Final Report
for the
City of Bellevue's Peer Review of Segment B7 of Sound Transit's East Link Light Rail Project

Bellevue, Washington

Prepared for:

CITY OF BELLEVUE

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1.0 Introduction

In March 2010, the Bellevue City Council (Council) updated its routing preference for Segment B (I-90 to SE 6th Street) of Sound Transit's East Link Light Rail Transit (LRT) Project to the B7 alignment. Council members have numerous questions about Sound Transit's data and analysis on the B7 alignment and are seeking an independent review of the analysis to evaluate its accuracy and completeness. Additionally, the Council would like to identify the critical areas for additional design and study, were B7 to be advanced for further analysis and engineering.

1.1 Purpose

David Evans and Associates, Inc. (DEA) has been tasked to conduct a peer review of Sound Transit's environmental analysis and conceptual engineering of the East Link B7 alternative to evaluate the sufficiency of analysis and identify areas for additional analysis and refinement.

1.2 Approach

The overall work approach, as directed by the City, was as follows:

1) Review the East Link Draft Environmental Impact Statement (DEIS) methodologies for individual discipline areas for best practices and consistency with industry standards. Review assumptions for reasonableness and consistency with local practices and industry standards. Industry standards included FTA environmental procedures as codified in CFR 49, Part 622 – Environmental Impact and Related Procedures. These procedures incorporate by reference Title 23 CFR – Federal Highway Administration (FHWA).

2) Review data sources for accuracy and completeness.

3) Review B7 conceptual engineering plans included in the DEIS and assess constructability of the design.

4) Evaluate overall sufficiency and completeness of the body of the B7 analysis for this stage of the project; compare to evaluation of other alignments in the DEIS and studies from other projects developed for the alignment selection stage of the environmental process.

5) Identify critical areas for additional analysis and refinement.

2.0 Conclusions

2.1 Draft Environmental Impact Statement

2.1.1 Traffic

The methodology used to analyze impacts in Chapter 3 Transportation Environment and Consequences was consistently applied to the B Segment Alternatives. The methodology resulted in three areas where the resulting analysis of the alternatives was either more conservative or less conservative.

Station Sizing

Park-and-Ride stations were not sized to match forecast ridership, rather they were sized based upon the development potential of the site (existing or acquired). This resulted in the South Bellevue Park-and-Ride being smaller than anticipated demand and the 118th Avenue Park-and-Ride being larger than anticipated demand. This method, while consistent, resulted in station costs that were not proportional to ridership forecasts for each station.

Station Impacts to Arterial Streets

The methodology used to evaluate station area arterial and intersection impacts was based on the station size (provided parking) not the forecast number of auto trips generated. This method results in a traffic forecast smaller than the ridership demand model for South Bellevue (station size is smaller than forecast demand) and a traffic forecast larger than the station demand (station size is bigger than forecast demand) at 118th Avenue. This results in underestimating impacts at South Bellevue and overestimating impacts at 118th Avenue.

Arterial and Intersection Operations Analysis

The HCM methodology used to evaluate intersection operations does not address observed queue spillback impacts on both South Bellevue Way and SE 8th from adjacent intersections and freeway ramp meters. This effect is more pronounced at South Bellevue, but occurs at SE 8th as well. The HCM methodology does not provide a worst-case analysis of the impacts of increased traffic on the arterials at these locations.

Intersection Mitigation

No mitigation is proposed for the South Bellevue Park-and-Ride access. The proposed mitigation at SE 8th is based upon an outdated intersection plan that does not reflect recent changes to I-405. The HCM methodology is limited as it applies to the analysis of intersections because it does not take into account the mitigation of additional queuing at either location.

2.1.2 Other Resources

DEA's review finds that Sound Transit's East Link DEIS fairly compares the B7 alignment with other Segment B alternatives. The technical approach and methodologies used to evaluate the environmental impacts of B7 are generally consistent with professional standards in the various disciplines. However, several specific items were identified as lacking, which DEA would typically expect in an EIS analyzing a project as large as East Link, and that, in most cases, are required by various guidance documents. These items are identified below:

Land Use – The DEIS did not highlight major differences in consistency with regional and local plans, goals, and policies between the No Build and Build Alternatives.

Visual and Aesthetic Resources – The DEIS did not provide a cross-walk between the standard FHWA numerical rankings and the visual quality categories used in the DEIS, so that the reader could understand what constitutes a significant visual impact.

Ecosystem Resources – The DEIS did not clearly explain and define the footprint of the project so the reader can easily understand what constitutes a permanent versus a short term impact.

Historic and Archaeological Resources – The archaeological field survey conducted as part of the DEIS did not include any of the publicly accessible portions of Mercer Slough adjacent to the B7 alignment.

2.2 Conceptual Engineering

DEA's review finds that the Conceptual Design drawings contained in the East Link DEIS exhibit the level of design and detail which would be expected for a light rail transit line at this stage of project development. Further, the level of design work appears similar between the B7 and B3 alignments. DEA's review of Sound Transit's Capital Cost Estimate for B7 finds that the estimate is supported by a Basis of the Estimate which explains the underlying assumptions and estimating methodology and provides a fair comparison of the estimated costs of the B7 and B3 alternatives. DEA finds that Exhibit 2-24 "At-Grade Track with Planned Trail in Former BNSF Railway Right-of-Way" that accompanies the description of B7 in Chapter 2 of the DEIS does not accurately represent the typical conditions found within the BNSF corridor and that a more accurate representation is presented in drawing B7 – KX02 found in Appendix G-1 of the DEIS. DEA's constructability review finds that B7 will be complicated by the construction of the elevated trackway structures in Mercer Slough and construction of extensive retaining structures and substantial regrading needed to accommodate the trackway and trail within the existing BNSF corridor which has limited access.

3.0 Peer Review of East Link DEIS

3.1 Approach

3.1.1 Preliminary Research

DEA reviewed pertinent background documentation as part of this project, including, but not limited to:

- East Link DEIS and technical appendices
- East Link Transit Integration Plan
- Conceptual design drawings for the B7 alignment
- Sound Transit's Capital Cost Estimate (Print Date 2/14/08)

In addition to document review, DEA participated in meetings with Council members, Bellevue Transportation Department staff, and Mayor Davidson to identify concerns with the DEIS. Also, various members of the Sound Transit East Link Project Technical Team were contacted directly for clarification on a variety of topics.

3.1.2 Field Reconnaissance

DEA staff conducted several trips to the project site to observe the alternative Segment B alignments. Photographs were taken and information from the DEIS was verified.

3.1.3 Peer Review

DEA conducted a peer review of the DEIS. Individual discipline areas were reviewed by technical specialists with expertise and experience pertinent to the resource in question. Noise was not included, because a separate review of this resource is being conducted simultaneously by another consultant. Technical specialists reviewed the analysis for each discipline for reasonableness and consistency with local practices and industry standards and for consistency with other B segment alignment alternatives. The reviewers also identified any omissions, corrections, and supplemental data sources.

The process for complying with the National Environmental Policy Act (NEPA) is defined in the joint FHWA/FTA Environmental Impact and Related Procedures (23 CFR 771). This regulation defines the roles and responsibilities of FTA in preparing environmental documents and managing the environmental process. However, the regulation does not provide discipline-specific guidance on the methodologies and topics to be discussed in the EIS. FTA has not produced a separate guidance for implementing NEPA, but does provide some discipline-specific Environmental Resources Information. FHWA does provide more detailed technical guidance in its Guidance for Preparing and Processing Environmental and Section 4(f) Documents (T 6640.8A). DEA conducted the peer review utilizing relevant NEPA guidance documentation from both FTA and FHWA.

3.2 Results and Conclusions

The following section summarizes the results and conclusions of the peer review of the East Link Project DEIS.

3.2.1 Traffic and Transportation

Regional Travel

Methodology and assumptions of Regional Travel in the East Link Project DEIS do not differentiate between alternative alignments, and provide a regional perspective on the effects of the East Link project on other surface transportation systems. The analysis uses screenlines that typically encompass all alignment alternatives for a segment and, therefore, does not provide a comparison of regional impacts at the segment alternative level. There is one screenline that is an exception—Screenline 3 includes I-90 between I-405 and Bellevue Way. It does not include any of the B Segment alignments. This screenline could be used to differentiate any regional impacts to I-90 associated with the B Segment alignments. Screenline 4 could be used to determine regional impacts to I-405 associated with the B Segment alignments. The East Link Project DEIS, however, treated this and the other screenlines in a more general build vs. no build analysis.

Transit

The Transit section in the East Link Project DEIS combines a system-wide analysis of transit using the same regional screenlines as those in section 3.4 Transit of the DEIS to evaluate the regional impacts of the East Link system and a more detailed segment-based analysis of ridership and construction impacts of the transit facilities.

The development of ridership forecasts and station demand utilized standard methodologies. The following key parameters described in section 4.4 of the DEIS influenced the ridership forecasts:

1) Land Use within 0.5 mile of Station (some east of I-405)

- B-3: South Bellevue Station – Suburban Residential – 50% single-family and 50% parkland.
  - Year 2000 Census Block Population = 2,254
- B7: 118th SE – 45% office, 35% single family, 10% parkland, 5% multifamily, and 5% light industrial.
  - Year 2000 Census Block Population = 5,064

2) Walk Access

- B-3: South Bellevue Station – Primary walk access available from single-family neighborhoods to the west.
- B7: 118th SE – Walk access available from adjacent office parks, multi-use residential and from neighborhoods to the east and west.

3) Transit Dependent Populations

- B3: South Bellevue Station – 13.8% minority
- B7: 118th SE – 4.7% minority
- Similar in low-income/senior populations and households with no vehicle

4) Drive Access Capture Area/Patterns and Ease of Site Access

- B3: South Bellevue Station – Many drive access trips approach from east via I-90 and south via I-405, from Newcastle, Issaquah, and Sammamish areas (East Link Project Update, April 8, 2010).
  - Requires approx. 0.5 mile out-of-direction travel each way from I-90 to make LRT connection for WB travel to Seattle.
  - Direct site access via I-90 and Bellevue Way SE.
- B7: 118th SE – Many drive access trips approach from east via I-90 and south via I-405, from Newcastle, Issaquah and Sammamish areas (Sound Transit presentation to Bellevue City Council, East Link Project Update, April 8, 2010).
  - Requires approx. 1.6 miles out-of-direction travel pattern from I-90 to make LRT connection for WB travel to Seattle.
  - Indirect site access via I-90, I-405, SE 8th Street and 118th Avenue SE
  - Some drive access demand diverts to Mercer Island station for more direct connection to Seattle.

5) Bus Access

- ST Route 550 would be eliminated under either scenario due to duplication of LRT service.
- B3: South Bellevue Station
  - Bellevue Way SE would continue to serve as an important route for local/regional service (Sound Transit presentation to Bellevue City Council, East Link Project Update, April 8, 2010).
  - ST Transit Integration Plan indicates several route connections would be made to station. Three routes terminate at station, others make through connections.
- B7: 118th SE
  - Section 3.4.3.1 of the East Link Project DEIS indicates that some bus routes would be rerouted to begin/end at this location, using 118th Avenue SE. Bus service would change to connect Mercer Island with South Bellevue Park-and-Ride Lot and Downtown Bellevue.
  - Many routes (i.e., routes along 112th Avenue SE) would not divert to station due to added travel time for passengers not transferring and increased costs for operating costs (Sound Transit presentation to Bellevue City Council, East Link Project Update, April 8, 2010). Routes serving 112th Avenue SE would connect to East Main Station instead.

   - ST Transit Integration Plan and East Link DEIS indicate transit routes from the Wilburton Park-and-Ride will be rerouted with one route terminating at the 118th Station, and the remaining routes serving SE 8th or the East Main Street Station.

6) Light Rail Travel Time in Segment

- B3: South Bellevue Station – 5 min.
- B7: 118th SE – 5 min.

7) Adjacent Station Proximity

- B3: South Bellevue Station to East Main Station = 1.67 miles
- B7: 118th SE to East Main Station = 0.46 miles

8) Station Mode of Access Breakdown (2030 PSRC Travel Demand Model)

- B3: South Bellevue Station by Mode to Station
  - Walk = 3.7% – will walk to the station from adjacent neighborhoods
  - Bus = 32.2% – will transfer from bus routes
  - Drive = 64.1% — will drive to the park-and-ride
- B7: 118th SE
  - Walk = 29.5% – will walk to the station from adjacent neighborhoods on 188th Avenue and office parks adjacent to SE 8th Street
  - Bus = 5.1% – will transfer from bus routes
  - Drive = 65.4% – will drive to the park-and-ride

9) Passenger Destination Observations at Existing Park-and-Rides

- B3: South Bellevue Station
  - Analysis of boardings data of South Bellevue Park-and-Ride users indicate approximately 70% of users are destined for Downtown Seattle and 25% are destined for Bellevue (pers. comm., Sound Transit and Ed Schumm, DEA).
- B7: 118th SE
  - Bus services at Wilburton Park-and-Ride are designed primarily to serve Boeing workers and are limited to peak-only services. Route 243 currently ends at Wilburton Park-and-Ride and originates in Jackson Park via downtown Bellevue, and not designed to serve park-and-ride commuters to Seattle (pers. comm., Sound Transit and Ed Schumm, DEA).

The assumptions made regarding transit integration contribute to the ridership forecast for the stations. A high level of transit integration is assumed for South Bellevue, while minimal transit integration is assumed for the 118th Station. Most of the transit service previously supporting the Wilburton Park-and-Ride is integrated into the East Main Station for the transit ridership forecast, which is part of Segment C.

Station boarding provides a useful comparison of station locations, and helps normalize productivity with cost effectiveness. At the segment level, it may be appropriate to add an effectiveness measure that includes passenger trips already on the train (in each direction) in addition to the boardings that occur at segment-specific stations. This would help represent the overall passenger demand that is enabled by the segment. An alternative/additional methodology for evaluating segment ridership would be to consider combining segments B and C due to the close proximity and inter-relationship of station capture areas.

Highway Operations and Safety

The analytical methods and assumptions for Highway Operations and Safety in the East Link Project DEIS focus primarily on impacts to I-90 resulting from inclusion of Segment A in the center roadway. Impacts to I-405 and I-90 related to variable travel patterns associated with the B Segment alternatives are addressed with respect to HOV direct access ramps at Bellevue Way. Impacts to interchange ramps and signals are also addressed in the Arterials and Local Streets section of the East Link Project DEIS.

The Highway Capacity Manual intersection LOS methodology used in Section 3.6 Arterials and Local Streets of the DEIS does not include a queuing analysis. The intersections affected on Bellevue Way and SE 8th are all affected by queue spillback from adjacent intersections and or ramp meters. The DEIS did not report the impact of station generated traffic on queuing at the intersections or ramp meters. The forecast station generated auto traffic is primarily focused to I-90 at the South Bellevue Station and I-405 at the 118th Station. The operational analysis of the freeway and ramp system focused on overall travel times. Ramp queuing onto the local system was not specifically addressed.

Arterials and Local Streets

The Arterials and Local Streets section in the East Link Project DEIS focuses on arterial street operations and differentiates impacts by Segment alternatives. Exhibit 3-22 identifies the intersections analyzed for Segment B and forecast LOS by segment alternative. The Highway Capacity Manual (HCM) methodology was consistently applied to the analysis of the segments. The HCM methodology does not address observed operational issues in the B Segment areas, including queues from ramp meters at freeway interchanges or adjacent arterial signals.

Arterial impacts identified included impacts from at-grade light rail operations and arterial impacts from increased vehicle traffic associated with stations that include park-and-ride facilities.

Traffic forecasts were developed based upon ridership forecasts and proposed station capacity. The methodology assumed that station capacity would govern the traffic forecasts at each station, not the ridership model. Table 3-24 summarizes the PM peak auto trips by segment alternative, including a comparison of the ridership model demand and capacity of the park-and-ride lot. This methodology resulted in a PM Peak auto demand at the South Bellevue Park-and-Ride that was about 8 percent less (1,750 vs. 1,910) than the ridership model forecast used for analysis of arterial impacts for alternatives B1, B2A, B2E, and B3. Applying the same methodology to 118th Park-and-Ride resulted in the PM Peak auto demand at the 118th Park-and-Ride that was nearly double (1,100 vs. 560) the ridership model forecast used for analysis of arterial impacts for alternative B7. This would appear to overestimate arterial traffic impacts for alternative B-7, and underestimate arterial traffic impacts for the other B alternatives, including B-3. The use of the HCM methodology grading scale of LOS A through F is limited in that intersection delay can continue to degrade once the LOS F threshold has been reached. There is the potential for significant variation in delay and congestion between an intersection that just exceeds the threshold delay for LOS E to F and an intersection that is overloaded to the point of gridlock, queuing from adjacent intersections and freeway ramp meters which can be observed today. The HCM methodology used does not address the observed queue spillback from adjacent intersections.

Section 3.6.5 Potential Mitigation excludes specific mitigation for the proposed South Bellevue Park-and-Ride access intersections. The mitigation proposed for 118th Ave SE is based upon an outdated intersection configurations (prior to the recent I-405 improvements) The mitigation proposed is based upon the HCM methodology described previously and does not address potential impacts of ramp meter or downstream intersection queuing.

Non-motorized Facilities

Analysis of Non-motorized Facilities in the East Link Project DEIS addresses non-motorized transportation. Impacts were identified for alternatives within segments. Non-motorized trip generation is identified by segment. The source of the non-motorized demand forecast is not specified and is assumed to be from the ridership model. The use of the BNSF corridor for trails is discussed in terms of what was known in 2008. The recent acquisition of the corridor by the Port of Seattle and associated use agreements with Sound Transit and others are not reflected in the East Link Project DEIS.

Freight Mobility and Access

Analysis and methodology of Freight Mobility and Access in the East Link Project DEIS does not generally differentiate between alternative alignments within segments and provides a general perspective on the affects of the East Link project on freight mobility at the regional and local arterial level. The use of the BNSF corridor for freight movement is discussed in terms of what was known in 2008. The recent acquisition of the corridor by the Port of Seattle and associated use agreements with Sound Transit and others are not reflected in the East Link Project DEIS. Construction impacts are differentiated at the segment alternative level.

Navigable Waterways

The Navigable Waterways section in the East Link Project DEIS states impacts on navigability are not anticipated for segments B1, B2A, B2E, or B3. The DEIS states the elevated profile of B7 would not block recreational navigability.

3.2.2 Acquisitions, Displacements, and Relocations

Section 4.1 of the DEIS identifies property acquisitions required for each alternative, including the number and type of permanent displacements and relocations and temporary construction easements. Data utilized for the analysis included right-of-way boundaries and parcel data from King County and the City of Bellevue and current land uses verified during the summer of 2007. Compliance with all relevant relocation policies is clearly stated, including the federal Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970, as amended; Sound Transit's Real Estate Property Acquisition and Relocation Policy, Procedures, and Guidelines; and the State of Washington's relocation and property acquisition regulations (WAC 468-100 and RCW 8.26). Although it was not specifically stated in the DEIS section, the methodology appears to be consistent with FHWA's Guidance for Preparing and Processing Environmental and Section 4(f) Documents (T 6640.8A) with the exception of detailed information regarding households and businesses to be displaced for each alternative, including: (1) family characteristics of households to be displaced; and (2) descriptions, types of occupancy (owner/tenant), and sizes (number of employees) of businesses to be displaced. However, this level of detail is not typically provided in an EIS during the alternative selection process. Therefore, the omission of this information in the DEIS is accepted as industry standard.

The methodology, analysis, and conclusions utilized in this section were applied consistently between alternative Segment B alignments, including Alignment B7. The conclusion that Alignment B7 would affect the least number of overall parcels is reasonable, since much of the alignment is located within BNSF right-of-way.

3.2.3 Land Use

Section 4.2 of the DEIS provides information on the existing land uses and current zoning, describes potential changes in land use, and evaluates the consistency of the project with local and regional planning policies. Data sources include all relevant local and regional land use policies and plans, zoning data, existing land uses, and right-of-way boundaries and parcel data from King County and the City of Bellevue. It was confirmed with Sound Transit that a land use discipline report was not prepared in support of the DEIS.

