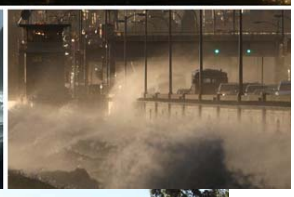
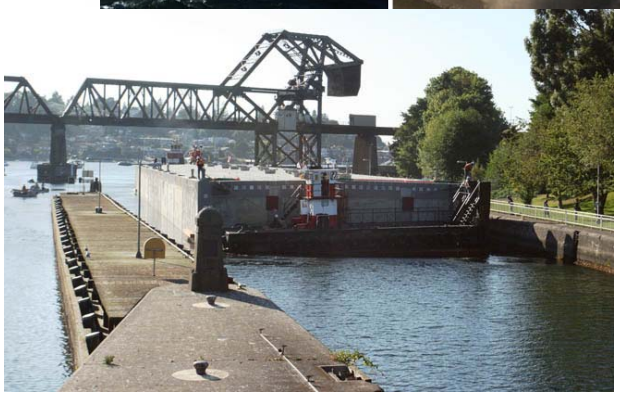
WSDOT SR520 PROGRAM
Pontoon Construction Project
Expert Panel Report, Phase 2

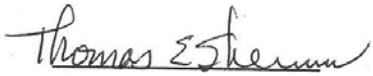


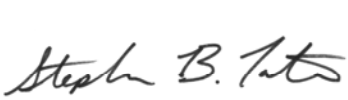


Submitted to WSDOT by the Expert Panel February 18th 2013

THE SR520 EXPERT REVIEW PANEL:

Chair: John Reilly, Panel: Dr. Neil Hawkins, Dr. John Clark





Mr. Tom Sherman, Mr. Mark Leonard Mr. Stephen Tatro

Panel Members' – Background (for details, See Appendix E)

The Expert Panel members were selected for their expertise in the design and construction of complex concrete and pre-stressed/post-tensioned[1] concrete structures. More than one member was required to have experience in floating bridges. An understanding of concrete materials and technologies was required.

Current Expert Panel members are:

Dr. Neil M. Hawkins, Ph.D., Dist. M. ASCE, Hon. M. ACI, Professor Emeritus, University of Illinois, and former chair of the Civil Engineering Department, University of Washington, specialist in reinforced and prestressed concrete.

Dr. John H. Clark, P.E., Ph. D., a consultant for the design and construction of long-span bridges and heavy structures, including pre-stressed concrete.

Mr. Tom Sherman, specialist in floating bridge design and construction, TES Enterprises

Mr. Mark Leonard, FHWA Resource Center, former Bridge Engineer for the Colorado Department of Transportation.

Mr. Stephen B. Tatro, consultant specializing in evaluation, testing, design, and construction of concrete structures and materials.

The panel is chaired by Mr. John Reilly, P.E., C.P. Eng., with experience in major project management, contracting and delivery, risk assessment / mitigation and prestressed concrete design.

Summary resumes of the Panel members can be found in Appendix E

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[1] Prestressing refers to beneficial stressing of the concrete structures before application of in-service loads. Post-tensioning is the process of applying that prestressing after the concrete has been placed and hardened.

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1. SUMMARY

The SR 520 floating bridge replacement program consists of several major construction projects. One of these is the Pontoon Construction Project (PCP), which includes the early construction, in Aberdeen, Washington, of 33 concrete pontoons which are needed to replace the existing bridge support structure.

On May 11th 2012, during post-tensioning (PT) of pontoon V, the first longitudinal pontoon to be post-tensioned, concrete spalling[2] was observed from forces created by post-tensioning tendons in the keel slab, adjacent to the bolt-beams at the end of the pontoon. WSDOT immediately designed and directed repair and strengthening of this area, for all longitudinal pontoons.

Subsequently, after repairs and re-application of the longitudinal PT, an unexpected level of cracking was observed over the depth and width of the end walls. WSDOT engaged this Expert Review Panel (ERP) to determine a) the most likely cause of the cracking, b) the need for, and character of, changes to the pontoon design to avoid similar cracking in future construction and c) to assist with measures which would allow the pontoons to meet their specified service life. The results of that initial work are reported in Appendix E.

WSDOT concurred with the initial ERP findings and began implementation of the changes, asking the ERP to review the on-going work and to report further on areas including a) design sufficiency b) quality of the pontoons, c) crack repair and minimization strategies and d) maintenance considerations. This report responds to that request.

KEY FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

1. Structural sufficiency/watertightness

After reviewing WSDOT's design criteria, loading and design calculations the panel found:

a. Loadings used for design were comprehensive and conservative in many respects – i.e. applied design loadings are more severe, in general, than will be experienced by the bridge in almost all cases.

b. The design criteria was designed to achieve long-term water-tightness by limiting crack widths, in accordance with the criteria used to design previous floating bridges in Washington State. This is appropriate and results in a significantly conservative design, in terms of available structural capacity.

c. The construction specification for epoxy injection of cracks of 0.006 inches or wider is appropriate. Lesser crack widths do not warrant epoxy injection.

d. The procedure used for calculating the typical pontoon's structural "demand to capacity" loading ratio is conservative.

e. The frictional shear capacity of the joint between longitudinal pontoons is significantly greater than the maximum shear and torsional load which will be applied to the joint – i.e. it is conservative.

f. The structural capacity of the pontoons is more than adequate for all anticipated loads.

Bolt-beam stresses and cracking

The bolt-beams are thickened concrete structures around the pontoon's perimeter keel and deck slabs and walls - at the ends of the longitudinal pontoons and at the mid-section of each end/cross pontoon. These bolt-beams encase the prestressing tendons and anchorages and the large bolts connecting the pontoons

[2] Spalling is defined here as disruptive or bursting concrete cracking caused by post-tensioning forces.

also pass through these structures. Very significant loads are applied from the prestressing tendons (PT) and the connecting bolts to the bolt-beams.

In combination, neglect of two factors: a) spalling stresses and b) shear lag effects [3] were the major contributors to the bolt beam spalling and cracking for the cycle 1 pontoons. In particular, bolt beam cracking at the end walls and under the pontoon is of concern regarding long-term service life.

These bolt beam cracks do not diminish the overall flexural or shear capacity of the pontoons but, if not sealed, constitute a potential source for leakage of water into the pontoons and increased risk of corroding reinforcing steel. The post-tensioning forces, if not resisted, will continue to "drive" these cracks which will tend to open and extend over time. These forces need to be resisted by transverse post tensioning, as has been recommended[4] and which is now being implemented for all pontoons.

Findings

a. The design of the bolt beam was inadequate to sufficiently resist the transverse splitting and tensile stresses caused by the longitudinal post tensioning forces.

b. Modifications to the tendon profiles during construction, which moved the tangent point of the tendons outside the bolt-beam, caused spalling.

End-wall cracking

The end walls above the lower bolt-beams close the ends of the pontoons – they span vertically between the keel and deck slabs and span horizontally over several internal longitudinal walls[5] that are constructed of precast concrete panels.

For the cycle 1 pontoons, flexure of these walls due to stresses from post tensioning, in addition to drying plus autogenous[6] concrete shrinkage of the end wall itself, plus shrinkage and thermal contraction forces from the top slab, produced cracking over the depth and width of the end walls.

These tensile forces could be reduced by better thermal control (which has been implemented for cycle 2), adjustments to the pour sequence (in discussion) and transverse post tensioning of the bolt beam (being implemented for cycles 2-6 and to be retrofitted to cycle 1).

Decoupling of the interior precast walls from the end wall by leaving a gap between the precast longitudinal walls and the end walls until after stressing the longitudinal PT was considered and modeled. It was determined that, while this reduced end wall stresses, the improvement was relatively minor and therefore not justified from a cost and schedule perspective, since there still would remain some cracking that would need to be sealed.

The basis for this determination was that, because the transverse post-tensioning is effective in reducing bolt-beam and some end wall stresses, it would be possible to eliminate the decoupling of the end walls from the interior walls for the remaining Type 3 (end/cross pontoon). End wall decoupling was also

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[3] Shear lag effects are related to the distance over which a force is transmitted from one structural element (e.g. the post tensioning anchorage) to another (e.g. the internal longitudinal precast walls of the pontoon cells). The force distribution over distance causes secondary force and stress effects.

[4] Several recommendations, such as the transverse PT and other recommendations referenced in this report, have been developed by WSDOT, the ERP, the contractor, and PB - in most cases working in coordination.

[5] These precast walls create the interior pontoon cells which strengthen the pontoon structure and support the top slab, side walls and keel slab. The cells are also used for ballasting the pontoons.

[6] Autogenous shrinkage is a volume change with no moisture transfer to the external environment. It is therefore different than drying shrinkage and most prevalent in high performance concrete where the water-cement ratio (w/c) is under approximately 0.32. The w/c ratio for the Aberdeen pontoons was as low as 0.28.

shown to have a minimal beneficial effect for longitudinal pontoons and was eliminated to improve schedule and lower cost.

Additionally, before the cycle 1 pontoons are joined longitudinally, the existing keel slab and end wall cracked areas (that will be below the waterline of the completed bridge) should be epoxy injected and waterproofed with fiber-reinforced sheets. This application is currently being studied by the contractor. The exact sequence of transverse post tensioning, epoxy injection, and waterproofing is currently under review and discussion.

Finding

The potential effects on watertightness, and resulting impact to service life, from cracking of the pontoon end walls due to spalling stresses and shear lag effects, created by the post-tensioning tendon layout, in addition to thermal and shrinkage stresses, were not adequately considered in the design.

2. Quality of the as-constructed pontoons

A review and comparison was made of the level of structural and non structural cracks found on SR520 pontoons, compared to cracks found on other WSDOT floating bridge pontoons.

Two cracking mechanisms affect cracking in the completed Pontoons: a) thermal and concrete shrinkage induced cracking, and b) post-tensioning induced cracking. The contract requires the contractor to:

1. Consider the results of the WSDOT ACME pilot project, which addressed the concrete mix and production related to crack control and construction quality of the pontoons and,
2. Seal all cracks with one of two methods – epoxy injection for cracks greater in width than 0.006 inches or application of a crystalline sealant for cracks smaller than 0.006 inches. Use of these methods is treated in detail later in this report.

Commentary on concrete cracking - mechanisms

a. Thermal and shrinkage cracking – this cracking is expected, cannot be avoided, but can be reduced and effectively sealed by the methods noted above - as has been WSDOT's practice and experience for all their floating bridges. It is the goal of WSDOT and the contractor to minimize these cracks. The proper detection, mapping and repair of these cracks by the contractor, as required by the contract, is expected to result in a service life similar to, or better than, the existing WSDOT floating bridges. Achieving the required 75 year service life can be accomplished with normal maintenance.

   Thermal cracking is not consistent in all pontoons. This is an indication that concrete placement and temperature conditions were different for each of the documented cycle 1 pontoons and indicates that the thermal control plan was not implemented consistently for those pontoon placements. See the body of this report for more detailed coverage relating to the thermal control plan.

b. Post-tensioning induced cracks occur generally at the ends of the post-tensioned pontoons as noted previously. The cause, location, consequence, and repairs of these cracks are detailed in Sections 3 and 5 of this report. Transverse post-tensioning and contractor designed repairs[7], discussed in Sections 3 and 5, are being implemented to provide cycle 1 and 2 pontoons with a durability similar, to or better than, that of existing WSDOT floating bridges. For later cycles, transverse post-tensioning is being implemented for construction in the casting facility, before float-out.

Comparison of cracking for the pontoons produced at Aberdeen and CTC Tacoma (thermal and shrinkage cracking) found that:

[7] Subject to WSDOT and ERP review and normal quality control for these applications.

a. Cracking for the SR520 cycle 1 pontoons was less severe than cracking experienced with the Hood Canal Bridge pontoons.
b. Cracking in the SR520 supplemental pontoons produced in Aberdeen was comparable to that of the SR520 Floating Bridge & Landing (FBL) supplemental pontoons produced the CTC facility, Tacoma.
c. Non post-tensioning related cracking for the SR520 cycle 1 pontoons was reported to be consistent with WSDOT's experience with the other Lake Washington floating bridges. This means that concrete cracking under service and extreme conditions is expected to be at a level similar to, or no higher than, that experienced for other WSDOT floating bridges.
d. Cracks in other WSDOT floating bridges have been successfully sealed and maintained with little or no leakage into cells from those cracks. In watertightness reports, water found in the pontoons of these bridges was suggested to be coming primarily from hatches that are not completely watertight, from condensation and from openings around anchor cable wells. The exception reported was for several pontoons of the existing Evergreen Point Bridge, where previously existing keel and side wall cracks did not close completely when the bridge was retrofitted with end to end post-tensioning.
e. Cracking has been, is being, or will be repaired using epoxy and crystalline sealing techniques similar to those used the for successful repairs made for other WSDOT floating bridges.
f. Following application of transverse PT and repair of cracking, the panel fully expects that the SR520 replacement bridge will be perform as well as existing WSDOT floating bridges, in terms of minor seeping, dampness and internal condensation.
g. Following application of transverse PT and repair of cracking it is very unlikely that there will be any cracks that "open" or "work" during service loading, such that water could then penetrate through the wall or keel slabs in any significant quantity.

3. Crack repair and minimization – methods and strategies

Crack types (noted above), their causes and repair methods were reviewed, consistent with current and historical repair methods used by WSDOT and by the industry for repair of concrete structures.

Concrete structures of this type presume cracking. Their design includes provisions to accommodate the occurrence of cracks and measures to seal them sufficiently for their specified service life.

The high level of reinforcement in the structure is designed to not only to accommodate service loads but to distribute cracking in a manner that results in more frequent but smaller cracks. The repair procedures specified are routine for these and similar structures where distributed cracking is a routine occurrence.

Non post-tensioning cracking - keel and deck slabs, walls, interior precast elements

The repair criterion established for the repair of cracks provides the necessary assurance that wall and keel slab cracks that develop before post-tensioning will not be detrimental. Epoxy injection of structural cracks[8] will seal these cracks. Leakage through sealed cracks was not observed in the cycle 1 pontoons.

Post-tensioning related cracking - bolt-beams, end walls

Several repair procedures are required to bring these structural elements to their required capability. Some of these procedures are adequately addressed by the normal, contract specified requirements at the Aberdeen and Lake Washington sites. Others require additional procedures, currently being designed and implemented by the Floating Bridge and Landings contractor (KGM) with the input of their specialized subcontractor (Gerwick). The Gerwick recommendations are currently being finalized and will be reviewed by KGM, WSDOT and the panel.

[8] Structural cracks are here defined as those greater than 0.006 inches in width.

Repair methods

Epoxy Injection is a recognized and well-proven method to structurally seal cracks in concrete and has been used by WSDOT for all their floating bridges. A previous report (*Wiss, Jenney, Elstner April, 1999*) evaluated WSDOT's method and concluded that the repair materials and methods were appropriate and effective. The panel has no reservations with respect to the use of epoxy methods, but offers the following recommendations:

a. Consideration should be given to the use of an epoxy that is known or demonstrated to be effective for crack widths as small as 0.006 inches (the current specification relates to 0.008 inches).

b. The effectiveness of full depth penetration for the epoxy injection should be demonstrated under actual placing conditions. The Aberdeen mockup could be used to evaluate this by core drilling of repaired cracks to determine the effective epoxy penetration. Should full penetration not be observed, there are other injectable epoxies that may be better suited for such repairs.

Surface coating sealant.

a. There is a body of literature and testing that supports the beneficial use of this type of material and it is known that a certain amount of leakage can be controlled by this material, provided water head pressures remain low (less than 30 ft.), which is the case for these pontoons, and crack movements are negligible – which is expected for this application (see structural loading and capacity findings).

b. No changes are recommended for the application of the crystalline surface coating sealant.

Crack reduction, concrete mixing and placement

The panel reviewed the concrete mixtures being used on this project. The current mixture is reported to be equivalent in proportions to that used for other similar projects and as tested for the ACME project, however the constituent materials are not the same.

