Review Discipline: Bridge Office

Comments Due By: 10/15/2007

Reviewed By: Tim Moore

Date Reviewed:

Comment Resolution Date: 10/25/2007

ALASKAN WAY VIADUCT & SEAWALL REPLACEMENT PROGRAM COMMENT REVIEW AND RESPONSE

Subject: Alaskan Way Viaduct Seismic Vulnerability Study

CODE: A-Will Comply; B-Consultant to Evaluate; C-Will Not Incorporate; D-WSDOT to Evaluate

Agency	Ini-tials	Item No.	Sec-tion	Drawing #/ Page #//Line #	Review Comments	Comment Level*	Design Responses	Designer's Initials	Review Action	**CR Disp.	Final Disp.
WSDOT	TMM	1	Exec	1	Third paragraph is nearly word for word duplication of the first paragraph. Please remove all sentences except for the portion regarding shear failure in footing. This could be supplemented with joint shear failure in footing and soil liquefaction.	3	Will Revise .	G. Inverso	A	A	
WSDOT	TMM	2	1.1	4	Return periods for seismic events must be viewed as statistical indicators of the likelihood of a seismic event occurring at a given time". Remove "as nothing more".	3	We will revise the last paragraph of section 1.1 to include: "A ground motion 'return period' expresses the annual rate at which a ground motion level is exceeded at a site. It is a convenient way to express the percent probability of ground shaking occurring or being exceeded for any period. Return periods do not imply that the ground motion occurs once every certain number of years at a site."	G. Inverso	A	A	

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WSDOT	TMM	3	1.1	4	Strike last three sentences in this Section. This only adds uncertainty in all statements made in the report and lowers the credibility of the findings.	3	Seismic forecasting is uncertain. The results are statistical in nature. Some qualifications of the findings are necessary. Due to limited knowledge and statistical uncertainty; 108-year and 210-year ground motions error ranges can overlap. A given event, like the Nisqually (See Figure F2), can perform in different return periods, given the structural period. Speaking with assurance about "108-year", "210-year" implies a precision in the results that is not present.	G. Inverso	B	C	
WSDOT	TMM	4	1.2	4	Strike *"robust"* seismic behavior.	3	"Robust" is a fairly standard term in design that implies 'defense in depth' or 'multiple redundancies'.	G. Inverso	B	A	

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WSDOT	TMM	5	1.2	5	Are you making a comparison of the Nisqually earthquake to the 1965 earthquake? If so, identify with magnitude or acceleration of both events.	3	Will Consider; The 1965 Seattle Tacoma Event had a magnitude (mb) of 6.5. The Nisqually Event had a magnitude (Mw) of 6.8. These are from the Shannon & Wilson's 10/2004 *Seismic Ground Motion Study Report* for the AWVSRP (Appendix A Section A.2.1). The type of "Magnitude" is different. The Seattle-Tacoma event is in Body Wave (mb) magnitude and the Nisqually event is moment magnitude (Mw). The Seattle Tacoma Event was closer to the Viaduct. A smaller magnitude event close in may have higher seismic demands on a structure than a larger magnitude event further away. The moment magnitude approach was developed because body wave magnitude tends to understate the earthquakes energy starting between magnitudes 6 and 7. Delving into the comparisons of the two events may become esoteric. For the purpose of the report, "similar size" may suffice.	G. Inverso	B	C	

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WSDOT	TMM	6	1.2	5	2nd Paragraph - "With these subsurface conditions and *expected* ground motions, the soils can liquefy…"	3	Per the PBQD; 3/2006; *Alaskan Way Seawall Without Project Conditions* ; *Corps of Engineers Feasibility Study;* Revision 0, the "expected" shaking from a 108-year ground motion does not liquefy sufficient soil to collapse the Seawall. The 200-year or higher ground motions may produce the "right" amount of shaking to liquefy sufficient soil to collapse the Viaduct.	G. Inverso	B	C	
WSDOT	TMM	7	1.2	5	2nd Paragraph - You have to define "downdrag" to the lay reader.	3	Will reword to replace "… down drag …" with "… as the ground settles it can drag the pile with it thus …".	G. Inverso	A	A	

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WSDOT	TMM	8	1.2	5	Third Paragraph - "If the soil does not liquefy, the Seawall will likely withstand an earthquake of sufficient magnitude to initiate collapse of the Viaduct". This sentence should be a separate paragraph. The remaining two sentences are relevant to the thoughts described in the earlier sentences of the paragraph.	3	Will make some revisions. As is, the text is sufficient to convey the meaning. This is a stylistic element of the report. The first sentence in each paragraph summarizes what is in the paragraph. If one reads the first sentence in each paragraph, it gives a succinct summary of the sections. The sentence with "If the soil does not liquefy …" is directly related to the sentence that follows "If a significant proportion of the soils do liquefy …" and is related to the topic sentence "The presence of the Seawall can also indirectly affect the stability of the Viaduct's foundations". A qualification will be added that the Seawalls are in their original conditions; not the current degraded state.	G. Inverso	B	A	

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WSDOT	TMM	9	1.2	5	Fourth Paragraph - Last sentence has no significance of the timing between	3	Comment appears incomplete. The stability of the Viaduct and Seawall are related. Failure of the Seawall can initiate a chain of events that can fail the Viaduct. In the lower level events, ground motions with return periods less than 500-years, maximum liquefaction that can collapse the seawall may occur after the maximum shaking that can collapse the Viaduct. To forecast the ground motion the Viaduct can withstand, the Seawall stability needs to be evaluated.	G. Inverso	B	C	
WSDOT	TMM	10	1.3	6	Fourth Paragraph - First sentence strike "because they do not have modern detailing for robust seismic behavior". Terminology is out of context and reads poorly.	3	The part of the sentence that states "… because they do not have modern detailing for robust seismic behavior" will be changed to "… because the arrangement of reinforcement is not sufficient to provide ductile seismic behavior".	G. Inverso	A	A	