The section is missing minor information as required by FHWA's Guidance for Preparing and Processing Environmental and Section 4(f) Documents (T 6640.8A) and FTA's Environmental Resources Information on Land Use and Development. While Section 4.2 does conclude that the project would not result in any changes in development patterns, it does not describe current development trends within the project area as part of the existing setting, such as real estate trends, redevelopment areas, or areas of transitioning land use. The consistency analysis (Table F4.2-1) also does not assess the consistency of *each alternative* with relevant development plans, but rather as a project as a whole. FTA requires that the land use consistency analysis include maps showing existing and planned future land uses (i.e., designated comprehensive plan land use) of the area around the proposed project alternative alignments. Section 4.2 of the DEIS provides generalized zoning maps only and a textual description of existing land use. The addition of a description in current development trends and existing and future land use maps is not expected to result in substantive changes to the conclusions of the DEIS.

The methodology, analysis, and conclusions utilized in this section were applied consistently between alternative Segment B alignments, including Alignment B7. The conclusion that Alignment B7 would result in the greatest land use conversion to transportation-related uses among the Segment B alignments is reasonable, since B7 would fully acquire 6 properties, 3 of which are associated with the 118th Station.

3.2.4 Economics

Section 4.3 analyzes the potential adverse and beneficial economic impacts of the project on local and regional economies. The analysis was based upon field observations and data from all relevant local, state, and federal sources, consistent with standard industry practices. The methodology and analysis is consistent with FHWA's Guidance for Preparing and Processing Environmental and Section 4(f) Documents (T 6640.8A), including discussion of business and employee displacement, tax revenues, accessibility, and retail sales. The methodology, analysis, and conclusions utilized in this section were applied consistently between alternative Segment B alignments, including Alignment B7. The conclusions are reasonable that Alignment B7 would affect the most employees in Segment B and would result in the least negative impacts from construction in Segment B.

3.2.5 Social Impacts, Community Facilities, and Neighborhoods

Section 4.4 of the DEIS evaluates potential project impacts to communities and neighborhoods. Data sources primarily include other DEIS sections and demographic data from the U.S. Census Bureau and local jurisdictions, which is consistent with standard industry practices. The methodology appears to be consistent with FHWA's Guidance for Preparing and Processing Environmental and Section 4(f) Documents (T 6640.8A) and FTA's Environmental Resources Information on Community Impacts. It was confirmed with Sound Transit that a socioeconomic discipline report was not prepared in support of the DEIS.

Environmental Justice is addressed in a separate appendix to the DEIS, although it is typically closely intertwined with the discussion of social and economic impacts. FTA and FHWA provide differing guidance on the relationship between social impacts and environmental justice. The FHWA Guidance requires an analysis of Environmental Justice impacts as part of the Social Impacts section of the DEIS. Environmental resources provided on the FTA website address Environmental Justice as a separate discipline. Although there are very few references to the Environmental Justice Appendix in Section 4.4 of the DEIS, the methodology, analysis, and conclusions does meet industry standards.

The methodology, analysis, and conclusions utilized in this section were applied consistently between alternative Segment B alignments, including Alignment B7. The conclusion is reasonable that Alignment B7 will result in minor impacts on communities and neighborhoods.

3.2.6 Visual and Aesthetic Resources

Peer review of the Visual and Aesthetic Resources discipline included a review of Section 4.5 of the DEIS and Appendix F4.5 – Visual Consistency and Key Observation Point Analyses. These documents assessed the existing visual conditions of the project area and the changes predicted to occur with construction of the various East Link Project alternatives, as well as consistency with the visual resource goals and policies of relevant local comprehensive plans and identification of key viewing locations where potential visual impacts of the proposed action were simulated. Approaches and terminology for this analysis were based on FHWA's Visual Impact Assessment for Highway Projects methodology. However, the DEIS uses a modified approach to the FHWA methodology, assigning general visual quality categories (high, medium, and low) to portions of each project segment instead of the typical numeric ratings (1 through 7). This approach was used to describe existing conditions and project impacts in a more reader-friendly manner. But, the DEIS does not provide a basis for how these numerical rankings were converted to the general categories, which prevents a repeatable application of the FHWA methodology. As a result, a significant visual impact may occur even if an overall category reduction does not (for example, from medium to low).

These methods appear to have been applied consistently between alternative Segment B alignments, including Alignment B7. Impacts were assessed and calculated in a consistent manner for all proposed alignments.

3.2.7 Air Quality and Greenhouse Gases

The analytical methods and assumptions for Air Quality and Greenhouse Gases appear to be consistent with industry standards. The Air Quality section covers criteria pollutants, Mobile Source Air Toxics (MSAT), and greenhouse gas, which are all the conventional issues associated with air quality. Most of these analyses are conducted at a corridor-wide level and so do not distinguish between the individual alignments, as is expected for a non-localized issue such as air quality. The one aspect of air quality that is analyzed on an individual alignment basis is air quality at intersections that are influenced by local vehicular traffic. Spatially, it initially appears that much less analysis was conducted on Alignment B7. However, 118th Avenue SE has no intersections between SE 8th Street and I-90. Therefore, this analysis has treated each alignment as equally as possible given the constraints of the alignment locations.

3.2.8 Noise and Vibration

Not reviewed; subject to a separate analysis by others.

3.2.9 Ecosystem Resources

The peer review of the Ecosystem Resources discipline included a review of Section 4.8 of the DEIS and Appendix H3 – Ecosystems Technical Report. These documents assessed the existing conditions and potential impacts to upland vegetation, wetlands, aquatic habitat, threatened and endangered fish and wildlife species, species of concern, and WDFW priority species. Analysis of Ecosystem Resources in the East Link Project DEIS appears to have utilized appropriate best available science background literature and methods, including adhering to requirements of City of Bellevue Municipal Code, U.S. Army Corps of Engineers Wetland Delineation methods, and the FHWA NEPA Handbook. These methods appear to have been applied consistently between alternative Segment B alignments, including Alignment B7. Impacts were assessed and calculated in a consistent manner and level of detail for all proposed alignments. No major gaps in the analysis were found.

The impact analysis was based on assumptions which were “worst-case.” That is, Sound Transit assumed a full 100-foot wide corridor of temporary disturbance for construction in undeveloped areas (for example, that portion of B7 across Mercer Slough), and a permanent footprint well outside the elevated section. This conservative approach is consistent with standard practice for NEPA/SEPA EISs when advanced design is not available. However, the analysis does not clearly define the footprint of the project, so it is difficult for the reader to understand what constitutes a permanent versus temporary impact. Nevertheless, it is clear that wetland areas in Mercer Slough would experience some long-term impacts, but would still function as wetland and provide wildlife habitat, water quality and hydrologic functions. The conclusion that Alignment B7 has the highest wetland impact is reasonable, given the above assumption. However, the DEIS does state that these impacts would be expected to be nearly eliminated by BMPs during final design.

3.2.10 Water Resources

Peer review of the Water Resources discipline included review of Section 4.9 of the DEIS. The Water Resources section covers water quality, stormwater drainage, floodplains, and groundwater. The analytical methods and assumptions for Water Resources appear to be consistent with industry standards. However, there is an absence of discussion about project effects on Washington State Department of Ecology §303(d) listed water bodies, which are water bodies that do not meet state water quality standards for beneficial uses such as drinking, recreation, aquatic habitat and industrial use. While the §303(d) listed water bodies are described in the context of what is present in the project area, there is no follow-up discussion in the effects section. Even though it is unlikely the project will affect the §303(d) listings, some discussion of this should be present in this type of document. With this exception, the analyses presented in this EIS adequately provide information typical for water resources.

3.2.11 Energy

Peer review of the Energy discipline included review of Section 4.10 of the DEIS. The analytical methods and assumptions for Energy appear to be consistent with industry standards. Effects on the region’s energy systems were evaluated by a comparison of the highest cost alternative to the lowest cost alternative in each segment. The B7 alternative was not selected as either, since it is mid-range in direct cost. This method evaluates the extreme cases presented by the project and assumes all other alternatives fall within the range presented by these extremes. While this provides a sound method for an overall generalization of the project, it allows only a limited comparison of individual alternatives.

3.2.12 Geology and Soils

Peer review of the Geology and Soils discipline included review of Section 4.11 of the DEIS and Appendix F4.11- Geologic Unit Summaries, Hazard Areas, and Boring Locations. The Geology and Soils section covered topography, regional geology, seismicity, geologic hazards, and site geology. The analytical methods and assumptions for Geology and Soils appear to be consistent with industry standards. The East Link Project DEIS states that Alignment B7 would: 1) have an elevated structure over Mercer Slough, "which would be at greater risk during a seismic event than other alternatives;" and 2) have steep slopes that "would be at greater risk during operations than they would be for other segment B alternatives." However, there is no detailed explanation in the DEIS of why B7 would be at greater risk than the others. Studies are available that document the challenges of constructing bridges in soft peat soils such as the ones common in Mercer Slough. These soils were studied as part of the construction of the I-90 Bridge as well as the reconstruction of SE 8th Street north of Mercer Slough. Additional discussion of this issue is provided in Section 4.2.5.

3.2.13 Hazardous Materials

Peer review of the Hazardous Materials discipline included review of Section 4.12 of the DEIS. This analysis was performed in accordance with guidance in the Washington State Department of Transportation's Environmental Procedures Manual M 31-11. The analytical methods and assumptions for Hazardous Materials appear to be consistent with industry standards. Only two of the B alternatives (B1 and B7) are associated with hazardous materials sites. The EIS describes the hazardous materials sites on both alignments equally. Comparisons of the two affected alignments appear fair and equitable.

3.2.14 Electromagnetic Field (EMF)

Peer review of the EMF discipline included review of Section 4.13 of the DEIS. The analytical methods and assumptions for EMF appear to be consistent with industry standards. The EIS identifies no sensitive receptors for EMF within segment B. Sensitive receptors are primarily locations with sensitive electronic equipment. The EIS also assumes that EMF will have negligible effects beyond 100 feet from the source. Since no sensitive receptors are present within segment B, these alternatives are treated equally with respect to the absence of any EMF effects.

3.2.15 Public Services

Peer review of the Public Services discipline included review of Section 4.14 of the DEIS. The analytical methods and assumptions for Public Services appear to be consistent with industry standards. The Public Services section covers fire and emergency medical services, police, postal service, schools, and solid waste and recycling collection. All Segment B alignment alternatives were treated equally with respect to this analysis.

3.2.16 Utilities

Peer review of the Utilities discipline included review of Section 4.15 of the DEIS. The Utilities section identified existing and planned utilities in the study area, including water, sanitary sewer, storm sewer, electrical power, natural gas, telephone and communications infrastructure, and petroleum products pipelines, then analyzed potential conflicts between these utilities and the proposed project alignments. Only one utility conflict (overhead electric power lines) was identified for the B7 alignment. The analytical methods and assumptions for Utilities appear to be consistent with industry standards. All Segment B alignments were treated equally.

3.2.17 Historic and Archaeological Resources

Peer review of the Historic and Archaeological Resources discipline included review of Section 4.16 of the DEIS and Appendix H4 – Historic and Archaeological Resources Technical Report. Analysis of this resource followed state and federal guidance as described in the National Historic Preservation Act of 1966, the Code of Federal Regulations Title 36, Part 800: “Protection of Historic Properties,” Section 4(f) of the U.S. Department of Transportation Act of 1966, and WSDOT’s Environmental Procedures Manual Section 456, “Historic, Cultural, and Archaeological Resources.” This analysis covered sensitive archaeological sites, inventory of historic buildings and structures and evaluation of eligibility for the National Register of Historic Properties, and coordination with Washington Department of Archaeology and Historic Preservation, local jurisdictions, and Indian tribes. No archaeological remains were identified on any segment, and no traditional cultural properties were identified in the project vicinity. No historic properties were identified along the B7 alignment. The analytical methods and assumptions for Historic and Archaeological Resources appear to be consistent with industry standards. However, the archaeological field survey focused on publicly owned land that was dispersed among the alternatives as practical. None of the surveyed archaeological tracts were within the sensitivity zone of alignment B7, but instead were just outside of it in publicly owned portions of Mercer Slough. In comparison, four survey tracts were within the sensitivity zone of the various Bellevue Way alignments. This lack of field survey within the B7 sensitivity zone leaves open the possibility of encountering previously undiscovered archaeological sites within the B7 alignment. Sound Transit appropriately addressed this issue by committing to a phased study approach which includes conducting subsurface testing prior to or during construction.

3.2.18 Parkland and Open Space

Peer review of the Parkland and Open Space discipline included review of Section 4.17 of the DEIS, Appendix D – Section 4(f)/6(f) Evaluation, and Appendix F4.17 – Park and Recreational Resources Inventory. The analytical methods and assumptions for Parkland and Open Space appear to be consistent with industry standards. Section 4(f) and Section 6(f) resources appear to have been identified and analyzed appropriately. All pertinent communication with local jurisdictions appears to have been conducted and documented in the report. Sound Transit should seek final mitigation commitments to ensure a *de minimis* finding for impacts to City-owned park property in Segment B should be included in the Final EIS.

4.0 Peer Review of East Link Conceptual Engineering

4.1 Approach

4.1.1 Preliminary Research

DEA reviewed relevant portions of the following documents as part of the preparation of this Task 2 report:

- Conceptual design drawings for the Segment B alignments, specifically the B7 and B3 alignments
- Sound Transit's Capital Cost Estimate (Print Date 2/14/08)
- East Link DEIS and technical appendices

In addition to document review, DEA participated in meetings with Council members, Bellevue Transportation Department staff, and Mayor Davidson to identify specific concerns with the DEIS.

4.1.2 Field Reconnaissance

DEA staff conducted a reconnaissance of the B7 and B3 alignments to observe conditions that are likely to affect the design and construction of the alternatives, and took photographs of conditions within the Burlington Northern Santa Fe (BNSF) corridor.

4.1.3 Geotechnical Briefing

A DEA structural engineer attended a briefing by WSDOT personnel in Olympia on July 1, 2010 regarding the results of additional geotechnical studies of structures in the immediate vicinity of Mercer Slough.

4.1.4 Peer Review

DEA reviewed the descriptions of alternatives B7 contained in Chapter 2 of the East Link Project DEIS and compared the description with the conceptual engineering plans contained in Appendix G-1 of the DEIS. DEA also examined the supplemental drawings and schedules contained in Appendices G-2 and G-3 related to potential right-of-way (ROW) acquisitions and potential hazardous materials sites. DEA reviewed the capital cost estimate for the B7 alignment in relation to the conceptual engineering plans and supplemental plans and schedules. These reviews were conducted by engineers experienced in the planning and design of light rail transit (LRT) systems. DEA reviewed the plans and cost estimates for reasonableness and consistency with industry practices for projects in the conceptual design phase and also looked for any obvious constructability issues.

4.2 Results and Conclusions

The following section summarizes the results and conclusions of the peer review of Sound Transit's East Link Project Conceptual Engineering Plans for Alignment Alternative B7 and the Constructability of Alternative B7.

4.2.1 Overview of Conceptual Engineering Design

The design of a rail transit system generally begins with identification of the purpose and need for high capacity transit service in a given corridor. After this basis has been established, the planning and design team identify key elements or destinations to be served within the corridor. This step is generally accomplished through a combination of internal work sessions and outreach to the public and local jurisdictions. Sound Transit conducted a "scoping" process during fall 2006 seeking input from the public and affected jurisdictions regarding alignment alternatives to be included in the East Link DEIS.

The team then begins the process of generating alternative alignments that "connect the dots" for desired destinations within the corridor. Often, the first step is to draw an alignment using a felt-tip pen on large maps or aerial photographs. The engineers will translate these crude drawings into conceptual engineering designs using computer aided design and drafting systems. The engineering development of the alignments will be guided by the sponsoring agency's design criteria, or generally accepted design criteria for the specific mode, if no agency guidance is available. At the conceptual design level, the engineers attempt to fit the trackway and stations into the built environment, avoiding obvious obstacles or known environmentally sensitive areas wherever possible. However, the alignment is conceptual and refinement of the design typically occurs later during the Preliminary Engineering (PE) phase. While the conceptual design is based on sound engineering judgment, its primary purpose during the DEIS phase is to permit the identification of environmental impacts and facilitate the comparison of alignment alternatives based on relative environmental impacts, ridership and cost.

The PE design will be based on information obtained during preparation of the DEIS, public comments, and additional engineering investigations. It is during PE that the alignment (both horizontal and vertical) is refined to respond to the environmental conditions identified in the DEIS and other information on the natural and built environment. Advancing the engineering design during PE allows for closer examination of impacts and trade-offs and can provide opportunities to avoid or minimize potential impacts. A major objective of the PE phase is the complete and accurate "definition" of the project's scope, schedule, and capital cost, as well as the sponsor's approach to managing the project. The generally accepted definition of a PE design is that the design is 30% complete; however, the alignment should generally be "frozen" during the PE phase. The phrase "freezing the alignment" implies that no further changes (e.g., shifting the track from one street to another) in the alignment occur after PE except for minor adjustments needed to satisfy engineering requirements.

DEA observes that the East Link DEIS is somewhat unique among recent LRT projects because of the large number of alignment alternatives being considered and evaluated.

4.2.2 Consistency between Plans and Alternative Descriptions

The general description of the B7 Alternative contained in Chapter 2 of the East Link Project DEIS generally agrees with the conceptual design plans for this alternative with the following exception: Exhibit 2-24 “At-Grade Track with Planned Trail in Former BNSF Railway Right-of-Way” that accompanies the description of B7 in Chapter 2 of the DEIS does not accurately represent the typical conditions found within the BNSF corridor and that a more accurate representation is presented in drawing B7 – KX02 found in Appendix G-1 of the DEIS. Exhibit 2-24 portrays a generally flat condition within the 100 ft. wide BNSF corridor. In fact, much of the BNSF corridor includes land that slopes steeply downward to the west of the existing tracks toward 118th Ave SE and rises to the east of the tracks toward I-405. A reader examining this cross-section could conclude that conditions within the BNSF corridor are more favorable for LRT and trail construction than actually exist. Exhibits 4-1 and 4-2 below illustrate actual conditions within the BNSF corridor.

4.2.3 Review of Conceptual Designs

The design drawings for B7 and the other Segment B alignments show equivalent levels of design development and are representative of the level of design expected from plans at the conceptual engineering stage of project development. With respect to the conditions identified in Section 4.2.2 above, the drawings indicate that it will be necessary to construct approximately 4000 ft. of retaining wall on the west side of the alignment and two retaining walls totaling 2370 ft. on the east side of the alignment within the 4800 feet of BNSF corridor. The BNSF corridor constitutes approximately 35% of the total B7 length of 13,872 ft.

4.2.4 Review of Capital Cost Estimates

DEA reviewed the Basis of the Estimate—the document that presents the underlying assumptions used in the development of the Capital Cost Estimates—and the individual estimates for both Alternatives B7 and B3, as specifically requested by Council members. The estimates appear to have been developed using a consistent estimating approach and consistent unit pricing, except where physical differences would dictate the use of alternative pricing. There is a greater level of detail in the estimates than is often found for projects at this stage of development. The estimates were compared to the Conceptual Engineering Drawings for consistency and were found to reasonably correspond to the information contained in the drawings for the respective alignment options. Several specific comments can be found in the matrix in **Appendix A-1**.

Exhibit 4-1
View looking south about Sta. 2090+00

Exhibit 4-2
View looking northwest about Sta. 2076+00

4.2.5 Constructability Issues

Mercer Slough

The LRT guideway for the B7 alternative is supported on elevated structure after it leaves the center of the I-90 roadway and heads east through the southern area of the Mercer Slough. Construction of the columns to support the elevated structure will likely require the use of a large crane and caisson drilling assembly or similar equipment to excavate for the foundations. In a wetland area, this equipment would require construction of a temporary trestle for access to each foundation (East Link Project DEIS Chapter 6.1.2.2). The trestle would also be used for truck access for delivery of concrete, steel reinforcing and other construction supplies unless WSDOT permits use of shoulder areas of the I-90 roadway for these purposes. Once the columns are constructed, the elevated guideway could be constructed using pre-cast segments and a travelling launcher assembly similar to that used to construct elevated portions of Sound Transit's Central Link system which would permit removal of the trestle. Construction and removal of the trestle will create impacts to the wetland and will require the use of more extensive Best Management Practices to minimize impacts than if the construction occurred outside a wetland. This type of work is inherently more complicated and risky than the construction of an at-grade alignment.

