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Trial Designer's Comments Received thru September 30, 2006

I would like to take this opportunity to thank all the Bridge Designers that participated in the trial designs and commented on the Guidelines for their efforts and useful suggestions.

The comments received through September 30, 2006 have been assembled into a single table and collated by section number of the Guidelines to facilitate compiling the responses and the review by others. The comments included in this tabulation cover only the Specifications, i.e. the comments on the Commentary have been removed. The Commentary as submitted was included as a first attempt to extract portions from specifications that were used, in part, to compile the Guidelines. The responses to the comments on the Guideline Specifications are being processed by section and will be posted for each section as they are completed.

Sincerely,

Roy A. Imbsen

	Section	State/ Name	Comment	Response
No.	Section1			
1		AR	It would be good for these guidelines to use ksi (as opposed to psi) units exclusively, such as the LRFD Specification does. Also, it would be good to use U.S. Customary Units exclusively.	
2		AR	It would be very beneficial in the final version to have the Section and the Section Commentary appear or at least start on the same page.	
3		AR	Guidelines are inconsistent throughout using both "Section" and "Article" interchangeably. LRFD Specs use "Article"	
4		TN/ Tim Huff	There is no provision for checking the rotation capacity of plastic hinges in the guidelines. The assumption is made that the only failure criteria for a hinge is strain in the confined core reaching the Mander model limit or reinforcing steel reaching it's strain limit. Is this the intent? Should other criteria be used in determining ultimate curvatures, displacements, and rotations?	
5		TN/ Tim Huff	It appears that design for unreduced seismic forces is no longer an option for Seismic Design Categories C and D. Hinging forces must be used even if they are greater than the seismic forces from an elastic analysis. Is this the case? If so, why?	
6		TN/ Tim Huff	The LRFD Specification uses ksi units exclusively now. It would be a nice convenience if the Guidelines did too. Most equations in the Guidelines are based on using psi units.	
7	1.1	AR	Art. 1.1, Background: Task 1 is not mentioned. Task 6, with its five sections, is mentioned first. Then… tasks 2 thru 5 are discussed. Why not discuss them in order?	

8	1.2.2	AR	Art. 1.2.2, 3rd paragraph under maps should read: "Alaska was based on USGS data…Hawaii was based on USGS data…"	
9	1.3, Figures 1.3C & 1.3F	BERGER/ Lee Marsh	The 'B or C' decision point in Fig 1.3C seems to prevent SDC C designs from getting to Fig 1.3F, which includes many capacity protection steps for C.	
10	**Section 2**			
11	2.1 / 2.2 Definitions m_D	BERGER/ Lee Marsh	Add a clear definition of 'local'	
12	2.2 SRSS	BERGER/ Lee Marsh	The second use listed (vector combination) is not 'statistical'.	
13	5-Feb	MO	2-5 S = Site coefficient specified in Article 3.5.1 (Article 3.5.1 does not exist)	
14	**Section 3**			
15	3.1	AR	The last paragraph of Art. 3.1 can be misleading. A better way to get the message across might be "Detailed seismic analysis is not required for a single span bridge or for any bridge in Seismic Design Category	
16	Pg 3-1	MO	3-1 Table C3.2-1 that is referenced in commentary does not exist	
17	Sec. 3.1 Paragraph 4	FHWA/Derrell	Add after "Design Category A." "Specific detailing requiremnts do apply"	
19	Pg 3-2	MO	Clarify "For sites with lateral flow due to liquefaction, significant inelastic deformation is permitted in the piles." (Pg 1-3 states "Design requirements for lateral flow are still debatable and have not reached a stage of completion for inclusion in the guidelines.")	
18	Pg 3-2—3-11	MO	"C3.3" is used on both pages	
20	3.2	AR	Art. 3.2 mentions a "one level design", what does this mean?	

21	3.2	AR	Art. 3.2 mentions "Life Safety for the Design Event", "Significant Damage Level" and "Significant Disruption to Service Level". These terms are somewhat cumbersome. They could be called Performance Level I, II, and III for example. Also, how does Figure C3.3-1 relate to these levels? What is the "operational objective"? How do the Design Levels differ, and what is specifically required of each?	
22	Sec. 3.2 Pg 3-1	CA/ Mike, Fadel, Mark, Lian	Not clear why "Significant Damage Level" and "Significant Disruption to Service Level" are defined. Per first paragraph of 3.2, bridges shall be designed for life safety for the design event.	
23	Pg 3-3 Several figures	CA/ Mike, Fadel, Mark, Lian	Strongly recommend that Earthquake Resisting Systems should be designed for plastic hinging of columns, NOT for elastic design using the 1000 year design earthquake. The probability of collapse for an event exceeding the 1000 year earthquake is very different for an ERS designed for ductile columns or elastic response without ductile detailing.	
24	Pg 3-4 Figure middle right pier wall and pg 3-5 middle left	CA/ Mike, Fadel, Mark, Lian	Plastic hinges may occur in the strong direction of pier walls. Is owner permission needed for use of all pier walls?	
25	Pg 3-4 Figure bottom right	CA/ Mike, Fadel, Mark, Lian	Design of abutment backwalls that are to resist dynamic forces of superstructure elastically without fusing is unconventional and should require the Owner's permission.	
26	Pg 3-5 top right	CA/ Mike, Fadel, Mark, Lian	No guidance is given on how to calculate the sliding displacement of a spread footing abutment with nonfusing shear keys.	
27	Pg 3-6 last paragraph	CA/ Mike, Fadel, Mark, Lian	Systems that do not fall in the listed permitted ERS should not be "not allowed", but instead "not recommended". There may be ERS that are not listed that are appropriate under certain circumstances with the Owner's permission as stated in the next sentence on pg 3-7.	

28	3.2 1st Para.	BERGER/ Lee Marsh	It would seem appropriate for the period of interest to match the design life in AASHTO LRFD, 75 years. This would result in about a 7% probablility of exceedence; perhaps this equivalence could be discussed	
29	3.3	AK	In Figures 3.3.1a, 3.3.1b, and 3.3.2 it would be convenient to have these elements numbered so they can be referenced in a design. For example, it could be said "This design uses Longitudinal Response #1 and Transverse Response #3, for the ERS" and …….for the ERE.	
30	3.3 Fig. 3.3.2	BERGER/ Lee Marsh	Use of reduced (70%) strength for abutment passive should only be for similar case for Fig. 3.3.1b for Permissible ERE. Full abument resistance should only be permitted with owner's permission. This is related to control of backfill placement.	
31	Sec. 3.3.1 Pg 3.3.1b	FHWA/ Derrell	Bottom left figure not legible	
32	Pg 3-10	MO	Fig. 3.4.1-2 through 14 do not exist but are referenced	
33	Pg 3-11 1st paragraph	CA/ Mike, Fadel, Mark, Lian	Recommend that an adequate geotechnical investigation should be performed such that the Site Class can always be determined rather than using Class D as the default.	
34	Pg 3-13		3-13 Fig X.X	
35	3.4	AR	Art. 3.4.1, Figures 3.4.1-2 thru 3.4.1-14 are not found in these	
36	3.4 2nd Para.	BERGER/ Lee Marsh	How will the hazard maps be controlled? Will a specific USGS version be referenced in the Guidelines? Will data from USGS website be permitted to be used?	
37	3.4.1	BERGER/ Lee Marsh	Will the long-period transition and constant-displacement spectral ordinates included in the 2004 USGS maps, FEMA-450 (NEHRP), 2003 and ASCE 7-05 be used? If so, why not include them now? Or is this data not available for the 1000-yr return period?	
38	Sec. 3.4.1 Pg 3.4.1-1	FHWA/ Derrell	S_{Ds} should be S_{DS}	
39	Sec. 3.4.1 Pg 3.4.1 (a)	FHWA/ Derrell	Figures 3.4.1-2 through 3.4.1-14 are not provided as indicated	

