4. MONORAIL AT GRADE WITHIN HOV LANES

Converting the two HOV lanes of the I-90 Homer Hadley Floating Bridge to monorail (refer to Figure 10) requires the evaluation of the rail system dead load. The dead load of the rail system will reduce the freeboard, and because the rail system is located a distance from the center of buoyancy of the bridge its dead load will also cause the bridge to list or rotate. Since any bridge list must be leveled or trimmed with offsetting ballast, the added ballast necessary to trim the bridge, further reduces the freeboard.

As the monorail rail system includes a substantial guide beam as part of its support structure, it was first necessary to design a guide beam which would meet all the necessary operational requirements of a monorail beam while at the same time being a lightweight structural element.

a. MONORAIL GUIDE BEAM DESIGN

The key criteria used in the evaluation of guide beam design concepts is as follows:

- Total support system weight shall be less than 1176 plf.
- Beam dimensions shall comply with the dimensional requirements of the monorail bogey system.
- The top surface of the beam shall be durable and shall have an appropriate coefficient of friction for the rubber tires of the monorail.
- The beam shall have the required strength to carry monorail loads, both horizontal and vertical.
- The beam shall meet appropriate vibration, deflection and fatigue criteria.

Two design concepts are identified that appear to meet the above criteria in this preliminary investigation. Concept A consists of an all steel box-beam with a metalized, spray-on, non-skid surface applied to the top steel plate of the guide beam. The non-skid surface is applied to the beam using plasma stream deposition, where molten metal alloy adheres to the top steel plate. Concept B consists of a concrete/steel composite box beam, with the top surface composed of lightweight concrete.

In both concepts, steel stanchions that are bolted to the top deck of the pontoon support the beams. Steel stanchions are used to reduce the overall weight of the monorail support system as well as to allow the current bridge drainage system to be maintained. Stanchions for the north beam will be located directly above the transverse bulkhead walls of the pontoon. The cantilever deck of the pontoon will support stanchions for the south beam.

To reduce weight, openings are cut in the guide beam sidewalls and bottom plate along the length of the beam. The weight of the monorail support system for each concept includes 70 plf for an elevated emergency walkway between the beams and 10 plf per beam for the weight of power rails. Beam details are shown in Figures 7 and 8. The weight of each concept is summarized below:

	Concept A	Concept B
Total System Dead Load, plf	1109	1177
Ratio of Total System Dead Load to 1176 plf Maximum Allowable	0.94	1.0

* "Total System Dead Load" includes the weight of two monorail guide beam structures and the elevated emergency walkway.

The beam design was primarily controlled by local plate stresses due to the monorail wheel loads rather than global stresses due to the beam span between stanchions. It should be noted that no contingency has been included in the weight of the monorail support system to account for additional weight that may be added as design progresses. The above ratios indicate that there is very little margin for additional weight to be added to the system.

b. DEAD LOAD ANALYSIS AND FREEBOARD LOSS MITIGATION

The weight and balance criteria established for the previous LRT study were that all loss of freeboard due to implementation of any design alternative be mitigated to a final net loss of zero. The bridge is also to remain trim. In the previous LRT study, a hydrostatic analysis was performed on the bridge to determine freeboard loss due to various LRT rail configurations as well as to identify a number of options to mitigate the loss of freeboard. Using a least cost approach, it was determined that the maximum amount of freeboard loss that can be mitigated without the use of auxiliary buoyancy is approximately 6.17 inches. Refer to Figure 10 for details. Mitigation measures include the following:

Mitigation Type	***Description***
M4	*Replace the existing concrete median barrier with a steel median barrier.*
M5	*Replace the existing concrete barrier along the south edge of bridge with a cable railing.*
M6	*Remove the existing pontoon gravel ballast.*
M12	*Remove 1-inch of existing overlay from the 40-foot wide dedicated HOV lanes on the south side of the bridge and replace with 0.25-inch of polymer concrete overlay.*

Using the monorail guide beam track location shown on Figure 4 it was determined that the maximum allowable dead load of the monorail support system that can be mitigated without the use of auxiliary buoyancy is 1176 plf. This is based on implementing all four of the mitigation measures indicated above as well as locating trim ballast in the north half of the north cell of the pontoon, as shown in Figure A. The dead load of the monorail support system includes the weight of the guide beams, beam stanchions, and emergency walkway.

c. LIVE LOAD ANALYSIS

A hydrostatic analysis of the floating bridge with the Monorail vehicle live load was performed to determine the bridge list, freeboard loss, vertical bending moment and torsion resulting from Monorail traffic. The simple 2-dimensional SAP model that was developed for the previous LRT study of the bridge was modified to apply Monorail vehicle live loads. The following load conditions were considered:

SAP Load Case	Description
Live1	(2) Trains bypassing at the mid-span with "push-train" on the outside beam.
Live2	(2) Trains about to bypass at the mid-span with "push-train" on the outside beam.
Live3	(2) Trains bypassing at one end of the bridge with "push-train" on the outside beam.
Live4	(2) Trains about to bypass at one end of the bridge with "push-train" on the outside beam.

