Memorandum

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Subject	**Initial Load Rating Evaluation**	**Project Name**	West Seattle Swing Bridge Rating
Attention	Kit Loo (SDOT)	**Project No.**	W3X88300
From	Sung Cheung, Adrian Corella		
Date	March 5, 2020		
Reviewed by:	Mark Johnson		

Per your request on March 2nd, 2020, we have performed an expedited analysis of the existing rating capacity of the West Seattle Swing Bridge. This analysis was done using information from the 1998 load rating of the bridge. The 1998 rating was performed based on prestress losses due to concrete time-dependent effects estimated using simplified equations in the AAHSTO Standard Specifications for Highway Bridges. Our complete future load rating will be performed using structural analysis models considering actual construction sequence with concrete time-dependent effects.

The 1998 load rating found an overall controlling rating factor of 1.04 for web shear at joint 20. This point is located approximately 60 feet towards the middle of the bridge from the centerline of each of the main pivot piers as shown in Figure 1. We therefore decided to focus on updating the load rating at this controlling location for our initial evaluation.

Below is a summary of our approach for this initial assessment:

1. The maximum forces produced from the dead and live load analyses in the 1998 load rating report were assumed to be accurate.
2. The prestress losses of 30 ksi in the top tendons used for the 1998 load rating were assumed to be accurate.
3. The LFR method was selected for the NBI rating to use the live load analysis results from HS20 loading (since the bridge was designed prior to October 2010).
4. Legal and Permit Load ratings were performed using the LRFR method.
5. Rating trucks included HS20, AASHTO 1, AASHTO 2, AASHTO 3, OL1, and OL2.
6. Rating was performed for the strength limit of shear only at Joint 20 in the center span, which was assumed to be the controlling location.
7. Load factors, resistance factors, and impact factors were determined per WSDOT BDM, July 2019.
8. Shear capacity was determined based on Section 12.2.12 of *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges* when using the LFR method.
9. Shear capacity was determined based on Section 5.12.5.3.8c of *AASHTO LRFD Bridge Design Specifications* when using the LRFR method.

Table 1 shows the results from the analysis. Note that per our calculations, the shear capacity of the webs has been reduced substantially from the capacity shown in the 1998 load rating report. In fact, the shear capacity is low enough that the dead load demands as calculated in 1998 exceed the shear capacity. This means that there is no additional capacity for live load and the load ratings are shown as zero. In order to get a sense of how critical the condition of the webs is, capacity-to-demand (C/D) ratios are also reported.

At the time the 1998 load rating report was prepared, there were two possible AASHTO specifications to use as guidance: the Standard Specifications for Highway Bridges and the Guide Specifications for Design and Construction of Segmental Concrete Bridges. Each code had different approaches to calculating shear capacity, but the key difference is that the Segmental Bridge Specifications provided an upper bound limit on the concrete contribution towards shear capacity. This upper bound was implemented by limiting the K value, which is the stress variable accounting for concrete compressive stress after prestress losses, shown in Table 1. The Standard Specifications did not provide an upper bound on the value of K, so in theory, the more prestressing that is provided, the higher the shear capacity, without a limit. The 1998 load rating used the unbounded shear calculation, so it reported higher shear capacities than are currently prescribed by the code. To examine the capacity of the bridge using the updated shear equations without an upper bound, the shear capacities were calculated without a limit on the K value and are shown in Table 2. For this case, the C/D ratios increase to more reasonable values but are still below 1.0.

The low C/D ratios for web shear are of high concern. During a site visit on 2/14/20, cracking of the webs at one of the exterior box sections was observed. The inclined cracks had an angle with respect the horizontal of approximately 20 degrees, which suggests cracking subject to combined shear and high compression. We recommend that a detailed inspection of the cracking at joint 20 and surrounding segments be performed and that the cracking be monitored as part of the ongoing inspections of the bridge.

Because these conclusions are based on limited analysis using information from the 1998 load rating, we recommend that the next step be to perform a thorough load rating of the structure to get more reliable answers.