A review of strength performance indicates that the current mixture achieves compressive strengths which are significantly in excess of those required. Ultimate concrete strength is not the only consideration nor is it necessarily beneficial for this application – for example the high concrete content generates significant heat, which affects thermal control and therefore frequency of cracking. Benefit might be realized by reducing the cement content which should be considered by WSDOT and the contractor.

Concrete Water Cement Ratio

A lower water-cement (w/c) ratio, while increasing strength, is not beneficial for this project. This has been discussed with WSDOT – who advised that the contractor understands and has been responsive to this issue.

Thermal Control

Compliance with the approved thermal control plan is essential to minimize thermal cracking. There has been a reduction of thermal cracking for cycle 2 following better contractor compliance with their thermal control plan, however the panel feels more improvements are possible, specifically regarding control of heating and cooling. This should continue to be discussed with the contractor, ensuring required procedures are followed and that steps are taken to further address the effectiveness of the contractor's thermal control plan and its implementation.

Time-dependent effects – current contract requirement

The contract specifications require the contractor to "Design for time-dependant effects associated with the construction sequence. This includes thermal, shrinkage, elastic shortening, and the design of the thermal control/concrete cure system."

This section of the contract also gives the contractor the option of constructing the pontoons based on WSDOT's M-11 drawings which are defined as the minimum requirements for the design of the pontoons for this contract. The contractor chose to use the M-11 drawings as the basis of design, therefore making WSDOT the responsible designer and Engineer of Record (EOR). There was no construction sequence specified in the contract documents for the pontoons but the contractor elected to use a sequence similar, in most areas, to that used in the ACME demonstration project.

This approach, utilizing the ACME sequencing application, and time dependant results[9] to develop their geometry control plan and pour sequencing, was determined by the WSDOT site construction office to be in accordance with the contract requirements regarding time dependant effects. It is likely that a different construction sequence could reduce thermal and shrinkage stresses and therefore cracking. This should be evaluated, in discussions with the contractor, and adopted if beneficial for reduction of cracking, and if consistent with schedule and cost considerations, as determined by WSDOT.

Crack reduction, bolt-beams and end walls

See above for a discussion of the forces driving cracking in the bolt-beam and end walls.

a. WSDOT and the contractor have made changes to the PT tendon profiles to eliminate slab spalling and to reduce end wall cracking, associated with overstressing concrete around the PT anchorages.

b. External transverse PT for the bolt-beams has been added for cycles 2-6, and will be retrofitted for the cycle 1 pontoons, to close and stabilize the bolt-beam cracks so that they can be effectively sealed.

4. Maintenance considerations

The current maintenance estimate for the new SR520 Floating Bridge was established based on WSDOT's historical experience for maintenance requirements of the existing bridge. That bridge had experienced significant leakage, requiring pumping of several cells at least 3 time a week, prior to the retrofit of external longitudinal post tensioning and epoxy sealing in 1999 to close and seal existing cracks. Following this, leakage was reported to be "very manageable" and in line with leakage experienced by other WSDOT floating bridges.

The panel observed, during a site visit to a representative number of cells in Pontoons V and U on Lake Washington, that repairs to the walls had been successful in sealing those cracks. These cracks were dry to the touch with no indication of weeping or seeping. However, the panel observed that some cracks, on the top of the bolt-beams in pontoons V and U, were moist, with water stains extending down the bolt beam slope from the crack locations. These cracks appear in the general vicinity of cracks mapped on the underside of the keel slab by divers. Investigations are proceeding by the contractor (KGM and Gerwick) with WSDOT regarding the most appropriate methods of crack sealing and repair for these areas.

[9] The panel notes that the ACME project mock-up did not include the application of PT, which could have a significant influence.

2. INTRODUCTION

Introduction

The SR 520 bridge replacement program consists of several major construction projects. One of these is the Pontoon Construction Project (PCP), which includes the early construction of 33 concrete pontoons for replacement of the existing SR520 floating bridge support structure. A follow-on design-build (DB) contract, the Floating Bridge & Landings Project (FBL) will produce an additional 44 pontoons and will add new bridge superstructure, including the road deck and pedestrian path / bikeway. The replacement bridge will bring the roadway up to current design standards, be much less vulnerable to storms and earthquakes and is capable of accepting Sound Transit light rail in the future.

The SR520 pontoons are long, hollow concrete units, strengthened[10] with reinforcing steel rods and prestressed in the longitudinal direction with high-strength prestressing tendons (PT), encased in the concrete walls and slabs. For the cross (end) pontoons, transverse prestressing tendons are used as well as longitudinal tendons.

The design of the pontoons is very complicated and specific, according to a design criteria which limits both tensile and compressive stresses in the concrete, steel and tendons, as well as limiting the width of expected cracks that will occur under service and ultimate loading conditions. The pontoons are designed to accept the weight of all structures, highway and other loadings, and to resist extreme wind, wave and earthquake loads.

WSDOT Strategy, Emergency Replacement, Contract costs, As-bid Savings

The original bid for the PCP project was $367 million, compared to an original estimate of $547 million. The savings of $180 million reflected the strategic approach taken by WSDOT regarding schedule, packaging, content, and form of this contract and the related FBL contract which was bid at $587 million, realizing a savings of $163 million compared to the WSDOT upset price estimate of $750 million. The very compressed project schedule was necessary in order to be prepared in the event of a catastrophic failure of the existing bridge. The PCP pontoons could be used for such an emergency replacement.

WSDOT is to be commended for that strategy and the cost and schedule benefits so obtained, for both the PCP and FBL contracts and the SR520 program. Given the cracking and delays, some of those benefits will now be offset by the cost and schedule impacts of the remedial actions for cycles 1 and beyond of the PCP contract and follow-on impacts to the FBL project. Those cost issues are being addressed separately.

Timeline, unexpected cracking

The PCP contract was awarded to Kiewit-General (KG) in January, 2010. They proceeded to construct the pontoon casting basin and gate and then to construct the 1st pontoon cycle – consisting of three longitudinal (Type 1) pontoons, one end/cross pontoon (Type 3) and two supplemental stability pontoons (SSPs).

In May of 2012, during and after the post-tensioning of pontoon V, the first longitudinal pontoon to be post-tensioned, spalling and cracking were observed in two locations: (1) on the line of the post-tensioning tendons and near the end walls; and (2) over the depth and width of the end walls.

[10] Concrete is strong in compression - 10-14 thousand pounds per square inch (psi) in this case, but relatively weak in tension (600-1000 psi). Reinforcing steel is used to resist tension and limit crack widths and prestressing tendons are used to compress the concrete to avoid or reduce tension cracking.

ERP Scope - Phase 1

As soon as the spalling was observed in pontoon V, WSDOT directed the contractor to repair the spalling (designed by WSDOT) and then proceeded to determine the cause of the spalling and cracking. WSDOT convened an Expert Review Panel (ERP) comprised of concrete design and construction experts which was asked to:

1. Determine the most likely cause of concrete cracking and spalling in the SR 520 Floating Bridge pontoons being constructed at the casting basin in Aberdeen, Washington.
2. Determine the need for, and character of, potential changes to pontoon design, details, and/or construction methods to avoid similar concrete cracking and/or spalling in future concrete pontoon construction cycles. Design and construction changes are the responsibility of the WSDOT Bridge & Structures Office (BSO) and/or contractor.
3. In coordination with WSDOT, identify and present considerations regarding the ability of the as-constructed pontoons to be repaired to a condition that will maximize their service life as an integral part of the completed new SR 520 Floating Bridge.

Results of the ERP Phase 1 work

WSDOT and the ERP identified design and construction concerns and made recommendations for changes and improvements to the currently constructed pontoons and for subsequent production cycles. These findings and recommendations were reported by the ERP in a memorandum of August 17th 2012 and WSDOT's response was defined in their memorandum of August 23rd 2013. Both memoranda are included in this report (Appendix D).

ERP work continued, Phase 2

WSDOT agreed with the ERP's Phase 1 recommendations and began the process of implementing design changes for subsequent cycles 2 through 6. WSDOT also initiated an independent design review by an outside consultant, Parsons Brinckerhoff as recommended by the ERP.

The ERP continued their involvement by reviewing the design and construction changes to be made for cycles 2 and cycles 3-6 by Kiewit-General for the Pontoon Construction Project (PCP) and repairs to seal cracks for the cycle 1 pontoons, to be proposed and implemented by the Floating Bridge and Landings Project (FBL) contractor, Kiewit-General-Manson (KGM).

ERP Scope - Phase 2

Reporting directly to the Secretary of Transportation through the Expert Review Panel Chair, the ERP was asked by WSDOT to evaluate areas related to the design, construction and long-term life-cycle performance of the pontoons of the SR520 Program. The scope of this phase addresses four general areas as follows (details of each scope area are reported in the introduction of each section following):

1. Review and assessment of structural sufficiency
2. Quality of as-constructed pontoons
3. Crack repair strategies
4. Maintenance considerations

This report

This report summarizes the findings and results of the ERP's work under the ERP Phase 2 scope with commentary and findings related to the overall design of the pontoons, quality of the work, crack repair strategies and long-term maintenance considerations.

ERP technical assistance

The ERP is also active in review and input related to corrective design and construction measures that are being taken, by WSDOT and its contractors, to close and seal cracks in order to allow the pontoons to meet their functional requirements and their 75 year service life. The ERP has been kept apprised of these actions and has given continuous input regarding technical considerations.

3. REVIEW AND ASSESSMENT OF THE PONTOONS' STRUCTURAL SUFFICIENCY

SCOPE FOR THIS SECTION OF THE REPORT

1. Review Bridge & Structures Office (BSO[11]) design calculation notebooks by selecting a sampling of more than 8 of (approximately) 33 volumes, plus portions of other volumes, to review.
2. Assess the completeness of the design calculations and their applicability to the overall structural sufficiency of the pontoons.
3. Review the design forces and deformations due to construction procedures and all in-service loadings over the pontoons' service lives in accordance with currently applicable design and performance criteria.
4. Determine whether the criteria are reasonable and whether the calculations appear to meet the intent of the design criteria.
5. Review and comment on the inputs, criteria, procedures, and results of finite element model analyses and use that data to assist in the assessment of the pontoons' structural sufficiency.

Review of Glosten report on wind and wave forces

The Glosten report on wind and wave forces to be applied used structural property data from the Bridge & Structures Office (BSO) as input for determining hydrostatic mass, damping and wave forces. Wave states were estimated for 1 year, 20 year, and 100 year Mean Recurrence Intervals (MRI) with a maximum fetch considered up the Mercer Channel directed to the west end of the SR520 bridge. This fetch has an angle of approximately 42 degree from normal to the bridge but for design purposes was applied as if normal to the bridge. This approach increases the design loads applied to the bridge, adding to the conservative nature of those design loadings.

Calculations of the dynamic bridge response were made using NASTRAN, a reliable and commercially available program developed by NASA. Bridge responses for both the 6 and 8 lane configurations were calculated using gross section properties for the 1 year and 20 year MRI storms and 60% of the gross section properties for the 100 year storm. The use of 60% gross section stiffness for the 100-year storm is consistent with observations on the existing SR520 bridge[12]. Several combinations of lake level, anchor cable tensions, and damage conditions were included. Resulting forces were reported for 10 nodes in each Type 1 pontoon including lateral shear, lateral moment, vertical shear, vertical moment, and torsion.

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[11] BSO – WSDOT Bridge and Structures Office

[12] According to the WSDOT BSO

Finding

The Glosten Report was comprehensive and was conservative with regard to the anticipated wind and wave forces imparted to the floating bridge structure.

Review of Design Criteria

Design criteria for the pontoons focused on achieving long-term water-tightness by limiting crack widths and associated reinforcing steel stresses, as appropriate for both service and ultimate load conditions.

The project specific load combinations and load factors are listed in Table 1. The notation for the specific loads are those specified in the AASHTO LRFD together with addition loads specified by WSDOT in its "*Floating Bridge Design Criteria of May 12, 2012*". For example DC is the dead load of structural components and LL is vehicular live load.

Table 1. Project Specific Load Combinations and Load Factors

Load Combination Limit State	DC DW WA BL EL CR SH PS	LL IM BR PL MA	WS_{IW} WWS_{IW}	WWD_{IW} IN_{IW}	WS_{IR} WWS_{IR}	WWD_{IR} IN_{IR}	WS_T WWS_T	WWD_T IN_T	WW_T IN_T	WL	TU	TG
Strength IA	γ_p	1.75	—	—	—	—	—	—	—	—	0.50/1.20	γ_{TG}
Strength IIIA	γ_p	—	1.20	1.40	—	—	—	—	—	—	0.50/1.20	γ_{TG}
Strength VA	γ_p	1.35	—	—	1.20	1.40	—	—	—	1.00	0.50/1.20	γ_{TG}
Extreme Event IIA	1.00	—	1.00	1.20	—	—	—	—	—	—	1.00	—
Extreme Event IIB	1.00	1.00	—	—	1.00	1.20	—	—	—	—	—	—
Extreme Event X	1.00	—	1.00	1.20	—	—	—	—	—	—	1.00	—
Service IA	1.00	1.00	—	—	1.00	1.20	—	—	—	1.00	1.00/1.20	γ_{TG}
Service IB	1.00	1.00	—	—	—	—	1.00	1.20	—	—	—	—
Service V	1.00	—	—	—	—	—	—	—	—	—	—	—
Service VI	1.00	1.00	—	—	1.00	1.20	—	—	—	1.00	—	—
Service VII	1.00	0.60	1.00	1.20	—	—	—	—	—	0.60	1.00	—
Service VIII	1.00	—	—	—	—	—	—	—	1.00	—	—	—

γ_p for BL (Ballast) and WA (Static Water) shall be 1.00.

γ_{TG} shall be determined in accordance with the AASHTO *LRFD Bridge Design Specifications*.

Service and Ultimate Load Cases

Service Load Case IA included dead load, live load, 20 year MRI wind and waves with appropriate load factors. Crack widths under this load case were limited to 0.004 inches, corresponding to a reinforcing steel stresses[13] of 14 thousand pounds per square inch (ksi[14]).

The Strength Load Cases included dead load, live load with 20 year MRI wind and waves, and 100 year MRI wind and waves, with appropriate load factors. Crack widths under these cases were limited to 0.010 inches and steel stresses to 25 ksi. Crack widths under Extreme Event Load Cases with damaged pontoons were limited to 0.016 inches.

Crack Widths

To calculate crack widths for prestressed concrete sections a strain compatibility analysis is required, which is the approach recommended by ACI Committee 224. A strain compatibility analysis is an iterative procedure to determine stresses in a cracked concrete section and takes into account the applied prestress loads, axial load and moment. The only assumptions are that plane sections before flexure remain plane, that strains are therefore linearly distributed over the depth of a section in bending and the resulting stresses are calculated from the strains using constitutive relations (stress versus strain) for the materials within the section. The BSO design used a strain compatibility analysis and assumed that the concrete carried no tension, a typically conservative assumption.

The limiting crack width of 0.004 inches for the Service Load 1A differs from the value of 0.006 inches used as a dividing line between "non-structural" and "structural" cracks, related to repair techniques to be used for the as-constructed pontoons. The following table summarizes the relationship between limiting crack widths discussed in this report, the maximum values recommended by ACI Committee 224 for various exposure conditions, the limiting crack widths used for design by the BSO and the limiting crack widths specified by the BSO for any required repair procedures for constructed pontoons.

CRACK WIDTH (IN.)	ACI 224 EXPOSURE CONDITION*	BSO DESIGN CONDITIONS	BSO REPAIR REQUIREMENTS
0.004	Water-retaining structures	Service Load Case 1A	"Non-structural" crack. Surface sealing required for crack widths less than 0.006
0.006**	Seawater and seawater spray, wetting and drying		Structural crack. Epoxy injection required for crack widths ≥ 0.006.
0.010	Significant flow of water through cracks starts***	Strength Load State	Structural crack. Epoxy injection required for crack widths ≥ 0.006
0.016	Dry air or protective membrane	Extreme Limit State	Structural crack. Epoxy injection required for crack widths ≥ 0.006

Table 2, Summary of Relevant Crack Width Criteria

* ACI 224 correlates the exposure condition in Column 2 with the crack width in Column 1. However, ACI 224 notes that the listed crack widths are reasonable values for service loads when the structure

[13] The relationship between crack width and steel stress was calculated in accordance with the recommendations of the Report ACI 224R.01 "*Control of Cracking in Concrete Structures*".