* All train loads are for the full passenger condition.

Where: "Push-train" represents a train used to push/pull a disabled train. The push-train load is applied immediately adjacent to the standard train on the outside guide beam.

All load cases also include an additional train load 500 feet away to account for 500 feet of headway between trains. In this study, the additional train was placed on the outside beam to envelope the worst case response of the bridge. A more likely situation is that the additional train is located on the inside beam while a disabled train is being pushed on the outside beam.

Bridge Freeboard Loss Due to Live Load

Freeboard loss along the south edge of the bridge and bridge list due to live load alone are summarized below. For comparison, freeboard loss and bridge list results for the LRT study are also included for similar load conditions. See Figures B through E for graphs of the displaced shape of the bridge for the various load cases.

	Current Study Results		**Previous LRT Study Results**	
Load Case	**Freeboard Loss due to Monorail (in)**	**Bridge List due to Monorail (degrees)**	**Freeboard Loss due to LRT (in)**	**Bridge List due to LRT (degrees)**
Live1	-4.68	0.177	-9.19	0.348
Live2	-4.45	0.173	-7.65	0.327
Live3	-6.43	0.194	-9.66	0.376
Live4	-5.85	0.250	-	-

As can be seen from the above results, the freeboard loss and bridge list due to the Monorail live load is about 50-percent of that due to the LRT live load. Corresponding bridge vertical bending moments and torsions due to the Monorail live load are also less than those due to LRT loading. As concluded in the LRT study, these bridge responses do not create structural performance problems on the bridge since they are transitory in nature, but they need to be reviewed for conformance to monorail train operation requirements. The monorail guide beam will need to be designed to accommodate the bridge deflections.

Bridge Response With Respect to the 1-year Storm Event

An evaluation of bridge response with respect to the 1-year storm event was made by combining live load moments and torsions with those for the storm event and then comparing them to the bridge ultimate capacity. Bridge ultimate capacities for vertical bending and torsion were obtained from a study performed by The Glosten Associates in 1994. Bridge responses to the storm events were obtained from a hydrodynamic analysis of the bridge performed by the Glosten Associates in 1983. The maximum moment obtained by combining live load with the 1-year storm event is approximately 30-percent of the ultimate vertical bending capacity of the bridge. The maximum torsion obtained by combining live load with the 1-year storm event is approximately 45-percent of the ultimate torsional capacity of the bridge. See Figures F and G for graphs of the bending and torsional response of the bridge in comparison to ultimate capacity.

Further analysis should be performed to compare the combined results of live load and the 1-year storm event to the service level limits of the bridge.

d. STRUCTURAL IMPACTS

Floating Pontoon

The service load stress levels and ultimate capacity of the cantilever portion of the top deck of the pontoons were checked against the load applied by the stanchions supporting the south Monorail guide beam. Structural impacts due to the stanchions supporting the north guide beam were not investigated because these stanchions are located directly over the transverse bulkhead walls of the pontoon. Service level and ultimate strength design loads were computed using the load combination tables presented in the Seattle Monorail Project design assumptions document. Load cases considered include monorail support system dead load, maximum monorail live load reactions, vertical and horizontal impact forces and wind load on the monorail vehicle. Various guide beam spans were investigated to determine the maximum stanchion spacing that is allowed by the capacity of the pontoon cantilever slab. These beam spans were selected to correspond to the approximate maximum spacing of the transverse bulkheads in the pontoon. Also, computation of forces applied to the pontoon deck assumes freeboard loss mitigation measures M5 and M12 are implemented. Analysis results are summarized below:

Ultimate Flexural Capacity

Beam Span/ Stanchion Spacing (ft)	Max Factored Moment (ft-k/ft) (Demand)	Design Strength (ft-k/ft) (Capacity)	D/C Ratio	Result
40	1151	1661	0.69	OK
70	1642	1661	0.99	OK

Ultimate Shear Capacity:

Beam Span/ Stanchion Spacing (ft)	Max Factored Shear (kips/ft) (Demand)	Design Strength, kips/ft (Capacity)	D/C Ratio	Result
40	17.70	22.38	0.79	OK
70	25.7	22.38	1.14	No Good

Service Level Stresses

Beam Span/ Stanchion Spacing (ft)	Max Service Level Moment, ft-k/ft (Demand)	Allowable Service Moment (ft-k/ft) (Capacity)	D/C Ratio	Result
30	642	648	0.99	OK
35	694	648	1.07	No Good
40	751	648	1.16	No Good
70	1072	648	1.65	No Good

Note: A "zero tension" limit is placed on the allowable concrete stresses.