40	Sec. 3.4.1 Page 3-11 (Item 1)	FHWA/ Derrell	"peak ground acceleration) is not defined. Possibly add PGA=0.4 S_{DS}	
41	3.4.3 4th Para.	BERGER/ Lee Marsh	Are active fault maps available for the entire country? Furthermore, does this section need to be limited to surface or shallow (definition?) faulting, or does it also cover deep faults, such as those in the Cascadia and New Madrid regions? The requirements for the near-fault effects	
42	Pg 3-14 first paragraph	CA/ Mike, Fadel, Mark, Lian	What attenuation relationship is to be used? How will T-3 incorporate the result of the PEER NGA (Next Generation Attenuation) relationships?	
43	Pg 3-14 last sentence of first paragraph	CA/ Mike, Fadel, Mark, Lian	"Peer Reviewed" needs to be defined as independent internal or external body. Internal peer reviews are likely adequate for Caltrans and other DOT's for typical bridges. Recommend either allowing internal peer reviews, or external as determined by the Owner to be necessary.	
44	Sec.3.4.4 Pg 3-15	CA/ Mike, Fadel, Mark, Lian	Is a near field adjustment made to the ARS curves?	
45	Pg 3-18	CA/ Mike, Fadel, Mark, Lian	Response spectra for construction sites that are "close" to active faults … "Close" needs to be defined.	
46	3-21,3-22.3-23	MO	Are there plans to provide guidance with detail for "SDC B, C & D level of detailing other than the information currently shown in the guidelines?	
47	Sec. 3.5 Pg 1-2 Task 3 No. 2 and Pg. 3-16	CA/ Mike, Fadel, Mark, Lian	Performing a displacement ductility capacity check provides minimal value without performing capacity design to ensure the plastic hinge occurs in the well detailed region.	
48	Sec. 3.5 Table 3.5.1	FHWA/ Derrell	add $S_{D1}=F_vS_1$ following the table	
49	Sec. 3.6 Pg 2-3 paragraph	FHWA/ Derrell	"those bridges…" should be on previous line. (Word wrap problem)	

50	4.7.1b Pg 4-11	CA/ Mike, Fadel, Mark, Lian	Typically there would be limited value in restricting the ductility demands as required for Limited Ductility Response for SDC B or C. This will result in stronger columns that will require stronger foundations according to Capacity Design principles, increasing the foundation cost. Typically Limited Ductility Response requirements are used to provide enhanced performance, despite the increased costs, to provide increased post-earthquake serviceability for an important bridge. Recommend deleting the last sentence of the last paragraph stating	
51	**Section 4**			
52	Section 4.1.1 Pg 4-1	WA/ Jugesh Kapur	states that the ratio of effective stiffness, as shown in Figure 4.1, between <u>any two bents</u> within a frame or between any two columns within a bent shall satisfy Equation 4.1. This equation limits the ratio to 0.5. However, Table 4.2 on page 4-6 allows the maximum stiffness ratio from span to span to vary from 2 to 4 depending on the number of spans. This table contradicts section 4.1.1 that limits the value between	
53	4.1.1 1st Para	BERGER/ Lee Marsh	How are the abutments included in the adjacent bent stiffness and mass considerations? The language is also non-mandatory; thus should this go into commentary?	
54	4.1.4 1st Para	BERGER/ Lee Marsh	Why does this section only apply to single-column bents? Is the intent to focus on superstructure torsional rigidity? Which shear demands are referred to here? Some commentary would be useful, particularly with the mandatory language of the section.	
55	4.2 Table 4.2	BERGER/ Lee Marsh	The limits in this table are somewhat more liberal than those in Sections 4.1.1 and 4.1.2. This seems to be inconsistent.	
56	Sec. 4.2 Table 41	FHWA/ Derrell	Table does not require time history analysis, but throught the entire document time history is required a multiple number of times. This requirement should be captured	

57	Sec. 4.2.2 Entire section	FHWA/ Derrell	AASHTO indicated that one category of bridges is desired in the specification. This section references Critical, Essential in addition to Normal bridges	
58	Sec. 4.2.2 4th paragraph	FHWA/ Derrell	Clarify "Saftey Evaluation Design Earthquake" has not been defined	
59	Chap 4 General	BERGER/ Lee Marsh	There is nothing in Chapter 4 about the abutments and whether to include them in the ERS. It seems that the material in Section 5.2.1 that relates to design choices (as opposed to modelling) should go into	
60		AR	There is an inconsistency in terms between Art. 4.2 and Art. 5.4.3. Procedure No. 2 is named "Multimodal Spectral" in Art. 4.2 and "Elastic Dynamic Analysis" in Art. 5.4.3.	
61	4.2.2 1st Para.	BERGER/ Lee Marsh	Active faults are referred to again, see comment for Section 3.4.3.	
62	4.2.2 1st Para.	BERGER/ Lee Marsh	Antecedent of 'these' in 3rd sentence is ambiguous, leading to the implication that bridges closer than 6 miles to an active fault must be analyzed with time history techniques. Is this the intent?	
63	4.2.2 4th & 5th Para.	BERGER/ Lee Marsh	SEE is not defined. Change to design seismic event?	
64	4th Para.	BERGER/ Lee Marsh	Should add a caution that the modified response spectra should still transition to the original PGA. Additionally, 30% damping seems quite	
65	4.3.3	AR	Art. 4.3.3: The "force reduction factor (R) is obtained by dividing the elastic spectral force by the plastic yield capacity". The plastic yield capacity is of the "bridge component where plastic hinging is expected". Hinging typically occurs in columns or beams due to high moment. Thus the "spectral force" is actually a moment. During a trial design using this method, an R value was found to be 0.9, much lower than the value of 3 required for SDC C. Should not the R value be greater in SDC D than in SDC C? More information on calculating R in SDC D	

66		AR	For Table 4.3, please include a USGS moment magnitude map in the guidelines. Also, see comment in Appendix D.	
67	4.3.3 2nd Para.	BERGER/ Lee Marsh	Is the maximum R selected bent-by-bent, bottom to top, and from each direction?	
68	4.3.3 2nd Para.	BERGER/ Lee Marsh	Define 'spectral force'. Note that only displacement is determined in Section 4.4, not forces as this section implies.	
69	4.3.3 2nd Para.	BERGER/ Lee Marsh	Should the passage read that R 'may be taken' equal to 2 and 3 for SDC B and C, rather than 'is'?	
70	4.3.3 Table 4.3	BERGER/ Lee Marsh	This table is OK for CA, but is this data available for the rest of the US? Is it readily available for the 1000-yr hazard? Is interpolation required? This seems complicated, particularly for an empirical approximation such as R_d. Why not use the corner of the response spectrum where the transition from constant acceleration to constant velocity occurs? Both ATC 32 and ATC 49 permitted that, although ATC 49 added a small	
71	Sec. 4.3.3 Entire section	FHWA/ Derrell	"R" is defined as an elastic force reduction factor, but it is also being usedin an unknown means in the formulas. What is "R"? It is defined as 2 or 3 for SDC B,C, but a calculated value is required for SDC D. Since elastic moments are typically larger than plastic values, this will make R <1. The spec is not clear which forces should be used to calculate the "R" value for SDC D. If this "R" is a true ductilty factor, then a note should be added require seismic detailing. How do these ductility factors tie into the true ductility factors in the remaining part of the document? Is the primary period for each direction used or one primary period used for calculating both Rd? AT what location is the deflection taken? Several different locations are required through specification.	
72	Sec. 4.3.3 Last paragraph	FHWA/ Derrell	Word wrap problem for "…in Article 4.4.."	