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WSDOT	TMM	11	1.3	8	Third Paragraph - Please provide a comparison of the incipient shear failure in the Lower Deck Floorbeam as it compares with the footing joint shear or shear in the footing failure scenario. This draft makes no mention of ultimate shear capacity of the lower floorbeam in Fig 1.5, 1.6 & 1.7. This reviewer believes it would add credibility to the findings of this report to be able to compare with the SSRC conclusions of failure due to acceleration of 0.26g and structural period of 1.5 sec. At what base shear will the lower floorbeam fail in shear? The information is relevant to the reviewer even if the author doesn't believe it's important information for the report.	2	See p 8 para 3 & 4; p 16 para 4 & 5; and Appendix D. There is not "an" ultimate shear capacity. Shear capacity is a function of concrete, steel, load and ductility. At each push increment, this analysis calculated capacities for the given load and ductility. In previous work 3.5√f'c was the principle tensile stress for joint shear failure limit, where cracking begins. This analysis used a 5.0√f'c joint shear failure limit per MCEER-06-SP10, 2006, *Seismic Retrofitting Manual for Highway Structures; Part 1 Bridges* . Although the 3.5√f'c limit was exceeded for certain foundation conditions, the 5.0√f'c limit was not exceeded before crushing failure of the column terminated the pushover analysis. SSRC procedures and loads were not given. A comparison is difficult. The UW; 7/1995; *Seismic Vulnerability of the Alaskan Way Viaduct: SED Typical Unit* used a similar shear capacity procedure with a 3.5√f'c limit. Again, push increment loads were not given; making a comparison hard.	G. Inverso	B	C	

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WSDOT	TMM	11	1.3	8	Comment 8 Third Paragraph (Continued): *This reviewer believes it would add credibility to the findings of this report to be able to compare with the SSRC conclusions of failure due to acceleration of 0.26g and structural period of 1.5 sec. At what base shear will the lower floorbeam fail in shear? The information is relevant to the reviewer even if the author doesn't believe it's important information for the report.*	2	Per a 26 Oct 2007 discussion: Comparing shear between SSRC and this work is difficult. SSRC shear load/capacity data are for exterior bents with a split column configuration that differs significantly from the Bent 83 interior bent. A direct comparison of interior bents would pair Bent 83 to Bent 98 of the SSRC structure. Bent 83 and Bent 98 heights are similar. Their column reinforcement and details are identical. Bent 98 is several feet wider. Cross girder dimensions differ significantly: 82.5”x29.0” for Bent 83 and 90.0”x37.0” for Bent 98 (HxW). The stiffer Bent 98 girders can draw ~26% more moment and corresponding shear than Bent 83. Additional moment drawn to the stiffer girder reduces the column moments. Thus, girder shear may govern over column moment failure in Bent 98. An analysis similar to that run for Bent 83 is need for direct comparisons. Figure 1.7 shows the general performance of Bent 83 and the SSRC structure relative to the EE is similar.	G. Inverso	B	C	
WSDOT	TMM	12	1.4	9	Findings - 1st Paragraph - "collapses" should be collapse.	3	Will revise.	G. Inverso	A	A	

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WSDOT	TMM	13	1.4.2	11	Strike Second Paragraph - Similar statements were made in Section 1.2, this paragraph adds no value to support the findings.	3	This is an element of style. Section 1.2 is an introduction/summary of seismic vulnerability. Sections 1.3 and 1.4 are more in-depth discussions of the Analysis Procedure and Findings respectively. Some paraphrasing of various sections is expected.	G. Inverso	B	B	

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WSDOT	TMM	14	1.4.2	11	Fifth Paragraph - Fig 1.8 - Please acknowledge that the findings also show that all models appear to have adequate strength and ductility to withstand "Zone A" ground motions.	2	The general statement cannot be made. The analysis was performed for the "Zone B" site-specific spectrum. The Zone "A" soils profiles are different. The structural configurations in terms of structural height and reinforcing configuration may also be different for Zone "A" structures. The Capacity Spectrum Method procedure was not run for Zone "A". Additional schedule and budget would be necessary to analyze a representative Zone "A" structures. Given what was seen in the analysis of a single Zone "B" bent combined with the SSRC data point, it is more reasonable to say the 108-year EE ground motion is sufficient to severely damage or collapse sections of the Alaskan Way Viaduct whether they are in site specific spectra zones "A" or "B". Additional analysis is necessary to support the contrary opinion. A statement identifying Zone "A" profile as reference only will be added.	G. Inverso	B	B	

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WSDOT	TMM	15	1.4.3	12	Second Paragraph - "Soil layers in this liquefies condition can exert three to five times the lateral force on an adjacent structure than the same soil when it is not liquefied". The equivalent fluid weight of non-liquefied granular soil for retaining wall design is 30 pcf. If soil is completely liquid the equivalent fluid pressure cannot exceed 62.4 pcf. Even if M/O forces are added to support the statement in the text, the unit weight of soil is linear and would not increase more than 2.1 times. Is there more clarification needed to support the statement that the forces increase by a factor of 3 to 5?	2	Sentence will be changed to: "Soil layers in this liquefied conditions can exert three times lateral force under non-liquefied, non-earthquake loading conditions (e.g., see earth pressures provided in the Berger/ABAM 2003b)." When a soil liquefies, or is completely submerged, the lateral static forces exceed the unit weight of water. Water does exert 62.4 pcf on the wall. In addition, the soil exerts a lateral wall load as a function of its buoyant unit weight time the lateral coefficient K (Ka, Ko). As a soil liquefies, K for the soil skeleton approaches 1.0. The static lateral pressure component can approach the saturate unit weight of the soil. In a liquefied state, excess pour water pressure and hydrodynamic pressures are also present. Liquefied earth pressures presented in project reports were developed per the US Army Corps of Engineers Technical Report ITL-92-11 *The Seismic Design of Waterfront Retaining Structures* , which recommends procedures other than M/O for liquefied soil conditions.	G. Inverso	A	A	
WSDOT	TMM	16	2.0	15	Second Paragraph - "substantial" should be substantially	3	Will revise.	G. Inverso	A	A	