[14] KSI is a level of stress measured in kips (1000 pounds) per square inch.

is first placed in service, a portion of the cracks in a structure can be expected to exceed the listed values, and crack widths will increase with time,

** ACI 224 notes that cracks wider than 0.006 in are generally visible to the naked eye and can generate a sense of insecurity if water retention is a concern

*** See Table in Chapter 4.

Findings

1. The design criteria applied by the BSO were in accordance with those used for previous floating bridges in Washington State.
2. The definition of service load limit, based on a crack width of 0.004 inches, is in line with that used for water-retaining structures and has been shown to result in structures which will not exhibit significant leakage.
3. The definition of ultimate strength based on a crack width of 0.010 inches will limit leakage in extreme events to a minor and manageable amount, consistent with the current performance of other WSDOT floating bridge structures.
4. The construction specification requirement for epoxy injection of cracks of 0.006 inches or wider is consistent with a threshold where leakage through cracks may be noticeable.

Review of Calculation Notebooks

Selected calculation notebooks, including relevant corresponding books from the design of the Lacey V. Murrow (LVM) floating bridge, were reviewed – including calculations of section properties, demand/capacity ratios, structural capacity calculations and details of the bolt-beam for anchorage of post-tensioning tendons and pontoon connecting bolts.

Load Cases, Strain Compatibility and Prestress level

Load cases covered included: the 6 and 8 lane configurations; different lake level elevations; 1, 20, and 100 yr storms; and multiple damaged conditions with flooded cells.

Seventy nine (79) separate cases were considered. Loads were combined with appropriate load factors according to the design criteria. The generic dead load vertical moments and shears that were included were based on the worst possible combination of superstructure loads since the pontoons were designed to be used anywhere in the length of the bridge (except for the cross pontoons). This “generic” dead load requires mobilization of 10% of the total vertical flexural capacity of the pontoons, based on a crack width of 0.01 inch. The vertical component of the anchor cable forces was included in the calculations.

The structural capacity of the pontoons - either the moment to produce a crack width of 0.004 inches or 0.01 inches depending on whether the load case was Service or Strength - was determined using a “strain compatibility analysis”. The prestress force varies along the length of the pontoon due to friction, wobble, and anchor set losses. The strain compatibility conservatively used the minimum prestress force, less assumed long-term losses of 27 ksi and an elastic shortening loss of 4.5 ksi for a final force after creep over a period of some 30 years of 170.5 ksi. PB’s long term analysis showed a minimum prestress force of 185 ksi after the same time period.

The strain compatibility analysis is sensitive to the level of prestress - a change of 1 ksi in assumed long term prestress loss results in a change of 3,600 kip-ft in the pontoon’s vertical moment capacity, based on maximum crack width of 0.01 inches. The strain compatibility analysis included a 500 kip tension from possible superstructure loads, but it neglected the compression force due to water pressure on the ends of the pontoons (1,081 kips). This adds to the conservative nature of the pontoon design.

The level of prestress in ksi in the longitudinal walls, and keel and deck slabs were similar to those for the pontoons of the LVM bridge

Findings

1. The strain compatibility analyses as conducted provide a conservative estimate of the available structural capacity.
2. The prestress load level used in the calculations is conservative in that the effective prestress load in the tendon, beyond the sharp curvature of the PT tendon within the bolt-beam, will be higher than at the ends of the pontoons. Long term prestress losses, as calculated by Parsons Brinckerhoff's finite element model, were less than those assumed by BSO

DEMAND / CAPACITY RATIO

The factored applied loadings, in both the lateral and vertical directions, were divided by the pontoon's structural capacity in the appropriate direction to determine a demand to capacity (D/C) ratio. The demand to capacity ratios for those two directions were then added arithmetically to determine a final demand to capacity ratio.

This procedure for calculating the demand to capacity ratio is conservative. The maximum demand to capacity ratio, and the only one found to be greater than 1.00, was 1.09 for one of the damaged pontoon cases (6 lanes, un-cracked, 7 cells flooded, Pontoon L). Given the design criteria used for ultimate strength, this is basically a very minor potential leakage issue and not a structural capacity issue.

Finding

The general procedure used for calculating the typical pontoon's demand to capacity ratio is conservative and the structural capacity of the pontoons is more than adequate for all anticipated loads with the caution that one condition, not thought to be a concern, has not yet been fully resolved, as described following:

INDEPENDENT STRUCTURAL SUFFICIENCY ANALYSIS (PB)

An independent pontoon structural sufficiency analysis is underway by Parsons Brinkerhoff. This effort is incomplete at this time, however a draft report for the Type 1 pontoons was issued Feb. 18th, 2013.

PB's findings in this draft report are summarized as follows:

1. A number of clarifications are needed to the design criteria and loadings in order to complete the analysis. These clarifications are in progress.
2. The structural strength of the pontoons is sufficient to withstand all loadings.
3. The crack resistance of the pontoons (affecting water tightness at normal loads) is within plus or minus 5% of the design criteria. This will be refined as the design criteria are clarified but plus or minus 5% is not of concern at this time.
4. The capacity of the bolt beam to resist transverse splitting forces from the post tensioning was not adequate.
5. The capacity of the pontoon to pontoon joint and resistance to general bending of the pontoons is conservative in comparison to the design loadings.

Conclusion

The bridge design criteria should be clarified and issued as a finalized document. Even with the criteria questions that are in discussion, the analysis indicates that the pontoons themselves and the bridge in total has sufficient structural capacity to withstand all expected loads.

Recommendation

1. The panel should review the design criteria and work with the BSO and PB to finalize the design criteria.
2. Based on the finalized design criteria, the BSO and PB should independently review the capacity of the bridge to meet the design criteria. A final review meeting with the BSO, PB and the panel will review the findings of that independent analysis.
3. If necessary, the panel will make recommendations to improve the performance of the pontoons to in order to meet all the design criteria.

Supplemental stability pontoons (SSPs)

Supplemental stability pontoons are added to the sides of the longitudinal pontoons to add buoyancy and rotational stability for the initial and final bridge configurations and loadings. However, the additional structural capacity provided to the longitudinal pontoons by the attached supplemental stability pontoons was not counted as part of the overall structural capacity by the WSDOT BSO.

Finding

The BSO design approach with respect to the SSPs is conservative.

Longitudinal Pontoons, connection bolts and joining

The bolts joining the pontoons were analyzed by the WSDOT BSO for combined tension and shear loadings. However these bolts will be subjected to shear loadings only if the cement grout in the joint[15] between the pontoons fails in shear.

Finding

The frictional shear capacity of the joint is significantly greater than the maximum shear and torsional load which will be applied to the joint. The BSO design approach is conservative.

BOLT BEAM / END WALL - FORCES AND CRACKING

The bolt-beams are thickened concrete structures around the perimeter keel and deck slabs and walls at the ends of the longitudinal pontoons and at the mid-section of each end/cross pontoon. The large bolts connecting the pontoons pass through these structures, which also encase the prestressing tendons and anchorages. Very significant loads are applied from the bolts and prestressing tendons to these structures.

The end walls are integral with the bolt-beams and seal the ends of the pontoons. The end walls span vertically between the keel and deck slabs and span horizontally from one side of the pontoon to the other supported over several internal longitudinal walls that are built up from precast panels. The end walls of longitudinal pontoons also contain alignment keys that protrude from one pontoon into the adjacent pontoon. In the following text, the forces acting on the bolt-beams and the end walls and the cracking for the bolt-beams and the end walls are discussed separately. However, because those two elements act as one, the forces they resist and their cracking patterns are interrelated.

The bolt-beams for this SR520 bridge are similar (almost identical) in size and reinforcing to those used on the Lacey V Murrow (LVM) bridge - even though the cross-sectional area of the pontoons for the SR520 bridge is approximately 100% greater than that for LVM. Further, while the average compressive stress levels from post tensioning on the pontoon cross-sections for the two bridges are similar, the

[15] The pontoons are joined and bolted together. The bolt load compresses a watertight gasket between the ends of the pontoons. The void between the resulting pontoon end walls is then filled and sealed with a cement grout.

individual post tensioning tendons and anchorage forces are significantly greater in size and loading for the SR520 bridge than was the case for the LVM bridge.

Additionally, the concentration and distribution of those post tensioning forces, across the keel and deck slabs for the two bridges, differs significantly - due to the need to provide greater stability restraint at the keel slab level for the taller internal precast walls in the SR520 bridge. The LVM tendons were distributed much more evenly. The SR520 tendons were grouped to avoid the feet and other elements at the base of the precast panels and the column bases for the eventual superstructure.

For the pontoons, longitudinal and transverse, detailed "strut and tie" method calculations were made to size the reinforcement required to resist the bursting and tendon deviation forces created by the longitudinal post-tensioning tendons, their curvature in plan and elevation, and for the forces from the bolts connecting the adjacent pontoons.

However, no calculations were found that addressed in detail the reinforcement (or transverse prestressing) required to resist significant lateral[16] splitting forces and tensions which were created in the area between adjacent tendon groups after stressing the PT tendons. This area is also centered on the internal precast walls which have a significant additive effect to the splitting forces and tensions in the bolt-beam and end wall. As a result, there were two significant cracking effects as noted following.

BSO - method of bolt-beam analysis

For the longitudinal pontoons the BSO proportioned and reinforced the bolt-beam so that it could resist the effects of the post-tensioning tendon forces in the longitudinal direction of the keel slab. They used the "strut and tie" procedure for modeling. They assumed that the same design details would also be adequate to resist the effects of the tendon forces:

1. In the side walls and deck slab, and
2. In the transverse direction of the bolt-beam for the keel and deck slabs.

Application of concepts from existing literature, and the extent of the vertical cracking in the end walls near and through the bolt-beams (keel and deck slab levels), demonstrate that the second assumption was unrealistic.

Analysis for the effects of the prestressing tendon forces on the bolt-beam for the transverse direction is a difficult task. While approximate methods of analysis may yield a design that is adequate in several respects, the task is best handled by finite element modeling (FEM). To be fully realistic the FEM model needs to be able to account for the effects of:

1. Restraints to keel slab movements by the casting dock floor,
2. Effects of shrinkage cracking along the length of the side walls and the deck of the pontoons,
3. Drying shrinkage and thermal stresses developed in the end walls, and
4. The greater stiffness of the interior precast walls relative to the other longitudinal elements of the pontoons at the time the post-tensioning is applied.

It also needs to be recognized that the design for tendon forces is only the first stage in a series of designs for the bolt-beam – which must also be able to resist the effects of towing of the pontoons to the bridge site, the effects of bolting of one pontoon to the next, and finally the effects of loadings when the bridge is in service. Problems in the performance of the bolt-beam during post-tensioning may lead to additional problems for its performance in subsequent loadings.

[16] Lateral relative to the longitudinal axis of the pontoon.

Finding

1. In combination, neglect of two factors:
   a) spalling stresses and
   b) shear lag effects [17]

   - were the major contributors to the spalling and cracking for the cycle 1 pontoons and in particular to bolt-beam cracking at the deck and keel slab levels, as discussed in the ERP memorandum report of August 17th, 2012. The cracks so produced do not diminish the overall flexural or shear capacity of the pontoons but, if not sealed, constitute a potential source for leakage of water into the pontoons.

2. This issue is particularly of concern for the cross-pontoons, whose end walls are exposed to the lake for their full service life. The longitudinal pontoon end walls are not exposed for their full service life, so for them, the issue is more of concern for the initial tow in seawater and subsequent exposure to lake water before they are joined and grouting pressures.

Application of the Bolt Beam "Strut and Tie" Model Calculations

The "strut and tie modeling" which was made to determine connection stresses and reinforcement within the bolt-beam and the bolt-beam / PT-Anchorage interaction was reviewed by looking at the scope of the calculations made by the WSDOT BSO as reported in Vols. 8, 23 and 24 of the BSO records.

The following matrix shows the scope of those calculations. While the connections for the cross pontoon and the SSP and the connections of those members to the longitudinal pontoon were analyzed and designed in depth, very little seems to have been done to examine the connections between adjacent longitudinal Type 1 pontoons. It is possible that this was done in another Calculation Volume but BSO has not advised that this was the case (as of this writing). ERP, with input from PB's independent global analysis (not yet received), will complete review of the structural sufficiency of the connection between adjacent longitudinal Type 1 pontoons. If there is also receipt of a BSO analysis, that will be considered.

[17] Shear lag effects are related to the distance over which a force is transmitted from one structural element (e.g. the post tensioning anchorage) to another (e.g. the internal longitudinal precast walls of the pontoon cells). The force distribution over distance causes secondary effects.

Pontoon Type	PT Anchorage, x-direction	PT Anchorage, y-direction	Bolt Anchorage x-direction	Bolt Anchorage y-direction	Bolt-PT Interaction
Longitudinal (Type 1)					
Keel (k)	✓				
Wall (w)					
Deck (d)					
Cross (Type 3) Long Wall					
Keel (k)	✓	✓	NA	NA	NA
Wall (w)	✓	✓	NA	NA	NA
Deck (d)	✓	✓	NA	NA	NA
Longitudinal (Type 1) To Cross-(Type 3)	✓ k, w &d	✓ k, w &d	✓ k, w &d	✓ k, w &d	✓ k, w & d
SSP to Long (Type 1)					
Keel (k)	✓	✓	✓	✓	✓
Wall (w)	NA	NA	NA	NA	NA
Deck (d)	✓	✓	✓	✓	✓

Table 3, PT Anchorage design analysis by pontoon type

Finding

Strut and tie modeling is an accepted procedure for ensuring that a connection has adequate strength. However, such modeling provides no information on what cracking may be associated with achieving that strength or the resulting extent and width of cracking at service loads. Further, for a structure in which strength considerations for the transverse direction (y-direction) are as important as that in the longitudinal direction (x-direction), the AASHTO LRFD Bridge Design Specifications require that three dimensional effects be examined and that as a minimum the connection be designed separately for two orthogonal directions.

It does not appear that, based on the calculations reviewed by the panel, that transverse direction in the bolt-beam was adequately analyzed.

Effect on the Bolt Beam

The effect of the PT and other forces on the deck and keel slab bolt-beams was to create vertical cracks in the bolt-beam, approximately in line with the interior precast walls. These cracks, of the order of 0.035 inches and greater, extend above the height of the bolt-beam for the keel and deck slabs and wrap around (under the keel slab and over the deck slab), extending some 12-15 feet along the length of the pontoon.

There are two issues with such cracks:

1. The prestress force will continue to "drive" these cracks, opening and extending them further over time. The FEM models confirm this. These cracks need to be stabilized in order for the cracks to be reliably sealed.
2. The sealing of these cracks is difficult since the keel slab cracks are not accessible until after pontoon float-out. The effective application of epoxy can be compromised if applied under water. See later in this report for discussion of epoxy and other sealing. WSDOT and the contractors are working on a method to solve this issue.

Finding

The transverse design of the bolt-beam was inadequate to sufficiently resist the splitting and tensile stresses caused by the post tensioning forces, combined with thermal and shrinkage effects and shear-lag effects.

Recommendation

The splitting forces from the post tensioning loads plus thermal, shrinkage effects and shear-lag effects need to be resisted by sufficient passive reinforcement and/or active transverse prestressing tendons. These are currently under design and installation for cycle 1 (retrofit), cycle 2 (in the casting basin) and for installation in the casting basin for cycles 3-6.

Effect on pontoon end walls

Post tensioning forces are applied at the ends of the pontoons but their effects only become relatively "uniform" a significant distance – exceeding more than half the depth of the pontoon – from each end of the pontoon and before this force is fully transmitted to, and effective in, the interior longitudinal (precast) walls. No calculations were found to address the effects of this force distribution (called a "shear-lag" effect – see earlier in this report). These forces cause the interior longitudinal walls, in the end cell of the pontoon, to have less compressive stress and thus less elastic shortening than the outside perimeter of the pontoon. Additionally, the precast walls are stronger and have more shortening resistance at the time of prestressing than the less-mature cast-in-place pontoon walls, deck and keel slabs. These factors result in "hard points" where the interior longitudinal walls resist inward movement of the end wall, causing additional exterior flexural tensile stresses in the end walls[18].