73	Sec. 4.3.3 table 4.3	FHWA/ Derrell	Clarify:" Commentary should be provided indicating that Mean Magnitude can be obtained from the USGS website	
74	Sec. 4.3.3 table 4.3	FHWA/ Derrell	The values for 6.75-7.0 and 7.5-7.75 are not included in the table	
75	Sec. 4.4 1st paragraph	FHWA/ Derrell	Add a statement that Displacement Magnification should be performed prior to combination of displacements	
76	Sec. 4.5 1st paragraph	FHWA/ Derrell	Dead load reaction should read "contributory mass".	
77	Sec. 4.5 1st paragraph	FHWA/ Derrell	The value 0.2 conflicts with 0.25 in section 4.13.2	
78	4.5	AR	Article 4.5 incorrectly references 4.13.2 or at least its unclear how 0.2 DL relates to the statements made in Art. 4.13.2.	
79		AR	For Equation 4.15, Should N be >=12 like SDC B, C, or D?	
80	4.5 1st Para.	BERGER/ Lee Marsh	Reference to Section 4.13.2 mis-states what that article says, which is 0.25 g.	
81	Sec. 4.6 1st paragraph	FHWA/ Derrell	"dead load" should read "contributory mass"	
82	Sec. 4.6 1st paragraph	FHWA/ Derrell	Add "The minimum support length shall be as specified in 4.12" to the last sentence. Also add "the force shall be carried through substructure"	
83	4.7.1 1st Para.	BERGER/ Lee Marsh	The ductility demand, md, is key to the whole process, but the definition is ambiguous. Is md the worst 'local' demand?	
84	4.7.1(b)	BERGER/ Lee Marsh	Are limited-ductility structures related to ERE where owner's approval is required? Or are limited-ductility structures SDC B&C bridges by	
85	4.7.2	AR	In Art. 4.7.2, there is a new requirement that at least 25% of the longitudinal top and bottom rebar shall be continuous for SDC D bridges, and spliced with couplers. Is this a requirement for concrete beams or for the slab? Prestressed girders are included in this. It seems that RC Slab, RCDG, and Precast Units would have to meet this	

86	Sec.4.7.2 Pg 4-11	CA/ Mike, Fadel, Mark, Lian	This provision does not ensure a minimum level of continuous mild reinforcement for cast-in-place prestressed concrete bridges for vertical acceleration. There may be only a nominal amount of mild reinforcement provided in a CIP P/s bridge. Caltrans SDC requires that additional mild reinforcement capable of resisting $\pm$0.25g be provided continuous over the length of the superstructure. This ensures a consistent level of mild reinforcement regardless of the amount or type of reinforcement used for service loads. I would recommend a similar minimum amount of continuous mild reinforcement be required.	
87	4.8 1st Para.	BERGER/ Lee Marsh	Need a clear definition of the displacement demand. Is this a local demand? What constitutes 'local'? It seems logical that both principal axes should be checked, but this is not stated directly. It should be. Also it would be logical to check the principal local axes of a bent or pier and not the principal axes of the bridge as a whole. For many bridges, this will require transformation of displacement data, since most programs only report global displacements. Should any account of additional uncertainty of displacements be accounted for due to the transformation?	
88	Sec. 4.8 all	FHWA/ Derrell	At what location on the column are deflections being compared?	
89	Sec. 4.8 all	FHWA/ Derrell	Clarify if the displacement demand is modified with displacement magnification	
90	Sec.4.8.1 Pg 4-12	CA/ Mike, Fadel, Mark, Lian	Is a reference available for equations 4-7a and 4-7b? I had problems when performing a quick calculation, getting a negative value for the displacement capacity. Is it possible there are missing or incorrectly placed parentheses? More information is needed to apply this to the many different applications that may occur in SDC B and C. Provide definitions, limitations and assumptions for the use of these equations.	

91	Sec. 4.8.1 definitions	FHWA/ Derrell	Is Bo taken as core or gross diameter?	
92	Sec. 4.8.1 Last paragraph	FHWA/ Derrell	If H is taken as distance from max moment to contraflexure, then if the corresponding deflection is used, then only 1/2 the true deflection is taken into account	
93	4.8.1 1st Para.	BERGER/ Lee Marsh	Provide a reference for the empirical capacity equations. Also it might be useful to clarify that 'Ln' is the natural logarithm. Perhaps use lowercase for the log function, because that is the most common way it is written.	
94	4.8.1 1st Para.	BERGER/ Lee Marsh	Have these expressions been calibrated against actual bridge designs? Are there any limits to their applicability, and if so, do those limits play into the procedure selection (i.e. send you to SDC D). For instance, I presume configurations such as bents with struts at mid-height should not be assessed directly with these equations?	
95	4.8.1 3rd Para.	BERGER/ Lee Marsh	I don't see where the second bullet option is addressed anywhere in the Guidelines (e.g. where the displacement capacity is a function of either longitudinal or transverse reinforcement for a concrete section.). See also comments for Section 4.9.	
96	Sec. 4.8.1	CA/ Mike, Fadel, Mark, Lian	Equations 4.7a and 4.7b do not show how detailing for SDC “B” and “C” will produce the desired ductility values.	
97	4.8.2 1st Para.	BERGER/ Lee Marsh	Why develop a new terminology, IQPA? Why not use Nonlinear Static Procedure, NSP, as other seismic specs are using?	
98	Misc, for internal use	CA/ Fadel, Mark, Lian	Investigate Mononobe_Okabe application in section 6.7.1 Investigate rocking application in section 6.3.4	
99	Figure 3.3.1a	CA/ Fadel, Mark, Lian	Permissible Earthquake Resisting Systems (ERS) allows elastic design of columns as an alternative to plastic hinges in inspectable locations. Regardless of the analysis method, bridges may form plastic hinges, which should be properly located and inspectable.	