Design of the foundations will present challenges because they will likely be founded on peat soils (East Link Project DEIS Chapter 4.1.1.2.3), which are generally considered poor for construction. WSDOT has reported movement in the I-90 piers for the HOV ramp (SB I-405 to WB I-90) located nearby. WSDOT has conducted geotechnical studies to identify the cause of the movement and presented a briefing on the topic to interested parties on July 1, 2010. A copy of the applicable reports is attached as **Appendices A-2** and **A-3**. Preliminary geotechnical studies would ordinarily be undertaken during the PE phase to support a preliminary foundation design and costing, and to determine the scope of detailed foundations studies needed in final design. Because of the uncertainty associated with the foundation conditions, a higher contingency should be associated with the related cost elements. The cost estimates for both B7 and B3 reflect a premium for elevated guideway construction in the slough Line item 10.04, Code EL65.

Southern LRT Crossing of 118th Ave SE (Mercer Slough to BNSF Corridor)

The elevated LRT guideway will cross over 118th Avenue SE as it transitions from Mercer Slough to the BNSF Corridor. The design and construction of this transition will require particular attention to avoid significant impacts to the buildings and businesses within the business park located south of SE 32nd Street on 118th Ave. SE. Maintenance and protection of traffic for vehicles using 118th Ave SE and SE 32nd Street and maintenance of business access will be considerations for the construction contractor.

BNSF Corridor

Construction of the at-grade alignment within the BNSF corridor will present several challenges. Before LRT construction can commence, the current tracks must be removed; the DEIS assumes that both the ties and ballast are contaminated and will require proper disposal as hazardous waste in a Subtitle D landfill (Basis of Estimate page 3). The BNSF corridor is shown to be 100 feet wide. Ordinarily, this would provide more than adequate room for a double track LRT line and associated facilities. When the corridor was in use by BNSF, only a single track occupied the corridor and the existing trackbed is approximately ten feet wide. Beyond the existing trackbed, the ground typically slopes steeply downward to the west and often rises to the east (see Exhibits 4-1 and 4-2). As noted in Section 4.2.3, the conceptual plans show that retaining walls will be required on one or both sides of the alignment in many areas. Construction of these walls may be difficult in some areas because of steep side slopes. Substantial re-grading or importation of fill will also be necessary to create a sufficiently wide condition to accommodate the two LRT tracks, the center catenary supports and duct banks for signals, electrification and communication cables, drainage structures and the proposed trail.

The corridor is marked as having buried fiber optic communication lines and an elevated manhole was seen near the southern end of the corridor; electrical transmission lines are also located immediately adjacent to the corridor on the east. These utilities may impact construction and/or require relocation. This issue of future maintenance access within the operating LRT ROW for utility crews must also be considered.

WSDOT recently widened I-405 in the area adjacent to the BNSF corridor between SE 8th Street and I-90. The mapping used for the DEIS and supporting design work does not appear to reflect the effects of the widening. Portions of the BNSF corridor may have been affected by the widening activities including the construction of a substantial drainage collection basin near the north end of the corridor.

Northern LRT Crossing of 118th Ave SE

The LRT guideway will transition from an at-grade condition at the north end of the BNSF Corridor to an aerial guideway which cross over 118th Ave SE and head north into the 118TH Ave Station. It appears that new construction has occurred in close proximity to the proposed alignment on the east side of 118th and adjacent to the BNSF Corridor which may result in an additional acquisition or a shifting of the alignment to avoid the impact. Maintenance and protection of traffic and maintenance of access during construction in the vicinity of the LRT overcrossing of 118th Ave SE will be a more significant consideration than at the southern crossing because of increased traffic associated with commercial activities near the intersection of SE 8th Street and 118th Ave SE. Two parcels identified as potential acquisitions for the 118th Station are shown as high risk sites from the standpoint of potential contamination by hazardous materials. Additional time will be needed to remediate these sites in advance of construction, and risk is increased as a result of the potential contamination. There is an existing fish ladder on Kelsey Creek in the vicinity of this proposed overcrossing; extra care will be needed to protect the creek from sedimentation and other construction related runoff.

5.0 Recommendations

This section describes study recommendations if the B7 alignment is carried forward for further analysis in the FEIS.

5.1 Traffic

The methodology used for station sizing skews the subsequent arterial traffic impacts and cost per segment boarding calculations of the B segments because the station sizes are not proportional to forecast station demands.

A more refined methodology is necessary to address intersection operations and impacts. Micro-simulation could be used to capture queuing impacts and the performance of mitigation measures on both South Bellevue Way and SE 8th. Mitigation should be developed to mitigate increases to delay and queuing at impacted locations.

5.2 Acquisitions, Displacements and Relocations

In order to accurately understand the potential impacts of the Segment B alignments, update the list of acquisitions, displacements, and relocations in the FEIS based upon the most recent available parcel data and current land uses.

5.3 Land Use

In the FEIS, update the discussion of consistency with regional and local plans, goals, and policies to reflect changes or updates to plans, goals, or policies that have occurred since preparation of the DEIS. In the consistency analysis with regional and local plans, goals, and policies (Table F4.2-1), provide a comparison between the No Build and Build Alternatives, and to the extent feasible, highlight differences in consistency between various alternatives and/or alternative segments.

5.4 Social Impacts, Community Facilities and Neighborhoods

In the FEIS, either incorporate the Environmental Justice Appendix into the Social Impacts section or add references to the Social Impacts section to more closely link the analysis and conclusions.

5.5 Visual and Aesthetic Resources

Establish at least two new KOPs, one looking north from the Mountains to Sound Trail toward the proposed B7 alignment, and another somewhere between KOP 8 and KOP 9 to capture potential visual impacts to multi-family residences along 118th Avenue SE.

5.6 Ecosystem Resources

The following list describes recommendations for improving analysis of the Segment B alternatives carried forward for analysis in the FEIS.

- Provide updated habitat and wetland mapping based on 2009 aerial photography to capture recent changes to vegetation
- Conduct formal wetland delineations in study area, or at least along preferred alternative
- Update wetland rating of WR-4 and WR-5 to more accurately reflect hydrologic connections to Mercer Slough
- Provide estimated area needed for compensatory wetland mitigation (not specific sites) according to local jurisdiction critical area ordinances

5.7 Update ROW Information

It appears that new construction has occurred along 118th Ave SE since the date of preparation of the DEIS. This new construction may affect or be affected by construction of the LRT on Alternative B7. It would be prudent to verify the ROW impacts and associated costs of B7 before a final decision is made.

5.8 Update Design Information

Updated mapping which reflects current conditions should be obtained prior to the commencement of PE.

5.9 Additional Geotechnical Studies

WSDOT has conducted geotechnical studies to determine the cause of movements seen in existing structures adjacent to Mercer Slough. These studies identified a thick peat layer, and movement of the peat layer in response to water level changes in Lake Washington as the cause of the movements. Additional geotechnical studies should be performed to support the design of the elevated guideway if the B7 alignment is selected.

5.10 Field Visit

The physical appearance of the BNSF corridor, specifically the topography within and adjacent to the corridor and the close proximity to the adjacent multi-family properties, may not be apparent to a reader of the East Link Project DEIS. If the Council is considering adopting the B7 Alternative and members have not had an opportunity to walk the BNSF corridor, it would be instructive to schedule a field trip of the corridor prior to a final decision.

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Appendices

Appendix A-1 Specific Comments on Issues Affecting Capital Cost

Comment	B3	B7
Environmental Considerations		
Hazardous material mitigation	Hazmat allowance is $281,250. Even though only three low-risk sites, the unpredictable nature of hazardous materials makes this seem slightly low. Cost estimate unit is lump sum.	Several high-risk sites on full-acquisition parcels at 118th Ave Station. Some medium-risk sites also along alignment, although not on land to be acquired. Construction would have to avoid. Hazmat allowance is $2.8M (combination of lump sum ($1.03M) and unit price (Route Foot) for tie and ballast disposal ($1.75M).
Cost Estimate		
No differences in unit prices or allowance/ contingency mark-ups		
From Table ES-4	$226.09M/mile	$196.15M/mile
From cost estimate (does not include SCC 30, 70 or 80, which are essentially the same across alternatives)	$128.81M/mile	$115.44M/mile
SCC = Standard Cost Categories. The cost estimate by CH2MHill is structured according to the Federal Transit Administration's SCC format. Category 30 is for Support Facilities, Category 70 is Vehicles, and Category 80 is Professional Services.		
SCC categories used in the cost estimate are 10 = Guideway and Track Elements; 20 = Stations, Stops, Terminals, Intermodal; 40 = Sitework and Special Conditions; 50 = Systems; and 60 = ROW, Land, Existing Improvements		
Utility Relocation		Amount shown does not appear to reflect the relocation of the fiber optic line which occupies the BNSF corridor.
Parking Garage	Higher "adder" shown for foundation – perhaps because building on peat/organics? Overall higher construction cost (not incl. ROW cost).	Discrepancy between drawing and report: report says 5-story (1030 spaces); drawing shows 4-story (1000 spaces).
Station	Line item to close and mitigate existing Park & Ride ($15.75M)	
ROW cost	$23.4M (24 impacted parcels)	Almost three times higher at $68.7M (14 impacted parcels); likely due to full-acquisition of four commercial properties.

Comment	B3	B7
Constructability		
Elevated guideway		Lower productivity through Mercer Slough. Additional expense to construct through Mercer Slough.
Staging and materials management	No clear or central location for material yard. Potential long-term construction staging areas in Mercer Slough are 4(f) issue.	Acquisition of 118th Ave Station property provides staging area for Segment B and perhaps also Segment C.

Appendix A-2
Geotechnical Report – Interstate 90: Mercer Slough Bridges

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Geotechnical Report

Interstate 90: Mercer Slough Bridges

Summary of Phase 1 Geotechnical Studies

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INTRODUCTION

Mercer Slough is situated along the eastern shore of Lake Washington (Fig.1). The slough occupies a north-trending trough about 2 miles long and half a mile wide that opens on the south into Lake Washington. Lake level is controlled by locks located to the west near Puget Sound that is annually drawn down to about elevation 6.9 feet in the winter and raised to 8.9 feet in the early summer (City of Seattle datum, which is 12.8 feet above Mean Lower Low Water U.S. Army Corps of Engineers). This seasonal draw facilitates salmon passage into and out of Lake Washington and minimizes seawater intrusion. The topography within the slough is flat with an elevation between 10 and 13 feet. Palustrine wetlands dominated by grasses/sedges, cattails, and willow comprise the southern portion of the slough in the vicinity of the interstate. A narrow, dredged channel drains the slough. North-trending ridges that steeply rise more than 150 feet bound the slough to the east and west.

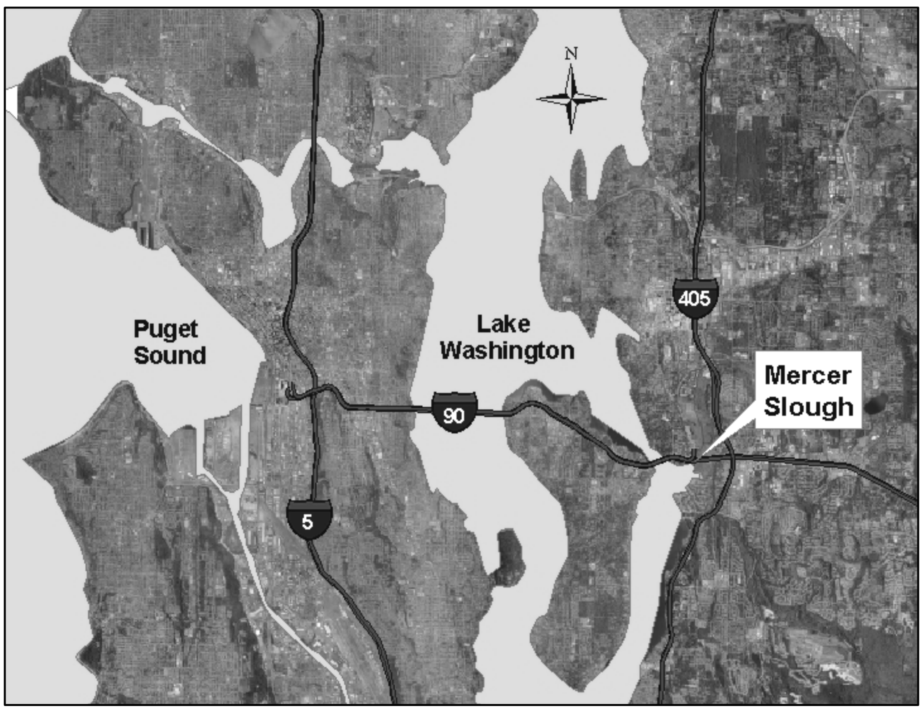

Figure 1. Mercer Slough is located on the east side of Lake Washington just east of Mercer Island.

Public State Highway (PSH) 2 was constructed across Mercer Slough in 1940, which consisted of a single, four lane bridge (90/43N) (Fig. 2). To keep pace with regional growth, additional bridges were completed in 1970 for eastbound traffic (90/43S), eastbound and westbound collector-distributor ramps (90/43ECD and 90/43WCD), and several ancillary ramps on the west side of the slough. The fifth and final major interstate structure (90/43W-W) was completed in 1993 for transit and high occupancy vehicles. Collectively, these structures carry nearly 125,000 vehicles per day. The 30 inch diameter waterline that serves Mercer Island, owned by Seattle Public Utilities, was initially sited within the existing interstate alignment. In 1968, prior to the major interstate improvements, the waterline was moved to its present location. Due to continuing deflections and perceived seismic vulnerability of the waterline, a second waterline was added in 1993 by suspending it from one of the interstate bridges.

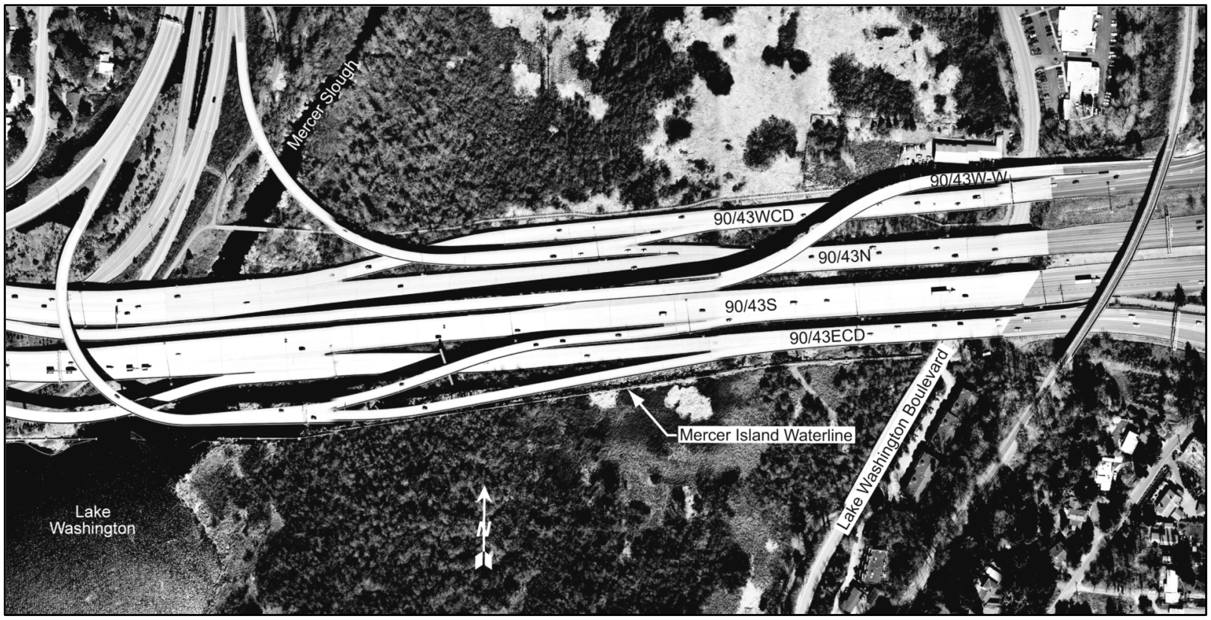

Figure 2. Interstate 90 crosses Mercer Slough and associated palustrine wetlands. Lake Washington Boulevard bounds Mercer Slough on the east. The 30 inch diameter waterline that serves Mercer Island is situated just south of the interstate bridges. Buildings just north of interstate on both sides of Lake Washington Boulevard were constructed around 1988 on 3 to 10 feet of fill.

Thick peat deposits in excess of 60 feet and locally thick soft clay that fill the slough have necessitated deep pile or drilled shaft foundations bearing on dense underlying sands to support the bridges and waterline. The phenomenon of westerly flowing peat, however, has resulted in detrimental lateral loads on these deep foundations. Since at least the late 1960s, deformation of the peat has caused large deflections of the bridges and waterline on the east side of the slough. Near collapse of several bridge spans occurred in 1983, 1987, and 2003, and more than 18 inches of deflection in the early 1970s severely compromised several pile bents of the 30 inch diameter waterline that serves Mercer Island. Additionally, up to 10 feet of settlement has been documented within an embankment section of Lake Washington Boulevard in the vicinity of the interstate. Peat deformation continues to the present day exhibiting an unpredictable and poorly understood effect on these critical facilities.

Previous geotechnical studies

Several geotechnical studies for both the Washington State Department of Transportation (WSDOT) and the Seattle Public Utilities (SPU), formerly the Seattle Water Department, have been undertaken to characterize the extent and causal mechanisms of the peat deformation, evaluate the resultant response of the sub- and superstructures, and develop mitigation alternatives (Shannon & Wilson, 1975; Rittenhouse-Zeman, 1989; Converse Consultants NW, 1995).

In the early 1970s, the Seattle Water Department employed Shannon & Wilson (S&W) to study the ground movement affecting the eastern portion of their waterline and to provide remedial concepts. The study included drilling and instrumenting five borings with slope inclinometers to monitor lateral ground deformation and to characterize subsurface conditions. Vane shear tests were also performed within the peat. S&W identified a number of contributing factors that included (1) the presence of thick peat and its adverse engineering properties (e.g., low strength, poor drainage characteristics, etc.), (2) presence of clean sand with high hydrostatic pressures beneath the peat, (3) fall-winter drawdown of Lake Washington coinciding with elevated groundwater pressures, and (4) instability within the adjacent Lake Washington Boulevard embankment. The lateral extent of the peat deformation was not investigated.

In 1988 after another near collapse of a span on a collector-distributor ramp, WSDOT hired Rittenhouse-Zeman & Associates (RZA) to investigate the mechanism, limits, magnitude, and direction of lateral movement causing the deflections of the 90/43ECD and 90/43WCD ramps on the east side of the slough. Several years prior, several spans within both structures had independently experienced near-collapse due to lateral movement within their foundations. This study was prompted, in part, because of another bridge (90/43W-W) being planned to improve interstate capacity. Subsurface explorations included nine borings and two cone penetrometer tests; seven inclinometers were installed. In situ tests, including vane shear tests, as well as triaxial, consolidation, and direct shear tests were performed. The study concluded that the embankment load from Lake Washington Boulevard and other fill placement coupled with the high hydrostatic pressures below and within the peat were the primary causes of the deformation. The limit of the deforming peat was estimated to extend nearly to the middle of the slough, where a zone of compression within the peat was assumed to exist.

SPU contracted for further geotechnical investigations in the mid 1990s to evaluate repair or replacement options for the 30 in waterline due to ongoing displacements and a developing awareness of its seismic vulnerability (Converse Consultants NW, 1995; Nelson-Couvrette & Associates, 1997). Subsurface investigations including geotechnical borings, both conventional and seismic cone penetration tests, and pressuremeter tests were performed at six locations along the eastern 300 feet of the waterline, where its displacement had been observed. Several pneumatic piezometers and inclinometers were also installed. No interpretations of the lateral extent or causal mechanisms for the peat deformation were offered.