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WSDOT	TMM	17	2	16	Last Paragraph - Characterization of structural performance with words such as "robust" is inappropriate. I suggest you strike the entire paragraph since your first statement is invalidated with the last.	3	This study is attempting to predict collapse of the structure. This is slightly backwards from design, which is to prevent collapse. A conscious effort was made to be on the liberal side of code interpretation regarding structural capacity; thus the “robustness” that has been mentioned in five comments. A more conservative capacity approach would be open to two criticisms. First, if the capacities are too conservative, then the conclusions are alarmist. Second, capacities that are more conservative would predict ground motions with return periods even less than the 108-year EE could initiate collapse, which differs from observations. Per Appendix F Figure F3 for two of the three performance points, the Nisqually event demands not only exceeded Bent 83's capacity it also exceeded the EE demand. The Viaduct structures suffered damage in the Nisqually event requiring significant repair (Bents 93-94). This indicated the estimate of collapse capacities are in the correct range.	G. Inverso	B	B	

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WSDOT	TMM	18	3	17	First Paragraph - first sentence - "inducing limit state stability demands" is awkward and is inconsistent with terminology used throughout this summary report which is intended for the lay person.	3	Will revise.	G. Inverso	A	A	
WSDOT	TMM	19	App D	D.4	The Example Joint Shear Calculation is a valuable addition from the previous draft. Why not consider providing the base of column and footing joint for the calculation? This is controlling failure scenario and the basis of the thesis of this report.	3	Additional development of the technical appendices would be good. However, this is outside the immediate scope of this report.	G. Inverso	B	B	

*	Comment Levels	By Definition, Means the Reviewer's Comment is:	Action Required
	1	A critical or policy issue	Must Include
	2	A factual issue to consider	Check on it
	3	Editorial Only	Consideration
**	P/S/E	Plans/Specs/Estimate	
***	CR Disp.	Comment Resolution Disposition	

Document Reviewed: **Draft Seismic Vulnerability Analysis - August 20, 2007**

No.		Page	Comment	Reviewer	Resp	Action Taken
1		1	Return period could be more accurately described along these lines - "The return period is average period of time at which a certain level of ground shaking is reached or exceeded." The "X/Y/probability" terminology seems likely to lose a lot of lay readers. If it was important to get that concept across, the suggested sentence could be followed with something like " - it can be used to predict the probability of reaching that level of shaking in any period of time."	SLK	G. Finn	Statement revised as indicated.
2		1	Remove the second half of 3rd paragraph, and present a clear statement that the Nisqually EQ event has been re-characterized in terms of local seismicity, and that event and the evidence gained therefrom points to a higher sesmic risk than previously considered for the Viaduct.	DG	G. Finn	Section revised. The higher seismic vulnerability is primarily based on the most recent, updated site-specific ground motions.
3		1	The term "risk" implies dollars (cost, loss) and is used herein to describe hazard and probability. Depends on your audience, but sticklers may object to looseness of terminology.	SLK	G. Finn	The term "hazard" in this context is more appropriate. Sentence revised.
4		2	Editorial in the 4th paragraph, 2nd line. "Soil Liquefaction, which may occur immediate…….seawall. Change to "may occur immediately……..seawall.	R. Imbsen	G. Finn	Stet. Current sentence is correct.
5		2	Clarification in the 5th paragraph 4th line. Should the analysis be risk or structural?	R. Imbsen	G. Finn	This paragraph discusses the concept of risk in broad terms.
6		2	If you arrived at the Nisqually spectrum in Figure F-3 based on a surface record, I believe for the general case this could be smoothed. But in any case, this key figure should be clearly referenced in the summary - perhaps in a reworded presentation at the top of page 2.	DG	G. Finn	Text will be modified to reflect Appendix F. Also, see comment 27.
7		2	The presentation in the last 2/3 of this page is a bit verbose and not really to the point for an exec summary. Following on the prior comments, the last 4 paragraphs should be replaced with a brief statement that the strength of the viaduct (whether referenced to the current or past analysis is not material), when compared with the the local seismicity, may be exceeded in a 108 year event, rather than the 210 year event previously forecast. The remainder can be omitted, pickup up again on page 3 with "Over a 10 year period,..."	DG	G. Inverso	Some modifications made. This executive summary is written to be stand-alone document in non-technical terms. It was written under the assumption that key non-technical decision makers may read may only read this part of the report.
8		2	In the last paragraph, consider "The probability of occurrence is a function of time" rather than "Risk is a function of time. The statement "For a short-period say 10 year, the risk has doubled, but the likelihood is smaller," is hard to understand.	TI	G. Finn	See response to comment 12.
9		2	Change to "Risk is a function of <u>exposure</u> time." That is what the periods of time described in the following sentences refer to. Then, in next sentence, say "For a long <u>exposure</u> period, …"	SLK	G. Finn	Statement revised as indicated.
10		2	"For a short period, say 10 years, the risk has nearly doubled, but the likelihood is smaller" - I don't understand what this is referring to (could be terminology problem -- risk vs likelihood).	SLK	G. Finn	Sentence deleted.
11		3	Change "goes" to "go"	SLK	G. Finn	Word changed.

Document Reviewed: **Draft Seismic Vulnerability Analysis - August 20, 2007**

No.		Page	Comment	Reviewer	Resp	Action Taken
12		3	Delete second to last paragraph. If this report is for the public, you just compromised the prior thesis with this paragraph.	DG	G. Inverso	In the end, we are dealing with statistical distributions. It is misleading to present return period, liquefaction potential, structural capacity and the complex interactions between them as precise numbers.
13		4	The second paragraph should be edited or omitted with respect to the Cypress Viaduct. The public has been told for some time that AWV is NOT a Cypress Viaduct, and other than being a double deck, there are few if any similarities in the structural system.	DG	G. Finn	We note that the AWV and Cypress Street Viaduct were superficially similar (i.e. both were double-deck viaduct structures) but the structural systems were different. Paragraph will be altered to clarify superficial nature of similarity.
14		4	Delete reference to the SSRC's commission to "and to estimate a return period…". That was not anywhere in the SSRC commission. The commission was simply to recommend to the DOT what to do with the viaduct. The data presented was in support of our recommendation, but estimating return period (in a 6 week time frame) was not in the charge.	DG	G. Inverso	Text will be revised
15		4	It would have been helpful to have access to the referenced reports. I tried to do a search on the University of Washington website, but was unable to locate the reports	R. Imbsen	G. Finn	The following reports were sent to R. Imbsen in an email (Finn to Imbsen) dated 8/29/07): Ryter, S., Eberhard, M., Colina, J., 1995. Seismic vulnerability of the Alaskan Way Viaduct: WSDOT typical unit. Report to Washington State Department of Transportation, (WA-RD 363.3). T.Y. Lin International, 2001. Alaskan Way Viaduct – Report of the Structural Sufficiency Review Committee. Knaebel, P., Eberhard, M., Colina, J., 1995. Seismic vulnerability of the Alaskan Way Viaduct SED typical unit. Report to Washington State Department of Transportation, (WA-RD 363.1).
16		4	Change "Eberhart" to "Eberhard"	SLK	G. Finn	Name corrected.
17		5	At the point where you note that past detailing " did not fully recognize the way structures can fail under seismic demand" as the basis for not having seismic details, it would be more accurate to convey that in the 1950's and essentially until 1971, earthquake was considered a force majeure, and not justified as a normal design load.	DG	G. Finn	It is sufficient that the lay reader simply understand that detailing practice has changed as a result of increased knowledge of the effects of seismic demands on structures.
18		5	In the second paragraph, I suggest that the stiffness discussion can be more brief, but that there should also be a clear statement that the current deteriorated state of the Viaduct makes it more vulnerable than when it was in like new condition. Keep in mind that you are now forecasting such a high risk as to generate public skepticism, given the age of the structure and the recurrence interval for significant events. It seems important to recognize that the viaduct is in poor condition for terms of reference other than earthquake.	DG	G. Inverso	Wording regarding deterioration and increased vulnerability added.
19		5	The "dot" representing the SSRC result should be referenced in the legend.	TI	G. Finn	Agreed. Label to be added to the SSRC results symbol on the graph.