100	Figure 3.3.2	CA/ Fadel, Mark, Lian	ERS requiring owner's approval: Ductile diaphragms in superstructure, yet yielding restricted to substructure! Why is the need for ductile	
101	Figure 3.3.2	CA/ Fadel, Mark, Lian	ERS requiring owner's approval: In-ground hinges in battered piles are not a good combination. The plastic hinge will most likely be not successful under the very large axial load of a battered pile while the vertical piles have very little participation!	
102	Sec. 4.8.2	CA/ Fadel, Mark, Lian	"Local Displacement Capacity" is a different concept than the push over analysis of a sub-system. The two cannot be mixed. The push over analysis is generally done on the most global level possible, say a bridge frame in the longitudinal direction. Local ductility requirement is appropriate for a single column of that frame.	
103	Sec. 4.9	CA/ Fadel, Mark, Lian	The local displacement ductility demand (allowance) of 6 for single column bents and 8 for multi-column bents is approximately 50% higher than Caltrans practice in certain cases.	
104	4.9 Overall	BERGER/ Lee Marsh	This section needs to be tightened with respect to the checks that are required. Literally as I read it, only the ductility demand needs to be checked and shown to be less than the listed values. A moment-curvature analysis is required to calculate the yield and plastic displacement capacity, but the latter is never used. Thus, I don't see why it is calculated. A literal reading seems to obviate the need to perform the pushover analysis at all, since the yield displacement could be	
105	4.9 1st Para.	BERGER/ Lee Marsh	The dispensation of the foundation and superstructure flexibilities in the calculation of the yield and total displacements must be clear. If such flexibility is included in the yield displacement, the resulting ductility demand will be unconservative relative to the limits prescribed.	

106	4.9 1st Para.	BERGER/ Lee Marsh	As stated elsewhere, the definition of 'local' must be clarified. It appears that equivalent cantilever local elements are to be derived. If this is so, clarifying figures, such as those included in Caltrans' SDC would be most helpful. These could perhaps be included in the commentary.	
107	4.9 1st Para.	BERGER/ Lee Marsh	What limits are intended to be used to calculate the plastic displacement capacity? Are the strain limits, for example those given in Chapter 8, intended to be used here? Are they meant to define an 'either/or' limit with respect to the ductility limits prescribed in Section 4.9.	
108	4.9 1st Para.	BERGER/ Lee Marsh	Is any conservatism built into the limits provided in Section 4.9, including strain limits if they are also to be used? (Caltrans' SDC provides 'reduced' strains that are less than the ultimates for each material.) Can a bridge as designed by these Guidelines be expected to endure larger seismic displacements, i.e. those caused by ground motions that are in the 5% exceedence category (in 50 yrs)?	
109	4.9 1st Para.	BERGER/ Lee Marsh	Should multi-column bents have a higher permissible ductility demand than single-column bents? I thought current thinking was 'no'?	
110	Sec. 4.9 Pg 4-14	CA/ Mike, Fadel, Mark, Lian	Equation 4.8: Is Δ_y the first rebar yield or the idealized yield. This can be significantly different, especially for circular rebar configurations commonly used in columns.	
111	Sec. 4.9 Pg 4-14	CA/ Mike, Fadel, Mark, Lian	The ductility demands specified are much higher than the target values used by Caltrans.	
112	Sec. 4.10 Last paragraph	FHWA/ Derrell	Last sentence is incomplete	
113	4.11	TN/ Huff	The paragraph directly under Section 4.11 on page 4-15 is incomplete.	
114	4.11 1st Para.	BERGER/ Lee Marsh	Sentence is not complete.	

115	Sec. 4.11 page 4-15	WA/ Jugesh Kapur	The paragraph is incomplete	
116	Pg 4-6	CA/ Mike, Fadel, Mark, Lian	Are the terms "important", "critical", and "normal" defined?	
117	Pg 4-6	CA/ Mike, Fadel, Mark, Lian	The maximum bent/pier stiffness ratio in Table 4.2 appears to be inconsistent with the requirements in 4.1.1	
118	page 4-9	WA/ Jugesh Kapur	Table 4.3. The range for the Moment Magnitude is not continuous. For example, there are no values for Mw between 6.75 and 7.0, and between 7.5 and 7.75.	
119	4.11	AR	The sentence in Art. 4.11 is incomplete	
120	Sec. 4.11 all	FHWA/ Derrell	Last sentence is incomplete	
121	4.11.1 1st Para.	BERGER/ Lee Marsh	Superstructures should be added to the list of elements that are to be capacity protected.	
122	4.11.1 Item c.	BERGER/ Lee Marsh	I believe the intent is that deep foundations that may experience lateral forces from collateral hazards may be permitted to be ductile or limited-ductility elements. Because lateral spreading forces due to liquefaction may likely occur after the peak vibration-induced displacements are developed, it seems that such deep foundations should be capacity protected for vibration-based loading and only permitted to yield for lateral spread displacements, and these would be considered as a	
123	Sec. 4.11.1	CA/ Mike, Fadel, Mark, Lian	Not all foundation elements are capacity protected. Shafts are allowed to "plastic hinge" under certain conditions.	
124	Sec.4.11.1 5 all	FHWA/ Derrell	Δ_D is not defined	
125	Sec. 4.11.2 all	FHWA/ Derrell	Overstrength factors are used to account for material uncertainties. However, the spec requires actual material properties when calculating Moment curvature. To do both appears to be over conservative.	

126	4.11.2	AK	Art. 4.11.2 mentions Table 3.3.2, where is this table located?	
127	Sec. 4.11.3M Pg 4-18	CA/ Mike, Fadel, Mark, Lian	Recommendations needed for the calculation of shear below ground in pileshafts.	
128	4.11.5 2nd Para.	BERGER/ Lee Marsh	I don't understand how a modal analysis will show out-of-phase motions between the top and bottom of a column, because the signs are stripped when the modes are combined. Perhaps the comparison should be based on the response of a single mode where the signs are preserved.	
129	4.11.5 Eqn 4.11	BERGER/ Lee Marsh	The use of 'm' for a subscript that is not associated with ductility is confusing.	
130	Sec. 4.11.5 all	FHWA/ Derrell	Non linear is required, but table 4.1 has no provisions for non linear analysis	
131	Sec 4.11.6 Eq 4.12	FHWA/ Derrell	"L" is not defined	
132	Sec. 4.11.7 2cd paragraph & second to last paragraph	FHWA/ Derrell	Clarify if core or gross cross section is used. "…where the moment exceeds…" Which moment is referenced? Plastic, elastic, overstrength?	
133	4.12 all	FHWA/ Derrell	Seat width requirements should not include the gap opening. If a gap larger than required is provided, then the minimum seat length is	
134	4.12 Figure 4.3	FHWA/ Derrell	N should be dimensioned to not include the gap. Upper right figure (pier) only provides 1/2 seat length the way it is dimensioned. N1 and N2 are not defined on this same figure. IF gap provided is large, then minimum seat length is not conservative	
135	Sec. 4.12.1	CA/ Mike, Fadel, Mark, Lian	The background info on equation 4.15 is limited. Why do they use 0.2 factor for H. Also equation 4.15 needs correction with the term (1+Sk^2)/4000 should be (1+Sk^2/4000)!!	
136	Sec. 4.12.2	CA/ Mike, Fadel, Mark, Lian	Correction in equation 4.16 similar to 4.15.	