For the cycle 1 pontoons, that flexure and the resulting tensile stresses following post tensioning, in addition to the drying and autogenous concrete shrinkage of the end wall itself, plus shrinkage and thermal contraction forces from the top slab, produced cracking over the depth and width of the end walls.

[18] Tensile stresses already exist in the end walls from a combination of thermal contraction plus drying and autogenous concrete shrinkage of the end wall itself, plus shrinkage and thermal contraction forces from the top slab. The PT forces noted here add to those tensile stresses, exacerbating the potential for cracking.

The FEM models show stresses which exceed the tensile strength of the concrete are derived from both:

1. Thermal, drying and autogenous concrete shrinkage and
2. Are exacerbated by the post tensioning forces.

The models indicate the relative contribution of these driving forces and stresses. Both thermal and concrete shrinkage stresses and the post tensioning forces need to be addressed to minimize end wall cracking.

Finding

End wall cracking is caused by a combination of thermal, drying and autogenous concrete shrinkage stresses, plus top deck shrinkage movements and the effects of the post tensioning "shear-lag" forces

Recommendations

Minimize the overall tensile stress levels in the end walls by a combination of

1. Reduction of the thermal differences between keel, wall and deck pours by consistent application of the thermal control plan. (improvements have been made for cycle 2).
2. Revisit the top slab thermal control plan and pour sequence to reduce end wall stresses due to top slab shrinkage movements.
3. Add transverse post-tensioning to the bolt-beam at the keel and deck slab levels. The magnitude and location of the transverse post-tensioning should be sufficient to reduce the extreme fiber stresses in the bolt-beam at the outer faces of the keel and deck slabs to levels less than will cause cracking.
4. It is expected that the transverse post-tensioning will be sufficiently effective in reducing end wall stresses so that it is possible to eliminate the decoupling of the end walls from the interior walls for the remaining Type 3 (the end/cross pontoon A).
5. End wall decoupling has been shown to have a minimal beneficial effect for the longitudinal pontoons and can be eliminated (to improve schedule and lower cost). The remaining end wall cracks will be sealed with the normal methods (epoxy or surface sealant)

SUMMARY OF ERP FINDINGS, STRUCTURAL SUFFICIENCY

1. Design criteria are appropriate, conservative and in conformance with prior floating bridges
2. Assumptions for loads and load combinations are comprehensive and conservative.
3. Assumptions for structural capacity are conservative.
4. The structural capacity of the pontoon cross-section is greater than all anticipated demands.
5. The localized panel stress in vertical reinforcing in the exterior walls needs to be verified[19].
6. The potential effects on watertightness, and resulting impact to service life, from cracking of the bolt-beam and adjacent keel and top slabs, were not adequately considered in the design
7. The potential effects on watertightness, and resulting impact to service life, from cracking of the pontoon end walls due to the cracking stresses and shear lag effects, created by the post-tensioning tendon layout, added to the thermal and shrinkage stresses, were not adequately considered in the design.

Recommendations

1. The bolt-beams adjacent to the keel and deck slabs should be strengthened against cracking by the application of transverse post-tensioning.
2. For cycle 3 and beyond, this transverse post-tensioning should be applied during the initial construction of those pontoons and before those pontoons are post-tensioned longitudinally.
3. For the cycle 2 pontoons, now under construction, transverse post-tensioning should be applied to the keel and deck slab bolt-beam areas. If necessary, the post-tensioning tendons should be external to (not embedded in) the keel and deck bolt-beams. Transverse post-tensioning should be applied before the pontoons are post-tensioned longitudinally.
4. For the existing cycle 1 pontoons, external transverse post-tensioning of the keel and deck slab bolt-beam areas should be retrofitted inside the end cells of the pontoons.
5. Before the cycle 1 pontoons are joined longitudinally, the existing keel slab and end wall cracked areas (that will be below the waterline of the completed bridge) should be epoxy injected and waterproofed with fiber-reinforced sheets. Specifics of this application are currently being studied by the contractor for recommendation and submittal to WSDOT. The exact sequence of transverse post tensioning, epoxy injection, and fiber-reinforced waterproofing is currently under discussion.
6. The thermal control plan, requiring limitation of thermal differences between adjacent pontoon elements during concrete pours should be reviewed and upgraded if practical improvements can be made, considering improved performance, reduction of cracking plus cost and schedule trade-offs.
7. Application of the thermal control plan on the site should be monitored in terms of adequacy, specifically the ability to produce uniform and timely control of heating (and cooling). Temperature sensors and control of heating and cooling should be aligned to allow smooth, reliable control.
8. Results of the current FEM analysis, recently discussed with BSO, should be examined and used to determine if there is merit (significant reduction in cracking) to use a modified (more restrictive) top slab pour sequence. Cost and schedule trade-offs should be considered.

[19] Subject to completion of review for the particular case of combined load methods relating to vertical stress in the exterior wall panels of longitudinal pontoons, which stress may be in excess of the design criteria.

4. QUALITY OF THE AS-CONSTRUCTED PONTOONS

Introduction

Published reports have questioned the amount of cracking and thus the "quality" of the construction of the SR 520 pontoons. This section of the ERP Phase 2 scope was undertaken to review and compare the level of both structural and non structural cracks found on SR520 pontoons and those which have been found on other WSDOT floating bridge concrete pontoons. The intent of this task is to quantitatively evaluate the quality of the new pontoons and to evaluate if the cracking observed on the first cycle of pontoons indicates any underlying quality issue that may affect the life expectancy of the pontoon, and their ability to serve their intended purpose for 75 years.

Scope for this Section

1. Review and comment on WSDOT Floating Bridge Operation, Inspection and Maintenance Manual as a tool to accurately determine the sufficiency of the pontoons.
2. Coordinate a rating inspection by WSDOT Bridge Preservation Office (BPO) utilizing appropriate criteria from the Maintenance manual
3. Compare the results of the inspection and rating survey with results of similar surveys of other floating bridges in the WSDOT system, in order to prepare a statement as to the quality of the SR520 Pontoons in comparison with pontoons on other bridges.

INFORMATION FROM OTHER WSDOT FLOATING BRIDGES

The panel undertook the following activities for this task:

1. Interviewed Dave Bruce of the Bridge Preservation Office (BPO)
2. Interviewed Archie Allen, Northwest Region Bridge Supervisor
3. Reviewed the proposed O&M budget for the new SR520 floating bridge
4. Reviewed the Operations, Inspection and Maintenance Manual for the existing SR520 floating Bridge
5. Reviewed the following Watertight Inspection Reports
   a. Evergreen Point Floating Bridge dated Oct. 2011
   b. Lacey V. Murrow Floating Bridge dated March 2012
   c. Homer Hadley Floating Bridge dated March 2012
   d. Hood Canal Floating Bridge dated June 2012
6. Reviewed the Floating Bridge inspection criteria for load rating, from the Washington State Bridge Inspection Manual M 36-64.03, dated November 2012
7. Visited Medina construction site and viewed pontoons U, V (internal inspection) and W (external inspection) to consider status of cracking, and effectiveness of repairs

In discussions with Dave Bruce regarding the possibility of a formal inspection of the pontoons by the bridge Preservation Office (BPO) the ERP was advised that the standard inspection criteria would only apply when the bridge was completed and turned over for normal operation. Therefore a formal inspection was not, and is not planned to be, undertaken by the WSDOT BPO office, with the pontoons in their current incomplete state.

Therefore, for the purposes of this report, a quality comparison of inspection and watertight reports from the other floating bridges, and personal inspections of the new pontoons by the panel, were used to evaluate the quality of the now pontoons for the SR520 Floating Bridge

The following is a summary of the results of the watertight inspection reports for the State's four existing floating bridge pontoons

Lacey V. Murrow Bridge Report dated March 2012

The bridge has 2 cross pontoons and 18 longitudinal pontoons. At the time of the most recent inspection in December 2011, 10 of the pontoons were entered and 928 cells were examined. Three cells were found to contain measurable amounts of water of 1 inches in depth. Two of those cells also contained gravel ballast and 25 cells contained a trace of water. An examination of five years of pumping records showed no conclusive evidence of on-going water infiltration in any cells.

Homer Hadley Bridge Report dated March 2012

The bridge has 2 cross pontoons and 16 longitudinal pontoons. At the time of the most recent inspection in December 2011, 9 of the pontoons were entered and 1080 cells were examined. Four cells were found to contain measurable amounts of water varying from 1 to 3 inches in depth and 35 cells contained a trace of water. The cause of the water infiltration is not conclusively known. Water seepage through hairline cracks was not visibly detectable.

Hood Canal Bridge Report dated June 2012

The bridge has 4 cross pontoons, 14 longitudinal pontoons and a central draw span. At the time of the most recent inspection in November 2011, 8 of the longitudinal pontoons were entered, two of the cross pontoons and 8 of the pontoons associated with the draw span. There was measurable water in 46, and a trace of water (less than 1 inch in depth) in 70 of the 1210 cells examined. Most of the cells with measurable water in them were associated with the draw span. Only 4 cells associated with longitudinal pontoons contained measurable water. While many epoxy injected cracks were present, no significant structural deficiencies were observed. Pumping records also showed that the area where most water was removed was associated with the draw span pontoons. The mating surfaces between the cross pontoons at the ends of the draw span and the longitudinal pontoons that frame into the cross pontoon allow the draw span to twist relative to the main pontoon line. The leaking at those joints was repaired in 2011 and no pumping has been recorded since at that location.

Evergreen Point Bridge Report dated October 2011

The bridge has 19 longitudinal pontoons, 4 cross pontoons and a draw span that contains 4 pontoons. At the time of the most recent inspection in July 2011, 15 of the pontoons were entered and 2276 cells were examined. 69 of the cells were found to contain measurable amounts of water varying from 1 to 4 inches in depth with most depths being about 1 inch. 753 cells were found to contain a trace of water. The pumping records show that there is on-going leakage in numerous cells with water having to be pumped out of 34 cells in 12 different pontoons at least biennially, and many annually. Most of the pontoons have transverse cracks across the keel slab and up the exterior walls. Most of the vertical cracks have been epoxy injected, however, some of those cracks continue to leak. Some of the water in the cells was attributed to water entering through the cable anchor ports.

CRACK CONDITIONS AND CONSEQUENCES FOR THE AS-CONSTRUCTED PONTOONS

Two cracking issues have been identified in the completed Pontoons: thermal/shrinkage induced cracking, and post-tensioning induced cracking. Cracking issues associated with the construction access openings in the end walls of the pontoons are influenced both by shrinkage and post-tensioning induced stresses and the consequence and necessary repairs are the same as for the two primary crack issues.

The thermal and shrinkage cracks occur to some degree throughout all pontoons. The cause, locations, consequence, and repairs of these cracks are detailed in section 5 of this report. The proper detection and repair of these cracks by the contractor, as required by the contract, are expected to provide an end

product similar to, or with better durability than, the other existing WSDOT pontoon bridges. For later cycles of the PCP contract, the need to repair these cracks, and the risks associated with these cracks, could potentially be reduced by revaluating the mix design and the thermal control plan, as discussed in section 5 of this report.

The post-tensioning induced cracks occur at the ends of the completed post-tensioned pontoons. The cause, location, consequence, and repairs of these cracks are detailed in Section 3 and 5 of this report. Transverse post-tensioning modifications and WSDOT/contractor designed repairs, discussed in sections 3 and 5, have and are being designed to provide cycle 1 and 2 pontoons with similar, or better, durability than other existing WSDOT pontoons. Transverse post-tensioning modifications are also being designed to reduce existing cracks and to prevent or minimize future cracks.

Shown in the figure below is a composite developed scale drawing of the cracking extent on the east end of cycle 1 Pontoon V. Both interior and exterior crack locations are noted. Where concrete was removed to permit the insertion of PT deviation reinforcement, and then replaced, it is indicated by hatching. The central portion of the diagram is the east end wall and the line of the top of the bolt-beam is clearly indicated by hatching.. To each side of that diagram and on the same horizontal line are diagrams for the adjoining east ends of the north and south longitudinal walls. Above the central diagram is the diagram for the adjoining east end of the deck slab and below is the diagram for the adjoining east end of the keel slab. The different cracks are identified as either Type 1, Type 2, and Type 3 cracks.

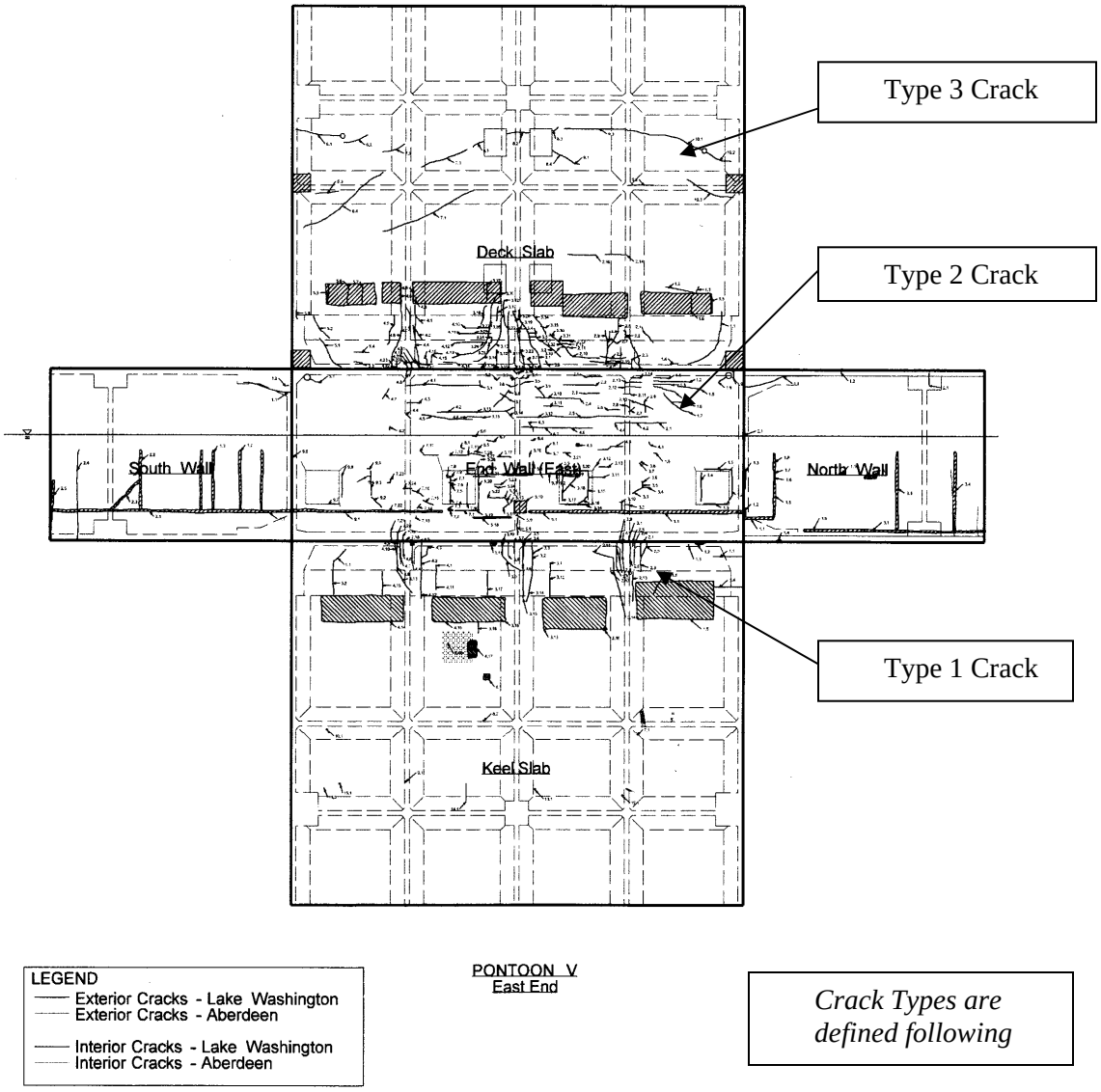

Figure 1, Crack Maps for Pontoon V

Commentary on these cracks

1. Keel Slab Cracks : (Crack Type 1 in figure). The most serious cracks are keel slab cracks that extend in the longitudinal direction of the pontoon. Similar longitudinal cracks are located sometimes in the exterior walls and frequently in the deck slab of the pontoon. Watertightness is of less concern for those wall and deck slab cracks than the corresponding keel slab cracks. The effects of the longitudinal post-tensioning are the same for all these cracks. Longitudinal cracks may be present due to the spalling stresses between tendon groups, bursting stresses in front of a tendon anchorage, and "shear lag" effects associated with the flow of prestressing forces into the precast interior walls . These cracks would be exacerbated by hydraulic loads in both service and extreme loading conditions.
2. End Wall Cracks, Pontoons A & W: (Crack Type 2 in figure). There is no bolt-beam inside the end walls for these two pontoons (the bolt-beams are located in the middle of the side walls). Existing cracks in the body of the wall will be exacerbated by both service and extreme hydraulic loads. They will have withstood the hydraulic loads associated with towing which may have further opened and/or extended existing cracks. These walls are only subject to hydraulic loads perpendicular to the plane of the wall in service. It should be a relatively simple task to estimate crack width under service and extreme loads and carry out appropriate repairs.
3. End Wall Cracks, Pontoons T through V (Crack Type 2 in figure) Existing cracks in the body of the wall inside the bolt-beam will be exacerbated by both service and extreme hydraulic loading conditions. Pontoons T, U, and V have withstood the hydraulic loads associated with towing which may have further opened and/or extended the existing cracks. Once the pontoons are connected these end walls are no longer subject to load except for the pressure due to grouting of the joints. The panel does not believe that these end wall cracks are likely to be a major cause for concern for regarding service life and maintenance.
4. Longitudinal Wall and Deck Slab Transverse Cracks (Crack Type 3 in figure): These cracks should close upon application of the longitudinal post-tensioning. The only question pertains to the width under extreme loads and the potential leakage amounts. The BSO estimation of the potential crack widths, due to longitudinal flexure of the pontoons, was based on a strain compatibility analysis. The results from such an analysis were compared with the service and extreme moment demands. Shear cracking due to torsion and vertical shear was predicted and is coincident with flexural demands. The effect of the longitudinal prestress in limiting such cracks and on the resulting slope of the cracks was neglected. This is conservative.