Document Reviewed: **Draft Seismic Vulnerability Analysis - August 20, 2007**

No.		Page	Comment	Reviewer	Resp	Action Taken
20		6	In second paragraph, last sentence…the reader was just informed that the structural detailing was key to survival. In this sentence, you say that soil liquefaction is key. I do not believe you want to imply that without liquefaction all is ok. This section should be reworded to clarify the parallel scenario with the sea wall.	DG	G. Inverso	Section 1.2 lists three sources of seismic failure. Each has various components and aspects that are discussed in subsequent paragraphs. One source of failure, say detailing, does not preclude the others say foundation failure. The sources are interrelated as is discussed. The paragraph in question is discussing the vulnerabilities of the foundation, of which liquefaction threatens the stability in several ways.
21		6	In the 3rd paragraph, I suggest a discussion of the mechanics of liquefaction in terms of pore water pressure and soil structure along the viaduct might be more instructive than the current text, especially for the public.	DG	G. Inverso	A brief general description of liquefaction is given in this section. A slightly more in-depth description is given in Section 1.4.3. As with all reports that have some general public consumption it is a trade off on how much depth should be given for technical issues. In this report, the general sections were kept on a less technical plane. The technical appendices are written with a great deal more technical detail.
22		*6*	As I understand both Bents 83 and 152 were used in the vulnerability report. As shown on the design plans they have different cross sections. It would be helpful to the reader if a more readable sketch of each is included in the report with the corresponding dimensions and longitudinal reinforcement. Also it would be helpful to give the percentage of steel for each cross section. The moment-curvature results shown in Figure G3 seems a bit more robust in both moment capacity and ductility capacity than I would expect from a column w/o confinement steel	R. Imbsen	G. Finn	Figures 2.2 and 3.2 are copies of the as-built drawings. Owing largely to the vintage of the drawings, the quality of the reproduction is poor. However, these figures represents the information available to those involved in the analysis. The moment-curvature relationship shown in Figure G3 is included for verification purposes only. The intention is to compare the m-κ results of a 'test' section obtained from GTSTRUDL with that calculated by XTRACT, which do show good agreement. However, the overall geometry of the 'test' section, longitudinal reinforcement content and confinement steel configuration do represent those of a Bent 83 column at the pile cap interface. As per telecon on 8/28/07 (Imbsen/Inverso/Kirandag/Finn), the XTRACT input file has been sent to Imbsen for verification.
23		6	Another mechanism we identified in our work years ago is loss of pile tip resistance due to downward migration of porewater pressures from liquefied zone. The piles have very little penetration into the dense soils below the liquefiable zone, and the effective stresses (hence, tip resistance) will decrease following shaking regardless of whether the seawall fails or not. Result would likely be limited vertical movement of pile tips, but it would be irregular between (and, to a degree, within) pile groups. I don't know if this is within the scope of this investigation, but I thought I would mention it.	SLK	G. Finn	The purpose of the study was to determine the likely earthquake return period that would cause instability of the viaduct leading to potential collapse. Damage resulting from post-liquefaction differential settlement was not deemed to be critical to immediate life-safety and was therefore excluded from the scope study.

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24		7	I question the validity of a SDOF characterization with only a first mode lateral pushover for the double decked viaduct for all stations, at least relative to a multiple time history approach. While in the end it appears the conclusion is the same, I would not hold out the SDOF pushover as anything more than a barometer, especially given the empirical evidence that the public has as a basis.	DG	G. Finn	Nonlinear static procedures (NSPs), which transform MDOF systems into SDOF oscillators, are commonly used for the seismic design and assessment of structures (FEMA 356, ATC-40, FEMA 440). While nonlinear dynamic analysis may reduce the level of uncertainty in the results, analysis in the time domain requires considerably more time, budget and effort to construct, exploit, interpret and validate than that associated with NSPs. Indeed Eurcode 8 Part 2, which deals specifically with bridges, implements the nonlinear static N2 method. MCEER-06-SP10, 2006, on seismic retrofit of highway structures specifically discussed the Capacity Spectrum method. The nonlinear static procedures used in the AWV Seismic Vulnerability Analysis are appropriate tools to estimate an earthquake return period, which could cause instability of the viaduct leading to collapse.
25		7	The third paragraph is repeated information. Suggest deleting.	DG	G. Inverso	Will consider
26		7	In the 5th paragraph, it is not clear what you are trying for with this discussion on conservatism. Recognizing that you are pushing the risk to a coin toss level for a structure that the public has driven on for 50 years, it seems you should be promoting reliability of the analysis, not conservatism. You can take the same engineering process and address the 'calibration' afforded by the earthquake and present a more convincing case. I would delete this paragraph as is.	DG	G. Inverso	Conservatism here refers to the structural analysis procedure using fixed based connection. Fixed based analysis is often used to get the maximum stiffness of the structure. Some assume that is the worst case. The paragraph discusses that fixed base analysis may mask other failure modes. Will modify the wording.
27		7	Need to explain why two non-linear analyses are required.	R. Imbsen	G. Finn	Both the N2 method and the CSM present similar performance-based engineering methods that rely on nonlinear static analysis procedures for the prediction of structural demands. While both procedures involve the generation of a pushover curve to predict the inelastic force-deformation behavior of the structure, they differ in the technique used to calculate the demand for a given earthquake ground motion. The N2 Method, which has been implemented in Eurocode 8, uses R-μ-T relationships to modify the elastic demand spectrum to create an inelastic spectrum, whereas the CSM, adopted in ATC-40 and FEMA 356, ductility/hysteretic damping relationships to determine a suite of equivalent damped elastic demand spectra. Each method has known limitations, so it was deemed prudent to adopt both methods to avoid reliance on the results obtained from a single analysis procedure. Explanation given during telecon on 8/28/07 (Imbsen/Inverso/Kirandag/Finn).