STRUCTURE DEFLECTIONS

The interstate bridges that cross Mercer Slough have been inspected by the WSDOT Bridge Preservation Office approximately every two years, as federally mandated. Evaluation of bridge movement has primarily focused on expansion joint aperture between individual deck units. This measurement, however, only provides an indication of relative movement. Deciphering actual structure movement is greatly complicated by thermal expansion and contraction of the deck units, the interaction of structural elements (deck, support column/bent, and foundation), as well as logistical complexities of a multi-decade inspection program, during which time numerous repairs and major improvements have been made to most of the bridges. Conservative estimates that account for these complications nevertheless indicate deflection in nearly all of the superstructures (the above-ground portion of the structure) since their construction. Figure 3 graphically depicts trends in relative deflections of expansion joints and deck units. These relative deflection trends represent varying time intervals over the last 10 to 20 years, and are not necessarily occurring simultaneously. Some reversals in deflection directions have also been observed. Reported measurements have been corrected for thermal effects. A brief discussion follows for each of the bridges summarizing observed deflections.

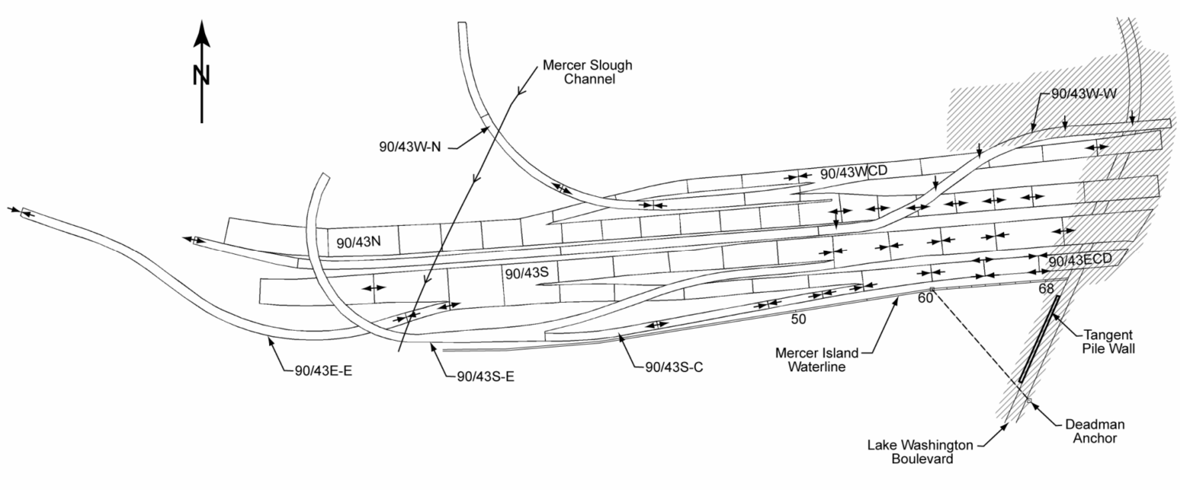

Figure 3. Plan view of interstate bridges and waterline that cross Mercer Slough. Note opening (divergent arrows) of easternmost expansion joints in bridges 90/43S, 90/43N and 90/43WCD and general closing of expansion joints (convergent arrows) toward the center of the structures. The outermost ramps, 90/43ECD and 90/43W-W, as well as the Mercer Island waterline have transversely deflected toward the centerline of the interstate corridor. Both extension and compression of deck joints are occurring in the western portion of the structures. Numbers below Mercer Island waterline reference pile-supported bents; bent No. 68 is the easternmost bent. Hachured area delimits approximate extent of fill.

The oldest bridge (90/43N), which now supports the westbound mainline, was constructed in 1940. Each bent is supported by timber piles with a reinforced concrete pile cap. As early as 1952, inspections reported many tipped rockers, which connect the deck to the cap of the supporting columns. The inspection also noted that expansion joints on the east end were opened on the south side but not on the north, indicating loads were being applied transverse to the bridge. The 1969 inspection noted that the rockers were tipped west across the entire structure, suggesting westward-directed loading/movement in the deck and/or the upper portion of the foundation near the pile cap. Until recently, joint measurements had focused only on the eastern portion of the bridge. The bridge was seismically retrofitted with transverse and longitudinal restrainers in 1991. Since at least 1988, expansion joints in this eastern portion have been opening up to about ½ inch (Fig. 3). The 1997 inspection also reported the compression seal falling out of the easternmost joint.

The eastbound mainline structure (90/43S) was built in 1970, and bents are supported by steel pipe piles with a reinforced concrete pile cap. Up until around 1987, expansion joints in the eastern portion showed compression, after which time no additional displacement has been noted. The bridge was seismically retrofitted with transverse and longitudinal restrainers in 1991. Between 1999 and 2004, opening of the east end up to about ½ inch and clockwise rotation of the westernmost joint have been observed (Fig. 3).

The westbound collector-distributor ramp (90/43WCD) was built in 1970 and is supported by steel pipe piles with a reinforced concrete pile cap. Beginning in 1974, the easternmost joint exhibited a gradual opening trend, culminating in 1987 when the inspection noted a 5.5 inch gap leaving less than 3 inches of bearing. The deck was jacked back into place, and wood blocks were installed in adjacent joints to the west to limit further movement. At that time, restraining straps were also installed across this easternmost joint (Fig. 4). The bridge was seismically retrofitted with transverse and longitudinal restrainers in 1992. Subsequent inspections between 1993 and 2001 noted crushing of the wood blocks. The 2003 inspection noted compression had lessened on some of the wood blocks.

Figure 4. Cable restraint fabricated in 1987 across Joint #9 on 90/43WCD to prevent loss of bearing.

The eastbound collector-distributor ramp (90/43ECD) was built in 1970 and is supported by steel pipe piles with a reinforced concrete pile cap. An inspection in 1983 found that the easternmost expansion joint had opened 7.5 inches and less than ½ inch of bearing remained. The deck was jacked back into place, and wood blocks were installed in two adjacent joints to prevent subsequent movement. The bridge was seismically retrofitted with transverse and longitudinal restrainers in 1992. Between 1993 and 2004, differential opening and closing of up to 12 mm of two eastern deck joints suggest transverse loading on the piers (Fig. 3).

The westbound high occupancy vehicle ramp (90/43W-W) built in 1993 and is founded on large diameter drilled shafts and steel pipe piles. The 2003 inspection reported up to 1.3 inches of southward movement within the eastern portion of the structure (Fig. 3).

The 90/43S-C ramp was built in 1970 and is supported on steel pipe piles with a reinforced concrete pile cap. Inspections between 1980 and 2003 noted up to 1 inch of closure in the center to eastern expansion joints and up to ¼ inch of opening in the westernmost joint (Fig. 3).

The 90/43W-N ramp was built in 1969 and is supported on steel pipe piles with a reinforced concrete pile cap. Between 1974 and 1999, several of the eastern expansion joints closed up to 1 inch, while the center joint showed up to ½ inch of opening (Fig. 3). No trend of movement has been noted since 1999.

The 90/43E-E ramp was built in 1993. Since its construction, the eastern joint has opened up to 1 inch, and the center and western joint have closed up to ½ inch (Fig. 3). Measurements further suggest the entire deck structure has moved west.

In 1968, the Seattle Water Department moved its elevated 30 inch diameter Mercer Island waterline to accommodate planned construction of new interstate structures (Figs. 2 and 3). Located just south of the existing interstate bridges, the waterline is supported by closely spaced bents consisting of a concrete pile cap with a pair of battered timber piles driven through the peat. Almost immediately, 2 inches of settlement and 4 inches of northwesterly movement was noted in the easternmost portion of the waterline between bents No. 60 to 68 over a 300 feet length (Shannon & Wilson, 1975). In early 1969, a deadman-type anchor was attached to the waterline to restrain the deflection. Later that year, a 5 foot deep outfall trench was excavated between the waterline and the interstate bridges from roughly bent No. 60 to No. 68. Its excavation reportedly had an immediate effect on the pipeline causing 5 inches of deflection at bent No. 63. Between 1968 and 1975 a maximum of 18 inches northwest movement and 6.5 inches of settlement occurred at bent No. 63 and at bent No. 65, respectively. The deflections dissipated by bent Nos. 60 and 68; no waterline movement has been observed west of No. 60. In 1992, a 16 in diameter auxiliary waterline to supply Mercer Island was suspended from the one of the interstate bridges (90/43WCD). A tangent pile wall was also constructed along Lake Washington Boulevard at this time to mitigate for the embankment load on the deforming peat and the persistent roadway deformation (Fig. 3).

SITE CONDITIONS

Land modification

Considerable modifications have been made within the slough and surrounding wetlands during historic time. The changes dramatically began in 1917 with the completion of the Lake Washington Ship Canal and the Hiram Chittendom Locks, connecting Lake Washington with Lake Union and Puget Sound, respectively, and the resultant lowering of the lake by nearly 10 feet. The lowered lake level exposed the slough. During the 1920s, the southern portion of the channel was dredged to make it navigable for development and agriculture on the newly exposed ground. Despite construction of an extensive network of shallow drainage ditches, the ground proved too wet for expanded agricultural use. A road and utility lines were constructed across the slough in the late 1920s to early 1930s, followed by the first highway bridge in 1940. An airphoto taken in August 1961 shows the southern portion of the slough (Fig. 5A). In 1967, a large channel was dredged across the slough, presumably in part for the relocation of the Mercer Island waterline and construction of a planned outfall ditch (Fig. 5B). Much of this new channel appears to have been backfilled prior to 1970. Construction of the interstate included relocating the lower portion of the Mercer Slough channel to the east (Fig. 5C). In 1988, construction of several small commercial buildings on both sides of Lake Washington Boulevard just north of the interstate bridges (Fig. 2) entailed placement of up to ten feet of fill on top of the soft saturated peat over an area of about two acres.

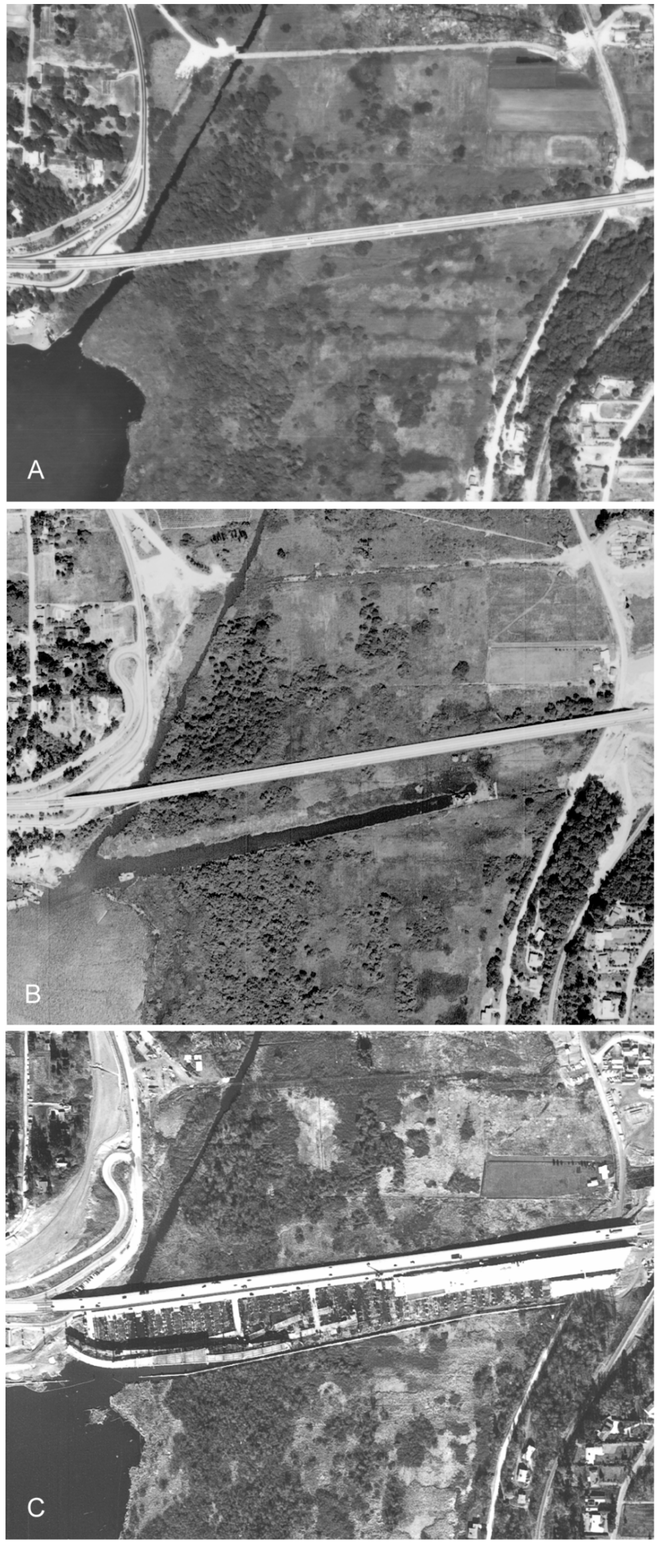

Figure 5. (A) WSDOT airphoto taken August 8, 1961 of the southern portion of Mercer Slough prior to relocation of Mercer Island waterline and interstate construction. (B) WSDOT airphoto taken July 15, 1967 shows excavation of outfall channel paralleling the south side of the highway. (C) WSDOT airphoto taken February 20, 1970 during construction of interstate bridges. Note relocated Mercer Island waterline to the south of the interstate bridges, eastward relocation of Mercer Slough outlet, and backfilled outfall channel.

Geologic setting

The Puget Lowland is a tectonic depression of Eocene origin formed by ongoing northward-directed compression and shortening within the upper crust (Johnson et al., 1994; Wells and Simpson, 2001). Numerous, W/NW-trending moderate to high angle reverse faults rupture the Puget Lowland, the most relevant of which is the active Seattle fault zone. Three inferred thrust faults principally define a 2 to 4 mile wide rupture zone that extends across Puget Sound and Lake Washington in the vicinity of Interstate 90 and terminates in the east near the Cascade foothills (Blakely et al., 2002) (Fig. 6). Recent paleoseismic studies of the Seattle fault zone have identified three, or possibly four, ground-rupturing earthquakes in the last 2500 years along a subsidiary strand of the frontal (Seattle) fault on Bainbridge Island, the most recent of which is dated around 1050 calendar years before present (Nelson et al., 2003). Earthquake magnitude near M7 is estimated from this most recent event. Oblique subduction of the oceanic Juan de Fuca plate under the North American plate is the source of large, moderately deep historic earthquakes (M>6) that have repeatedly occurred in the region, most recently on February 28, 2001 (Nisqually earthquake M =6.8).

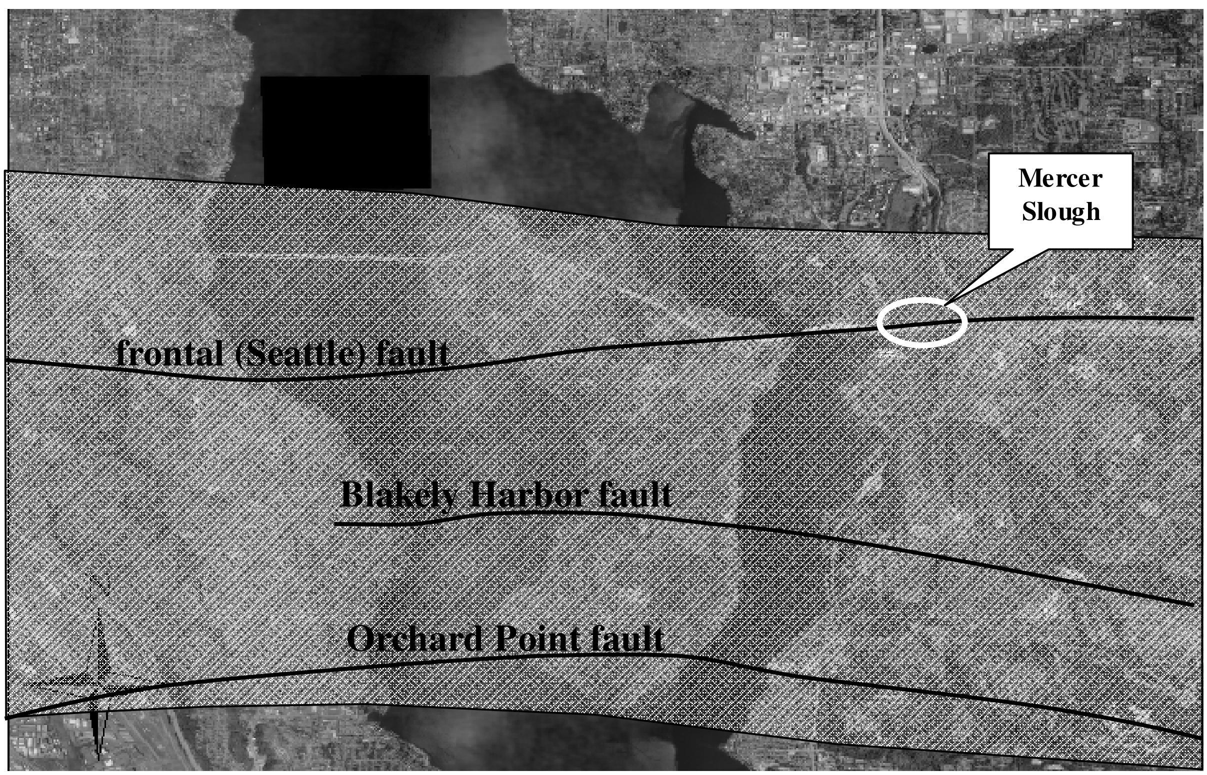

Figure 6. Hachured zone approximately delimits east-trending Seattle fault zone. The inferred location of the frontal (Seattle) fault lies beneath Interstate 90 where it crosses Mercer Slough.

While regional tectonism has locally exposed Tertiary bedrock in the central portion of Puget Lowland, glacial sediments dominate the upper several hundred meters. During the Pleistocene, at least five continental glaciations have inundated the lowlands and shaped the topography, with the most recent and final retreat (Vashon- age) beginning about 16 ka (thousand years before present) (Porter and Swanson, 1998). Characteristic of the Puget Lowland are steep-walled troughs that were cut by subglacial meltwater and ice erosion prior to 14 ka (Booth and Hallet, 1993). Lake Washington occupies one such trough. Around 13 ka, the trough was isolated due to a rapidly prograding alluvial fan/delta of the Cedar River at the southern end of the lake, and the previous marine embayment became freshwater Lake Washington (Leopold et al., 1982). Lake level remained stable until around 9 – 7.5 ka, when a period of transgression and rising lake level occurred as a result of rising Holocene sea level and prograding of the Duwamish River delta and Cedar River alluvial fan (Thorson, 1998). During this time, a thick peat deposit formed in the protected embayment of Mercer Slough. In 1917, the lake level was lowered by nearly 10 feet following completion of the ship canal and the locks exposing the peat within the slough.

Subsurface conditions

Numerous geotechnical investigations provide a detailed picture of subsurface conditions through the interstate corridor (Shannon & Wilson, 1975; Rittenhouse-Zeman, 1989; Converse Consultants NW, 1995; Kramer, 1996). Four engineering geologic units are depicted in a generalized cross-section across the slough (Fig.7).

