements and expected damage for various sizes of columns. The two column parameters that were investigated were the 20-inch (50.8cm) column the 24-inch (60.96cm) column. Each column had 1% longitudinal reinforcement. The results are shown in Figure 5 and Figure 6 respectively. From these figures, both columns exceed the yield displacements in both single and double curvature for the design level earthquake. The 20-inch (50.8cm) column experiences approximately 1 inch (2.54 cm) of greater displacement at the top of the column than the 24-inch (60.96-cm) column for the design level earthquake. This displacement difference is further increased for the earthquake that is 150% of the design level earthquake. Based on these results, even though the larger column has a lower yield displacement, it would be expected to have less damage in the column.

Conclusions

The design and selection of a column has a significant impact on a bridge system. The column size will have a direct impact on the period, capacity, damage, and design of surrounding elements. Determining the optimal column design can vary dramatically based on the design goals of the bridge. Determining the limit states of an extreme event and the reserve capacity for strength and service states will drive the design of a system from a relatively small column that will experience a larger amount of damage from the design earthquake, to a larger column system with reduced damage from the design earthquake.

Acknowledgments

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References

1. Seismic Design Course, Design Example No. 6, BERGER/ABAM Engineers Inc. September 1996.
2. AASHTO-LRFD Bridge Design Specifications, fourth Edition, 2007 and interims through 2008.
3. AASHTO Guide Specifications for LRFD Seismic Bridge Design, May 2007
4. XTRACT v3.0.5, Imbsen Software Systems
5. SAP 2000 v14.2.0, Computers and Structures, Inc.

Table 1: Prototype Bridge Properties

Subtended Angle	104° (1.8 rad)
Total Length at c.l.	362.5 ft (110.49m)
Span Lengths at c.l.	105 ft – 152.5 ft – 105 ft (32m-46.48m-32m)
Radius at c.l.	200 ft (60.96m)
Total Width	30 ft (9.14m)
Plate Girders	3 spaced @ 11 ft o.c.
Column clear height	20 ft (6.10m)
Piers	Single column, drop cap

Table 2: AASHTO Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*IC*	*CT*	*CV*
STRENGTH I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
EXTREME EVENT I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
SERVICE III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
FATIGUE— *LL, IM & CE* ONLY	—	0.75	—	—	—	—	—	—	—	—	—	—	—

Table 3: Parametric Column Design Properties

	16in	20in-1%	20in-1.5%	20in-2%	24in-1%	24in-1.5%	24in-2%
Design Parameters							
p	1.99%	1.02%	1.58%	1.97%	1.10%	1.56%	1.95%
ps	0.94%	0.97%	0.97%	0.97%	0.99%	0.99%	0.99%
Period							
T1 (sec)	1.430	1.109	1.014	0.961	0.776	0.683	0.681
T2 (sec)	0.623	0.588	0.575	0.567	0.532	0.507	0.506
Xtract values							
P_{dl} (k)	124	126	126	126	129	129	129
M_{pcol} (k-in)	2142	2688	3498	4039	4652	5866	6853
V_{pcol}(k)	54	67	87	101	116	147	171
$M_{ultimate}$ (k-in)	2204	2867	3724	4322	5068	6399	7436
ALR	11.2%	7.3%	7.3%	7.3%	5.2%	5.2%	5.2%

Table 4: Elastic Response Spectrum Demands

	16in	20in-1%	20in-1.5%	20in-2%	24in-1%	24in-1.5%	24in-2%
Column Demands Design RSA							
SRSS (in)	3.48	2.50	2.32	2.23	1.73	1.57	1.57
μ demand $_{single}$	2.64	2.53	2.22	2.09	2.05	2.07	1.76
Elastic Response Spectrum Analysis							
Mu (k-in)	5390	7016	7669	8161	10433	12066	12095
Vu (k)	39	57	60	61	78	90	91
Mu/R (k-in)	1797	2339	2556	2720	3478	4022	4032
Vu/R (k)	13	19	20	20	26	30	30