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28		7	At the bottom of the page the foundation conditions for the non-liquefied and liquefied conditions are described as two of the three cases investigated. Since these two conditions include the soil/structure interaction it would be helpful to mention it here.	R. Imbsen	G. Finn	The analysis of both liquefied and nonliquefied soil conditions included soil springs to model the soil-foundation interaction. The derivation of such is included in Appendix E. A reference to Appendix E will be added to the report.
29		7	Pg. 7 - 4th paragraph - The soils around the footing caps are well above the water table per Fig. 8.1 and therefore not susceptible to liquefaction. Does the lateral soil analysis in DFSAP consider the water table elevation? The last sentence in the aforementioned paragraph may be in error.	TM	G. Finn	The geotechnical engineers provided DFSAP parameters for each bent location, considering subsurface profiles and water table elevations. The last sentence in this paragraph will be revised. The list of soil parameters used will be added to Appendix E.
30		7	Pg. 7 - Sixth paragraph should be located in front of fifth paragraph as a more appropriate description of various foundation modeling techniques.	TM	G. Inverso	Will comply.
31		8	Consider moving most of the GT-Strudl info to an appendix.	DG	G. Inverso	Will consider.
32		8	Last paragraph should address the condition of lateral spreading vs local liquefaction. You have previously addressed the sea wall and said how critical lateral spreading (your 'mass movement') is, and now it appears you are saying all liquefaction remains in place. Also, it is not clear that pile gap elements (no connection) is either conservative or unconservative in isolation, since gap elements can affect the dynamic response.	DG	G. Inverso	The last paragraph on Page 8 (Section 1.3) discusses the procedures used to develop the lateral soil springs used in the model for liquefied and non-liquefied soils. The liquefied soil spring values establish a boundary condition for lateral stiffness of the footing/pile cap system. The fact that the soil can move, flow, or spread lateral once liquefied is a function of confinement. Confinement is related to the seawalls integrity, which is independent of the procedure used to develop spring values.
33		8	As discussed in the middle of the page the GT-STRUDL model has some limitations. The limitations are in the member and joint shear. As discussed these limit states are investigated using the demands obtained from the incremental pushover analysis. Additional information is needed to explain the models and results of these analyses conducted for the "T" and knee joints at the lower and upper levels, respectively. Intuitively, I would expect, based on what was observed on the Cypress Street Viaduct failure, i.e., problems with the "T" joint. I would also expect that the footing with no tensile capacity would be a bit more forgiving than the "T" joint.	R. Imbsen	G. Finn	Information on the determination of member and joint shear capacites is included in Appendix D. Owing to the different structural configurations between the AWV and the Cypress Street Viaduct, Tee joint overstress was not identified as the primary failure mechanism in the analysis. Since principal tension joint stresses were calculated at each step of the pushover analysis, it would be impractical to provide the results of the joint stress calculations within the report. An example calculation will be provided in Appendix D.
34		8	Pg 8 - 3rd paragraph - relations should be relationships	TM	G. Finn	Will comply.
35		10	There is a typo in the second paragraph "Theses bents…….defined in SW_10/04." should be The bents………..defined in SW_10/04."	R. Imbsen	G. Finn	Correction made.
36		11	In the last paragraph, it is not clear what you are trying to convey through this discussion. A lay reader will wonder why you are worried about damage, since damage is good. I do not see the benefit of this discussion to the presentation, and suggest deleting this paragraph.	DG	G. Inverso	The discussion is a prelude to how ductility (N2) and damping (CSM) affect the structure. It is understood this may be obscure for the non-technical reader. The problem is more obvious for Bent 152, which "fails" for lateral load ~150 kips, but it rides out the EE, all be it at a longer period with similar performance as Bent 83 that "fails" at ~610 Kips and does not ride out the EE. Even the uninitiated technical reader finds these types of results paradoxical.