Localized punching shear strength in the vicinity of the stanchion base plate was also investigated and determined to be adequate for the 70-foot stanchion spacing.

As indicated above, stanchion spacing is limited to 30 feet on-center for the south guide beam due to the "zero tension" limit on allowable service level stresses.

Elevated Superstructure

The elevated superstructures consist of a composite concrete/steel structure with a concrete deck and steel box girders. The structures provide the vertical transition from the level pontoon deck to the grades of the transition spans, and the approach roadways from the east and west. Concrete piers at approximately 59 feet on-center rise from the pontoons to support the superstructure.

The proposed monorail guide beam system is located above the concrete deck, and spans to steel stanchions located over the concrete piers. One of the guide beams is located at approximately the midpoint between box girders and the other is located approximately at the centerline of a box girder (See Fig.5).

Calculations show that the existing concrete deck has the bearing capacity to support a monorail stanchion. For the stanchions located between the superstructure box girders, steel bearing-stiffeners would be added between the bottom of the deck and the top of the pier. For the stanchions located at the centerline of the box girder, the girder bearings would need to be modified or augmented with additional bearings to accommodate the increased dead and live load reactions. See Figure 9 for details.

In this study it was assumed that the concrete piers have the capacity to carry the stanchion reaction loads to the pontoons below. Further analysis is required to verify this assumption.

e. *ESTIMATED CONSTRUCTION COSTS OF IMPROVEMENTS*

Schematic-level cost estimates for structural and weight mitigation measures required for the two monorail guide beam design concepts are summarized below:

Bridge Conversion Cost		
	Concept A	**Concept B**
Mobilization	$ 1,346,895	$ 1,352,395
M4, Steel Median Barrier	$ 2,996,000	$ 2,996,000
M5, Cable Railing	$ 2,240,000	$ 2,240,000
M6, Remove Existing Ballast	$ 280,000	$ 280,000
M12, Remove 1-inch overlay/install polymer concrete overlay	$ 5,712,000	$ 5,712,000
Add Gravel Ballast	$ 1,720,000	$ 1,775,000
Elevated superstructure retrofit	$ 270,946	$ 270,946
Traffic control	$ 250,000	$ 250,000
Subtotal	$14,815,841	$14,876,341
Contingency at 30%	$ 4,444,752	$ 4,462,902
Total Estimated Cost (2004$)	**$19,260,593**	**$19,339,242**

Monorail Support Structure Cost		
	Concept A	**Concept B**
Mobilization	$ 1,879,755	$ 1,613,278
Guide beam system	$15,707,171	$13,042,401
Emergency walkway	$ 3,090,383	$ 3,090,383
Subtotal	$20,677,311	$17,746,062
Contingency at 30%	$ 6,203,193	$ 5,323819
Total Estimated Cost (2004$)	**$26,880,504**	**$23,069,881**

All costs are in 2004 dollars. Costs do not include Washington State sales tax, engineering and construction management, electrical modifications/temporary services and monorail system installation costs. Costs reported in the previous LRT study included bridge conversion costs only and were in 2001 dollars. Costs for the LRT rail system were not included.

It should be noted that the long-term maintenance requirements for the running surface on the top of the guide beams for the proposed options might be different. It is recommended that a comparison of life-cycle maintenance costs be used as one of the methods to evaluate the feasibility of using one guide beam system over the other.

5. MONORAIL ELEVATED OVER THE CENTRAL MEDIAN

During the course of the monorail conversion study, it was suggested that the feasibility of centralizing and elevating a future monorail system over the I-90 floating bridge should be investigated, as shown in Figure 11. The main purpose for such a study would be to center the dead load of any additional public rapid transit system close to the center of buoyancy of the floating pontoons and also to maintain the existing traveled way for vehicles across Lake Washington.

a. MONORAIL GUIDE BEAM DESIGN

The support structure will consist of an elevated lightweight monorail beam, similar to the Concept A beam designed for the monorail at grade option in Section 4a above. This guide way beam consists of an all steel box-beam with a metalized non-skid surface applied to the top plate of the beam and weighs 1,109 plf.

b. DEAD LOAD ANALYSIS AND FREEBOARD LOSS MITIGATION

A number of options to mitigate the loss of freeboard were identified in the previous LRT study. As the elevated monorail option will maintain traffic flow within the two HOV lanes on the south side of the I-90 Homer Hadley bridge, certain weight mitigation measures (M5 and M12) utilized for the monorail at grade scenario will no longer apply to the elevated monorail. It was determined that the maximum amount of freeboard loss that