137	4.12.2 definitions	FHWA/ Derrell	Are displacement multipliers required for Δeq?	
138	4.12.2 all	FHWA/ Derrell	Define how to measure "N" with respect to skew	
139	4.12.2 Eqn 4.16	BERGER/ Lee Marsh	The use of three significant figures (1.65) seems rather precise for this empirical expression. Why not use 2?	
140	4.12.2 Eqn 4.16	BERGER/ Lee Marsh	Have the expressions for seat width been calibrated against those used in the current provisions? Are Eqns 4.15 and 4.16 more or less conservative? It should also be made clear that Deq must include the effects of foundation flexibility; otherwise this approach is unconservative. The Div I-A and ATC 49 approaches used approximate methods that allowed for some foundation rigid body movements and asynchronous ground and frame movements. The expressions given in Eqns 4.15 and 4.16 appear to rely on accurate predictions of the	
141	4.13 Pg 4-13	MO	4-13 Give guidance for "Where foundation and superstructure flexibility can be ignored, the two dimensional plane frame "pushover" analysis of a bent or a frame can be simplified to a column model (fixed-fixed or fixed-pinned) if it does not cause a significant loss in accuracy in estimating the displacement capacities"	
142	4.13.1 all	FHWA/ Derrell	Commentary is required to explain how the cable restrainers are determined in the event a case occurs where 5 restrainers are not appropriate. Specifications for the materials and details required are also needed. Define how restrainers are placed with respect to skew and	
143	4.13.2 all	FHWA/ Derrell	.25g is unconservative if a structure is in high seismic. Need to add comment that the larger of elastic analysis or .25 g is required. Also, add .25g times contributory mass.-This section conflicts with4.13.1 which specifies how many restrainers are required and 4.13.2 now requires the design of them. Conflict should be resolved.	

144	4.14 2nd Para.	BERGER/ Lee Marsh	Are there any limits on the steel type used in shear keys covered by this section? Should A706 recommended/required?	
145	4.14 all	FHWA/ Derrell	Non linear is required, but table 4.1 has no provisions for non linear analysis	
146	4.15	MO	See Article 7.4.9" Article 7.4.9 does not exist. Numerous references to Article X.X	
147	Pg 4-21	MO	4-21 What is the background for the development of the Plastic Hinge Length? It appears the diameter of the column would influence this length and should be included in the calculation. Is the accuracy of the equation justified or could 31 or 36 inches or column diameter be	
148	Pg 4-23	MO	Seat or support width: If one is doing a pushover analysis, what method is suggested to obtain the delta eq value? Could additional commentary be added for this requirement? It appears that we are getting erroneous results using the equation and the "skew factor" for long spans comparing skewed bridges and bridges without skews.	
149	**Section 5**			
150	5.2.1 3rd & 4th Para.	BERGER/ Lee Marsh	Consider moving these paragraphs to Chapter 4. See the 'general' comment regarding Chapter 4, above.	
151	5.2.3.2 Figure 5.2	BERGER/ Lee Marsh	In the text that references Figure 5.2, state that an approach slab in not required.	
152	5.2.3	TN/ Huff	Section 5.2.3.3 regarding abutment stiffness calculation is difficult to interpret and apply. There may be a unit problem or a typographical error. For example, consider an abutment wall 10 feet high and 60 feet long. Applying the equations as stated gives a stiffness of : $K_{eff1} = \frac{P_p}{0.02H_w} = \frac{\frac{2}{3}H_w L}{0.02H_w} = \frac{100L}{3} = \frac{100 \times 60}{3} = 2{,}000\ k/ft = 167\ k/inch$	

			which is a very low number. Perhaps the intent is for the capital “Pp” to be a lowercase “pp” so that the equation becomes $$K_{eff\,1} = \frac{P_p}{0.02H_w} = \frac{\frac{2}{3}H_w^2 L}{0.02H_w} = \frac{100H_w L}{3} = \frac{100\times10\times60}{3} = 20{,}000\,k/ft = 1{,}667\,k/inch$$ This still seems a bit low compare to previous values which would be on the order of 40 kips/inch/ft x 60 feet x 10/8 = 3,000 kips/inch.	
153		FHWA/ Derrell	Paragraph should be as commentary since it gives no requirements	
154	5.2.3.1 1st paragraph	FHWA/ Derrell	The word active should be "passive" pressure	
155	5.2.4.1 1st & 2cd paragraph	FHWA/ Derrell	Replace "dead load reaction" with contributory mass	
156	5.2.4.1 1st paragraph	FHWA/ Derrell	Minimum lateral force = 0.2DL, but section 4.13.2 requires 0.25g	
157	5.2.4.2 all	FHWA/ Derrell	need commentary	
158	5.3.1 1st paragraph	FHWA/ Derrell	For spread footings, the mass should be EXCLUDED since it is extremely stiff and obtaining 90% participation will require numerous	
159	5.2.3.3 1st Para.	BERGER/ Lee Marsh	Delete the word 'pressure' in the first sentence, and in the sentence that begins with 'Thus'. In the definition of lowercase p_p, delete 'per lineal foot of wall unit length along the wall'.	
160	5.2.3.3 a Heading & Bullets	BERGER/ Lee Marsh	Delete the word 'pressure' in the heading. Change the uppercase P_p to lowercase p_p. In the second bullet, delete the words 'per foot of wall length'. (Throughout this section uppercase denotes the total passive force on the wall and lowercase denotes the passive pressure, assumed to be uniformly distributed. The definitions in Section 2.1 are correct.)	

161	5.2.4	AR	Art. 5.2.4 mentions "Fusing". More information on this concept would be beneficial.	
162	5.2.4.1	AR	Art. 5.2.4.1 states the design force for Shear Keys: "Shear keys shall be designed for, a lateral force, equal to the difference between the lateral force demand and 0.4DL". Does this mean that as the earthquake force becomes larger, the shear key design force decreases? Needs	
163	Table5.1	AR	Table 5.1: The estimated depth to fixity is a possible foundation modeling method and can be determined with simple equations. Should these equations be included in the guidelines?	
164	5.3.1 1st Para.	BERGER/ Lee Marsh	If a foundation is modelled as rigid, including the mass of the foundation seems unnecessary, because the displacement degrees of freedom for the foundation would be eliminated from the stiffness matrix. If foundation flexibility is included, the foundation mass may cause problems getting to 90% mass participation. Suggest deleting the 3rd sentence of the	
165	Sec. 5.3.1 Pg. 5-10	CA/ Mike, Fadel, Mark, Lian	Foundation Modeling Method I should be the minimum required for SDC B&C. The designer should always have the latitude to more accurately model the foundations using FMM II.	
166	Sec. 5.3.1 M Pg. 5-10	CA/ Mike, Fadel, Mark, Lian	Recommend using FMM II for soft soils in SDC B and C.	