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37		11	Clarify whether the pushover analysis was conducted for a typical interior bent or a typical exterior bent. Clarify that the pushover was in the transverse direction only, not in the longitudinal direction. What results could be expected for other bent types or for the longitudinal direction?	TI	G. Finn	Analysis details (transverse pushover of an interior bent) are presented in Section 1.3 and in the series of Appendices. While it is recognized that there are some differences in reinforcement configurations between bents, particularly those designed by SED, the overall geometry, lap splice details, and bar embedments are similar for the double-deck viaduct throughout its length. Bent 83 (SED) was selected due to the typical nature of its geometry, its proximity to the seawall and the potential for the underlying soils to liquefy. Bent 152 represents a typical WSDOT interior bent, is founded on potentially liquefiable soils and was assessed previously by Eberhard et al. Regarding the performance in the longitudinal direction, the studies by Eberhard et al report on ultimate base shear and drift ratios associated with SED and WSDOT units. For the SED unit, the ultimate base shear in the transverse direction was 23% higher than that associated with the longitudinal direction. However, the ultimate drift ratio in the longitudinal direction was 255% greater than the transverse
38		11	I suggest to discuss how exceedance of column/footing joint shear capacity relates to "collapse" of the viaduct. The shear strength of the joint will degrade with increasing deformation demand, but the viaduct may not collapse as long as the footing can carry vertical loads equal to the dead load. Likewise with shear failure of the footing, if the core piles underneath and around the column can carry the dead load, the viaduct may not literally collapse. Certainly the damage would be severe and require closure of the viaduct, but the performance criteria seems to be "collapse." Maybe it's the word "collapse" that is not the best choice, since damage requiring closure should mark the end of this structure's useful life.	TI	G. Finn	The analysis techniques employed identify limit state instability leading to potential collapse, they do not predict the exact point of physical collapse. While it is recognized that joint capacity is unlikely to be completely lost as a result of reaching the joint "failure" stress and that further joint stiffness degradation is unlikely to cause catastrophic failure, there is uncertainty regarding the available rotational capacity beyond this limit. Furthermore, GTSTRUDL does not have the capability to model stiffness degradation. It may have been possible to convert the joint degradation relationships contained in Prestley *et al* to equivalent moment-curvature relationships using techniques described in same. Again, GTSTRUDL does not have the capability to model moment-curvature elements, therefore 5(fc')^0.5 was taken as the (conservative) limit. The text will be reviewed for references to "collapse".
39		11	In the fourth paragraph, the first sentence says that column/footing joint shear failure occurred in the liquefied soil case, but the pushover curve in Figure 4.3 does not show that.	TI	G. Finn	Sentence will be revised. Shear failure does not control the response of the structure with liquefied soil conditions.
40		12	Third and fourth paragraphs: From the figures, the accelerations (Sa) at failure are markedly different between the N2 method and the CSM method. Are the failure mechanisms the same as mentioned on the previous page? Suggest to discuss why the results differ or are not equivalent.	TI	G. Finn	The accelerations presented for the N2 Method are the equivalent elastic values (S_{ae} - see Figure A4). That is, they factor up the instability limit state accelerations (S_y - identical for both CSM and N2 methods) using the R-μ-T relations (equal displacement rule).
41		12	Fourth paragraph: Apparently only the Zone B spectrum was considered for the CSM method. How would results for Zone A vary?	TI	G. Finn	Bent 83 is in Zone B soil, therefore only its performance relative to Zone B demand was presented. Based on the N2 results presented on Figure 6.0, it is likely that the CSM will predict the point of instability (potential collapse) farther from the locus of performance points. That is, an earthquake of return period less than 108yrs may cause instability of the viaduct.

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42		12	"… it dampens the earthquake-induced movement." Many readers will equate movement with displacement, in which case this statement is not literally true. The softening associated with damage will reduce forces and energy within the structure, but displacements (movement) will generally increase. Could cause confusion.	SLK	G. Finn	Agreed. Sentence revised: "movements" will be replaced with "force demands".
43		12	Kinematic bending demands can also damage piles even without permanent seawall deformations. Free-field curvature of soil profile will be very high at top and bottom of liquefiable zone. Low penetration means piles are nearly pinned at bottoms, but high bending moments will be induced at top of liquefiable zone (essentially, at groundwater surface).	SLK	G. Finn	Kinematic interaction has not been considered in this study. The following statement will be added to Appendix E: "*The seismic response of pile foundations is a very complex process involving inertial interaction between structure and pile foundation, kinematic interaction between piles and soils, seismically induced pore-water pressures and the nonlinear response of soils to earthquake motions. In contrast, the nonlinear static procedures used in engineering practice and adopted in this study neglect several of these factors that could potentially affect pile response. The treatment of soil-structure interaction in this study is limited to the use of nonlinear springs representing the pile foundation system* ."
44		12	I'm not sure what happened to the discussion of Bent 152, but piles supporting southern portion of viaduct are not bearing in strong materials (liquefaction could occur beneath them, resulting in severe loss of pile capacity).	SLK	G. Finn	Bent 152 analysis is pending but will be included in the final version of the report. However, the analysis assumes that the piles are embeded in competent (i.e. nonliquefiable) material as per geotechnical recomendations.
45		13	See earlier comments on liquefaction and lateral spreading (second paragraph). There is a circular argument being presented here, saying on one hand that soil will liquefy without spreading (analysis) but that liquefaction will fail the wall and then the viaduct. This paragraph also implies foundations sitting on liquifiable soil instead of piles (last sentence), which is not generally the case. This same logic is carried forward to the presentation of conclusions in section 2.0.	DG	G. Inverso	Paragraph 2 on Page 13 is the thumbnail summary of liquefactions. Paragraph 3 discusses the relevance of liquefaction to seawall stability. Paragraph 4 summarizes the results of the 2003 and 2006 seawall reports relative to seawall stability. The conclusions of the 2006 Seawall report are carried to the conclusion in Section 2.0.
46		13	I have not seen the referenced reports on the seawall, so I cannot comment on potential for seawall failure due to liquefaction, as determined in those investigations.	SLK	G. Inverso	Acknowledged.
47		14	The last sentence in the first paragraph says that liquefaction due to a 108-year motion is "marginal." The fourth paragraph on p. 13 says that 200-year ground motions result in a 70% chance of collapse. These statements seem somewhat contradictory.	TI	G. Inverso	Per the 2006 Seawall report, in Shannon & Wilson's (SW) opinion base on qualitative data, insufficient soil liquefy during the 108-year ground motion to cause the seawalls to collapse. SW has set 200-year ground motions as necessary to liquefy sufficient soil to collapse the seawall. The decision matrix in the 2006 Seawall report assigns a 70% chance of Viaduct collapse if the Seawall collapses. Text modified to clarify.
48		14	I would consider rewording the first sentence - could be taken to imply that one of three indicated that 108-yr motion is NOT capable of causing big problems, which is not your intent.	SLK	G. Finn	Acknowledged. Paragraph revised.
49		14	Pg 14 - First paragraph (first sentence) - Please rewrite into simple statement, "inducing limit state stability demands"?	TM	G. Finn	Acknowledged. Sentence to be revised.