   The project team should use a check list similar to the table below to evaluate each crack, and use the results to determine the appropriate repair strategy.

CRACK TYPE & LOCATION (SEE ABOVE)	1	2	3	4
1. Do these cracks penetrate to the interior surface of the pontoon?				
2. Will these cracks continue to grow in width, length, depth with time?				
3. Will these cracks experience autogenous healing?				
4. Have these cracks been repaired?				
5. Has the repair procedure closed these cracks?				
6. Will these repairs hold under service loads?				
7. Will these repairs hold under extreme loads?				
8. What is the crack width under service loads?				
9. What is the crack width under extreme loads?				
What are the leakage rates under service load and, extreme load?				

Table 4, Crack Type summary / issues checklist

Quantity of water related to crack width

The rate at which water flows through cracks in concrete is primarily a function of crack width and water head. The following table summarizes that relationship as reported in the report "Investigation of the Sinking of the I-90 Lacey V. Murrow Floating Bridge" by Wiss, Janney, Elstner Associates April 6, 1991.

CRACK WIDTH (INCHES)	DIFFERENTIAL HEAD (FT)					
	2	4	6	7	8	10
.01	0.58	1.44	2.29	3.00	3.15	4.00
1/32 (0.03)	15	26	33	35	38	43
1/16 (0.06)	27	40	49	53	57	65
1/8 (0.125)	61	97	123	134	144	162
1/4 (0.25)	113	179	226	247	265	300

Table 5: Flow Through Cracks of Various Widths (Gal/hr per inches of crack length)

The draft of the pontoons for the SR 520 bridge is projected to be of the order of 20 ft compared to a draft of about 10 ft. for the original LVM pontoons. Extrapolation of the flows listed in the foregoing table to a 20 ft. depth yield flows of about 9 gal/hr per inches for a crack width of 0.01 inches. That value corresponds to 1.2 cubic ft./hr and 28.8 cu. ft./day. The typical end cell of a SR 520 pontoon has an interior size of approximately 28 ft. by 17 ft. in plan and can therefore be expected to be filled with water to a depth of about ¾ inches in one day by a 0.01 in wide crack that is one inch long and extends through the depth of the keel slab. The criticality of properly sealing cracks of widths greater than 0.006 inches is obvious.

When there is water seepage through concrete cracks up to about 0.004 inches in width, the cracks are likely to self heal - a natural tendency of cracks in concrete rich with cementitious materials, as is the PCP concrete. The self-sealing process occurs under slow water infiltration which promotes the generation of calcium carbonate that will seal such cracks.

The crack sealing method used by WSDOT for cracks with widths up to 0.006 in has been previously found effective for water retaining structures. The method works in a similar manner to self-healing by having the crystalline waterproofing material move into the crack and seal it, as water attempts to flow into the crack.

Concern is sometimes expressed about the possibility of erosion of the concrete by water passing through cracks. The panel does not believe that this concern is cogent except for cracks that may open significantly under service loads. However, the length of time that extreme loads would be applied is short and even then the foregoing table of water flows suggest that water velocity through the cracks will be small unless the cracks width is large – i.e. 0.10 inches or more.

CRACKING, SR520 PONTOONS COMPARED TO OTHER WSDOT FLOATING BRIDGES

A comparison was made by WSDOT (see Appendix C for details) of the level (number and extent) of structural cracks ("cracks greater than 0.006 inches in width") and the number and length of non-structural cracks ("cracks less than 0.006 inches in width") observed for the Hood Canal Bridge pontoons (the concrete pontoon bridge most recently constructed) and for the PCP cycle 1 pontoons constructed in the Aberdeen casting basin:[20].

Aberdeen Cycle 1 compared to Hood Canal

It was found that the SR520 pontoons exhibited an 88.6% reduction in structural cracking and a 45.6% reduction in total cracking compared to the Hood Canal Bridge pontoons as tabulated in Appendix C (non post tensioned cracking). This is a significant overall improvement for the SR520 pontoons compared to the Hood Canal pontoons.

However, the average length of non-structural cracking in the SR520 cycle 1 pontoons was greater by a factor of 4 compared to the Hood Canal pontoons as shown in the following table below. This probably means that the concrete proportions, mix, transport, placement and thermal controls for the Aberdeen pontoons need to be reviewed to reduce non-structural cracking. Length of concrete pours is also a factor.

SR520 Cycle 1 Pontoons			Hood Canal Pontoons[21]		
Length of crack <.006 per 100 ft^2 of wall	Length of crack >=.006 per 100 ft^2 of wall	Total length of crack per 100 ft^2 wall	Length of crack <.006 per 100 ft^2 of wall	Length of crack >=.006 per 100 ft^2 of wall	Total length of crack per 100 ft^2 wall
8.2	2.3	10.5	0.4	13.1	13.5
7.0	2.5	9.6	0.3	9.4	9.7
5.1	0.2	5.3	2.1	12.4	14.4
2.2	0.1	2.3	2.1	11.9	14.0
			2.0	9.8	11.8
Averages			Averages		
5.6	1.3	6.9	1.4	11.3	12.7

Table 6 - Cracking, SR520 Cycle I pontoons from Aberdeen compared to Hood Canal Project

Aberdeen Cycle 1 Supplemental pontoons compared to those at CTC Tacoma

Appendix C of the WSDOT report also contains a summary comparison between the PCP cycle 1 supplemental pontoons in the Aberdeen casting basin and the FBL supplemental pontoons constructed at the CTC facility in Tacoma. The same mix proportions we are used for both construction sites but the material sources were different. The CTC work did not use thermal controls but kept the pour lengths to 30 feet. No thermal controls were used for the Aberdeen supplemental pontoons. No post tensioning was used in either group of supplemental pontoons.

[20] Data for other WSDOT floating bridges is not available at this time
[21] FBL – Floating Bridge and Landings Project

In general the CTC vs. Aberdeen supplemental pontoons are comparable for structural and non-structural cracking

			MEASURED CRACKING	
Location	Type	Name	Non-structural (lf)	Structural (lf)
Aberdeen	4	VNW	1,500	5.34
Aberdeen	4A	VSW	1,885*	10.08
CTC	6	VSE	1,488	15
CTC	6	VNE	967	5

* One precast panel included 535 lineal feet of non-structural cracking

Table 7 Comparative crack lengths, by type, Aberdeen vs. CTC sites

Summary / conclusions

1. Cracking for the SR520 cycle 1 pontoons is less severe, in general, than cracking experienced by the Hood Canal Bridge pontoons.
2. Cracking for the SR520 Aberdeen supplemental pontoons constructed by KG is comparable with that of the SR520 CTC pontoons constructed by KGM.
3. Non post-tensioning related concrete cracking for the SR520 cycle 1 pontoons is reportedly consistent with WSDOT's experience with the other Lake Washington floating bridges. This means that concrete cracking under service and extreme conditions is expected to be at a level no higher than has been experienced for other WSDOT floating bridges.
4. The cracks in other WSDOT floating bridges have been successfully sealed and maintained with little or no leakage into cells from such cracks. In the watertightness reports, discussed previously, water in the pontoons for these bridges has been suggested as coming primarily from hatches that are not completely watertight from condensation and from openings around cable tensioning galleries. An exception is that in several pontoons of the existing Evergreen Point Bridge, previously existing keel and side wall cracks did not close completely when the bridge was retrofitted by end-to-end post-tensioning.
5. Cracking has been, is being, or will be repaired using epoxy and crystalline techniques similar to those used for successful repairs made for other WSDOT floating bridges. (See Section 5 of this report following).
6. Following application of PT and repair of cracking, the panel expects that the SR520 replacement bridge will be comparable, or better, in terms of quality of construction, minor seeping, dampness and internal condensation than existing WSDOT floating bridges.
7. It is very unlikely that there will be any cracks that "open" or "work" during service loading such that water can penetrate the full depth of the wall or keel slab in any significant quantity.

5. CRACK REPAIR & MINIMIZATION STRATEGIES

Scope for this Section:

1. Review current WSDOT contract specifications for crack repair in pontoons. Review and comment on materials, procedures, and effectiveness of crack repair planned on the pontoons
2. Review and identify potential state-of the-art concrete crack procedures as they apply to pontoon crack repair, and comment/recommend if modifications to the current WSDOT crack repair strategies are needed.
3. Recommend appropriate actions to minimize future cracking of the structure.

Types of cracks, characteristics, causes

This section will address concrete cracking which can arise from one or more effects, as discussed previously and following. For the purposes of this section of the report, we will address the following cracking environments.

1. Restraint cracking of interior precast wall partitions. These cracks result from volume changes due to thermal cooling and other shrinkage that is restrained by adjacent or internal structures.
2. Restraint cracking of interior partition closure pours. These cracks result from volume changes due to thermal cooling and other shrinkage that is restrained by adjacent or internal structures.
3. Restraint cracking of the pontoon perimeter elements , These cracks are caused by thermal effects, drying shrinkage or concrete autogenous shrinkage.

Restraint cracking is expected cracking. It cannot be eliminated, but can be reduced by the use of several methods. WSDOT routinely and successfully seals such cracks which have appeared in all floating bridges. This cracking appeared in the longitudinal walls, keel and top slabs of all pontoons in cycle 1

4. Stress cracking caused by forces and/or stresses, including prestressing, but not classified as spalling or bursting cracking.

   Such cracking may be exacerbated by stresses arising from thermal effects, drying shrinkage or autogenous shrinkage but which stresses, by themselves, may or may not cause cracking. This was the cracking which appeared at :

   a) The end walls of the longitudinal pontoons and the cross-pontoon in cycle 1 and
   b) The bolt-beams adjacent to the keel and top slabs – with cracks which extended into the adjacent keel and deck slabs.

   This type of cracking can be reduced, but not totally eliminated, by the use of several methods as discussed elsewhere in this report..

5. Spalling or bursting cracking, caused by concentrated and/or unrestrained prestressing forces.

   This was the spalling caused by stressing the PT Tendons in Pontoon V on May 11, 2012. It was caused by the curvature of the PT tendon exerting insufficiently restrained forces on the keel slab, outside the bolt-beam.

With respect to these cracks, the ERP found that:

1. Restraint cracking caused solely by thermal effects, drying shrinkage or concrete autogenous shrinkage was expected, was consistent with the experience of cracking in other WSDOT floating bridges and could be sealed effectively using normal WSDOT and industry crack sealing techniques.

The WSDOT Bridge & Structures Office (BSO), designer of record for the pontoons, correctly anticipated this condition.

2. Cracking caused by forces and or stresses, including prestressing, but not spalling or bursting cracking was more than anticipated. This cracking was caused by uneven stress distribution from PT forces, plus the effect of deck slab shrinkage plus resistance from the interior pre-cast cell walls.

   This should have been anticipated by the BSO both with respect to cracks in the upper and lower bolt-beams which are difficult to seal completely (especially under the keel slab) and which will continue to open with time under the action of the prestress forces, as well as cracks in the field of the end walls which, for the cross-pontoons, are exposed to the lake in service. A reliable method of sealing these cracks is being investigated, but will cost substantial additional time and money over the initial (bid) contract sum.

   These cracks, forces and stresses are the subject of extensive design changes[22] including modifying the bolt-beam PT geometry and materials, adding rebar and transverse prestressing for cycles 2 thru 6 and retrofitting the cycle 1 pontoons with transverse prestressing, epoxy injection and, probably, use of crack sealing membranes.

   The WSDOT Secretary has advised the Governor and Legislature (House and Senate Transportation Committees) that costs due to WSDOT's design deficiencies are WSDOT's responsibility. These costs are currently being negotiated with the contractors.

3. Spalling or bursting caused by concentrated and/or unrestrained prestressing forces was unexpected and should have been avoided. The contractor and his subcontractors made changes to the PT geometry, PT tangent length and duct material in order to attempt to have the PT system meet industry guidelines. As a result, they requested that the tangent point of the PT tendon be extended 2'-6" outside the bolt-beam. WSDOT modified that request, but approved moving the tangent point of the PT 1'-9" outside the bolt-beam. See RFI[23] 111 for details[24].

   The WSDOT Secretary has advised the Governor and Legislature (House and Senate Transportation Committees) that costs due to WSDOT's design deficiencies are WSDOT's responsibility. Costs arising from contractor non-compliance with requirements are the responsibility of the contractor. These costs are currently being negotiated with the contractors.

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[22] Since the Bridge & Structures Office is the Engineer of Record for the pontoon design, WSDOT is responsible for changes and costs associated with this condition.

[23] RFI is a "Request for Information" normally used to clarify a design or specification requirement or circumstance. In this case, it was used inappropriately to modify contractual requirements.

[24] The documentation to move the tangent point outside the bolt-beam by use of an RFI is not consistent with WSDOT construction management procedures which require such a deviation from the M-11 drawings to be handled by a contract change order. See WSDOT "Performance Audit Report SR520 Design-Build Project", August 2012.

EVALUATION OF CRACK REPAIR PROCEDURES

The following cracking types by location were observed or have been reported for the cycle 1 pontoons. They are addressed in the following section in terms of crack repair procedures.

1. Restraint cracking of interior precast wall partitions. These cracks result from volume changes due to thermal cooling and other shrinkage that is restrained by adjacent structures or internal structures.
2. Restraint cracking of interior partition closure pours. These cracks result from volume changes due to thermal cooling and other shrinkage that is restrained by adjacent structures or internal structures.
3. Restraint cracking of the keel slab and exterior walls. These cracks also result from volume changes due to thermal cooling and other shrinkage that is restrained by adjacent structures or internal structures. It was intended to minimize these cracks by implementing a thermal control plan.
4. Stress cracking of the keel slab and end wall from post tensioning of tendons and post-tensioning shear lag effects for interior walls.
5. Spalling cracking of the Bolt Beam after Post-Tensioning (PT)

Cracks in Interior Precast Wall Panels

Inspection of the pontoons under construction showed that many of the precast panels exhibited edge cracking very similar to the cracking observed in the Kiewit-General PCP Contract mockup structure. The extent of cracking on the wall panels appeared to vary. However, the existence of cracks was more obvious when water was applied to the panel surfaces and allowed to dry. The cracks remain wet as the surrounding concrete surfaces dry.