167	Sec. 5.3.2 Pg. 5-11	CA/ Mike, Fadel, Mark, Lian	Caltrans does not allow rocking of new bridges pending results of ongoing research. Reasons include: Rocking response is less predictable than other traditional types of response, effects of soil "rounding" under the footing - changing the rocking response under multiple cycles is not well understood, effects of paving, sidewalks and other surface features are not well understood, distribution of nonlinear response between column and foundation rocking can be difficult to determine due to sensitivity to variables with dispersed values that are difficult to precisely predict. If nothing else, Owner's Permission should be required.	
168	Sec. 5.3.4M Pg. 5-12	CA/ Mike, Fadel, Mark, Lian	The use of Group Reduction Factors for a single row of pileshafts or pile extensions is the subject of ongoing debate in the bridge engineering community. The GRF can have a significant effect on the flexibility and thus the overall response of the structure. While practices vary, many engineers are now analyzing the structure with and without the GRF's, similar to what is done for liquefaction.	
169	5.4.1 second paragraph	FHWA/ Derrell	critical and essential bridges is not defined-Table 4.1 does not require non linear time history	
170	5.4.2 2nd Para.	BERGER/ Lee Marsh	The Single-Mode Spectral Method is no longer defined. If it is permitted, then it should be defined. It is not clear from the paragraph whether the ESA is an alternate to the ULM and SMSM or envelopes	
171	5.4.3 1st Para.	BERGER/ Lee Marsh	The mandatory language requiring specific numbers of elements in the last sentence conflicts with the non-mandatory language to the same effect in Section 5.5.3.	
172	5.4.3 2cd paragraph	FHWA/ Derrell	Delete the words "on the other hand"	
173	5.4.4 all	FHWA/ Derrell	Time History analysis is not required in table 4.1	

174	Pg 5-6	BERGER/ Lee Marsh	Clarify "In this case a check of the abutment displacement demand and overturning should be made."	
175	5.6.1 Heading	BERGER/ Lee Marsh	Suggest adding the words 'reinforced concrete' between effective and section.	
176	5.6.1 1st Para.	BERGER/ Lee Marsh	Delete the words 'in reality'.	
177	5.6.2 Heading	BERGER/ Lee Marsh	Add either 'R.C.' or reinforced concrete before 'ductile'.	
178	5.6.2 1st Para.	BERGER/ Lee Marsh	Delete the word 'initial'.	
179	5.6.2	BERGER/ Lee Marsh	Consider adding a note permitting/suggesting that the unfactored axial gravity load be used when determining the effective properties.	
180	Pg 5-9	MO	5-9 What are sacrificial concrete shear keys used to protect the piles?	
181	Pg 5-14	MO	We are interpreting the recommendation of the 100 year event to be the seismic loading for the elastic design. Although a separate issue from these proposed Guideline, it does not appear that the 100 year event acceleration data is available. Using the 100 year for elastic design and reviewing displacement capacity for the 1000 year event seems very	
182	**Section 6**			

183	6.3.3 & 4 Overall	BERGER/ Lee Marsh	The mandatory requirement to base spread footing design on the rocking analysis, outlined in 6.3.4, seems to introduce a performance objective that is somewhat inconsistent with what has been required by the Guidelines in earlier chapters. This rocking approach also is less conservative than the approach that has traditionally been used, 'half uplift' under the plastic forces. The rocking analysis represents a fundamentally different behavior than that otherwise included in the analysis of the bridge. Basically, the system is being reanalyzed bent-by-bent. To exploit such behavior should be a choice the designer makes intentionally. Additionally, the apparent allowance of behavior right at the edge of stability seems unconservative, and would potentially place some structures at the threshold of toppling, because stability is likely not solely a function of the elastic response spectra. This approach could be included, but it should be done so as an option, not as a mandatory	
184	6.3.3 3rd Bullet	BERGER/ Lee Marsh	For calculation of the inertial forces, the superstructure weight should be the effective seismic weight, which depending on articulation of the bridge may include more than the gravity weight tributary to the bent.	
185	6.3.3 5th Para	BERGER/ Lee Marsh	The calculation of ductility in this section is effectively based on a Rd of 1.0. Is this the intent?	
186	6.3.3 Overall	BERGER/ Lee Marsh	The definition of D in the equations appears to require a 'T' subscript to be consistent with Figure 6.1.	
187	6.3.3 5th Para	BERGER/ Lee Marsh	It is not clear what the ductility calculation is for. Is the intent of this requirement to calculate the ductility demand assuming no rocking?	
188	6.3.3 Figure 6.1	BERGER/ Lee Marsh	The weights provided must also consider potential buoyancy effects. Omission of these would be unconservative, because the weights at the base help resist overturning.	
189		MO	Does “mu” or “ductility parameter of a rocking column/footing system” intended to be the same for all three pages.	

190	6.4.4 1st Para.	BERGER/ Lee Marsh	In the last sentence, insert 'geotechnical' between ultimate and capacity.	
191	6.4.4 2nd Para.	BERGER/ Lee Marsh	The first sentence seems to be the only place that potential tension in piles is discussed. It would seem appropriate to have an entry in the concrete section alerting the designer to consider appropriate anchorage of the piles into the cap and to consider these effects on shear in the cap.	
192	6.3.2	FHWA/ Derrell	add Forces corresponding to overstrength moment"	
193	6.3.3	FHWA/ Derrell	add Forces corresponding to overstrength moment"	
194	6.3.4	FHWA/ Derrell	Figure 6.1 has Δ_T but the formulas have Δ	
195	6.3.4	FHWA/ Derrell	"Recalculate Δ considering 10% damping…" Commentary should provide method of changing the response spectrum	
196	6.3.4 Below equation 6.5	FHWA/ Derrell	"…soil passive resistance.." should be "…soil weight (mass).."	
197	6.3.4 Last paragraph	FHWA/ Derrell	P-Delta analysis is required, but this conflicts with P-Delta requirements in 4.11.5	
198	6.3.4 Last paragraph last sentence	FHWA/ Derrell	column plastic hinging is now required as a design force, but section 6.3.2 & 6.3.3 only require rocking analysis forces	
199	6.3.4 Figure 6.1	FHWA/ Derrell	Figure 6.1 has Δ_T but the formulas have Δ	
200	6.3.4 Figure 6.1	FHWA/ Derrell	Locate "F" arrow at the CG of the structure	
201	6.3.4 Figure 6.2	FHWA/ Derrell	Clarify ΔT or Δ. Ductility factor of 8 has no consideration for structure types. Ductility factor, beta factor and minimum footing size are specified in the figure, but not placed in the actual specification	

202	Sec. 6.3.4 Pg 6-4 Eqn. 6.5M	CA/ Mike, Fadel, Mark, Lian	Should specify that μ should be determined based on local ductility, not global, due to the significant effect rocking will have on the curvature demands on the column. In addition Δshould be defined as Δ_f and the yield displacement defined locally.	
203	Sec. 6.3.4 M Pg. 6-3	CA/ Mike, Fadel,Mark , Lian	The same as Sec. 5.3.2, Pg. 5-11	
204	Pg. 6-6 Fig. 6.2 Logic box m<8 M	CA/ Mike, Fadel,Mark , Lian	Unclear how widening the footing will reduced the μ demand. As the footing size increases, rocking is reduced, footing stiffness increases, and the local ductility demand on the column will increase.	
205	Pg. 6-7 Third para. from the bottom M	CA/ Mike, Fadel,Mark , Lian	Where is "the simplified foundation model" defined? Unclear why that affects the use of Mp or Mpo to design the foundation. Seems as though Mpo should be used to be consistent with the rest of the specifications for capacity protected members.	
206	Pg 6-8	MO	Appears to be an error in format with the numbering system. . .C7.4.3.3 is incorrect.	
207	Pg 6-8	MO	Standard size piles are considered to have a nominal dimension less than or equal to 16 inches." Could you provide commentary for the use of larger piles (20 and 24" diameter concrete filled steel shell piles)?	
208	Pg 6-9	MO	6-9 "For conforming to capacity design principles, the distribution of forces on these piles shall be examined about the X and Y axis in addition to the diagonal direction of the foundation cap. (Should the loading be 100% & 30% or 100% and 100% OR 70% in both directions and what loading is associated with the "diagonal direction"?)	
209	Pg. 6-10 2nd para.	CA/ Mike, Fadel,Mark , Lian	Why use 50% of the ultimate capacity of the pile which is comprised of both skin friction and end bearing? Why not just use the skin friction?	