Table 5: Column Capacities and D/C Ratios

	16in	20in-1%	20in-1.5%	20in-2%	24in-1%	24in-1.5%	24in-2%
Column Capacities							
ϕVn (k)	100	157	157	157	221	221	221
Δc_{single} (in)	7.21	8.70	8.00	7.79	8.45	7.69	7.23
Δc_{double} (in)	4.37	5.41	5.47	5.31	5.86	5.73	5.32
Δy_{single} (in)	1.31	0.99	1.04	1.07	0.84	0.76	0.89
Δy_{double} (in)	0.54	0.41	0.43	0.44	0.35	0.31	0.36
Column D/C Ratios Design Level							
Shear D/C	53.7%	42.9%	55.8%	64.5%	52.6%	66.4%	77.5%
$(Mu/R)/M_{pcol}$	83.9%	87.0%	73.1%	67.3%	74.7%	68.6%	58.8%
$SRSS/\Delta c_{single}$	48.2%	28.8%	29.0%	28.6%	20.5%	20.4%	21.7%
$SRSS/\Delta c_{double}$	79.6%	46.2%	42.4%	42.0%	29.5%	27.4%	29.5%

Conversion Table

1k-ft	=	1.36 kN-m
1k	=	4.45 kN
1in	=	2.54 cm

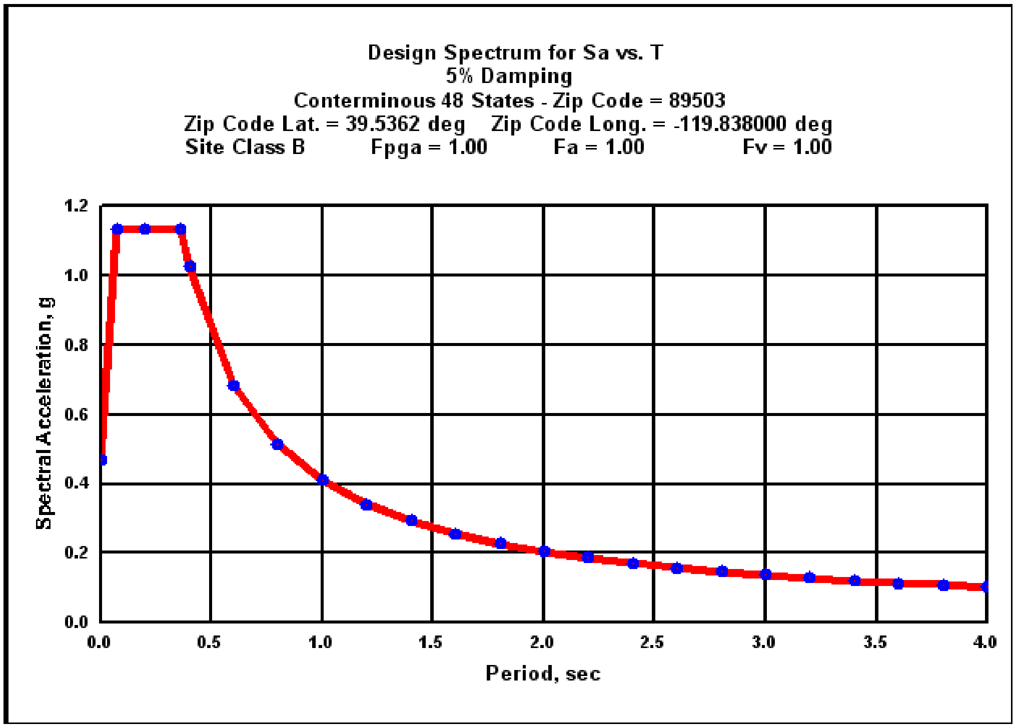

Figure 1: Reno Response Spectrum

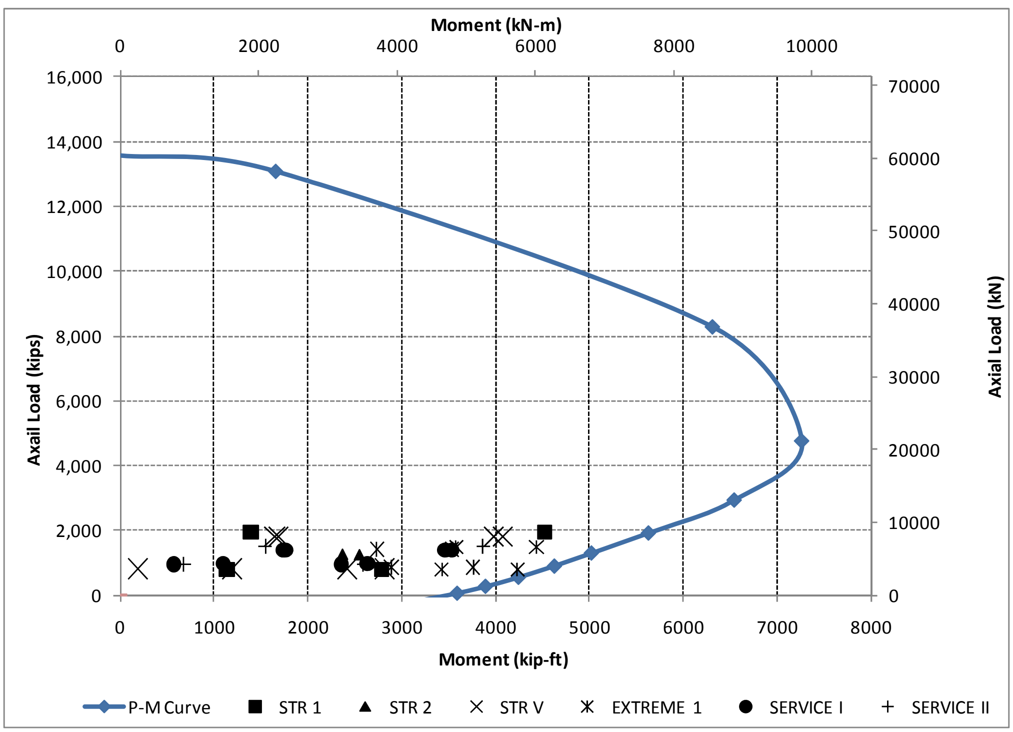

Figure 2: AASHTO P-M Interaction Diagram for 60in (152.4cm) Column

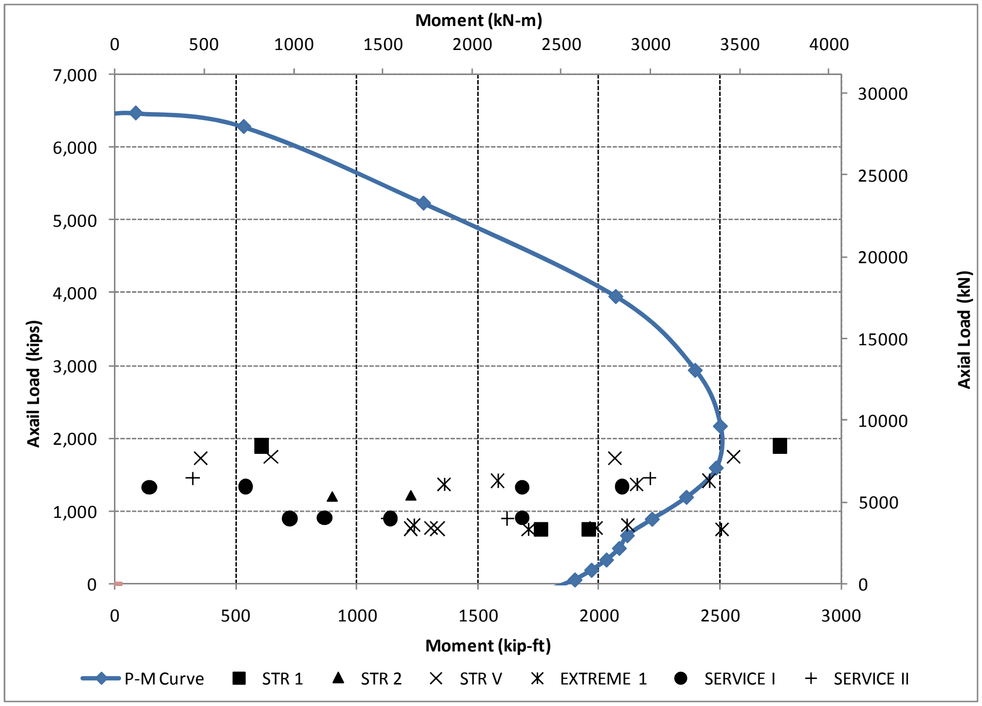

Figure 3: AASHTO P-M Interaction Diagram for 40in (101.6cm) Column