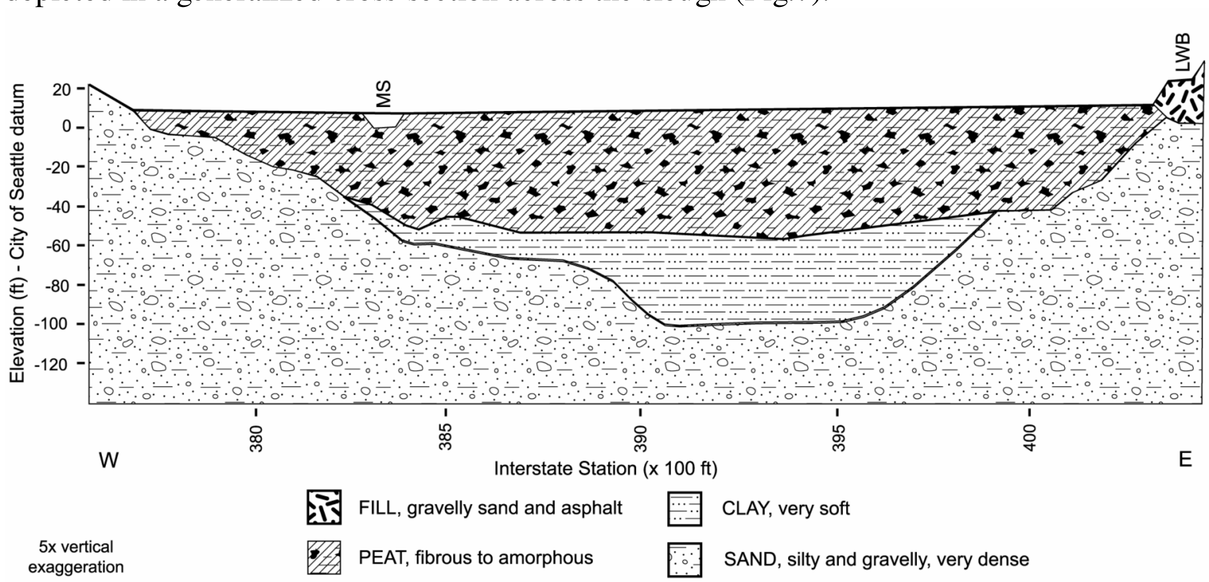

Figure 7. Generalized cross-section of Mercer Slough through the interstate corridor with 5x vertical exaggeration. Horizontal axis is stationing (in hundreds of feet) of interstate and vertical axis is elevation (in feet) using City of Seattle datum. The Mercer Slough channel (MS) and Lake Washington Boulevard (LWB) are also shown.

Peat fills much of the slough up to a maximum observed thickness of 60 feet. Geotechnical descriptions characterize the peat as fibrous near the surface becoming increasingly decomposed and amorphous with depth. Moisture contents range from 500 to 1200 percent with no apparent trend with depth (Kramer, 1996). Standard Penetration N-values are typically zero. Void ratio generally ranges between 5 and 10. Vane shear tests yielded peak undrained strengths 10 to 160 psf and remolded strengths of 8 to 35 psf. Piezocone penetration tests exhibited a uniform tip resistance of about 350 psf and a friction ratio of 1.5 to 6.0 percent; pore pressures during penetration were nearly hydrostatic. Unconsolidated-undrained triaxial tests produced undrained strengths ranging 50 to 180 psf. Consolidated-drained triaxial and direct shear testing yielded friction angles between 9° and 13°; long-term stress-controlled creep testing yielded higher friction values. During pile load tests, the peat also demonstrated significant creep and stress relaxation behavior. Dynamic response of the peat was evaluated with resonant column tests, which showed increasing stiffness with confining stress and a highly nonlinear stress-strain behavior that decreased with increasing effective stress, characteristics similar to cohesionless soils,. The low strength and stiffness of the peat is expected to amplify long-period ground motions and to develop large strains in response. Also as in cohesionless soils, damping of the peat decreased with increasing confining pressure.

Underlying the peat in the middle of the slough is a very soft, low to high plasticity silty clay with scattered organics and shell fragments, and minor interbedded silt and fine sand. Loose to medium dense sands with interbeds of soft clay were locally encountered beneath the peat on the east side, which may also correlate with the clay unit in the middle of the slough. This unit is interpreted to be Vashon-age glaciolacustrine recessional deposit. Its maximum observed thickness is about 50 feet, with the unit thinning toward the basin margins.

Beneath the soft clay and peat is a medium dense to very dense, clean to silty sand with minor gravel, cobbles, and boulders and thin beds of hard sandy silt and silt. Borings typically do not extend more than about 30 feet into this unit. This glacially over-consolidated unit represents either an advance outwash deposit of Vashon age or older glacial and/or non-glacial deposits. Encountered at the top of this sand unit is pressurized groundwater that consistently produced artesian flow 2 to 8 feet above ground elevation. Winter – spring equivalent head levels are typically about one foot higher than summer – fall levels. In one test boring (BRZ-5) on the east side of the slough near the 90/43WCD ramp, artesian flow was also encountered within the peat that was interpreted to be hydraulically connected to the underlying aquifer of the dense sand unit (RZA, 1989). A piezometer in this boring yielded the greatest head (eight feet above ground elevation) measured within the slough. During recent site reconnaissance in 2005, two small springs were noted in the same general location. A small pile of clean sand and gravel was observed below the discharge of one spring that flowed from a 8 inch diameter cavity on the side of a shallow excavated channel. It is assumed that the source of the flow and sand discharge is the sand unit that underlies the peat.

Much of Lake Washington Boulevard in the vicinity of the interstate is founded on about 10 feet of granular fill that in some areas includes considerable thickness of both broken and intact asphalt and concrete pavement associated with decades of pavement repairs (Fig. 7). In 1988, development of parcels on both sides of Lake Washington Boulevard on the north side of the interstate involved the placement of 3 to 10 feet of fill (Fig. 3). Where the fill has been placed over the peat, considerable settlement and lateral deformation has occurred.

Ground deformation

Ground deformation along the east side of the slough is evidenced in deformed fills along and adjacent to Lake Washington Boulevard and in numerous slope inclinometers installed within the deforming peat. Pavement repairs to Lake Washington Boulevard on both sides of the interstate are clearly visible in airphotos from 1936 through 1987. South of the bridges and waterline in the vicinity of the tangent pile wall (Fig. 3), Lake Washington Boulevard is founded on about 15 feet of fill containing older asphalt and concrete roadway surfaces. Fresh tension cracks in the fill exhibiting both horizontal and vertical offset have recently been observed in front of the tangent pile wall, indicating ongoing deformation. Beneath the interstate bridges, 2 to 3 inches of recent settlement has occurred in concrete barriers along Lake Washington Boulevard that are founded in roughly 10 to 15 feet of fill. On the north side of the 90/43W-W ramp, a large area of peat was covered with 3 to 10 feet of fill in 1988 for a two-story, pile-founded commercial building. Nearly three feet of fill settlement is observable beneath the building. No tension cracks have been observed in the slough west of Lake Washington Boulevard. However, many of the pile caps of the mainline bridges (90/43N and 90/43S) are exposed to at least the middle of the slough, suggesting that considerable settlement and/or depletion (flow) has occurred within the peat in this area.

Since the deflection of the relocated waterline was first noted in 1968, about 19 inclinometers have been installed in the eastern portion of the slough in the vicinity of the interstate. Despite the large number of inclinometers, most have a short monitoring history of only several years providing an interrupted history of the long-term, but clearly on-going peat deformation. The interrupted monitoring is due to the use of different uniquely calibrated probes by numerous investigators, shearing of the inclinometer pipe resulting from large peat deformations, and construction damage. In a few cases, when inclinometers have excessively constricted to prohibit further monitoring, new inclinometers have been installed in a proximal location. Despite the shortcomings of the historical record, trends in direction, depth, and magnitude of the peat deformation are well established in the eastern portion of the slough. Figure 8 illustrates the direction and magnitude of movement for representative inclinometers, and Table 1 provides a summary of the depth below ground elevation (BGE), direction and magnitude of near-maximum displacement for the monitoring periods depicted in Figure 8.

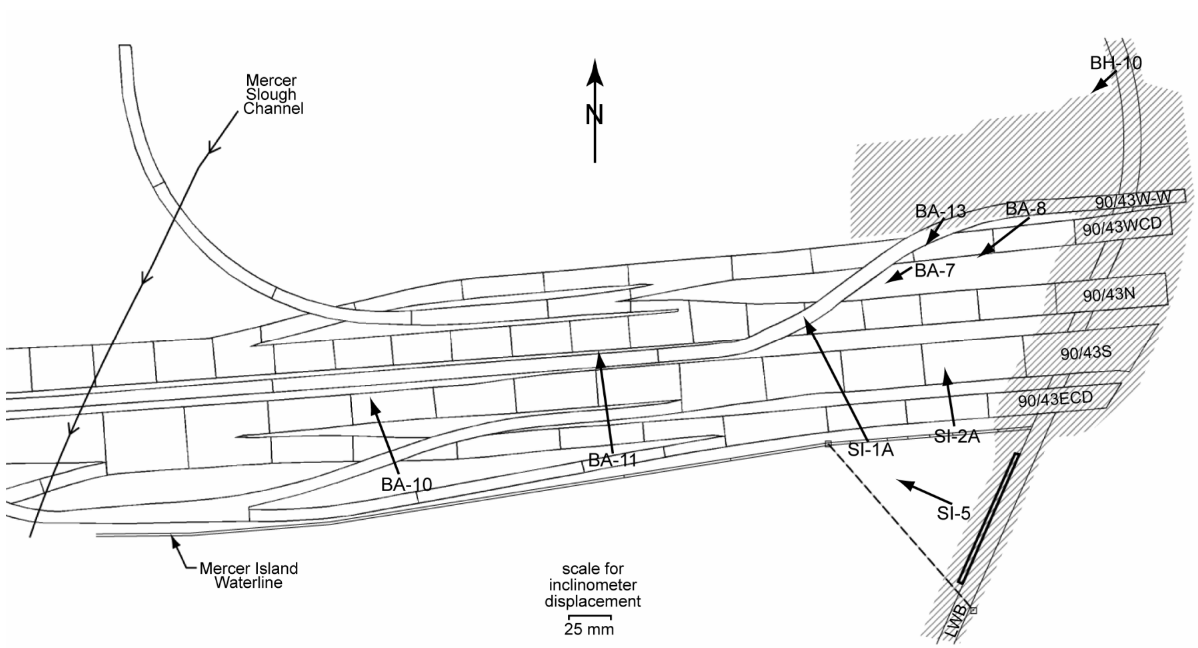

Figure 8. Plan view showing observed peat deformation east of Mercer Slough channel. Arrows depict direction and magnitude of displacement of representative inclinometers summarized in Table 1. Hachured area delimits approximate extent of fill.

Table 1. Summary of inclinometer data

Inclinometer	Monitoring Period (m/d/y)	Depth BGE (ft)	Azimuth (°)	Displacement (in)	Displacement Rate (in/yr)
BA(BRZ)-7	12/9/88 – 1/5/93	10	240	0.8	0.2
BA(BRZ)-8	1/3/89 – 1/5/93	12	223	1.5	0.5
BA-10	12/7/88 – 6/30/89	14	340	1.9	3.5
BA-11	12/7/88 – 6/30/89	18	352	2.5	4.6
BA-13	6/25/98 – 10/3/00	10	216	0.9	0.3
BH-10	11/2/89 – 1/5/93	8	229	0.9	0.3
SI-1A	11/1/93 – 2/6/95	10	333	3.3	1.5
SI-1	10/3/74 – 4/21/75	10	349	2.6	4.7
SI-2A	10/3/74 – 4/21/75	12	350	1.4	2.6
SI-5	10/3/74 – 4/21/75	12	294	1.5	2.7

Inclinometers on both the north and south sides of the interstate record converging peat flow toward the centerline of the interstate structures with the largest displacements recorded on the south side. RZA (1989) summarized inclinometer data between 1971 and 1988 for several locations adjacent to the waterline, and reported a total cumulative displacement up to 30 inches resulting in an annualized rate of about two inches. Prior geotechnical investigations neglected to report data from inclinometers BA-10 and BA-11, in part because the instrument displayed considerable instability probably due to the reported artesian flow through and around the inclinometer casings. Another reason for their omission may have been the somewhat inexplicable direction of movement. Despite the ambiguous north-south axis results for both inclinometers, BA-10 shows a characteristic displacement plot toward the west that was observed in a number of the inclinometers closer to Lake Washington Boulevard and the waterline.

All inclinometers record displacements that initiate near the basal contact of the peat. In some cases, the peat deforms somewhat uniformly through its full thickness, such as is shown in the plot of BA-13 prior to the May 9, 2001 reading (Fig. 9), suggesting a discrete zone of shearing. Another typical displacement profile consists of increasing deformation from the base of the peat to within roughly 15 feet of the surface (creep-type behavior), at which point the peat is uniformly deformed to the ground surface.

Several investigations (S&W, 1975; RZA 1989) noted accelerated movement during wet winter months. This corresponds to both slightly higher hydrostatic pressures within the underlying dense sand and a fall-winter lowering of lake level. Where several decades of data are available for a discrete area, such as inclinometer SI-1/SI-1A, rates of movement appear to have slowed considerably since the late 1980s. The overall trend in the direction of movement, however, has not changed. One notable deformation path observed in a few inclinometers entails an abrupt, short-term reversal (Fig. 10). Periods of expected northwest-directed movement coincide with fall-winter drawdown and low lake level, while the reversal corresponds with spring filling and high summer lake level.

S&W (1975) made an illuminating observation about the nature of the peat deformation, when they noted that movement initiated in inclinometers SI-1 and SI-2A, located farther out in the slough, several months prior to inclinometers located closer to Lake Washington Boulevard.

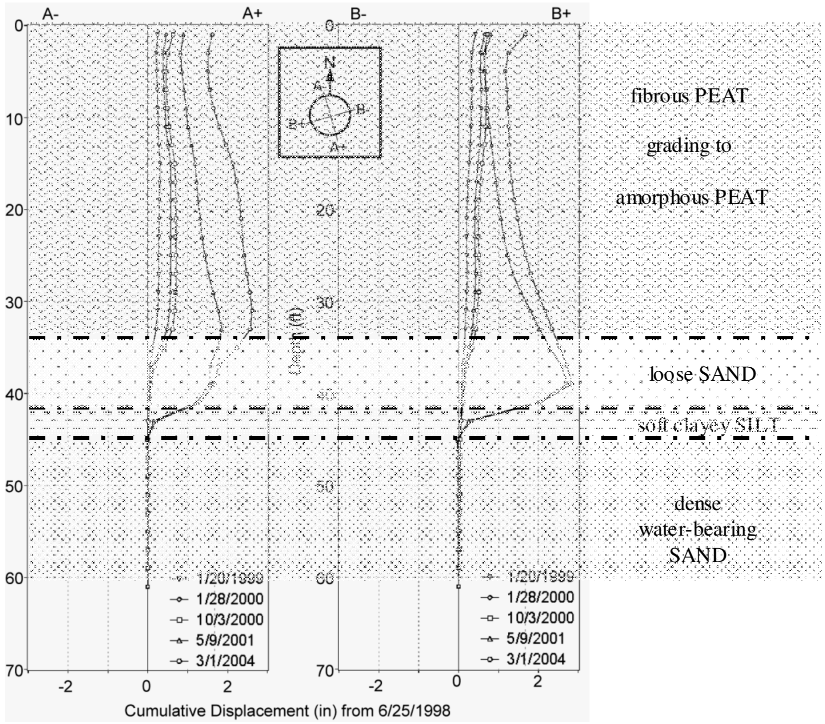

Figure 9. Inclinometer BA-13 plot of successive readings between initial on June 25, 1998 and March 1, 2004. Orientation of the A- and B-axes is depicted in upper portion of figure; the actual direction of movement is toward the southwest. Vertical axis represents depth below the ground surface in feet; horizontal axis is displacement in inches. Soil units and associated thicknesses are depicted on right side of inclinometer plots. The large anomalous displacement about 10 feet below the contact that occurred between October 3, 2000 and May 9, 2001 is attributed to the magnitude 6.8 Nisqually earthquake on February 28, 2001.

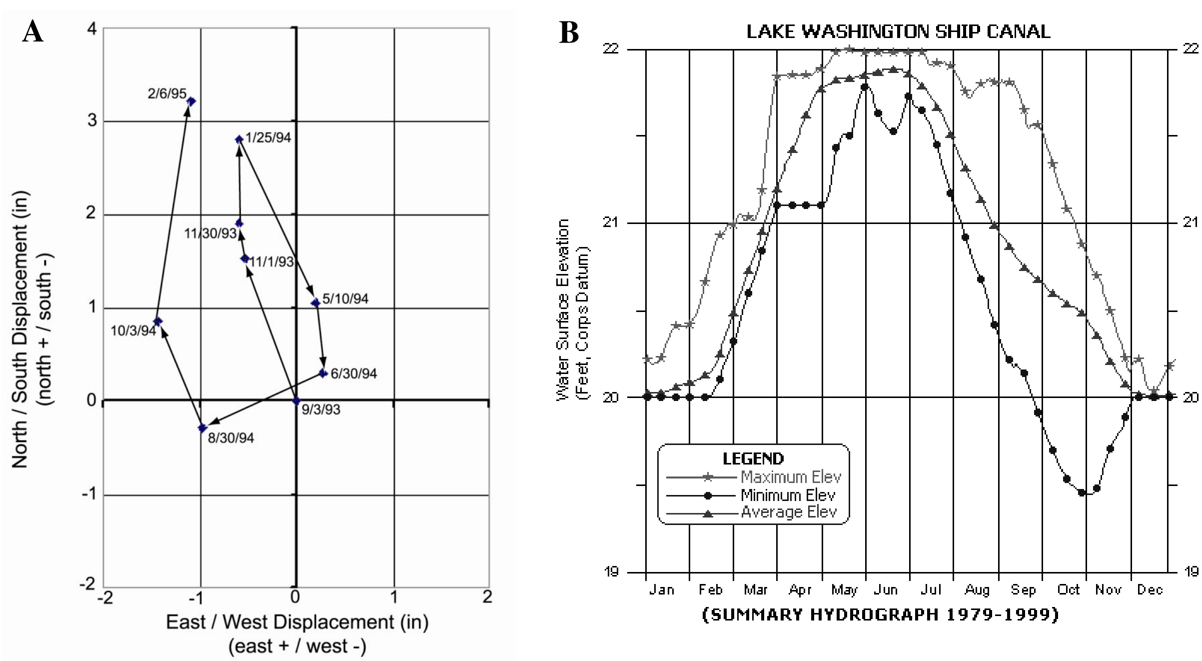

Figure 10. A) Displacement plot of inclinometer SI-1A at 4 foot depth (Converse Consultants NW, 1995). Initial reading taken on September 3, 1993 with subsequent readings through February 6, 1995. Expected direction of movement is northwest. B) Note movement coincides with lake drawdown and reversal occurs during filling.

Fortuitously, a closely spaced set of readings of inclinometer BA-13 between October 3, 2000 and May 9, 2001 recorded an anomalous accelerated displacement of comparable direction beneath the peat in a loose sand unit (Fig. 9). Similar anomalous displacement and direction was noted in an adjacent inclinometer BA-12. This time interval spans the magnitude 6.8 Nisqually earthquake event of February 28, 2001. Peak horizontal accelerations in the east-central portion of Lake Washington were estimated to be 0.1–0.2 g (Pacific Northwest Seismograph Network). Assuming that the anomalous displacement is seismogenic and the peat was rapidly loaded, it is especially noteworthy that the largest displacements were in the lower amorphous/decomposed peat and much less displacement occurred in the upper fibrous peat. It supports the laboratory testing (Kramer, 1996) that under more rapid loading fibrous peat is stiffer even under low confining stress than the deeper amorphous peat under higher confining stress.

One repeatedly expressed concern is the potential for consolidation of the peat as it flows laterally around the pile and shaft foundations. Consolidation, and thus strength gain, could result in increasing lateral loads on the foundations over time. To investigate this potential, cone penetrometer tests were performed on the upstream and downstream sides of a pile cap on the 90/43ECD ramp (RZA, 1989). The tests yielded nearly uniform bearing resistance and sleeve friction that appeared independent of depth and upstream/downstream location.

FINDINGS

Peat flow

The extensively documented lateral deformation of the peat can be classified as peat flow. As defined by Hungr et al. (2001) and references therein, "peat flow is a slow to very rapid flow-like movement of saturated peat, involving high pore-pressures". The authors note that excess pore pressure may not be required for flow due to the low weight of peat. Slow movement is defined by a velocity of around 1.6 m per year. With velocities ranging between 6 and 120 mm/yr, peat flow in Mercer Slough would be classified as very slow, as defined by Hungr et al. (2002). Examples in the literature of more rapid, catastrophic movement of saturated peat have been termed "bog slides", "bog flows", or "bog bursts" (e.g., Tomlinson and Gardiner, 1982). A review by Hungr and Evans (1985) found many peat flows involved failure within the underlying amorphous/decomposed peat, which later breached the more fibrous cover. Apparently, deformation initiating within the more fibrous peat is less common. Triggers have included natural events like an intense period of precipitation, and artificial causes such as embankment construction. The latter has been reported by Lucas and MacLain (1967) involving levee failures on the Sacramento River delta, California on relatively flat slopes where movements in the underlying peat extended more than 1000 feet beyond the levee. Little to no evidence of heave or other deformation common to landslide accumulation zones was observed in the toe areas.