The cracking of the precast concrete interior wall panels is consistent with cracking of the Kiewit-General mockup structure constructed in advance of the pontoon construction. It is not surprising that thin panels with a high degree of reinforcement exhibit a high degree of cracking.

It has been reported that thermal cracking of the ACME structures was less extensive than observed on some of the cycle 1 and 2 pontoons. Review of the ACME[25] photographs was inconclusive. If ACME thermal cracking was, in fact, less, a construction process cause is likely the cause. It appears the ACME construction used tilt-up construction processes where interior slabs were individually cast on the base slab. The SR520 PCP project utilizes the process of stack casting where subsequent panels are cast on top of previous panels. This practice could allow for the accumulation of heat in the panels followed by adverse cooling later.

Ultimately, the observed cracking of the precast concrete interior panels has little consequence on performance of the pontoon structure. In some cases the interior chambers may be flooded to provide ballast and the partition cracks are subject to hydraulic head and may potentially allow minor water leakage into adjacent chambers. The volume of possible leakage is expected to be inconsequential.

Conclusion:

The repair criterion established for the repair of cracks provides all the necessary assurance that such panel cracks are not detrimental. In addition, the natural tendency of cracks in a concrete rich with

[25] SR520 – ACME Project Final Findings Report, T.Y.Lin International, July 28, 2010, Document No. TYLI-002 Rev 00. The ACME (Advanced Construction Methods and Engineering) was a demonstration project to evaluate concrete mixtures and placement techniques for the Pontoon Construction Project to determine the most suitable mix and placement processes. The structure has been demolished.

cementitious materials, as these are, is for a self-sealing process to occur. Over a period of a few weeks or months, cracks subject to leakage should heal as water and air promotes the generation of calcium carbonate that can seal many cracks.

Cracks in Closure Pours Adjacent to Interior Precast Panels

The construction sequence is to place all the precast interior panels in a designated section. The panels are supported on the base slab form by integral concrete supports. Protruding reinforcing steel is tied to adjacent panel and wall steel forming continuous lapped steel corners. Up to four panels can lead to the intersections of the interior partitions. The intersection area is then formed and concrete placed (the closure pours). These full-height placements, conducted in lifts, are approximately 26 feet in height for the longitudinal pontoons, approximately 31 feet for the cross-pontoons, and approximately 5 feet in width. Horizontal cracking of the closure pour concrete occurs frequently and with regularity. The cracks can mirror the cracks existing in the precast panels. However placing conditions may create circumstances where more or less volume change and restraint occurs resulting in more or less cracking in the closure concrete.

Finding:

Cracking of interior wall panels and closure pours is not a critical element. These cracks are difficult to avoid given the geometry (concrete restraint and boundary conditions) and placement processes. Changes in placement processes for these elements are not necessary – however if simple measures can be effective, such as lower cementitious materials contents in the mixture or the use of shrinkage reduction admixtures are possible they should be considered.

Repair conclusions for these closure pours are the same as for the interior precast concrete wall panels.

Cracks in Exterior Walls and Keel and Deck Slabs, prior to post-tensioning

The keel slab is placed in advance of the exterior walls that are in turn completed in advance of the construction of the deck slab. These perimeter concrete elements are constructed in a sequence of successive longitudinal pours and all are ultimately post-tensioned. The many interior partitions described earlier are not post-tensioned although in longitudinal pontoons the interior longitudinal precast walls will become prestressed for most of their length once the longitudinal post-tensioning is applied. The exception is that part of the longitudinal interior wall between the end wall and the first interior transverse wall. The keel slab thickens near its intersection with the exterior walls and this thickened exterior concrete placement includes several steel pipes through which temperature-controlled water is pumped to control volume change while the concrete is curing.

Inspection of wall and keel slab cracking was difficult. Wall cracks were observable because of thorough marking of cracks, otherwise they would be difficult to detect in the low light conditions. Keel slab cracks were more difficult to observe because of water ponded in the invert of most of the cells. In most instances the crack widths are less than the specified structure crack threshold.

Cracking of the keel slab and adjacent exterior wall slabs prior to post-tensioning is the result of restraint conditions. The specified and developed techniques to minimize restraint were shown to be effective in minimizing cracking. It is very likely the excessive cracking is due to thermal conditions that were not well controlled for that placement. Keel slab cracking is likely the result of thermal conditions far outside of target parameters. This cracking should be controlled by better implementation of the thermal control plan.

Thermal cracking is not consistent in all pontoons. This provides good evidence that placing and temperature conditions were different for each of the documented pontoons and seems to be ample evidence that the thermal control plan has not been implemented consistently for all pontoon placements. See the paragraphs below for more on the thermal control plan.

Conclusion:

The repair criterion established for the repair of cracks provides the necessary assurance that these wall and keel slab cracks that develop before post-tensioning are not detrimental. Epoxy injection into the substantial cracks controls the overall length change that can occur when cracks close during post-tensioning. Significant leakage through these cracks has not been observed in the cycle 1 pontoons.

Cracking of the End Wall after Post-Tensioning (PT)

Review of crack mapping diagrams and photographs of the end walls and personal accounts indicate that cracking of the end walls was very minimal prior to PT and that the observed cracks occurred primarily after PT, with the understanding that the PT forces and stresses were additive to the thermal and shrinkage restraint stresses in the end wall. A remedy is being developed and implemented to adjust the post-tensioning process to reduce resulting stresses for future pontoon placement cycles.

The panel toured the Lake Washington (Medina) construction site where 3 cycle 1 pontoons were moored. With restricted access, it was only possible to observe the top deck cracking and cracking of part of the end walls and inside several of the interior cells. Contract specified repairs (epoxy injection and application of crystalline crack sealant) had been completed for each pontoon prior to their ocean tow from the Aberdeen casting basin. The extent of cracking of the end walls and other accessible surfaces has been thoroughly mapped (See prior Figure 4). Seepage of water through the cracks into the inspected cells was observed to be negligible and other cells were reported to be similar. Only one interior location in Pontoon V was observed where very minor seeping of water could be visually detected. The seepage was through to the inner surface of the bolt-beam about 4 ft in front of the end wall and about 2 ft to one side of the face of an interior wall . It appeared that most accumulation of water in the pontoon cells was due to condensation rather than seepage through cracks.

Conclusions:

A variety of repair procedures are required to restore these structural elements to full service. Some are adequately addressed by the specified treatments observed at the Medina site. Others require a repair currently under design by KGM / Gerwick which is being reviewed and discussed. The Gerwick recommendations, and concurrence by KGM and WSDOT, are not yet finalized. See following.

Cracking and Repair of the Bolt Beam after Post-Tensioning (PT)

Cracking into the bolt-beam level at the keel and deck slabs was observed as a result of the longitudinal post-tensioning of the longitudinal pontoons of cycle 1. The post-tensioning tendons were placed in groups of five or six symmetrically about the centerlines of the interior cells. Vertical cracking occurred along the length of the bolt-beam between the outermost tendon anchorages of adjacent tendon groups. Crack widths were greatest adjacent to the lines of the edges of the interior longitudinal walls. Cracks extended vertically to approximately the top of the taper on the bolt-beam at the interior face of the end wall (about 4 ft up the wall).

On the exterior faces of the keel and deck slab levels cracks typically extended back about 6 ft in the longitudinal direction and were often inclined in plan as a result of the shear lag effect caused by the interior precast walls. In several instances these cracks extended as much as 12 ft. in the longitudinal direction, extending them into the area where concrete had been temporarily removed to install the reinforcement necessary for the fix for the deviation point spalling.

While many of these cracks were repaired in accordance with WSDOT procedures before the pontoons left the Aberdeen casting basin, some were observed to have redeveloped and extended in length during inspections of the pontoons in Lake Washington. However, at the time of that inspection, none of those cracks were observed, as yet, to have penetrated through to the interior surfaces of the deck and keel slabs except as discussed in the prior section. Analyses by Parsons Brinckerhoff (PB) and SC Solutions have

demonstrated that due to creep and shrinkage effects these cracks can be expected to continue to grow over time if that growth is not effectively restrained by some active methods of repair.

Cracks similar to those described in the foregoing for the longitudinal pontoons also developed in end wall and the adjacent keel and deck slabs of the cross pontoon W as a result of its longitudinal post-tensioning and the presence of its interior longitudinal precast walls. However, the panel has been unable to fully review documentation on the extent of those cracks due to on-going construction involving Pontoon W on Lake Washington.

The panel, in conjunction with the WSDOT BSO and PB, have concluded that, to partially or fully close the vertical and longitudinal cracks in the bolt-beams (deck and keel slabs) the cycle 1 and 2 pontoons will require active repair with transverse post-tensioning placed immediately above the keel slab bolt-beams and immediately below the deck slab bolt-beams.

The design for this closure PT has been developed by PB and the WSDOT BSO. It is in the process of being finalized and implemented for cycle 1 (retrofit) and cycle 2 (installation in the casting basin).

The development of a repair process is ongoing to treat and seal keel slab cracks in the vicinity of the bolt-beam. The proposed process is considering epoxy sealing of cracks greater than 0.006 inches (before or after the transverse PT has been applied, the sequence is still in discussion) followed by the application of a bonded membrane - FRP (Fiber reinforced plastic) sheet or other material - to the concrete surfaces on the underside of the keel slab. Ben C. Gerwick, Inc. has been retained by KGM to design the repair and installation methodology. They are a reputable design firm with significant experience in underwater construction and concrete repairs.

Conclusions:

At the time of submitting this report (February 18 2013), insufficient information was available to review the KGM proposed repair process and provide meaningful comments. This will be fully addressed when the final KGM recommendations are received, following submittal (to KGM) of the Gerwick report and subsequent transmittal to WSDOT. Even with the planned improvements to the end walls, better concrete placement and thermal control, cracking in the end walls will probably still occur.

Comments on Contract Specified Crack-Repair Requirements

Structures of this type presume cracking and include provisions to accommodate the occurrence of cracks and seal them sufficiently for the specified long-term service life. In fact, the high degree of reinforcement in the structure is designed to not only carry service loads but to distribute volume changes in a manner that result in more cracks but at a reduced width. Repair requirements are a routine part of this work for similar concrete structures as distributed cracking is a routine occurrence.

For cracks less than 0.006 inches in width, the WSDOT specifications do not require treatment that reduces the crack width. The specifications require that an application of a surface seal be applied to these cracks. The surface sealant is intended to provide a continuing reaction that will further choke water passage and therefore allow the crack to seal with hydration products.

Cracks greater than 0.006 inches require treatment that ultimately reduces the crack width to below the specified threshold. The specified approach is to inject each crack using an epoxy injection repair process commonly used in the industry. The crack surfaces are sealed with a surface sealer that prevents extrusion of the injected epoxy. Injection ports are drilled into the crack and along the crack alignment at a specified spacing. A low viscosity epoxy material is sequentially injected into the crack, resulting in epoxy that fills the full crack width to the full depth. The goal is to perform this work after the majority of crack width development has occurred.

Epoxy injection

The SR520 PCP contract specification states:

> "*Structural cracks, defined as greater than or equal to 0.006 inches in width, at the discretion of WSDOT, shall either be repaired by epoxy injection at no additional expense to the WSDOT or may be cause for rejection of affected unit. Epoxy injection of structural cracks shall be done prior to any post tensioning of the pontoons*".

The epoxy injection process is designed to achieve full penetration of the concrete section and provide full bonding. Many structural elements, where bond across the crack must be reestablished, are repaired in this manner. For most situations, bonding of the crack faces is not required for effective repair. The epoxy must fill the space so subsequent post-tensioning will not result in crack closure in the concrete and relax the PT stress. The performance of this work can be done in either dry or wet conditions. Generally dry conditions are preferred since access is easier and conditions more stable. The more relevant factor is that for elements where PT is to be done, cracks must be filled prior to initiating the PT.

Currently it is reported that the contractor(s) are self-performing most of the epoxy injection repairs rather than subcontracting the work to specialty contractors. The repairs have been made using Sikadur 52. The material is a 2-part injection epoxy advertised for crack widths of 0.2 mm (0.008 inches) or greater. The viscosity varies more than 4 fold over a recommended temperature range of 5-30°C. There is some concern that the epoxy may not penetrate the full depth of the crack.

Conclusion:

1. Evaluate the epoxy injection process on the mockup. Core drill cracks to determine penetration. Change epoxy materials if full depth penetration is not observed. Repeat until successful.
2. Consideration should be given to the use of an epoxy that is known or demonstrated to be effective for crack widths as small as 0.006 inches rather than the current 0.008 inches. Further the effectiveness of full depth of injection for any epoxy used should be demonstrated for the actual placing conditions. Should full penetration not be observed, there are other injectable epoxies that may be better suited for the repairs.

Surface coating agent

The SR520 PCP contract specification states:

> *"Non-structural cracks, defined as less than 0.006 inches in width, shall be coated with an approved crystalline waterproofing agent with a brush-on or spray application."*

The Contractor submitted and received approval to use a product advertised as a blend of silane and siloxane to be trowel-applied similar to mortar. Two products were reviewed. Both appeared to utilize the same mechanism of sealing. While experience varies with the use of these products, the application of this product is not detrimental to the goal of further reducing the effect of cracks on the performance of the structure.

Conclusion:

1. There is a body of literature and testing that supports the beneficial use of this type of material. It appears that a certain amount of leakage can be controlled by this material provided the head pressures remain low (less than 30 ft.) and the crack movement is negligible.
2. No changes are recommended for the application of the crystalline surface coating sealant.

CRACKING REDUCTION ACTIONS – CONCRETE RELATED

Concrete Mixture Proportions

The panel also reviewed the concrete mixtures used on this project. The current mixture is reported to be equivalent proportions to those used on other similar projects. However the constituent materials are not the same.

A cursory review of strength performance indicates that the current mixture achieves compressive strengths significantly in excess of the required strength. Ultimate concrete strength is not the only consideration; it has been pointed out that time of set, early strength gain, and consequent form pressures are mixture parameters that may also affect construction schedule. It appears the concrete mixtures generate significant heat and that some benefit in reduced cracking could be realized by appropriately reducing the cement content.

When this subject was discussed with WSDOT at the construction site, the panel was advised that mixture changes were not being considered by WSDOT at that time for the cycle 2 concrete. This should be further discussed with WSDOT since the panel believes that incremental improvements can be made at small or no cost that will improve the performance of the mix, specifically reduction of shrinkage and associated thermal cracking for cycles 3 -6.

Conclusion:

1. Review mixture performance to evaluate and determine if a lower cement content in the concrete mix will improve the quality, and reduce cracking of the pontoons. The panel feels that this is an area of improvement that should be investigated – it has the potential to add value at low or no overall cost (better concrete crack control will reduce crack sealing costs).
2. Using lower strength mixtures for the lower exterior wall sections would be the most beneficial in reducing cracking due to thermal stresses. Exterior wall cracking would most benefit by this change while keel slab cracking may be least impacted by such a change.

Water/cement ratio

A previous ERP comment recommended that a minimum w/c ratio of 0.33 be required. Currently the requirement is that a maximum w/c ratio of 0.36 not be exceeded. While lower w/c ratios can be achieved and compressive strength can increase, lower w/c ratios result in an increasing amount of unhydrated cement. Stated simply, there is not enough water to fully react with all of the cement. Consequently less strength per unit of cement is achieved, rendering the process less and less efficient.

Furthermore, there is a limit to which the water content can be reduced. The specific surface area of the aggregate requires a certain amount of water to achieve workability and hydration. Admixtures influence this amount but do not replace this action. So to achieve lower w/c ratios, only the cement part of the w/c ratio can increase. That increases the paste volume and increases many of the negative aspects of mixture performance equivalent to increasing water content.

Conclusion:

1. The specification or agreement on a lower limit on w/c ratio would be is useful in preventing the negative results discussed above. The panel has been advised that the contractor understands this concern and is not now using a low w/c ratio.