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50		14	Pg 14 - Third paragraph (first sentence) - Please make a comparison to the conclusions of the analysis of the SSRC Report and the Bent #97 to 100 analysis with comparison of C/D ratios for the transverse floorbeam bent.	TM	G. Finn	A discussion section will be added to the report. However, a direct comparison to pervious studies would be difficult to relate and would thus not be appropriate. In previous studies demand had been generated from design events (TH, Response Spectrum) whereas demand in the context of this study was calculated at each load step generated from a pseudo static load applied transversely to a given Bent (i.e. from pushover analysis).
51		1-3	Figures 1-3 are not mentioned in the body of the report	TI	G. Finn	Additional references to figures and appendicies will be added to the next revision of the report.
52		11&12	It is not clear how the plots in Figures 4,1. 4.2 and 4.3 are correlated with Figures 7.1, 7.2 and 7.3. Should also explain in more detail how the instability is formulated.	R. Imbsen	G. Finn	The capacity curves (ADRS format) in Figures 7.1, 7.2, and 7.3 have been calculated from the pushover curves (in ARS format) terminated at the identified shear failure point where appropriate. The transformation procedure will be provided in Appendix A.
53		4.1-4.3	In Figures 4.1, 4.2, and 4.3, what displacement is being plotted? From the top level of the structure; or the bottom level?	TI	G. Finn	The reference displacement being plotted on the pushover curves is the horizontal displacement of the top right-hand node. A note will be added to the figures.
54		B1	State the concrete and steel properties used in modeling. Are they the specified properties, or expected properties.	TI	G. Finn	A table of material properties will be added in the next revision of the report. Material strengths are *expected* values and are the same as those used in the Eberhard *et al* reports.
55		B3	2nd paragraph: What does a "uniform" disbribution of lateral load mean for this structure? Why was this load pattern chosen, and not one proportional to the first mode shape?	TI	G. Inverso	The uniform force distribution applied lateral force at each level of the structure that is proportional to the mass at that level and acceleration determined from a specific shape vector assumption. Per FEMA 440 and other procedures, two load distribution need to be considered. Uniform load is one. First Mode distribution is another. When modal analysis were run and the first mode deflection were normalized (1.0 as maximum) the distribution was nearly uniform – 1.00 for the top level and 0.85 for the lower level. This is close to the uniform load. A check case using Bent 83 with a fixed base and first mode distribution will be run.
56		B3	Discuss the relationship between the displacement of the structure (at the top of the frame?) versus the displacement of the equivalent SDOF oscillator. Are these the same? Likewise the force: Is there a one-to-one relationship between the base shear in Figures 4.1-4.3 and the acceleration of the equivalent SDOF oscillator (after dividing the base shear by the weight of the frame). (Generally the relationship involves a participation factor.)	TI	G. Finn	The MDOF pushover curve was transformed to an equivalent SDOF system by dividing both the base shear and displacement ordinates by a modal participation factor. The SDOF capacity curve (ADRS format) was then constructed by dividing the equivalent SDOF base shear by the equivalent mass of the system. The displacement vector used in the transformation assumed the elastic first mode shape. A flowchart will be added to Appendix A to outline steps involved in the N2 Method and CSM.
57		B4	Piles themselves were assumed linear here? Worth mentioning?	SLK	G. Finn	Pile elements were modeled as compression only (linear) elements. Foundation modeling is described in Appendix E.
58		C1	You should clarify where the Ld formula is used, and comment on the types of bars in the viaduct. This formula does not address square bars with fillet welds.	DG	G. Inverso	Clarifications will be added. See Comment 60.

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59		C2	At top of page, a code-type reduction in yield for deficient development length does not model accumulated damage in a section (allowing yield unstead of bond failure).	DG	G. Inverso	Clarifications will be added. Per FEMA 273 approach the bar slips at a stress equal to the tension yield stress of the steel times the development length provided divided by the development length required (Ld). Once the bar starts to slip, it can maintain 20% of the original slip force. This 20% force does not go forever. It was truncated at ~8 times the slip strain to reflect cumulative damage per Priestley et.al. Priestley data was for lap splices lose of strength as cracks widen that correlates to curvature ductility.
60		D3	The assumption that a stress of 5 $\sqrt{f`c}$ leads immediately to collapse is conservative	TI	G. Finn	See response to comment 46.
61		E1	Change "has" to "have" in last sentence.	SLK	G. Finn	Noted. Word changed.
62		E1	This section is very confusing regarding DFSAP, GROUP, and GT-STRUDL. The role of each program and the relationship between models is not clear.	TI	G. Inverso	Clarifications will be added.
63		E2	Lateral stiffness of foundations appears to be very sensitive to stiffness of soil placed around footings (in passive wedge). Were stiffness measurements available? If not, was stiffness estimated from SPT or other tests? If so, were effects of uncertainty accounted for? Maybe too many questions for a short document like this, but issues are significant.	SLK	G. Inverso	Soil profiles were supplied by Shannon & Wilson for the various bent locations considered in this analysis. The data were based on two or more adjacent borings. The profiles provided the parameters needed to develop P-Y curves for the DFSAP software. The information included unit weight; effective unit weight; friction angle; horizontal modulus of subgrade reaction. For the liquefiable layers additional information provided included: average equivalent STP blow count N-Values (N1)60; average percent fines; and particle angularity. From these the software develops P-Y curves to track loads and deflections of the piles and the cap.
64		F1	The third paragraph is confusing regarding the relationship between attenuation of earthquake motion and return period	TI	B. Perkins	The third paragraph does not address a relationship between ground motion attenuation and return period. It is an introduction to two basic concepts that must be understood in a discussion of the return period of the Nisqually Earthquake ground motions at the site. These two concepts are discussed in the two following paragraphs (paragraphs 4 and 5). In the appendix draft sent to PB, the paragraphs 4 and 5 were bulleted (numbered) to help make clear that the third paragraph was an introduction to the two concepts in paragraphs 4 and 5. We suggest using the bullet format provided in the draft to help clarify.