Extent and style of deformation

Since the most notable area of ground and structure deformations was on the east side of the slough along Lake Washington Boulevard, geotechnical investigations to date have focused almost exclusively on this area. As a result, the extent of deformation of what would be considered the depletion zone of this unusual type of landslide is fairly well defined within the interstate corridor. The headscarp extends approximately to where the peat pinches out beneath Lake Washington Boulevard. Airphotos dating back to 1936 that document extensive pavement damage a hundred or more meters beyond both sides of the interstate bridges suggest a wider zone of peat deformation than what has been investigated to date. The potential width of the headscarp area may thus approach 1200 feet. Consolidation of the peat with no associated lateral displacement could also explain this lengthy section of pavement deformation.

Inclinometers located near Lake Washington Boulevard indicate that the peat is deforming through its full thickness (40 to 50 feet), often with the greatest displacements occurring in the upper 15 feet. Both discrete shear zones near the basal contact of the peat and thick zones of creep-type movement in the lower portion of the peat deposit have been observed. In the vicinity of Lake Washington Boulevard on both sides of the interstate bridges, the peat is curiously flowing both toward the interstate bridges as well as westward (Fig. 8).

Although some of the previous studies speculated about the distal extent of the peat deformation, its extent has not been conclusively determined. While artesian flow around and within the casings may have contributed to the somewhat confusing movement recorded in inclinometers BA-10 and 11, increasing displacement trends and a characteristic westward displacement profile in BA-10 strongly support the occurrence of peat flow in the central-western, as well as the eastern, portion of the slough. The observed structure deflections in the central and western portion of the slough may be related to peat flow in this area, or they could originate in the eastern portion of the slough and transmitted westward through the superstructures. Airphotos from 1936 to 2004 indicate significant dredging along the Lake Washington shoreline and within the lower Mercer Slough channel. Noteworthy is that the channel width is relatively uniform along the length of the channel (Fig. 11), with the exception of a constricted reach just north of the interstate. This constriction, as well as other unexplained changes to the Lake Washington shoreline south of the outlet, suggests a distal limit of the peat flow. Unfortunately, the scale of the aerial photography, dense shoreline/channel vegetation, and ongoing dredging complicate detailed comparison of airphotos. For example, assuming a deformation rate of 1 to 2 inches per year uniformly distributed across the entire slough, only about 5 to 15 feet of potential movement would have occurred between the time Mercer Slough was exposed and the channel first dredged around 1920. Presuming that Mercer Slough channel is the distal extent of the peat flow, the distance between the headscarp around Lake Washington Boulevard and the channel is about 2000 feet, defining a potential area of deformation approaching 60 acres. If the thickness of the deforming peat averages 30 feet, the volume of flowing peat would exceed 3 million cubic yards.

Some previous investigations surmised that the fills along Lake Washington Boulevard were significant, if not primary drivers for the peat deformation. If this were true, deformation of a low strength, fluid-like material should initiate near the fills and propagate westward. Shannon & Wilson (1975) observed the opposite behavior. Inclinometers farther out in the slough were displaced months before those located near/within the Lake Washington Boulevard fill. Furthermore, measured displacements out in the slough appear to show higher rates of movement than those closer to the slough margin. While this phenomenon could be explained by rheological changes throughout the deforming peat, an occurrence common to many landslides, a retrogressive style of deformation would produce a similar pattern in displacement rates. In other words, peat flow appears to be initiating in the west, possibly in the channel or lake shoreline, and propagating eastward in a retrogressive style. This retrogressive style would be expected given the observed drawdown effect of the lake on the peat movement.

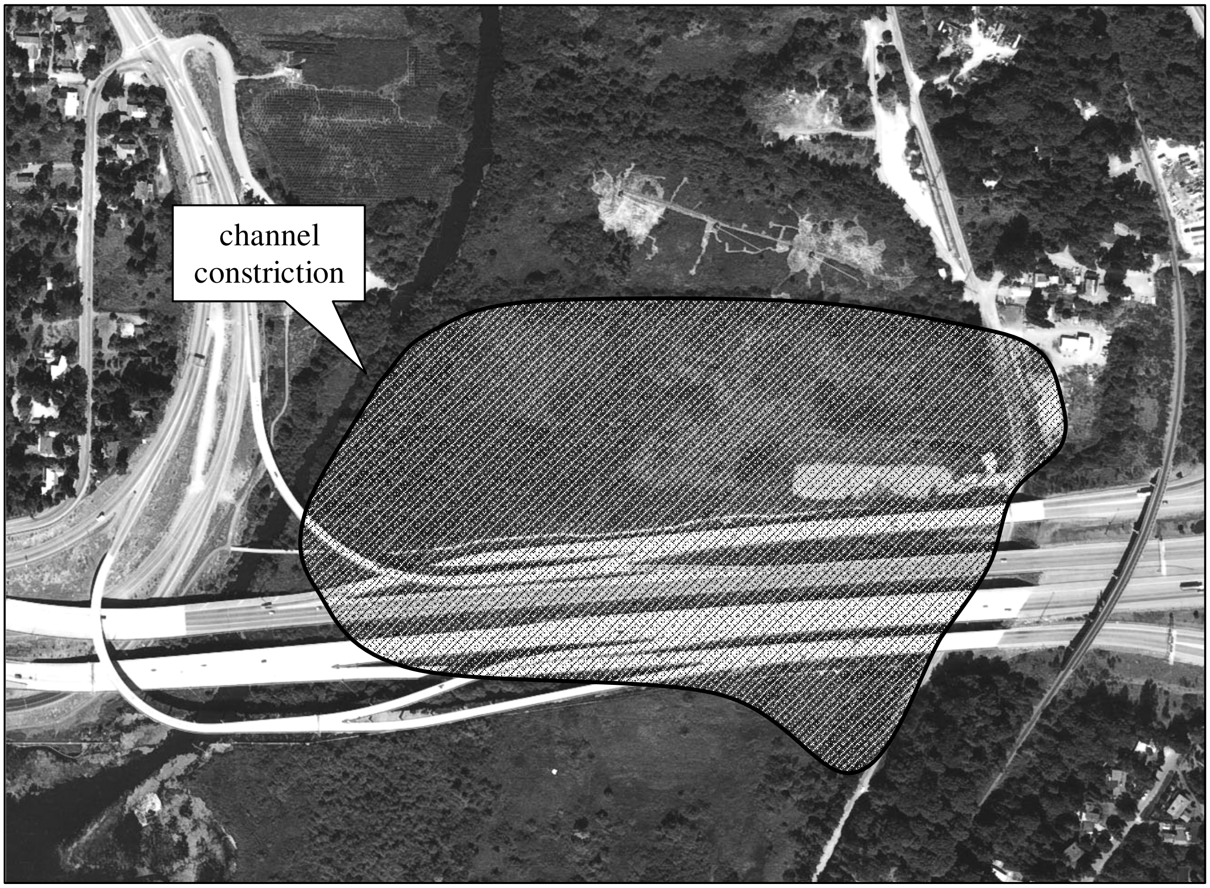

Figure 11. WSDOT airphoto (dated August 25, 1987) shows channel constriction about 600 feet in length located just north of interstate bridges. Hachured area depicts possible limits of peat flow.

Causal mechanisms

A number of potential causes have been identified by investigators, some assigning different levels of importance to their contributory effect of the peat deformation in the eastern portion of the slough. These include:

- Presence of the peat and its overall poor engineering properties (e.g., low strength, high compressibility, low unit weight/buoyancy, low permeability; rate-dependent behavior);
- Presence of an underlying sand unit with hydrostatic pressure transmitted beneath and within the peat;
- Fall-winter lake drawdown corresponding with seasonally elevated hydrostatic pressure in the underlying sand unit; poor drainage characteristics of the peat would result in hydraulic gradients and seepage (tensional) forces within the very low strength peat; and,

- Surcharge load of the fill along and adjacent to Lake Washington Boulevard on the underlying peat.

Shannon & Wilson (1975) also noted that excavation of an outfall channel adjacent to the waterline corresponded with accelerated deflections. Excavation of a large diameter drilled shaft for the 90/43W-W ramp in the early 1990s resulted in similar accelerated peat flow toward the shaft that persisted for several years. Previous geotechnical investigations do not mention the extensive dredging of the lower Mercer Slough channel and its potential destabilizing effect of the peat.

The flow direction of the peat, particularly in the eastern portion of the slough, converging toward the pile-supported bridge suggests that bridge foundations may also be contributing to the peat deformation. Artesian flow has been noted around many of the inclinometers that terminate in the dense sand unit. Since all of the piles are founded within this same unit, similar upwardly and horizontally directed groundwater flow (presumably toward Lake Washington) may be occurring around the numerous piles. Such concentrated groundwater flow might influence the observed flow toward and, possibly, along the interstate centerline.

Additional subsurface characterization is needed in the central-western portion of the slough to prioritize the relative importance of these destabilizing effects on the peat. The priority of one, however, can probably be moved to the bottom of the list. In 1992, a roughly hundred meter long tangent pile was constructed along Lake Washington Boulevard near the waterline (Fig. 3), in part, to isolate the surcharge load of the fill. Peat deformation has continued, and no determinable reduction in displacement rates can be directly attributed to the wall. If peat deformation is occurring in a retrogressive style, then the fill settlement would be only a sympathetic, secondary effect.

Seismic loads are also a suspected mechanism for accelerated deformation of the peat, as observed in inclinometer BA-13 (Fig. 8). Given the large anomalous displacement observed in BA-13 is the likely result of the moderately deep and distal Nisqually earthquake, a proximal shallow source such as the Seattle Fault with much larger predicted ground motions (Weaver, 2005) would likely result in even larger displacements within the peat.

Lateral loads on foundations

Another poorly understood but critically important issue is the effect the peat flow has on the bridge and waterline foundations. Bridge inspections have found that sections of the superstructures have deflected in an erratic manner. After conservatively accounting for temperature-related deflections in the bridge decks, the remaining and damaging deflections can only be attributed to lateral loads placed on and the resultant deflection of the pile foundations by the flowing peat.

To a large extent, the peat appears to flow around the bridge foundations. This deduction is based on several observations. First is the fact that tens of inches of displacement has occurred within the eastern portion of the peat over the lifespan of most of the bridges; superstructure deflections typically have been much less. Credence is also provided by the lack of apparent peat consolidation on the upstream side of a pile foundation determined from a pair of cone penetrometer tests; more tests are probably warranted. Further, inclinometer BA-13, located within 10 feet and on the downstream side of a large diameter drilled shaft, detected comparable directional but somewhat reduced lateral displacements as an adjacent inclinometer fully exposed to the flowing peat. The stress-relaxation and creep behavior as well as the overall low strength appear to be the primary constitutive controls on the peat's ability to effectively flow around the foundations. Based on the displacement path observed in inclinometer BA-13, lake drawdown and elevated hydrostatic pressures in the underlying sand unit suggest that the highest lateral loads would occur during fall and winter (Fig. 10).

While it is assumed that the peat provides little to no effective passive earth pressure, the superstructure appears to demonstrate some stiffness in resisting the lateral loads. Disregarding the recent seismic retrofitting of bridges, the long superstructures likely provide more rigidity to lateral loading of the foundation in the longitudinal direction than in the transverse direction. Note that the largest superstructure deflections have occurred within the waterline, which has very little transverse stiffness. Here, the dominant component of peat flow is nearly perpendicular to the waterline. The largest deflections in the bridge structures have occurred on the eastern end of the outermost bridges, 90/43ECD and 90/43WCD. Here, the dominant directional component of peat flow is perpendicular to the structures. Much reduced deflections have occurred in the interior mainline structures, where the peat flow appears to converge and is directed more westerly along interstate centerline.

Some apparently conflicting observations exist concerning peat strength over varying levels of strain. In the upper, more fibrous peat, stiffness was found to increase with higher strain levels presumably by mobilizing the tensile strength of the fibers (S. Kramer, 2005, personal communication). This interpretation seems to be supported by lesser displacements in the fibrous peat, which likely occurred as a result of the Nisqually earthquake, as compared to the greater displacements in the lower amorphous peat (Fig. 9). The spatial variation in displacement rates of the peat and the corresponding structure deflection is also confusing. Based on the inclinometers, the peat within Mercer Slough is deforming at very to extremely low displacement rates of around 5 to 120 mm/yr. These displacement rates are lowest near the eastern margin of the slough and appear to rapidly increase toward the west. Yet, the largest deflections in the structures are occurring where the displacement rates are lowest, which corresponds with lower strength. Conversely, where the peat appears to be more rapidly deforming (i.e., a higher strain rate), which presumably corresponds with higher strength (S. Kramer, personal communication), relatively little structure deflection has been observed.

CONCLUSIONS AND RECOMMENDATIONS

Over the last four decades, ongoing deformation of a thick peat deposit within Mercer Slough has resulted in damaging deflections, and near collapse in three cases, of pile-supported interstate bridges and a waterline that cross the slough. Several geotechnical investigations have been undertaken to characterize the extent and nature of the deformation, and to develop remedial options to safeguard these critical facilities. These investigations have focused on the east end of the structures, where structure deflections have been most noticeable. Recent compilation of periodic bridge inspection reports and analysis of expansion joint measurements have revealed structure deflections in the central and western portion of the slough. Reevaluation of previous geotechnical studies suggests that the area of deforming peat appears very much larger than what has been previously considered. Additionally, there are strong indications that the peat flow is initiating in the west and retrogressing eastward, as opposed to being driven from subsurface conditions and surcharge loads in the east.

Potential casual mechanisms for the deforming peat include:

- Presence of the peat and its overall poor engineering properties (e.g., low strength, high compressibility, low unit weight/buoyancy, low permeability, rate-dependent behavior);
- Presence of an underlying sand unit with high hydrostatic pressure transmitted beneath and within the peat;
- Fall-winter lake drawdown corresponding with seasonally elevated hydrostatic pressure in the underlying sand unit; poor drainage characteristics of the peat would result in hydraulic gradients and seepage (tensional) forces within the very low strength peat;
- Dredging in the western and southern portion of the slough; and,
- Extensive puncturing of the underlying pressurized aquifer by the numerous piles.

If the peat flow is initiating in a retrogressive style, the surcharge load of the fill along Lake Washington Boulevard on the peat is probably not a significant contributor.

To a large extent, the peat is flowing around the pile foundations. Yet, excessive lateral loads are being imparted to the foundations with mostly unpredictable spatial and temporal effect on the structures. Heterogeneity, creep, and stress-relaxation of the peat are the likely influencing properties for the largely unpredictable effects on these critical facilities.

Based on our review of the available information, three outstanding and critically important questions have arisen that need further investigation to better evaluate the risks to these structures.

1. What are the limits and timing of the peat deformation, and how and when are the structures responding to this deformation?

2. What are the associated lateral loads from the flowing peat?

3. Is accelerated flow of the peat induced by a nearby earthquake, such as from the Seattle fault zone, an additional or even dominant seismic hazard to the structures?

As a cover to this report, we have attached a proposed scope of work/cost estimate for a Phase 2 geotechnical study to attempt to answer these questions and better quantify the risks to these bridges.

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Appendix A-3
Interim Report for the Sound Transit Board

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Interim Report for the Sound Transit Board

I-90 South Bellevue Interchange Structure and Soil Monitoring Program July 12, 2010

Background

The Mercer Slough/South Bellevue Interchange consists of nine bridges, two of which carry I-90 mainline traffic, with the remaining seven serving as ramps or collector-distributors connecting I-90 to South Bellevue Way and I-405. See Figure 1. These bridges are almost exclusively pile-founded, though the 90/43W-W structure has two drilled shafts near the west end. The two I-90 mainline structures carry approximately 120,000 vehicles per day, with the other structures ranging from approximately 26,000 to 1,600 vehicles per day. See Figure 2.

Records maintained by the Bridge Preservation Office show a history of excessive and unusual structural movement. In December 1983 a span of 90/43ECD nearly lost bearing support because one span moved significantly relative to the adjacent span that supported it, reducing the bearing seat width to less than ½ inch. Load restrictions were imposed until a temporary repair was put in place in 1984. In September 1987 a similar but less severe loss in bearing seat width was found on 90/43WCD, and again, a temporary repair was put in place shortly after discovery of the problem. Between November 1992 and May 1994 significant movement was detected on 90/43WCD, possibly caused by foundation work for the 90/43W-W structure, though no repair was initiated in response to these findings. In November 2003 Bridge 90/43W-W showed signs of excessive movement at bearings. In December 2003, bridge 90/43S was found with loss of bearing seat width at two joints, and in January 2004 the seismic restrainers were checked and calibrated to ensure a minimum of 1 inch bearing width.

Several studies have been done over the years. The earliest documented soil measurements were taken in the early 1970's and findings documented in a 1975 report by Shannon & Wilson to the Seattle Water Department. This study evaluates soil movement affecting the waterline foundation running along the south side of the I-90 corridor. There have been several other studies and less formal evaluations of bridge and soil movement since then, performed by various WSDOT offices. With the discovery of new significant structural movement in 90/43S in December 2003, and the recognition that temporary repairs were still in place, BPO commissioned the WSDOT Geotechnical Division to prepare a Phase 1 study focused on reviewing existing data in the record and developing a plan for a Phase 2 study. This Phase 1 report was completed in August 2005, and the recommended Phase 2 study was initiated in 2006 with coordinated data collection between structural and soil monitoring. Soil measurements started in November 2006 and structural measurements started in May 2007. This study is on-going as of June 2010, and has led to the findings described below. See Appendix 1 for a timeline of events through June 2010.

Geology

Mercer Slough is a ½-mile-wide, north/south-trending embayment on the east side of Lake Washington. The slough is filled with 40 to 60 feet of peat, which grades from relatively fresh and fibrous at the surface to completely decayed and amorphous at depth. On the margins of the slough, the peat rests directly on sand and glacially over-consolidated sand and gravel. In the center of the slough, up to 60 feet of soft clay is sandwiched between the peat and the sand. While the peat and clay are fully saturated due to their association with the lake, a confined and pressurized aquifer exists in the underlying sand, which produces artesian flow in the numerous borings that have been drilled across the slough. Over the decades, fill has been placed on the peat along the eastern margin for the construction of Lake Washington Boulevard and various commercial developments, and as a result, extensive settlement and lateral deformation has occurred and continues to occur within this fill.

The various geotechnical studies of Mercer Slough geology at the I-90 corridor have consistently attributed structure deflections to widespread movement of the peat. The Phase 1 Geotechnical Study by WSDOT Geotechnical Division compiled available information and identified a number of outstanding questions for a Phase 2 study to better assess the nature/extent/timing of the peat and structure movements, attempt to estimate lateral loads on the pile foundations, and assess their seismic vulnerabilities. In 2006 the Phase 2 study was initiated, and the Geotechnical Division installed and has since been monitoring six slope inclinometers and the deflections and loads on two 12 inch "test piles". See Figure 3. The following findings summarize our current understanding of the geotechnical issues influencing the structures:

- Lake level is controlled at the locks by the ACOE and involves an annual 2-ft cycle of drawdown July through November and filling February through April. This drawdown-filling cycle induces the peat to move generally westward/lakeward during drawdown, and the peat reverses direction or stalls during filling. Inclinometers in the middle of slough typically move up to two inches annually, while inclinometers on the margin move ½" to ¾" annually. While much of the peat movement is recovered during filling, permanent peat deformation is occurring at varying rates and locations.

- Other geologic/geotechnical factors are also influencing cyclic and permanent deformation of the peat, including the adjacent fill along Lake Washington Boulevard, the pressurized aquifer beneath the peat, and the poor engineering properties of the peat.

- While the peat is mostly flowing around the pile foundations through a complex stress-relaxation, time-dependent behavior, significant lateral loads are still being imparted to the foundations sufficient to cause comparable movements in the superstructures. In addition, the test pile closest to Lake Washington Boulevard is experiencing progressive bending at an annualized rate of ~0.1 inches/yr since it was installed in February 2007.

Extrapolation of this rate to the 40 to 70-year-ages of the structures could equate to as much as 4 to 7 inches of bending.