Thermal Control Plan and Implementation

The thermal control plan is a comprehensive and detailed approach developed to minimize differential volume change between keel slab elements and the exterior walls. The overall goal is to reduce edge restraint to the wall placements by heating the keel slab to near the peak wall temperature during curing and control the cooling so both cool together. If adjoining structures have no differential volume change, no cracking restraint at the interface can occur.

The challenge is in coordinating activities preceding, during and after wall placement so that target temperatures are achieved and rates of heat gain and loss are controlled. It appears from the data collected and conclusions drawn in the ACME report, that the specified processes can be successful in minimizing wall cracks. Conversely, less controlled conditions can lead to wall cracks and may in some cases cause keel slab cracks. The panel was unable to assess actual conditions experienced by the pontoons during their construction because the data that was available could not be correlated with cracking conditions due to a lack of information on the precise location of temperature sensors.

The Contractor is responsible for preparing and submitting a Concrete Thermal Monitoring and Control Plan (Section 2.14.5.2.5). This requirement is further discussed in Section 2.14.5.2.9, where the contractor is told "Concrete curing and thermal control methods in accordance with Contract 7812 (ACME) may be used in lieu of the following procedures." KG has chosen to base their thermal control plan on the ACME project. The most recent submittal is dated Nov. 2011.

Anecdotally the panel has been told by the Aberdeen Project Staff that in several instances during cycle 1 pours, the thermal control plan was not fully implemented. Following these instances, we were advised that the contractor has reemphasized efforts to comply with the thermal control plan. In cycle 2, it has been reported that non structural cracking has occurred in various areas of the pontoons, typical to what might be seen from thermal effects during the concrete placing and curing process. This is an area of concern to the project, and steps are being taken to further investigate the effectiveness of the contractor's thermal control plan and its implementation. Discussions with the construction office indicate that thermal controls on cycle 2 are now in conformance with the contract requirements.

Recommendations:

1. Review the temperature controls and the temperature records for specific placements to confirm if cracking was the result of improper thermal control plan implementation.
2. The thermal control on site should be monitored to better control concrete temperatures. In particular, sensor locations and related control of the heating water supply should be reviewed and adjusted.
3. The thermal control plan needs to be implemented consistently for all pontoon placements, in conformance with contract documents. If cracking typical of thermal impacts continue to occur, the thermal control plan must be revisited to determine what steps should be taken to improve performance of the thermal controls.

Time-dependent effects – current contract requirement

Section 2.14.1.1.1 Option A, requires the Design Builder to "Design for time-dependant effects associated with the construction sequence. This includes thermal, shrinkage, elastic shortening, and the design of the thermal control/concrete cure system."

This section also refers to the components of time dependant effects, saying "Development of a geometry control plan to account for construction tolerances, construction sequence, thermal movements, elastic shortening, and shrinkage."

This section of the contract gives the contractor the option of building pontoons based on the M-11 drawings as the "minimum requirements for the design of the design of pontoons in this contract." The contractor chose to use the M-11 drawings as the basis of design. There was no construction sequence specified in the contract documents for the pontoons and the contractor has elected to use a sequence similar in most areas to what was used in the ACME demonstration project. This approach, utilizing the ACME sequencing concepts, and using the time dependant effects components to develop the geometry control plan, has been determined by the project construction office to be in accordance with the contract requirements for considering time dependant effects.

It is possible that improvements could be made in this area – this should be further discussed within WSDOT and the contractor to determine the benefit of changes related to cost and schedule trade-offs.

CRACK REDUCTION ACTIONS – STRUCTURALLY RELATED

Changes to bolt-beam PT geometry and reinforcement

The cycle 1 post tensioning profile changes, which resulted in curvature of the post tensioning outside of the bolt-beam area, have been documented as the reason for the original spalling observed in Pontoon V. This was corrected in Pontoons V, U, T, and W by adding reinforcing steel and additional depth of concrete in those areas of the cycle 1 pontoons before float out. However, upon post tensioning these pontoons, other PT related cracks appeared, primarily in their end walls. In order to attempt to control this end wall cracking, several changes have occurred in cycle 2 Pontoons.

Recommendation

Implement PT tendon modifications to eliminate or control the keel slab spalling and to reduce the end wall cracking associated with overstressing concrete around the PT anchorages experienced in cycle 1.

Changes

First, the post tensioning profile in the cycle 2 pontoons was moved back so all of the tendon curvature is within the bolt-beam. "Hat bars" (bent reinforcing steel) were added over all ducts beyond the end of the bolt-beam to ensure that PT deviation forces, if existent for any reason beyond the bolt-beam, would be resisted by the reinforcing instead of resulting in spalling. "Strong backs" or reinforcing bar bent to the specific design curvature for the post tensioning tendon ducts were added to all ducts to help hold the ducts in the correct location.

Based upon the panel's input, other steps were taken to reduce stresses in the cycle 2 end walls that contributed to the observed cracking. The post tensioning anchorages have been moved toward the perimeter of the pontoon by approximately 6 inches, reducing their angle of placement and the residual tension in the end wall concrete. Additional reinforcing bars have been added in key areas, between anchorages, to help control the structural cracks that appeared in this area after post tensioning in cycle 1.

Addition of Transverse Post-Tensioning cycles 1 & 2

Finite Element Modeling (FEM), recommended by the panel for analysis of the end walls and bolt-beams, resulted in identifying significant stresses that explained the structural cracks between post tensioning anchorages in the longitudinal and cross pontoons. The panel and the project team have evaluated several possible strategies to control this cracking. The result is a strategy, currently being implemented, to add transverse post tensioning to the upper and lower bolt-beams in the ends of the pontoons and to apply this transverse post-tensioning prior to the application of the longitudinal post-tensioning. The transverse post-tensioning effectively strengthens the bolt-beam by reducing tensile forces and stresses. For the cycle 2 pontoons and the pontoons of cycles 3-6, the magnitude and location of the transverse post-tensioning is designed to reduce the tension stress levels in the extreme fibers of the bolt-beams for the keel and deck slabs, caused by longitudinal post-tensioning, to values less than that which would initiate concrete

cracking. For the cycle 1 pontoons, where bolt-beam cracking has already occurred, the transverse post-tensioning should, in large measure, close the existing cracks and effectively eliminate the possibility of their growth with time. The use of transverse post-tensioning is a method of "actively" controlling crack formation and growth. By contrast, fiber sheets placed over existing cracks, while important for improving watertightness, are only a "passive" method of controlling cracks because the crack must attempt to increase in width in order to stress the fiber sheet.

Recommendation

Implement the PT external transverse tendon modifications to repair the cycle 1 and 2 pontoons.

Commentary/Status

This is a proven method to control cracking and to provide additional strength to concrete structures. Because the cycle 1 and 2 pontoons are already cast, it has been necessary to develop post tensioning strategies appropriate for the cycle 1 pontoons already floating, and cycle 2 pontoons, already cast but currently in the Aberdeen casting basin. This strategy is currently in design, with the intent of adding it to all cycle 1 pontoons currently floating, and to all cycle 2 pontoons before float out occurs.

Addition of Transverse Post-Tensioning cycles 3-6

Planning for remaining cycles of the pontoons includes the addition of internal transverse post tensioning in the bolt-beam areas of all longitudinal pontoons.

Commentary/Status

The transverse PT designs are being completed and the project team is working with the contractor to verify the design and installation procedures. It is anticipated that all longitudinal pontoons of Aberdeen cycles 3-6 will have transverse PT added to the end wall bolt-beams, which will minimize cracking in this area of the end walls.

6. MAINTENANCE CONSIDERATIONS

Scope for this section

1. Work with the STATE to jointly develop a revised estimate for Operations and Maintenance (O&M) costs over the service life of the Floating Bridge, considering the State's rating for the cycle 1 pontoons, and extrapolating to consider the entire Floating Bridge.
2. Compare the baseline estimate (to be provided by the WSDOT) with the revised estimate.

Activities/Investigations

The following activities were undertaken by the panel for this task:

1. Interviewed Dave Bruce, Bridge Preservation Office.
2. Interviewed Archie Allen, Northwest Region Bridge Supervisor.
3. Reviewed the proposed O&M budget for the new SR520 floating bridge.
4. Reviewed the Operations, Inspection and Maintenance Manual for the existing SR520 floating Bridge.
5. Reviewed the following Watertight Inspection Reports.
    a. Evergreen Point Floating Bridge dated Oct. 2011.
    b. Lacey V. Murrow Floating Bridge dated March 2012.
    c. Homer Hadley Floating Bridge dated March 2012.
    d. Hood Canal Floating Bridge dated June 2012.
6. Reviewed the Floating Bridge inspection criteria for condition rating, from the Washington State Bridge Inspection Manual M 36-64.03, dated November 2012.
7. Visited Medina construction site and viewed pontoons U, V (internal inspection) and W (external inspection) to consider status of cracking, and effectiveness of repairs.

Findings

The current maintenance estimate for the new SR520 Floating Bridge is established based on the experience of the maintenance activities on the existing bridge. For the first biennium after construction, (2015-2017) it is estimated the O&M budget is approximately $515,000. The budget estimate does not provide specific details for energy costs for example, so the panel is not able to make a recommendation on any adjustment that may be appropriate for any increase in energy usage.

The estimate also does not identify any future cost for crack repair. The existing bridge did experience significant leakage requiring pumping of several cells at least 3 time a week (per conversation with Archie Allen) prior to the 1997 crack repair contract (bid price of $2.2 million) and external longitudinal post tensioning contract in 1999 (bid price of $8 million). After those two contracts, the leakage through cracks was reported to be very manageable and in line with the leakage experienced by the other floating bridges. Section 4 summarizes leakage currently being experienced on the existing SR 520 bridge, as well as other floating bridges. The majority of leakage occurring now has to do with water coming through hatches that do not quite seal properly, through the anchor cable ports during storm conditions and in the vicinity of the drawspan.

At the site investigations of Pontoons V and U the panel observed the crack repairs completed on those pontoons. In general the repairs on the walls appeared to be successful and had sealed the cracks. The cracks were dry to the touch with no indication of weeping or seeping.

The exception was the cracks on the top of the bolt-beams adjacent to the interior walls that have not yet been repaired, in pontoons V and U. These cracks were moist, with water stains extending down the bolt-beam slope from the crack locations. These cracks appear in the general vicinity of cracks mapped on the underside of the keel slab by divers. Investigations are proceeding on the appropriate method of crack sealing/repair in these areas.

Conclusions

The existing SR520 Floating Bridge experienced significant leaking cracks prior to crack sealing and post tensioning repairs made in the late 1990's. Following those repairs, water leaking into the pontoons has been manageable, and no additional repairs have been needed. Any water accumulation in the pontoons has been associated with hatch leakage or anchor cable well overflow, and is similar in magnitude (i.e. insignificant) to other floating bridges.

The current O&M budget estimate for the new floating bridge does not include any specific line item for crack or other repairs.

The wall cracks previously repaired in accordance with the contract specifications have sealed the cracks with no additional moisture appearing on the inside of the pontoons. It is possible that the repaired cracks may grow over time, or these and other cracks may appear and "work" as the new bridge undergoes storm events. After storm events, the cracks are anticipated to close, but both situations may result in occasional, intermittent intrusion of small amounts of water that will need to be monitored, in accordance with Operations and Maintenance manuals and normal WSDOT practice.

The cracks in the bolt-beam are moist and have very minor amounts of water seeping into the pontoons. Unrepaired, the current water volume is insignificant in comparison to the volumes seen in the existing SR520 floating bridge before repairs. If repaired successfully, it is expected that water intrusion into the new pontoons will be occasional and will not require any modification of the Operation and Maintenance procedures, or budget estimate. If however, the crack sealing in the keel slab at the bolt-beam area is not accomplished or is not effective, or existing cracks grow or work, it should be expected to continue to see minor seeping into the pontoons. In order to minimize the risk of corroding the reinforcing steel, additional efforts of crack sealing over the life of the structure should be planned and budgeted.

APPENDIX A

Expert Panel, Specific Scope for Phase 2

1. Review and Assessment of Structural Sufficiency

    a. Review BSO design calculation notebooks

    b. Review results of Independent Pontoon FEM Analysis

    c. Based on the above, prepare a statement or assessment of pontoon sufficiency for the SR520 Floating Bridge

2. Quality of As-Constructed Pontoons

    a. Review and comment on WSDOT Floating Bridge Operation, Inspection and Maintenance Manual as a tool to accurately determine the sufficiency of the pontoons

    b. Coordinate a rating inspection by WSDOT BPO utilizing appropriate criteria from the Maintenance manual

    c. Compare the results of the inspection and rating survey with results of similar surveys of other floating bridges in the WSDOT system, in order to prepare a statement as to the quality of the SR520 Pontoons in comparison with pontoons on other bridges.

3. Crack Repair Strategies

    a. Review current WSDOT contract specifications for crack repair in pontoons. Review and comment on materials, procedures, and effectiveness of crack repair planned on the pontoons

    b. Review and identify potential state-of the-art concrete crack procedures as they apply to pontoon crack repair, and comment/recommend if modifications to the current WSDOT crack repair strategies are needed.

4. Maintenance Considerations

    a. Review the existing O&M estimate for the SR520 Floating Bridge. Work with WSDOT to compare pontoon conditions of the new SR520 pontoons, and other floating bridge pontoons, and how that might indicate if the current O&M estimates are appropriate or need modification to reflect the as built condition of the new pontoons.

    b. Compare any new O&M estimate to the baseline estimate for evaluation by WSDOT

Related SR520 Pontoon Construction Review Activities

The following work elements are also in process, resulting from the initial Expert Panel recommendations and on-going engineering and construction needs which have been identified. These activities are being managed and coordinated by the SR520 program with the participation, review and recommendations of the Panel as appropriate.

Independent Pontoon Analysis Activities

1. Cycle 2, Pontoon Type 1 Time Dependant Analysis
    a. Prepare FEM of Type 1 Pontoon to include time dependant effects of sequencing, shrinkage, thermal and post tensioning, for two conditions, with and without planned decoupling and closure pours.
    b. Prepare a report documenting results of the FEM.
2. Cycle 2, Pontoon Type 3 Time Dependant Analysis
    a. Prepare FEM of Type 3 Pontoon to include time dependant effects of sequencing, shrinkage, thermal and post tensioning, for two conditions, with and without planned decoupling and closure pours.
    b. Prepare a report documenting results of the FEM.
3. Independent Pontoon Design Review
    a. Analysis of top slab, end walls, side walls and keel slabs and bolt-beam for service load cases
    b. Analysis of overall pontoon cross-section, joints, and anchorage forces for service and extreme load case moments and shears
    c. Prepare a Report of Findings

Concrete Quality Investigation (proposed)

1. Specifications Review
    a. Review of LVM mix, ACME Report, and PCP concrete specifications to determine opinion on sufficiency of specifications for intended use and stated goals of PCP project
    b. If warranted, recommend possible changes in mix and possible trial mix and testing program to evaluate suggested changes
2. PCP Production Concrete Evaluation and Performance Review
    c. Review of available data for production concrete at PCP, including mix approval documents, constituent materials characteristics, curing techniques, thermal control plan, and field observations of cracking patterns and mix, placement, and cure procedures, identify and comment on possible reasons for cracking observed in PCP pontoons.
    d. In consideration of both above tasks, recommend any additional testing, forensic analysis, core sampling or other systematic evaluation to help determine why cracking in pontoons exceeded the expected level of cracking, and if modifications to mix are appropriate to improve concrete performance.
3. Prepare Report
    e. Prepare and present findings from investigation activities, identifying if possible probable reasons for observed cracking and recommendations for changes in materials and procedures that will improve the cracking performance of the pontoon concrete.

APPENDIX B

SR520 PONTOONS, CONCRETE INVESTIGATION RECOMMENDATION

The following memorandum proposed elements to be investigated related to improving the performance of the SR520 concrete including its composition, mixing, placing, curing and thermal control, for pontoons in cycles 3 and beyond.

Some elements of this work and recommendation are in process of being implemented, but the full scope of the Technical Memorandum has not been addressed at this time.