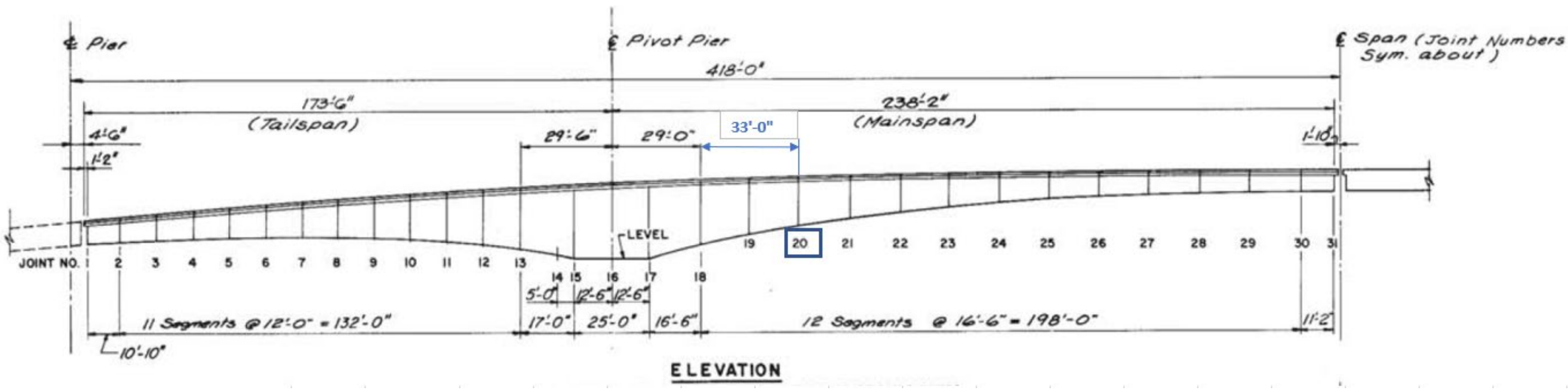

Figure 1: Joint 20 Location

Table 1: Shear rating using bounded concrete shear capacity

Methodology	Rating Trucks (1)	Load & Resistance Factors					Forces in section					Shear reinforcing			Ultimate loads			Shear capacity							Rating Factor			C/D ratio
		Dead load factor	Live load factor	Prestress load factor	Impact factor	Shear resistance factor	Shear due to prestress	Shear due to dead load	Max shear due to one lane	Max shear associated with max flexure	Prestress force	Shear reinforcing area	Shear reinforcing spacing	Shear reinforcing assumed to work in transverse direction	Live Load multiplier	Ultimate shear	Ultimate shear associated with ultimate flexure	Unfactored compressive stress after losses at centroid	AASHTO Eq 5.12.5.3.8c-5	AASHTO Eq 5.12.5.3.8c-3	Shear reinforcing area available to take vertical shear	AASHTO Eq 5.12.5.3.8c-4	AASHTO Eq 5.12.5.3.8c-1					
-	-	γ_D	γ_L	γ_P	μ	ϕ	V_p	V_d	V_{LL}	$V_{LL,i}$	P	A_v	s	-	-	V_u	V_i	f_{pc}	K	V_c	A_v'	V_s	V_n	ϕV_n	$\phi V_n - V_D{*}\gamma_D$ (4)	$V_{LL}{*}\gamma_{LL}$	RF (3)	
-	-						kips	kips	kips	kips	kips	in^2	ft	in^2		kips	kips	ksi		kips	in^2	kips		kips	kips	kips		
LFR Inventory	HS-20 (2)	1.30	2.17	1.00	0.08	0.9	0	3,546	145	129	33,780	1.76	0.83	0.6	4.70	5291	5216	1.59	2.00	1895	1.2	1331	3226	2903	-1706	681	0.00	0.55
LFR Operating	HS-20 (2)	1.30	1.30	1.00	0.08	0.9	0	3,546	145	129	33,780	1.76	0.83	0.6	2.82	5018	4973	1.59	2.00	1895	1.2	1331	3226	2903	-1706	408	0.00	0.58
LRFR Legal	AASHTO 1	1.25	1.45	1.00	0.10	0.9	0	3,546	46	27	33,780	1.76	0.83	0.6	3.19	4579	4519	1.59	2.00	1895	1.2	1331	3226	2903	-1529	147	0.00	0.63
LRFR Legal	AASHTO 2	1.25	1.45	1.00	0.10	0.9	0	3,546	62	36	33,780	1.76	0.83	0.6	3.19	4630	4547	1.59	2.00	1895	1.2	1331	3226	2903	-1529	198	0.00	0.63
LRFR Legal	AASHTO 3	1.25	1.45	1.00	0.10	0.9	0	3,546	72	43	33,780	1.76	0.83	0.6	3.19	4662	4570	1.59	2.00	1895	1.2	1331	3226	2903	-1529	230	0.00	0.62
LRFR Legal	Lane	1.25	1.45	1.00	0.10	0.9	0	3,546	145	129	33,780	1.76	0.83	0.6	3.19	4895	4844	1.59	2.00	1895	1.2	1331	3226	2903	-1529	463	0.00	0.59
LRFR Permit	Overload 1	1.25	1.20	1.00	0.10	0.9	0	3,546	79	47	33,780	1.76	0.83	0.6	1.32	4537	4495	1.59	2.00	1895	1.2	1331	3226	2903	-1624	104	0.00	0.64
LRFR Permit	Overload 2	1.25	1.20	1.00	0.10	0.9	0	3,546	180	114	33,780	1.76	0.83	0.6	1.32	4670	4583	1.59	2.00	1895	1.2	1331	3226	2903	-1624	238	0.00	0.62