210	page 6-14	FHWA/ Derrell	Page 6-14 either mis-numbered or is missing	
211		MO	6-5 Information for Mo > Mr is lacking.	
212	Sec. 6.5 Pg. 6-12 Last para. M	CA/ Mike, Fadel, Mark, Lian	Using the 1.5 multiplier to determine the tip elevation of the drilled shaft is adequate for homogenous soil conditions, but can be extremely conservative and costly if the tip elevation was controlled by a rock layer at the bottom of the pile. Recommend using the elevation that has a depth that is the lesser of 1.5 the stable length for Vo, or the stable	
213	Sec. 6.7.1 M Pg. 6-13	CA/ Mike, Fadel, Mark, Lian	Use of the Monobe-Okabe method is much too conservative for areas of high seismicity. Caltrans does not design for seismic earth pressures pending the results of the ongoing NCHRP project on this subject.	
214	6.7.1 2cd paragraph	FHWA/ Derrell	"0.4 times dead load reaction" should read 0.4 times contributory mass times g or force from analysis"	
215	6.8 Item b.	BERGER/ Lee Marsh	Is the mean magnitude information (i.e. deaggregation data) available for the entire U.S. for the 1000-yr event?	
216	6.8 3rd paragraph	FHWA/ Derrell	Define California DMG. This should also be provided in the Appendix	
217	6.8 Item 2	FHWA/ Derrell	Do not mention proprietary software "DESRA"	
218	Page 6-17	CA/ Mike, Fadel, Mark, Lian	a) Second paragraph item1 should read passive pressure instead of “active pressure” b) Second paragraph item 2 should read active pressure instead of “passive pressure”	
219	Page 6-19	CA/ Mike, Fadel, Mark, Lian	Detailing of Splicing for liquefaction should cover the case where mechanical or lap splicing can not be avoided due to the extent of zone comprising the location of hinging in the liquefied and non-liquefied cases	
220		MO	6-10 Clarify “In no case shall the uplift exceed the weight of material surrounding the embedded portion of the pile?	
221		MO	Page 6-14 is missing	
222	**Section 7**			

223	section 7	CA/ Mike, Fadel, Mark, Lian	Design provisions for shear connectors between the end diaphragms and concrete deck shall be provided to ensure the critical load path during seismic events.	
224	7.1 Figure 7.1	BERGER/ Lee Marsh	The figure seems to imply that inelastic action in both the superstructure and substructure is acceptable. Section 7.2 states otherwise. Add a clarifying note the drawing.	
225	7.2.2 1st Para.	BERGER/ Lee Marsh	This is one of the few places where R factors are used. However, there is no guidance regarding how to use them. Designers understand R factors today, but without the knowledge from using Div I-A, mention of a R factor alone is not clear.	
226	Pg 7-3 7th Para.	CA/ Mike, Fadel, Mark, Lian	"LRFD Design Specification for Single Angle Members" is superseded by ANSI/AISC 360-05, *Specification for Structural Steel Buildings* , March 9, 2005, American Institute of Steel Construction, Chicago, IL.	
227	Pg. 7-5 3rd & 4th Para.	CA/ Mike, Fadel, Mark, Lian	AWS/AASHTO D1.5-96 *Structural Bridge Welding Code* is superseded by AASHTO/AWS D1.5M/D1.5:2002 *Bridge Welding Code*	
228	Pg. 7-5 5th Para.	CA/ Mike, Fadel, Mark, Lian	Statement "An effective length factor K of 0.85 shall be used unless a lower value can be justified by an appropriate analysis" is incorrect. It is only valid for compression members in braced frames. It shall be revised to read as ""An effective length factor of compression members in braded frames, K of 0.85 shall be used unless a lower value can be justified by an appropriate analysis"	
229	Pg 7-7	CA/ Mike, Fadel, Mark, Lian	Table 7.1. Column 1- Row 2 "Ductility" shall read as "Ductile" Row 3 - "Mn" shall read as "Mns"	
230	Pg. 7-7 Pg. 7-12	CA/ Mike, Fadel, Mark, Lian	Both Table 7.2 and 7.3 have same title "Limiting Width-to-Thickness Ratios". For ductile components, there are two different requirements. Which one shall be followed?	

231	Pg 7-8	CA/ Mike, Fadel, Mark, Lian	AISC-LRFD (1993) and AISC-Seismic Provisions (1997) are superseded by ANSI/AISC 360-05, *Specification for Structural Steel Buildings* , March 9, 2005, and ANSI/AISC 341, "*Seismic Provision for Structural Steel Buildings*" March 9, 2005, American Institute of Steel Construction, Chicago, IL., respectively. Table 6.2 shall be updated.	
232	Pg 7-10 Line 19	CA/ Mike, Fadel, Mark, Lian	There is no publication titled as "LRFD AISC Seismic Provisions for Structural Buildings 1997". The correct title shall be "Seismic Provisions for Structural Steel Buildings".	
233	**Section 8**			
234	8.1 3rd Para.	BERGER/ Lee Marsh	If different permissible ductilities are retained for single and multi-column bents, add clarifying language regarding the treatment of multi-column bents in their strong and weak directions.	
235	8.1	AR	Article 8.10 says "The column overstrength moment.... shall be distributed to the left and right spans of the superstructure". Are these left and right spans the spans to the left and right of the bent that is transferring seismic load or the end spans? Please explain further.	
236	8.2	AR	Es in Figure 8.2 reads 28,5000 ksi instead of 28,500 ksi. There is an extra zero.	
237	8.2 1st Para.	FHWA/ Derrell	Add"….and the connection force shall be carried through the substructure.."	
238	Sec. 8.3.1M Pg. 8-1	CA/ Mike, Fadel, Mark, Lian	Recommend that $D_{col} \leq D_{superstructure}$	
239	Sec. 8.3.2	CA/ Mike, Fadel, Mark, Lian	Design of columns for unreduced elastic forces is very risky, particularly in shear.	
240	Pg 8-2	MO	Article X.X	