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65		F2	The objective of this analysis is clear, but it seems to me that the results could be made a lot clearer with a little more analysis. Looking at the recorded MAR spectrum, it seems likely that a significant portion of the rapid change in Sa from about 0.6 to 0.7 sec (and the fluctuations in spectral shape on either side of that range) is due to local site response at the MAR site. If information on subsurface conditions at MAR is available (or can be estimated), why not obtain the Site Class B spectrum at MAR by deconvolving the measured motion through the MAR soil profile. It seems likely that this would remove some of the site-specific response that is making the inferred (by using simple period-insensitive scaling factors) MAR Site Class B spectrum so irregular. This could give you a spectrum that is more parallel to the ARP spectra and thereby not cut across such a wide range of return periods. It is not guaranteed to work, but it wouldn't be difficult to give it a try - benefits (improved resolution of return period) could be worth spending a couple hrs playing around wi	SLK	B. Perkins	The more rigorous deconvolution suggested by the reviewer could be accomplished with additional budget and schedule beyond those presently allotted for this task. We do not poses either site-specific shear wave velocities or subsurface profile for the MAR site as these would improve the deconvolution analysis. Before proceeding with the more rigorous deconvolution and obtaining site-specific subsurface profile and velocity measurements to support the deconvolution, PB should assess whether a difference on the order of +/-10 percent in MAR spectrum modified for Site Class B would significantly change their conclusions.
66		F2	If analyses suggested in previous comment are not performed, I would consider using a smoothed version of the MAR spectrum. This would tend to remove rapid fluctuations that are driving some of the variability in return period without being really meaningful.	SLK	B. Perkins	Upon receipt of the reviewers suggestion, we smoothed the MAR spectrum visually by selecting constant acceleration, velocity, and displacements for various portions of the spectrum, as is common in engineering practice (log-log tripartite spectrum). While the spectrum was smoother, the range in return periods for the smoothed spectrum was not appreciably different than for the un-smoothed spectrum. Other methods to develop a smoothed spectrum are not immediately obvious. For example smoothing by averaging over period increment ranges could be done; however different period-increments/window lengths would produce different smoothed spectra (i.e various ranges of return periods could be obtained depending on the width of the smoothing window). Without justification of the period increment ranges, we are reluctant to use this method.
67		F3	Section F.4 seems to be comparing Site Class E spectrum (recorded KMK motion) with a Site Class B spectrum. Why? I don't see how you can infer anything about return periods from this comparison because of the apples/oranges factor. To get around this, you could run a suite of motions consistent with the Site Class B spectrum (scaled to match at some Sa, or synthetic spectrum-compatible motions) through a site response analysis of the KMK soil profile. That would give you an estimate of the 108-yr spectrum for the conditions at the KMK site, and would allow a meaningful comparison with the spectrum from the recorded KMK motion.	SLK	B. Perkins	The comparison is between the EE spectrum for **Zone B** (*not* Site Class B), and the Nisqually motions recorded at station KDK. As described in section F.2, Zone B is geographic area defined in the 2004 AWV Seismic Ground Motion Study Report in which site-specific ground response analyses were performed. Station KDK is practically located in Zone B, therefore the Nisqually ground motions recorded there are directly comparable to the site-specific Zone B spectrum. In hind-sight, the geographic zones in the 2004 report could have been designated some other way (e.g., "X" and "Y") to avoid confusion with Site Classes "A" through "F" used in the various design codes.

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68		F3	Is is possible to make a more definite characterization of the Nisqually Earthquake? This section compares it to 50-year, 108-year and 200-year motions, leaving the reader unclear how to characterize the earthquake.	TI	B. Perkins	Unfortunately given the lack of nearby recorded Nisqually ground motions and large structural period range of interest, the data do not support a more definitive characterization than what is summarized in section F.2. The intent of the section is to have the reader understand that the Nisqually ground motions recorded at the site can not be characterized by one return period but that the ground motion return period varies by structural period. This was one of the key concepts provided in the introduction.
69		Fig 1.0	Label locations of bents of interest	SLK	G. Finn	Noted. Bents labeled.
70		G3	The comparison of XTRACT to GT-STRUDL doesn't say anything about the "joint" performance, as alluded in the text.	TI	G. Inverso	Comment noted.
71			The three page executive summary used the word "collapse" eighteen times. While not adverse to it's use, the potential "collapse" mechanism should be clearly described and illustrated.	TM	G. Inverso	The purpose of the analysis is to identify the ground motion that will collapse the structure. This is slightly backwards from design, which attempts to “prevent collapse”. “Collapse” and “Prevent Collapse” are seismic performance levels that have definitions or at least expectation. The more esoteric “limit state stability demands” says the same thing but may unwisely mask the concept of “collapse” that most people understand. It is true the analysis techniques employed identify limit state instability leading to potential collapse; they do not predict the exact point of physical collapse. Text will be reviewed with "collapse" being replaced by "potential collapse" where appropriate. See Comment 46.
72			Three different "collapse" mechanisms have been identified per notations on Fig. 4.1, 4.2 & 4.3. It would suit the reviewer and reader, to further describe these failure scenarios. Incidently, these failure scenarios are different than those reported by the SSRC Report and Bents 97-100 Damage Rating Analysis. The two noted reports found shear capacity deficiencies in the upper and lower transverse floorbeams. What are the capacities of these elements per the pushover results in the reported analysis? It would add credibility to the analysis described if the shear capacity of the floorbeam are plotted on Figs. 4.1, 4.2 & 4.3.	TM	G. Finn	A discussion will be added to the report which will deal with the results of the study and how they compare to previous analyses. The initial and final shear capacities of the main structural elements by-and-large concurred with those of the University of Washington Reports (Eberhard *et al*). In order to communicate the shear failure mechanisms identified in the analyses, the pushover curves will include all predicted shear failures.
73			Footing joint failure could be explained by diagram/photograph such as that shown in Fig 5.86 (a) & (b) per reference Appendix D.4 (2) "Seismic Design and Retrofit of Bridges" Priestley, Seible & Calvi. All parameters used to determine principal tension stresses of 5*(f'c)^0.5 should be included in Appendix D.3.	TM	G. Finn	A sample joint shear stress calculation together with effective joint dimensions will be added to Appendix D.
74			Footing shear failure parameters should be included in calculation with reference drawing to represent "collapse" mechanism. It's unclear as to shear strength value used for footing concrete that represents a "collapse". Please provide a sketch of the location of the shear failure plane and the associated remaining footing and piles capable of supporting dead load to support claim that bent can "collapse".	TM	G. Finn	A statement on the assumed shear strength of the pile cap will be added to Appendix D. The critical section for shear of the pile cap was assumed to be at an effective depth away from the face of the column. Also, an effective width, somewhat less that the full width, was assumed to resist shear.
75			Please elaborate on the term soil failure. How does this relate to "collapse"?	TM	G. Finn	The term "soil failure" refers to a loss of stiffness in the soil resulting in structural instability.