- The structures are also seismically at risk from soil loading. An inclinometer monitored between October 2000 and May 2001 detected an anomalous ~3 inch pulse of permanent movement within a liquefiable sand zone beneath the peat, representing what we believe to be a lateral spread/flow failure associated with the 2001 Nisqually earthquake. Concurrent with such rapid (i.e., seismic) loading events, we believe that the upper fibrous peat develops greater shear strength, making it less prone to yield (flow) around the foundations, and, thus, imparts additional lateral loads to the foundations. An additional seismic hazard is the location of the structures within the rupture zone of the Seattle Fault.

Bridge Movement

As part of the on-going Phase 2 study, 20 GPS-based monitors were installed at various locations on several bridges, providing structural movements to approximately 1/10 inch accuracy recorded daily, with data downloaded to a website. See Figure 3. With data from May 1, 2007 through June 30, 2010, the following was observed:

- A few monitored areas show only expected longitudinal movement associated with seasonal thermal expansion/contraction, but the majority of the monitors show a pattern of movement that is either transverse or diagonal to the roadway centerline. This "non-thermal" movement follows an annual cycle generally consistent with the soil movements documented above, which in turn are consistent with seasonal changes to the lake levels. Structural movement in 2007 and 2008 indicated that there was some permanent deformation in a few locations, but mostly the structures returned to their original locations annually.

- In August 2009, consistent with starting the Lake Washington drawdown, most structural monitors broke the cyclic movements recorded in 2007 and 2008, and showed a definite and significant movement toward the west or northwest, which continues to the present. This relatively sudden change in cyclic movement appears to be reflected in the soil measurements as well, though they do not consistently move in the same west or northwest direction and the correlation between the soil and structure movement will take more evaluation.

- Several monitors clearly indicate that compression and tension linkages are created across deck joints at certain times of the year. A compression linkage occurs when a deck joint completely closes and a tension linkage occurs when the seismic restrainers are engaged.

- One monitor indicates there is approximately ¾ inch of vertical movement occurring on an annual cyclic pattern. This finding was initially difficult to accept, and every effort was made to validate the accuracy of the data. A preliminary judgment is that the data is valid, but the cause of this apparent movement has not yet been determined.

- Regrading and new fill added to the west side of Lake Washington Boulevard in February and March 2009 can be associated with movements detected in the bridges near this work. Prior studies have indicated that the Lake Washington Boulevard fill may be causing some peat flow, and this fill has settled and additional fill has been placed over the years.

Unresolved Questions

The on-going Phase 2 study has largely confirmed the findings from previous studies, and has added significantly to the understanding of the nature and extent of the soil and structure movement, particularly the close association with Lake Washington water levels. However, many questions remain that will not be answered by the current study underway:

- Very little is known about how the soil and structures will respond to an earthquake. A report completed in February 1994 indicates that the westbound mainline structure (90/43N) built around 1940 is highly sensitive to seismic forces, but there is no study of the newer structures that are the primary focus of the current study.

- An accurate assessment of the magnitude of the forces applied to the foundation by lateral movement of the peat still eludes us. The Phase 2 study provides a basis for making very approximate estimates only, and only for pile founded structures. There is virtually no information about how a drilled shaft will respond to these soil conditions.

- The possibility of permanent deformation in the pile foundations of the existing structures remains only a possibility, though a strong one. Permanent deformation of the superstructure (which is just now becoming evident as part of the Phase 2 study) strongly indicates that pile deformation is occurring, but the nature of that deformation remains totally unknown, including the total cumulative deformation, where it's occurring along the pile lengths, and if this deformation is compromising the structural integrity.

- While the causes of peat movement is now generally understood to be a combination of changes in lake level and fill on the edges of the slough, our current understanding isn't sufficient to understand how these causes interact. More importantly, we are not able to predict movement in the long term.

- There is some indication in the bridge records that construction of the 90/43W-W bridge in 1992-1994 caused structural movement of the 90/43WCD, but no clear understanding of the nature and extent of this movement has yet been found in our records. We have reason to assume that new construction adjacent to existing structures will cause structural movement, but cannot provide any significant information on how to quantify and mitigate this expected interaction.

Based on what is currently not known and won't be answered by the current Phase 2 study, we recommend careful evaluation of these unknowns, particularly for the expected lateral loading conditions for any future construction across Mercer Slough in the vicinity of the I-90 corridor.

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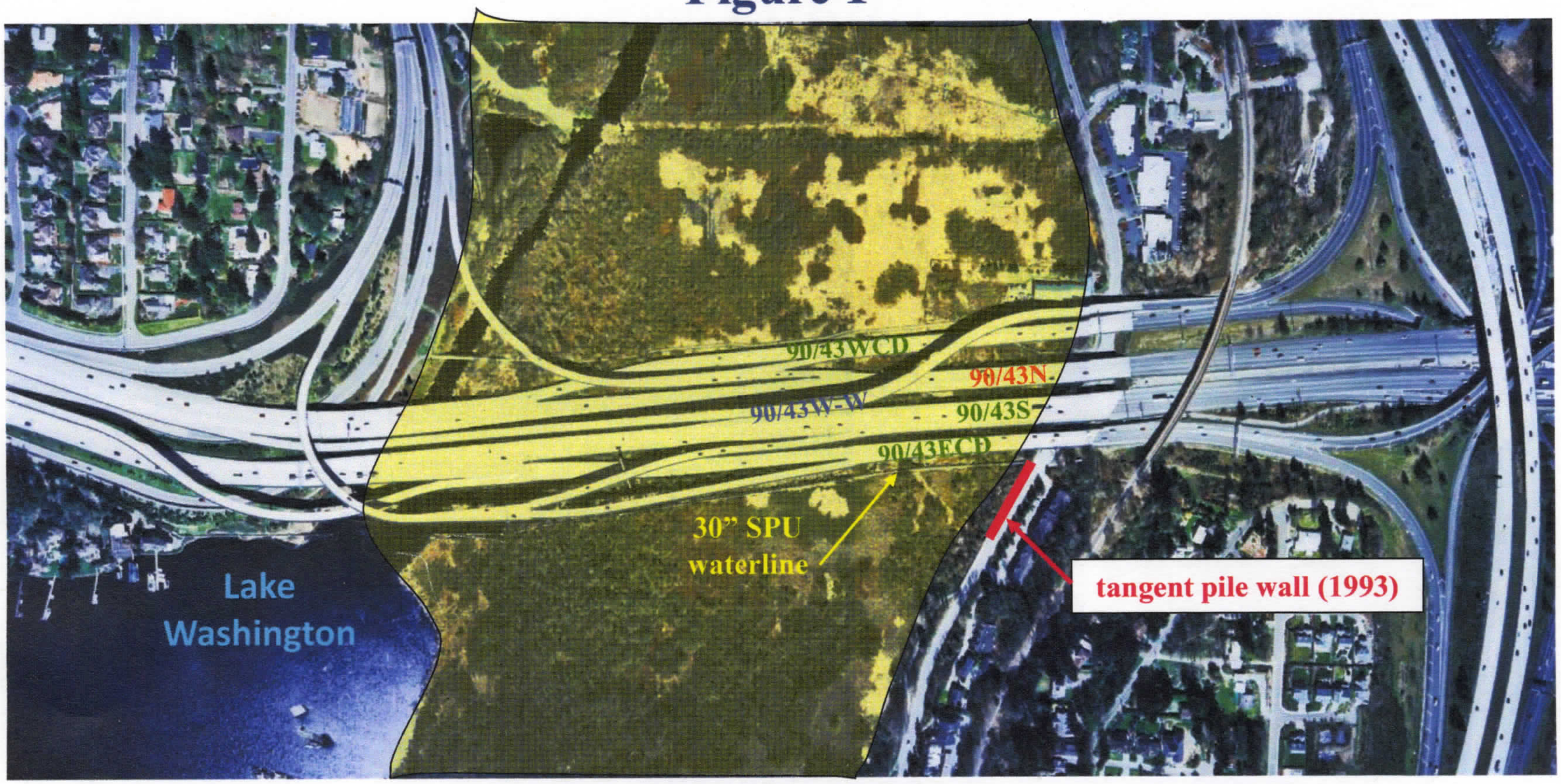
Mercer Slough

Figure 1

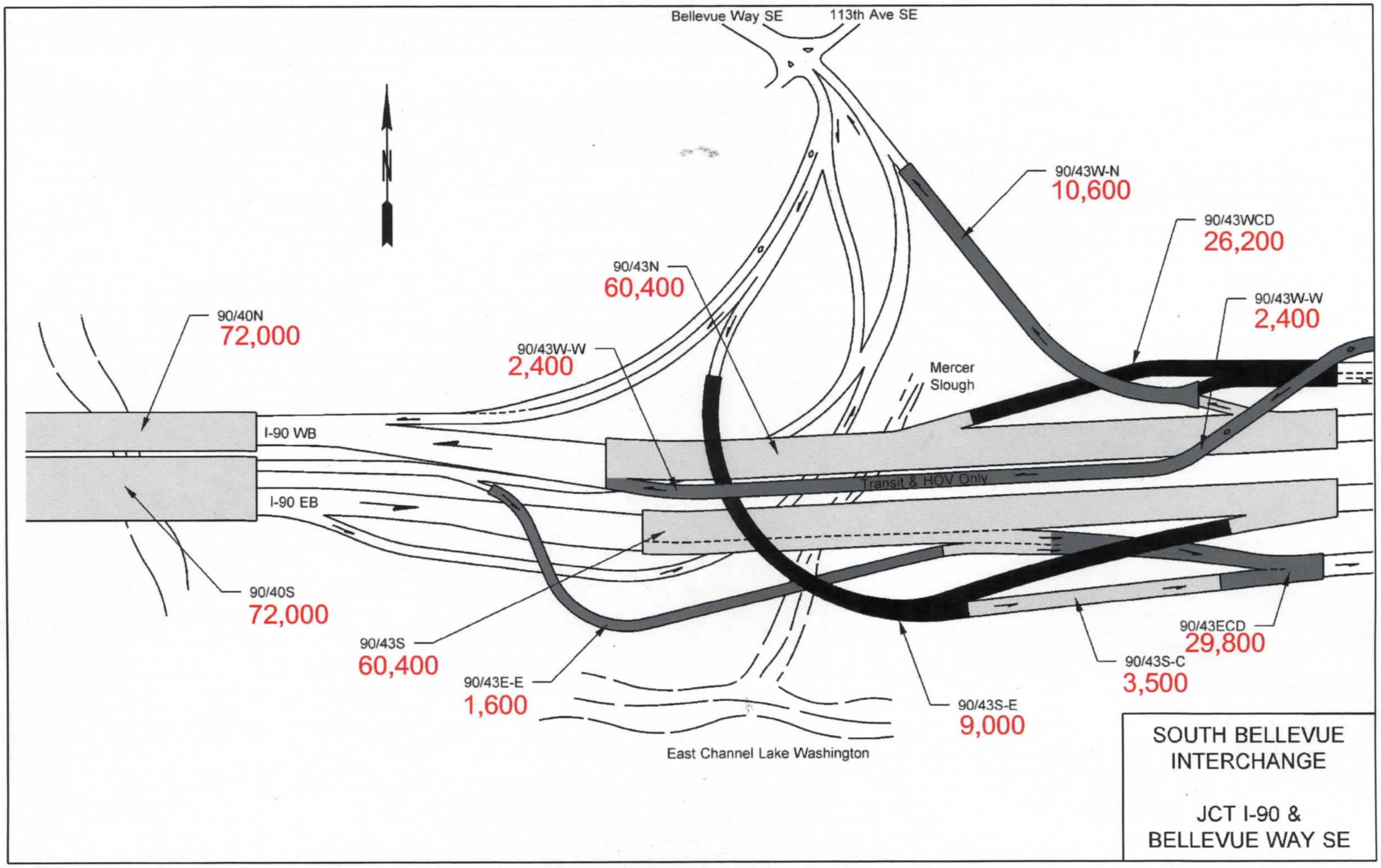


90/43N – 1940; timber piles; seismic retrofitted in 1991

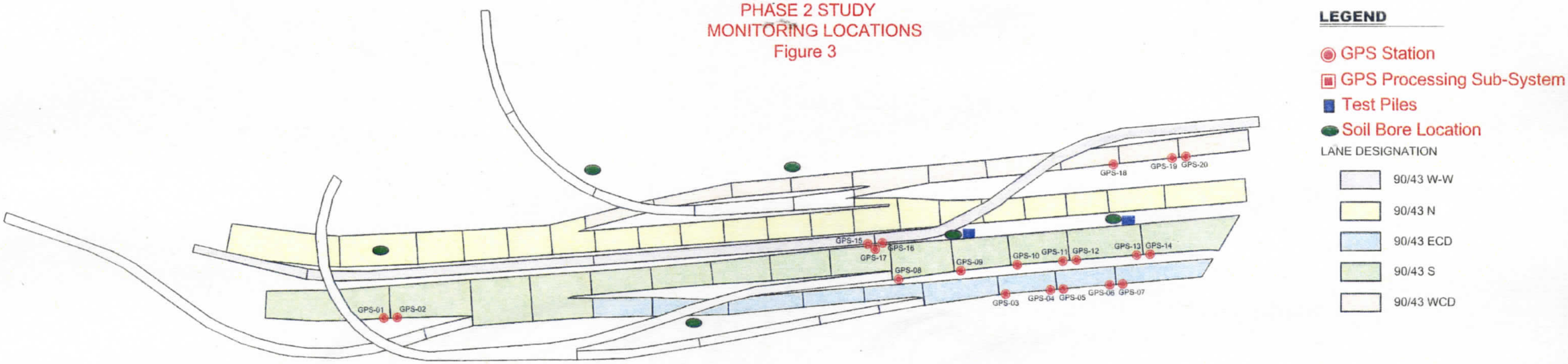
90/43S – 1970; steel pipe piles; seismic retrofitted in 1991-2

90/43W-W – 1993; drilled shafts

Figure 2
2007 Average Daily Traffic (ADT)



MERCER SLOUGH
PHASE 2 STUDY
MONITORING LOCATIONS
Figure 3



APPENDIX A

Mercer Slough/South Bellevue Interchange Timeline of Events Relating to Bridge Structural Movement December 1983 through June 2010

- December 1983 – The expansion joint #9 at Pier 35 of Bridge 90/43ECD opened to approximately 7.5 inches, resulting in all the bronze bearing plates falling out. Measurements taken at this time indicated ¼ to ⅜ inches bearing remaining at each of the seven brackets. The three adjacent joints to the west were closed tightly indicating that the bridge was translating to the west.

- January 1984 - Bridge 90/43ECD restricted to a maximum gross load of 80,000 lbs until further notice.

- January 1984 - District 1 maintenance forces jacked Bridge 90/43ECD back into position by pushing units 7 and 8 eastward, closing the joint 2". Timber blocking was placed at each end of units 7 and 8 to prevent the joints from closing.

- January 1984 - The load restriction on Bridge 90/43ECD was lifted.

- April 1984 – Bridge Condition requested assistance from District 1 in developing a monitoring program for determining the movement of Bridge 90/43ECD over the next two years using expansion joint measurements, tilt plates, and slope indicators.

- April 1984 - Materials lab learns that the City of Seattle has had a similar problem with their waterline adjacent to Bridge 90/43ECD.

- May 1984 - A copy of the Seattle monitoring program was received by the Bridge office (Located in the 90/43ECD letter file).

- June 1984 - District 1 maintenance crews removed timber blocking at Joint 7 and trimmed the blocking 1" at Joint 8 on Bridge 90/43ECD.

- October 1984 – Bridge Condition received a request from District 1 for a restrainer system at Joint 9 of Bridge 90/43ECD.

- October 1984 and November 1984 – Photo documentation of bearing seats and jacking operation at Bridge 90/43ECD. See Exhibits 1 and 2.

- November 1984 – Design of seismic restrainers for Bridge 90/43ECD started.

- November 1984 - Timber blocking at Joint 7 was replaced ½" shorter than original blocking at Bridge 90/43ECD.

- May 1985 – Intra-Departmental Communication (IDC) from Keith Dodge stating District 1 maintenance would install seismic restrainers and that I-90 design group would be responsible for monitoring the bridge stability and foundation materials for Bridge 90/43ECD.

- Summer 1985 - Earthquake restraint system installed at the East Joint of Bridge 90/43ECD.

- October 1985 – Summary of ground movement activity for Bridge 90/43ECD prepared by Materials Lab (Located in the 90/43ECD letter file). It was discovered that the City of Bellevue had recently removed pavement from part of Lake Washington Boulevard adjacent to the bridge and that they are leveling this area with gravel about every 3 to 4 months as the soil settles. Materials lab recommends that the district hire a geotechnical consultant to work with the structural engineer to determine a method to stabilize the existing bridge.

- September 1987 – Expansion joint #9 adjacent to Pier 36 on Bridge 90/43WCD opened to 5.5 inches.

- November 1987 – District 1 maintenance jacked Bridge 90/43WCD back into place and installed timber blocking in the expansion joints to keep them from closing.

- December 1987 – Plans for using cable ties to strap Piers 36 and 37 together across joint #9 on Bridge 90/43WCD issued. This repair characterized as temporary. Photo documentation of cable tie installation shown in Exhibit 3.

- February 1988 – Plans for installing leaded survey nails in the tops of the curbs for Bridges 90/43N, 90/43S, 90/43ECD, and 90/43WCD were issued. The nails were to be used for monitoring expansion joint movements.

- January 1989 – Report covering Bridge 90/43ECD and 90/43WCD movement issued by Rittenhouse-Zemen & Associates, Inc (Located in the letter file for Bridges 90/43ECD and 90/43WCD).

- May 1990 – Plans for bridge widening and seismic restrainer installation for Bridges 90/43N, 90/43S, and 90/43ECD issued.

- August 1991 – IDC issued stating that overextended rocker bearings exist at Bridge 90/43N. The IDC states that the rockers are mostly tipped toward the west.

- August 1992 – Plans for Seismic Restrainers for Bridge 90/43WCD issued.

- November 1992 to August 1993 – As much as 5 inches of movement detected in an inclinometer near Bridge 90/43WCD, possibly caused by foundation work for the 90/43W-W structure.

- August 1993 to May 1994 – An additional 2 to 3 inches of movement detected in an inclinometer near Bridge 90/43WCD.

- February 1994 – Seismic analysis of 90/43N

- February 1996 – New reference points (RP's) installed prior to this date on Bridge 90/43N to replace the leaded survey nails installed earlier for monitoring joint movements. The leaded survey nails were lost during a recent construction project that modified the joints, rail, drains, and overlay.

- May 1997 – Bridge 90/43W-W first noted with damage to concrete lateral restrainers at Pier 6 (labeled Pier 12 in plans and 5/97 through 10/01 inspection reports). See November 2003 below.

- July 1998 – Tilt plate monitoring starts at five different locations on Bridges 90/43N, 90/43S, 90/43ECD, and 90/43WCD.

- August 1998 – Expansion joint measurements taken about 3 axes (x, y, z) begins at Bridges 90/43N, 90/43S, 90/43ECD, and 90/43WCD.

- December 1999 – Date of last tilt plate measurement at Bridges 90/43N, 90/43S, 90/43ECD, and 90/43WCD.

- March 2001 – Date of last expansion joint measurement about 3 axes at Bridges 90/43N, 90/43S, 90/43ECD, and 90/43WCD.

- November 2003 – 90/43W-W at Pier 6 and East Abutment found offset of superstructure relative to substructure at Pier 6 bearings and East Abutment bearings. This movement indicates that the substructure is shifting south relative to the superstructure. At Pier 6, this relative movement is estimated at 1 ¾ inches. See Exhibit 14.

- December 2003 – Inspection of 90/43S finds enlarged openings at Joint 2 near Pier 5 and Joint 14 near Pier 55, with Joint 2 showing a wider opening on the north end when compared with the south end. Sliding bearings under joints also have compromised bearing seat width. See Exhibit 4.

- February 2004 - Seismic restrainers tightened on 90/43S at Joints 2 and 14 to maintain 1 inch bearing. Repair No 15031. One bearing plate fallen out at Joint 14 and a second near falling out. See Exhibits 5, 6, and 7.

- October 2004 – Interim inspection of 90/43S. Second bearing plate fallen out at Joint 14, and exterior bearing seat widths at Joints 2 and 14 documented. See Exhibits 8 through 13.