241	Pg 8-3	MO	Should the size of the transverse hoops and ties shall be equivalent to or greater than #4 rather than #3 as shown? Clarify "Ties shall be used to provide lateral restraint to intermediate longitudinal bars within the reinforced concrete cross section." 8-2	
242	8.4 Heading & 1st Para.	BERGER/ Lee Marsh	The full development of displacement capacity, as referred to in this section, is only used in SDC B & C as an option. This should be clarified.	
243	8.4.2 Overall	BERGER/ Lee Marsh	Consider building in some conservatism to the permissible ultimate strain limits. It seems that the expected ultimate strain is permitted to be used. Both Caltrans and the CA Marine Oil Terminal (MOTEMS) criteria use reduced allowable strains.	
244	8.4.2 1st paragraph	FHWA/ Derrell	The sentence requiring A706 steel should be located in 8.4.1 since 8.4.2 is how to model steel.	
245	8.4.2 1st paragraph	FHWA/ Derrell	is A706 steel required in the entire structure or only in the hinging locations?	
246	8.4.4 equation 8.7	FHWA/ Derrell	The strength of 5000 psi is based on an assumed initial concrete strength. This needs to be spelled out since all States do not use the same initial strength concrete for substructures	
247	8.4.4 4th Para.	BERGER/ Lee Marsh	Include a reference citation for Mander's model.	
248	8.4.4 last paragraph	FHWA/ Derrell	Manders model needs to be referenced or commentary provided	
249	8.5 last paragraph	FHWA/ Derrell	The overstrength factor of 1.2 is to account for material uncertainties. Since the actual material properties are required in calculating capacities, isn't applying the overstrength factor in addition to actual properties too conservative?	
250	8.5 Overall	BERGER/ Lee Marsh	Add a requirement that appropriate (e.g. dead/permanent) unfactored axial forces must be included in the M-f analysis to obtain the correct capacities, and no resistance factors should be included with this.	

251	Sec. 8.6.	TN/ Huff	The units seem to be off in Equation 8.13 of Section 8.6. *P* is stated to be in kips, but I believe it should either be in pounds or the factor of 2000 in the denominator should be changed to 2: $$v_c = \alpha'\left(1 + \frac{P}{2000 A_g}\right)\sqrt{f'_{ce}}$$	
252	Sec. 8.6 Page 8-8	AK/ Elmer	In all locations where the concrete member capacity is calculated the expected concrete strength, f'ce, is specified. It would seem appropriate to use f'c when calculating a member capacity and f'ce when calculating a member demand. This comment is applicable to most of Section 8 and parts of Section 6	
253	8.6.1 & 8.6.2 & 8.6.3 most equations	FHWA/ Derrell	fonts not uniform	
254	Sec. 8.6.1	CA/ Mike, Fadel, Mark, Lian	The shear demand for column Vd SHOULD NOT be the force obtained from elastic analysis. It should always be the force corresponding to plastic hinging.	
255	8.6.3 Eqn 8.25	BERGER/ Lee Marsh	Suggest using an alternate term to A_v for spiral sections to avoid confusion with the shear area for rectangular sections. Perhaps A_{nsp} since this applies to both spirals and interlocking spirals?	
256	8.6.8 1st Para.	BERGER/ Lee Marsh	Are there any minimum overlap requirements for the interlocking spirals? (e.g. max center-to-center of spirals of 0.75 dia of spiral)	
257	8.7.2 1st Para.	BERGER/ Lee Marsh	Is the maximum axial load permitted with or without seismic overturning effects? Suggest without just for simplicity.	
258	8.8.3	AR	In Article 8.8.3, does Sentence 2 mean that Lap Splices and welded splices in rebar in SDC D are forbidden for use?	

259	8.8.4	AR	Article 8.8.4 does not mention the 1.25 factor that increases the development length of the column reinforcing into the footing or cap. This factor has been in the LRFD and LFD Specifications for a long time. Do we want to be less conservative in this area? Also, why is it not desirable to have hooks in SDC D?	
260	8.8.7 2nd Para.	BERGER/ Lee Marsh	Does the second paragraph mean that if the ductility demand is less than 4, then no special requirements are necessary?	
261	8.8.7 3rd Para.	BERGER/ Lee Marsh	Does the wording of this paragraph also permit the use of spirals welded back onto themselves (with fillet welds) to facilitate the placement of steel at joints?	
262	8.8.8	AR	Article 8.8.8 mentions ending a spiral with 1 turn as opposed to the historical 1 ½ turns. Is there evidence that indicates 1 ½ turns was too	
263	8.8.8 last paragraph	FHWA/ Derrell	Two paragraphs in this section require different amounts of steel outside the plastic hinge. The first says50% and the second states "same amount". This is conflicting	
264	8.8.9 all	FHWA/ Derrell	Since plastic hinging can also occur with SDC B, maximum spacing requirements should also apply	
265	8.8.10 all	FHWA/ Derrell	Add to last sentence "…for SDC C & D, respectively"	
266	Sec. 8.9 Page 8-17	AK/ Elmer	For members that are designed to remain essentially elastic, it does not appear to be appropriate to design for a concrete strain of 0.005 (spalling strain limit) and esu (ultimate tensile steel strain limit) as defined in Article 8.4 -- as both of these limits are beyond an elastic limit. Perhaps a concrete strain of 0.002 and a steel strain less than the tensile yield strain would be more appropriate.	

267	8.12		Section 8.12. The second sentence states "The minimum lateral transfer mechanism at the superstructure/substructure interface shall be established using an acceleration of 0.4g in addition to the overstrength capacity of shear keys or the elastic seismic force whichever is smaller." What is the intent here? (i) 0.4 times the reaction plus the minimum of (a) the key capacity and (b) the elastic force or (ii) the minimum of (a) 0.4 times the reaction plus the key	
268	8.12	AR	The designation for Articles C8.8.4.3.2, C8.8.4.4, C8.8.5.3, C8.8.6, and C8.8.6.1 is incorrect or these articles are out of place. They are near Articles 8.12 and 8.13.	
269	8.13 Figure 8.7 & 8.8 & 8.9, 8.11	FHWA/ Derrell	Clarity of text is not sufficient, larger fonts needed on some text	
270	Sec. 8.13.4.2	CA/ Mike, Fadel, Mark, Lian	The joint shear reinforcement may be provided in the form of column transverse steel or exterior transverse reinforcement. Need to add "exterior transverse reinforcement:.	
271	Sec. 8.13.4.2	CA	Need to make reference to the additional reinforcement not required for SDC C	
272	Sec. 8.13.4.2 Page 8-24	AK/ Elmer	Recent publications [Sri Sritharan, J. Struct. Engrg., Volume 131, Issue 9, pp. 1334-1344 (September 2005)] indicate that the principal tension stress, pt, should be limit to 3.0*sqrt(f'c) as opposed to the 3.5*sqrt(f'ce) provided in the proposed specifications.	
273	8.16.1 1st Para.	BERGER/ Lee Marsh	It is not clear what 'not designed as capacity protected members' means. I think this means if plastic hinging is expected. Perhaps reword this as such. I presume that this section also covers pile bents (i.e. pile extensions) where plastic hinging would be expected at the top of the pile and potentially in-ground.	
274	Figure 8.9	CA/ Mike, Fadel, Mark, Lian	There in no mention in the text where the extra 12" width is required.	

275	Page 8-26	CA/ Mike, Fadel, Mark, Lian	Paragraph D: J dowels are only required for integral caps.	
276	General	CA/ Mike, Fadel, Mark, Lian	There is no a list of “References”	
277		AR	Appendix C is called "Guidelines for Modeling of Footings". A more appropriate name might be “Guidelines for Modeling of Footings and	
278		AR	Appendix C has the same spring constant graphs for translation of piles in the longitudinal and transverse directions that have been previously used. A discussion on their use and/or an example would be beneficial.	
279		AR	Appendix Art. D.2.3: The USGS web address for finding earthquake magnitude is outdated. It is not obvious what distribution is being discussed, or what map to use for earthquake magnitude.	