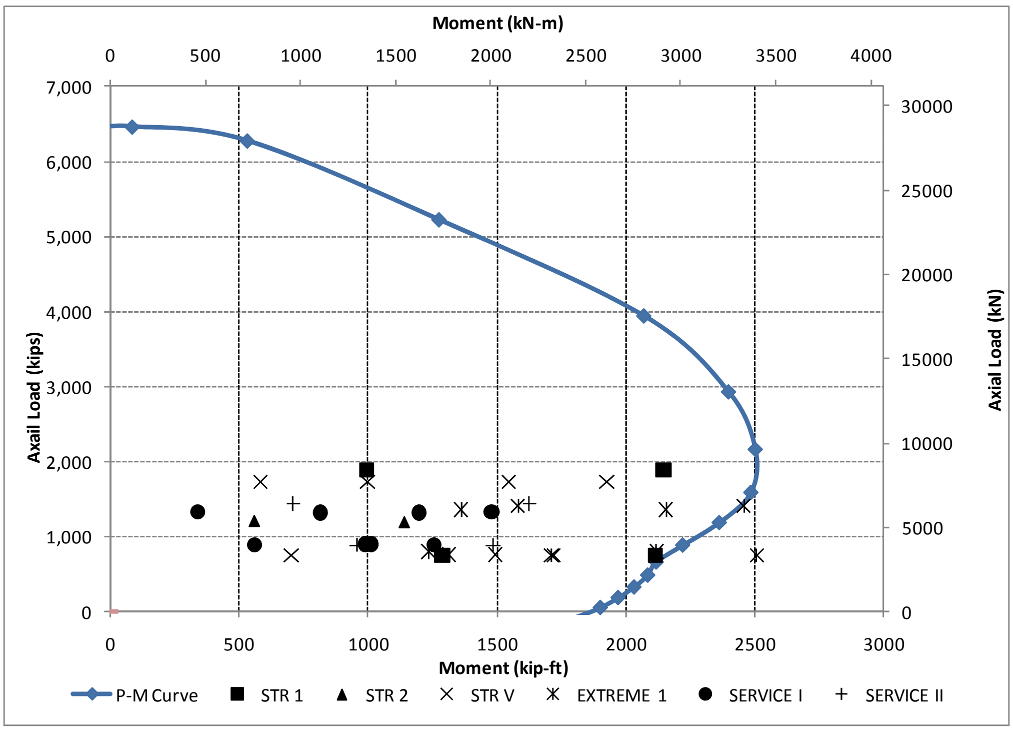

Figure 4: AASHTO P-M Interaction Diagram 40in (101.6cm) Column New Temperature

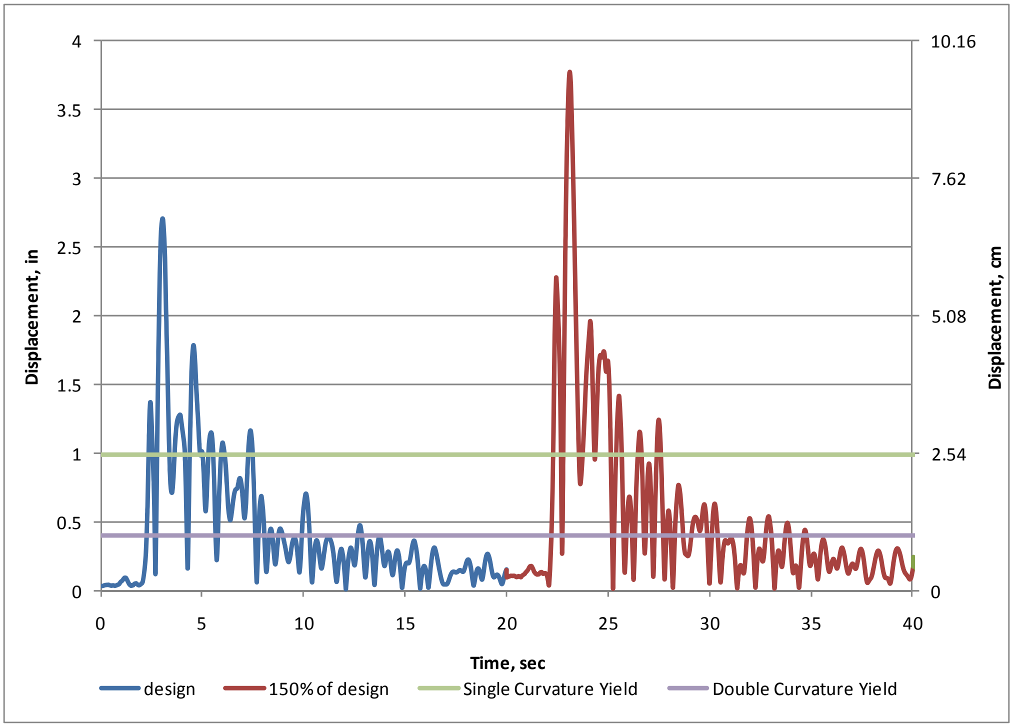

Figure 5: 20in (50.8cm) Time History Displacement Results

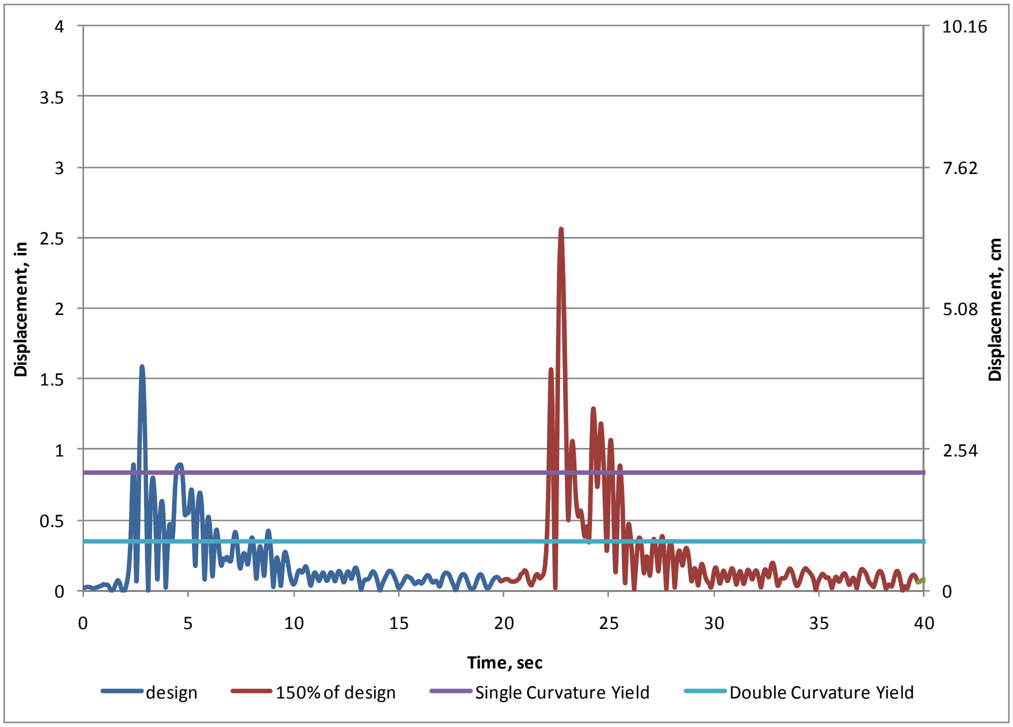

Figure 6: 24in (60.96cm) Time History Displacement Results