(1) Rating has been performed only for the rating trucks with live load analysis results available in the earlier rating report, 1998

(2) For bridges designed prior to October 1, 2010, NBI ratings can be based on either the LFR or LRFR methods. LFR is selected and the rating factors are based on HS loading per WSDOT BDM.

(3) Rating factor with a value of zero indicates that the permanent load demand exceeds the section capacity

(4) Numerator for rating factor uses a different equation for overload vehicles that considers a standard truck on the other lane.

Table 2: Shear rating using unbounded concrete shear capacity

Methodology	Rating Trucks (1)	Load & Resistance Factors					Forces in section					Shear reinforcing			Ultimate loads			Shear capacity							Rating Factor			C/D ratio
		Dead load factor	Live load factor	Prestress load factor	Impact factor	Shear resistance factor	Shear due to prestress	Shear due to dead load	Max shear due to one lane	Max shear associated with max flexure	Prestress force	Shear reinforcing area	Shear reinforcing spacing	Shear reinforcing assumed to work in transverse direction	Live Load multiplier	Ultimate shear	Ultimate shear associated with ultimate flexure	Unfactored compressive stress after losses at centroid	AASHTO Eq 5.12.5.3.8c-5	AASHTO Eq 5.12.5.3.8c-3	Shear reinforcing area available to take vertical shear	AASHTO Eq 5.12.5.3.8c-4	AASHTO Eq 5.12.5.3.8c-1					
-	-	γ_D	γ_L	γ_p	μ	ϕ	V_p	V_d	V_{LL}	$V_{LL,i}$	P	A_v	s	-	-	V_u	V_i	f_{pc}	K	V_c	A_v'	V_s	V_n	ϕV_n	$\phi V_n - V_D*\gamma_D$ (4)	$V_{LL}*\gamma_{LL}$	RF (3)	
-	-						kips	kips	kips	kips	kips	in²	ft	in²		kips	kips	ksi		kips	in²	kips		kips	kips	kips		
LFR Inventory	HS-20 (2)	1.30	2.17	1.00	0.08	0.9	0	3,546	145	129	33,780	1.76	0.83	0.6	4.699	5291	5216	1.59	3.36	3184	1.2	1331	4516	4064	-546	681	0.00	0.77
LFR Operating	HS-20 (2)	1.30	1.30	1.00	0.08	0.9	0	3,546	145	129	33,780	1.76	0.83	0.6	2.815	5018	4973	1.59	3.36	3184	1.2	1331	4516	4064	-546	408	0.00	0.81
LRFR Legal	AASHTO 1	1.25	1.45	1.00	0.10	0.9	0	3,546	46	27	33,780	1.76	0.83	0.6	3.19	4579	4519	1.59	3.36	3184	1.2	1331	4516	4064	-368	147	0.00	0.89
LRFR Legal	AASHTO 2	1.25	1.45	1.00	0.10	0.9	0	3,546	62	36	33,780	1.76	0.83	0.6	3.19	4630	4547	1.59	3.36	3184	1.2	1331	4516	4064	-368	198	0.00	0.88
LRFR Legal	AASHTO 3	1.25	1.45	1.00	0.10	0.9	0	3,546	72	43	33,780	1.76	0.83	0.6	3.19	4662	4570	1.59	3.36	3184	1.2	1331	4516	4064	-368	230	0.00	0.87
LRFR Legal	Lane	1.25	1.45	1.00	0.10	0.9	0	3,546	145	129	33,780	1.76	0.83	0.6	3.19	4895	4844	1.59	3.36	3184	1.2	1331	4516	4064	-368	463	0.00	0.83
LRFR Permit	Overload 1	1.25	1.20	1.00	0.10	0.9	0	3,546	79	47	33,780	1.76	0.83	0.6	1.32	4537	4495	1.59	3.36	3184	1.2	1331	4516	4064	-463	104	0.00	0.90
LRFR Permit	Overload 2	1.25	1.20	1.00	0.10	0.9	0	3,546	180	114	33,780	1.76	0.83	0.6	1.32	4670	4583	1.59	3.36	3184	1.2	1331	4516	4064	-463	238	0.00	0.87

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