- August 2005 – Phase 1 Geotechnical study completed by WSDOT Mats Lab, summarizing findings to date and detailing a proposal for a Phase 2 study.

- October 2006 – Leica Contract DP01200 initiated for structural monitoring at 20 locations on various bridges as part of Phase 2 study.

- November 2006 – Phase 2 geotechnical study started by HQ materials lab.

- March 2007 – Leica structural monitors installed on bridges.

- May 1, 2007 – Structural monitoring data calibrated and this date considered start of historical data.

- November 2008 – Interim inspection of 90/43S. Third bearing plate fallen out at Joint 14, and four additional bearing plates cracked or broken.

- March 2009 – Bike path along Lake Washington Boulevard under I-90 bridges is rebuilt with new fill and a rock blanket over embankment.

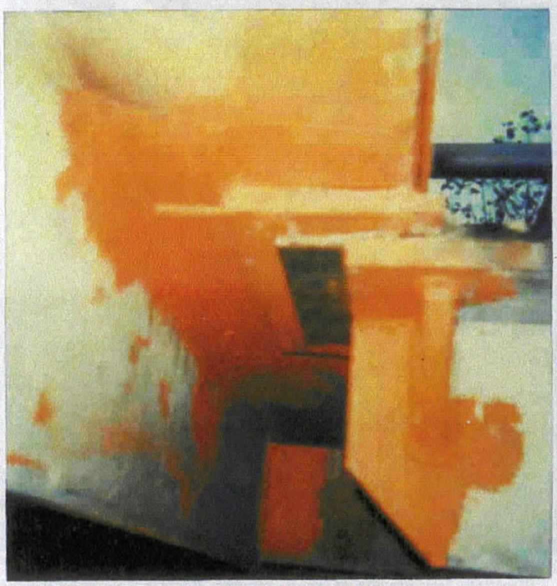
- September 4, 2009 – Structural monitoring reference station disturbed.

- September 25, 2009 – Structural monitoring reference station reset on solid foundation at slightly different location.

- May 2010 - Structural monitoring data from October 2009 recalibrated on data website to account for new reference station location.

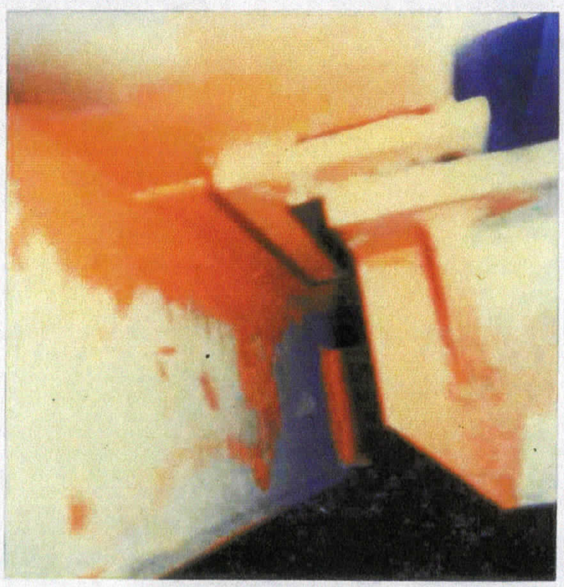
- June 2010 – Review of structural and phase 2 soil monitoring data evaluated with 3 years of data. Structural monitoring verified cyclic movement annually, with permanent displacement in some areas. Soil monitoring suggests possibility of permanent deformation of piles.

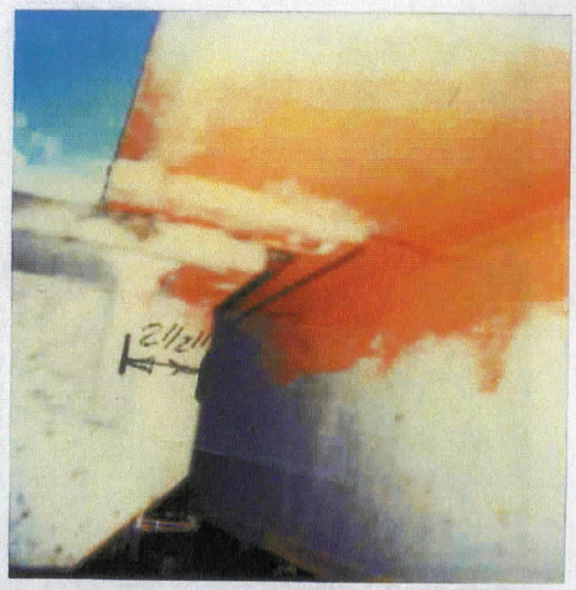
90/43 ECD Expansion Joint



Looking South at East Joint at Right Fog Line → Same

10-22-84





11-7-84

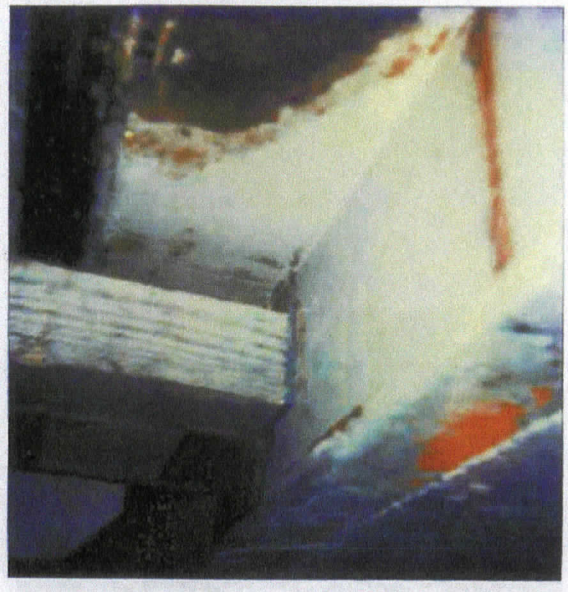
Looking South at East joint at Right Fogline

11-7-84

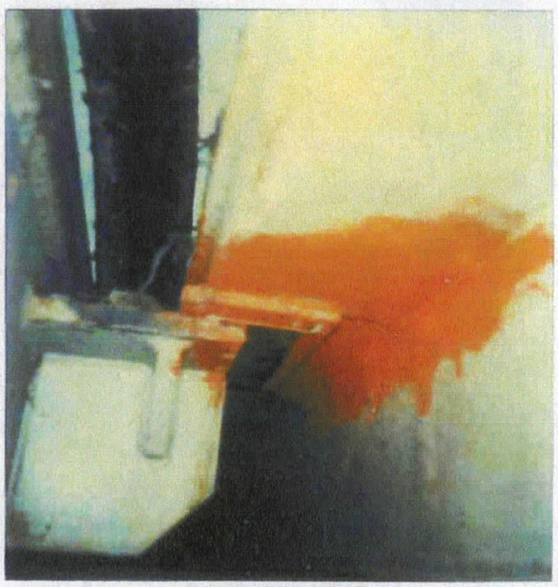
Looking North at East Joint at Right Fog Line

Exhibit 1

11-9-84
Looking North at 3rd Joint From East



11-9-84
Looking North at 3rd joint From East



10-22-84 Looking North at East Joint at right Fog Line

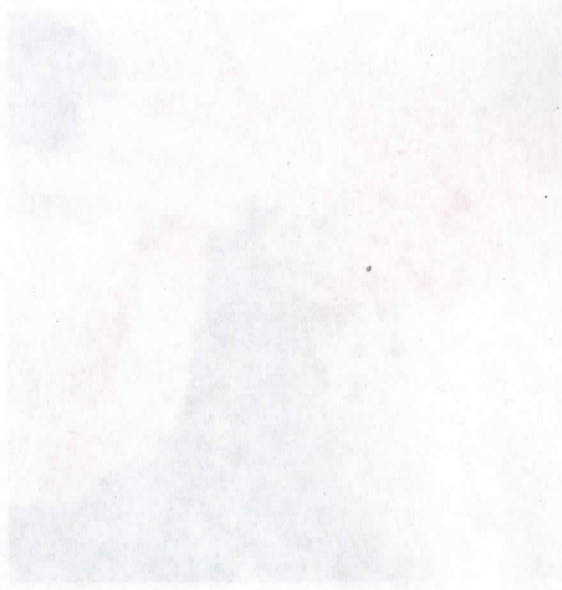
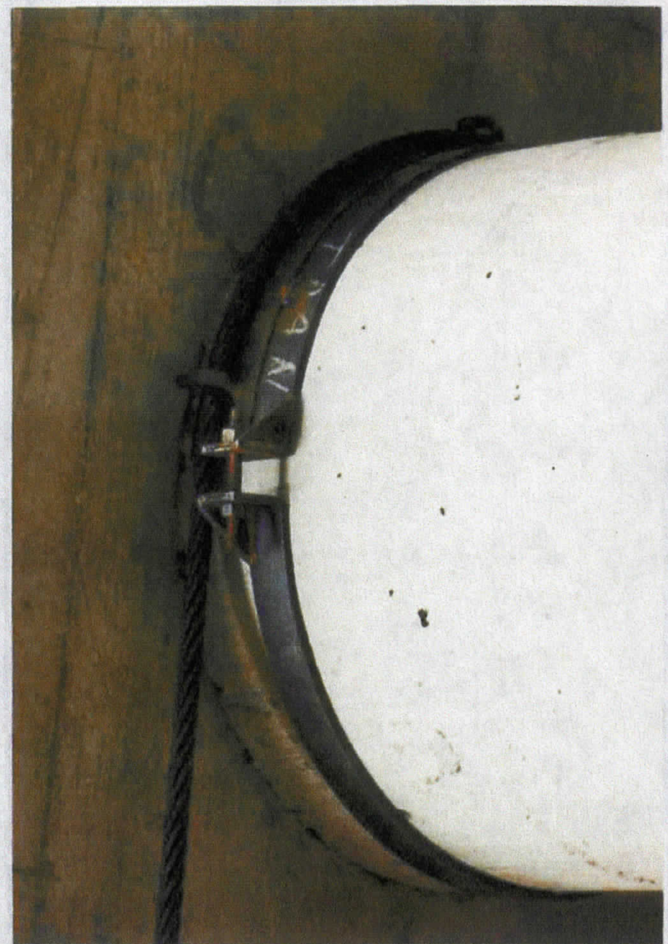


Exhibit 2



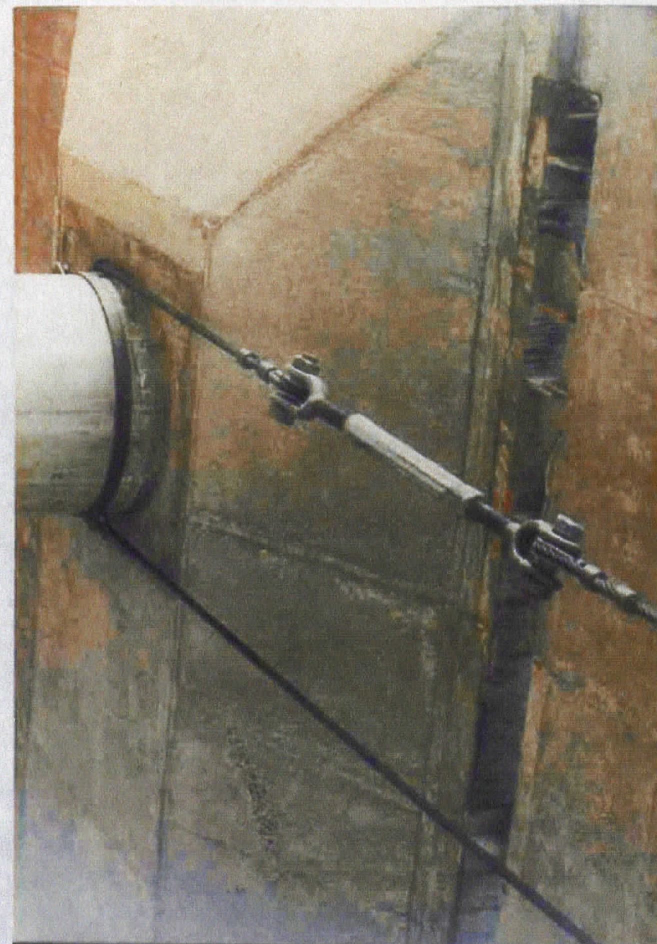


Exhibit 3

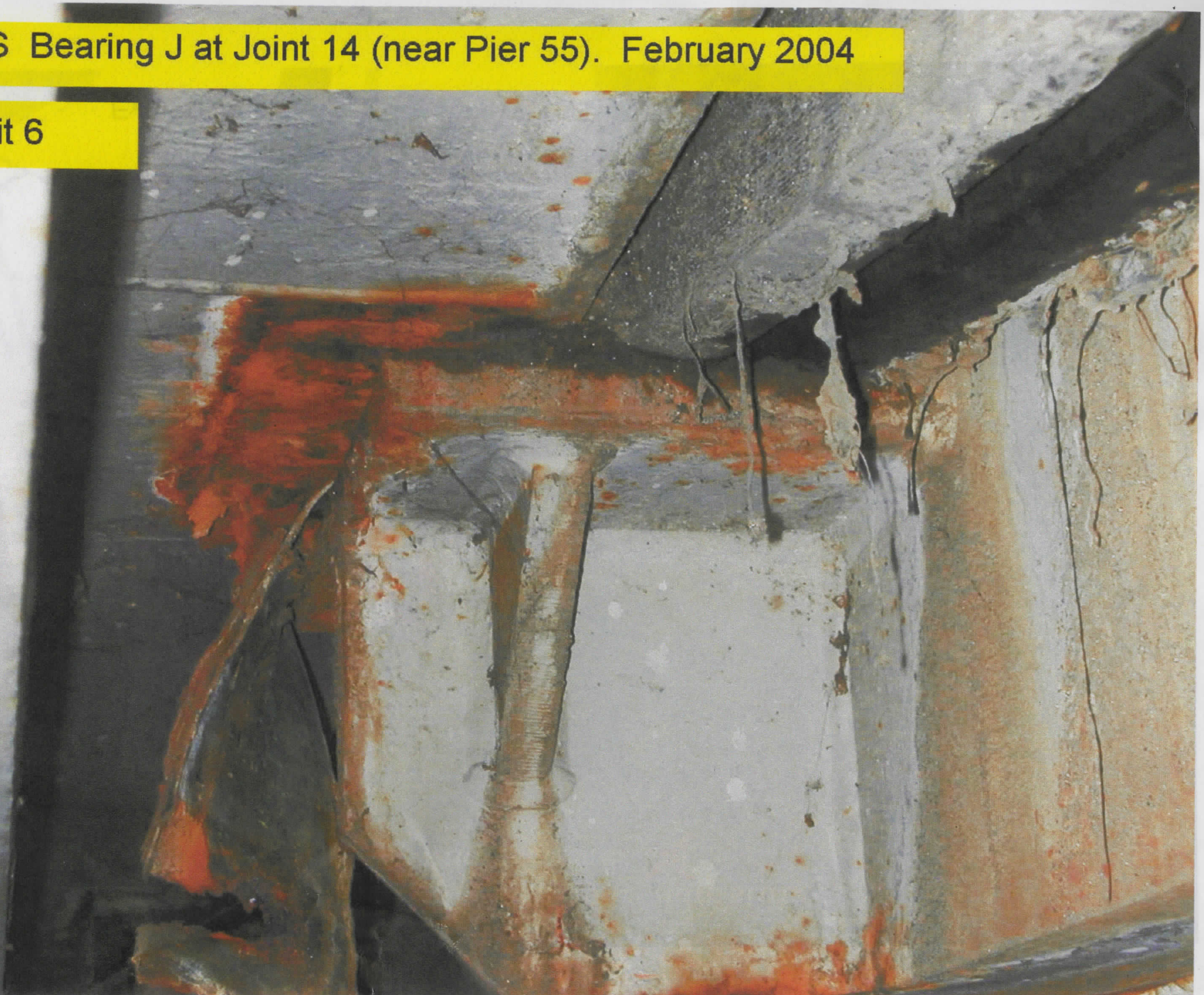
90/43S Bearing F at Joint 2 (near Pier 5) December 2003

Exhibit 4


Mike Tachell & Tony Messmer adjusting
earthquake restrainers @ pier ~~54~~ 55
90/43S
Mercer Slough
02 21 2004
Exhibit 5

90/43S Bearing J at Joint 14 (near Pier 55). February 2004

Exhibit 6



90/43S Bearing G at Joint 14 (near Pier 55). February 2004

Exhibit 7

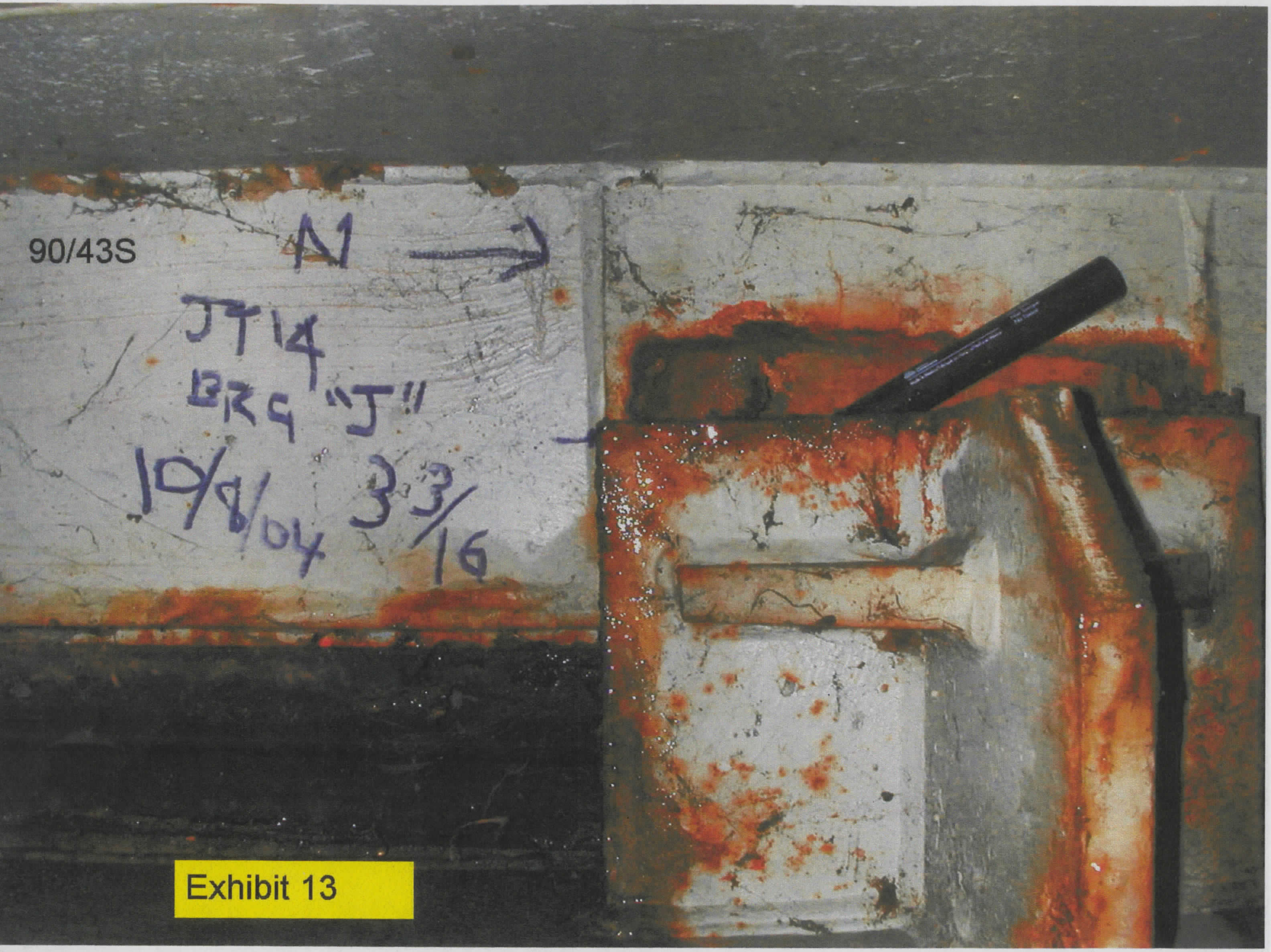


Exhibit 13