Action: Discuss with WSDOT to address the cost/performance/benefits of this proposal in order to determine actions required.

Technical Memorandum

To: Julie Meredith, SR520 Program Director
From: John Reilly and Larry Kyle
Cc: Mike Cotten, Tom Horkan
Date: October 8th, 2012
Re: SR 520 Pontoons, Concrete Cracking Investigation – recommendation to proceed

SUMMARY:

The first cycle of pontoons experienced unexpected cracking and spalling[26,27] that exceeded the desired goals of reduced or no cracking and no spalling. Accordingly, Julie Meredith, SR520 Project Director, convened an Expert Panel to address the causes of the spalling and cracking and to make recommendations.

The Panel reported their findings and recommendations in their report of August 17th, 2012.

The panel found that spalling resulted from the applied prestressing and that concrete cracking related to the length of pour, the concrete mix and many other factors specific to the composition of the concrete, its production and placement. They recommended changes to design and construction procedures and further investigation of concrete in the areas of materials, composition, mixing, transport, placement, thermal controls and curing.

This memorandum[28] addresses specific areas which should be investigated to determine those factors which led to the increased cracking in the cycle 1 pontoons and which, if addressed, will reduce cracking in cycle 2 and subsequent cycles.

Recommended by : The Expert Review Panel, in consultation with Tom Baker

[26] We use ”cracking” here to refer to cracks which are due to thermal, drying, or autogenous concrete shrinkage and will use “spalling” to indicate cracking and spalling due to prestressing or other forces.

[27] See crack maps produced by the contractor, available from WSDOT site staff.

[28] The full memorandum details specifics of the proposed work. It can be made available if needed.

APPENDIX C – CONCRETE CRACKING COMPARISONS

SR520 compared to Hood Canal

Comparisons were made for non-prestressed cracking experienced for the SR520 cycle 1 pontoons in Aberdeen and the most recent Hood Canal replacement pontoons. The results are as follows:

SR 520 Pontoon Construction Project
Summary of SR520 Cycle 1 Pontoon Cracking compared to Hood Canal
Non-post-tensioning cracking only

Data for SR520 Cycle 1 Pontoons

Pontoon	Pontoon Type	Pontoon Length (ft)	Pontoon Width (ft)	Pontoon Height (ft)	Length of crack <0.006 (ft)	Length of crack >=.006 (ft)	Total crack length (ft)	# Panels > 30 FT structural crack	Exterior wall concrete surface area (ft^2)	Length of crack <.006 per 100 ft^2 of wall	Length of crack >=.006 per 100 ft^2 of wall	Total lengthof crack per 100 ft^2 wall
T	1	360	75	29	2032	575	2607	6	24781	8.2	2.3	10.5
U	1	360	75	29	1741	629	2370	8	24781	7.0	2.5	9.6
V	1	360	75	29	1255	53	1308	0	24781	5.1	0.2	5.3
W	3	240	75	33	457	20	477	0	20790	2.2	0.1	2.3
									Averages	5.6	1.3	6.9

Data for Hood Canal Bridge Pontoons

Pontoon	Pontoon Type	Pontoon Length (ft)	Pontoon Width (ft)	Pontoon Height (ft)	Length of crack <0.006 (ft)	Length of crack >=.006 (ft)	Total crack length (ft)	# Panels > 30 FT structural crack	Exterior wall concrete surface area (ft^2)	Length of crack <.006 per 100 ft^2 of wall	Length of crack >=.006 per 100 ft^2 of wall	Total lengthof crack per 100 ft^2 wall
PA	N/A	312	40	21	66	1937	2003	N/A	14784	0.4	13.1	13.5
PB	N/A	312	40	21	43	1388	1431	N/A	14784	0.3	9.4	9.7
NA	N/A	193.5	40	21	204	1213	1417	N/A	9807	2.1	12.4	14.4
NB	N/A	193.5	40	21	208	1166	1374	N/A	9807	2.1	11.9	14.0
W	N/A	325	60	18	281	1356	1637	N/A	13860	2.0	9.8	11.8
									Averages	1.4	11.3	12.7

Table 7 – Crack lengths, Aberdeen Site compared to Hood Canal Project

Reduction in cracks >=.006 from Hood Canal to SR520	**88.6%**
Reduction in total cracks from Hood Canal to SR520	**45.6%**

Compare and Contrast CTC Flankers to Aberdeen Flankers

A. cycle 1 SSP Construction

Aberdeen: cycle 1: 2 SSP's, VNW (type 4), VSW (type 4A)
CTC: cycle 1: 6 SSP's, VNE (type 6), VSE (type 6), UNE (type 5), USE (type 5), RNE (type 2), RSE (type 2)

B. Differences in Design

The most directly comparable pontoon types created at the two sites is the two Type 4 pontoons built at the Aberdeen site in cycle 1 with the two Type 6 pontoons built at the Concrete Tech facility at the Port of Tacoma. The Type 4 and Type 6 pontoons are dimensionally the same – 98'2" long, 60' wide and the depth of the pontoons ranges from 28'6" at the joining side to 27'9" at the water side of the pontoons. The differences accounts for the cross slope of the pontoon for drainage.

A design difference is the Type 4's from Aberdeen include anchor galleries for anchorage to the lake bottom. The Type 6's do not include anchor galleries and are not intended to be anchored to the lake bottom.

Aberdeen : Type 4 - 60 feet wide supplemental stability pontoons without center drainage wells and with anchor galleries. Dimensions - 98'2" x 60' x (28'6" to 27'9")

CTC: Type 6 – 60 feet wide supplemental stability pontoons without center drainage wells and without anchor galleries. Dimensions - 98'2" x 60' x (28'6" to 27'9") with anchor galleries

C. Differences in Construction (including means and methods between the two projects)

In cycle 1 in Aberdeen and cycle 1 at CTC the concrete mixes were the same, although sources of materials were different. The difference in the construction of the cycle 1 Type 4's from Aberdeen and the cycle 1 Type 6's from CTC is that in Aberdeen K-G made use of precast panels for some of the interior walls, longer pour lengths and managing the pour lengths by the use of thermal control in the pouring sequences. K-G-M at the CTC site did all the work cast in place, did not use thermal control and kept pour lengths to 30 ft.

D. Compare – Cracking Data

- 100% comparison is difficult
- Anchor Gallery presents some differences
- Degree of accuracy of measurement
- In some cases not complete data for every cell
- However, in general the Pontoons seem comparable for structural and non-structural cracking
- The pontoons have approximately 51,000 sf of surface area (not including bottom of keel slab). Surface area includes flooring, walls, roof, top deck and all exterior walls.

			MEASURED CRACKING	
Location	Type	Name	Non-structural (lf)	Structural (lf)
Aberdeen	4	VNW	1,500	5.34
Aberdeen	4A	VSW	1,885*	10.08
CTC	6	VSE	1,488	15
CTC	6	VNE	967	5

* One precast panel included 535 lineal feet of non-structural cracking

Table 8 Comparative crack lengths, by type, CTC vs. Aberdeen sites

APPENDIX D - AREAS ADDRESSED, ERP PHASE 1

REPORT OF THE ERP, PHASE 1 MEMORANDUM

To	Julie Meredith, SR 520 Program Director	
From	John Reilly, Chair SR 520 Pontoon Expert Panel	
cc:	Panel Members:	Neil Hawkins, Tom Sherman, John Clark,
	SR520	Larry Kyle, Mike Cotten, Tom Horkan, Dave Ziegler
Date:	August 17, 2012	
Re:	SR520, Expert Panel Recommendations	

Thank you for the opportunity to convene the SR 520 Pontoon Expert Panel in order to offer an independent opinion on the most likely cause of the concrete cracking and spalling in the SR 520 floating bridge pontoons, discovered May 11, 2012 in one of the Cycle 1 pontoons being constructed in Aberdeen, WA.

The Panel members - Dr. Neil Hawkins, Mr. Tom Sherman, and Dr. John Clark - have worked closely and consistently since their appointment in June to investigate the cracking, determine probable causes and to recommend changes which could better limit the extent of future cracking.

Background:

On May 11, 2012, unexpected spalling and cracking of concrete occurred inside Pontoon V, one of the longitudinal pontoons constructed in Cycle 1. The cracking was discovered after the pontoon's longitudinal post-tensioning was complete. WSDOT, working with the contractor, Kiewit-General, immediately developed a repair and modification procedure to apply to all three longitudinal Type 1 pontoons in Cycle 1. The Panel considered the additional concrete and steel reinforcement specified by the WSDOT Bridge and Structures Office and concluded: "…the added bars were adequate to effectively fulfill the AASHTO requirements as demonstrated by their successful performance when the pontoons were subsequently again post-tensioned."

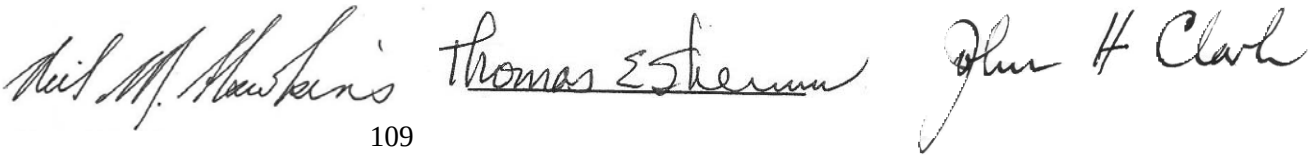
The following document lists the essential recommendations of the Panel which, we understand, are being addressed by WSDOT. We understand that WSDOT and the contractor are working to identify and implement specific changes related to these recommendations, and are addressing those construction, materials, processes and procedures necessary to minimize cracking of the pontoons for construction Cycle 2 (currently in preparation). More refinements are anticipated for Cycle 3 and beyond. Changes for Cycle 2 and subsequent cycles are the subject of a design analysis that is under way, as recommended by the Panel, including an independent review of the current design to identify and evaluate potential design changes.

Please call to discuss the specifics of the Panel recommendations if and as necessary. The Panel is currently monitoring the design analysis work in process and will review changes for Cycle 2 and beyond. They stand ready to assist as possible within their remit.

Yours sincerely,

John Reilly, Chair

Panel Members:



Dr. Neil Hawkins Mr. Tom Sherman Dr. John Clark

SR 520 Pontoon Construction Project, Summary of Expert Panel results

The SR 520 Bridge Replacement and HOV Program requires large concrete pontoons built to withstand the loadings of launch, transport, assembly and to support the SR 520 Evergreen Point Floating Bridge on Lake Washington. During construction of the pontoons in the casting basin in Aberdeen, WA, unexpected cracking and spalling of concrete was observed in one of the Cycle 1 pontoons. As a result, a panel of concrete and construction experts was convened to conduct an expert review of the design and construction process for the pontoons – related to the observed cracking and spalling.

Panel Member's - Background

The Expert Panel members were selected for their expertise in the design and construction of complex concrete and pre-stressed/post-tensioned[29] concrete structures. More than one member was required to have experience in floating bridges. An understanding of concrete materials and technologies was required.

Panel members are Neil M. Hawkins, Ph.D., Dist. M. ASCE, Professor Emeritus, University of Illinois, and former chair of the Civil Engineering Department, University of Washington; Tom Sherman, specialist in floating bridge design and construction, TES Enterprises; and John H. Clark, P.E., Ph. D., a consultant in long-span bridges and heavy structures. The panel is chaired by John Reilly, P.E., C.P. Eng., with experience in management, risk assessment and pre-stressed concrete.

Scope of the Panel's work

Determine, as feasible:

1. The most likely cause of concrete cracking and spalling in the SR 520 floating bridge pontoons being constructed at the casting basin in Aberdeen, WA.
2. The need for, and character of, potential changes to pontoon design, details, and/or construction methods to avoid similar concrete cracking and/or spalling in future concrete pontoon construction cycles. Design and construction changes are the responsibility of WSDOT and/or the contractor.
3. In coordination with WSDOT, identify and present considerations regarding the ability of the as-constructed pontoons to be repaired to a condition that will maximize their service life as an integral part of the completed new SR 520 floating bridge.

Responsibility for pontoon design, construction, towing and assembly:

The design of all SR 520 pontoons, in terms of their final configuration and performance in-service on the lake, is the responsibility of WSDOT. The Contractor is responsible for specific design elements of the pontoons including but not limited to, the design of the post tensioning systems, pontoon access and walkway systems. Design and construction of the casting basin and the physical construction of the

[29] Prestressing refers to beneficial stressing of the concrete structures before application of in-service loads. Post-tensioning is the process of applying that prestressing after the concrete has been placed and hardened.

pontoons in the Aberdeen facility are the responsibility of the Contractor, Kiewit-General. The Contractor is responsible for Quality Control (QC) and Quality Assurance (QA) for the entire project.

Other related contracts, such as the Floating Bridge and Landings (FB&L) contract affect decisions related to towing, construction and assembly of the SR 520 floating bridge. Design of elements such as the road deck superstructure – including construction and integration with the pontoons – is the responsibility of the FB&L contractor (Kiewit/General/Manson, A Joint Venture) which also has responsibility for construction of anchors, the remaining pontoons at the Tacoma facility, and towing and assembly of all pontoons on Lake Washington.

Summary - causes of cracking and spalling

The Panel found the following basic causes for the cracking and spalling of the Cycle 1 pontoons:

1. The placement and location of the longitudinal post-tensioning ducts and tendons for the Type 1 pontoons deviated from the contract drawings to such an extent that the tendon forces caused cracking and spalling of the slabs adjacent to the end bolt-beam.
2. Resistance to longitudinal post-tensioning from the interior precast concrete walls caused vertical cracking of the bolt-beam, adjacent to these precast walls.
3. End walls experienced a combination of thermal and autogenous[30] concrete shrinkage, radial tension stresses from the post-tensioning end anchorages and forces from the longitudinal post-tensioning which led to cracking of these walls. Additionally, construction access block-outs in these end walls contributed to the cracking.
4. Contract requirements for concrete curing and thermal control were not rigorously followed resulting in more extensive thermal and shrinkage cracking.
5. For some concrete, water/cementatious (w/c) ratios were lower than those recommended following the ACME project. The ACME project was a test project developed prior to pontoon construction that allowed WSDOT to test mix designs for strength and durability, test form methods for efficiency and to expedite pontoon construction. Procedures for control of the w/c ratio on site (e.g. moisture measurements of aggregates, water added at the site) did not appear to be sufficient.
6. The long pour length for some longitudinal walls (133.9 feet in some cases) was a major cause of the extent of shrinkage and associated vertical cracking in these walls. Adverse cement and concrete properties, curing and thermal control issues potentially added to this cracking.

[30] Autogenous shrinkage is a volume change when there is no moisture transfer to the external environment. It is therefore different than drying shrinkage and most prevalent in high performance concrete where the water-cement ratio (w/c) is under approximately 0.32. The w/c ratio for the Aberdeen pontoons was as low as 0.28.

SUMMARY - RECOMMENDATIONS

Cycle 1 Pontoon repairs (completed)

1. **Longitudinal Pontoons T, U and V:**
   a. The Panel concurs with the WSDOT Bridge and Structures Office that repairs made to the bolt-beam/slab spalling and pulled-out post-tensioning ducts are adequate for structural capacity. Pontoons T, U and V were successfully re-tensioned longitudinally after this repair.
2. **Cross Pontoon W**

Future work noted in this section applies to the design and construction of the second cross-pontoon (A).

   a. Longitudinal tendon re-tensioning was completed. Some additional cracking was observed at four locations and was repaired. The cause of this cracking needs to be understood and measures taken.
   b. Repairs to spalling at the bolt-beam transverse post-tensioning were completed. The spalling in this area appears to have occurred even where the post-tension tendons were reported to be in the correct location although significant local deviations were noted. The cause of this cracking needs to be understood and measures taken.
   c. External wall cracking (at the bolt-beam location) is a cause for concern. Cracking is also evident at the end wall which will be exposed to the lake in the in-service condition. The cause of this cracking needs to be understood and measures taken.
   d. Issues relating to the bolt-beam and associated transverse prestressing, plus the effect of deck hatch locations, need to be understood and measures taken.
3. **Repair of all cracks** per contract procedures is reported to have been completed. These repairs follow WSDOT's practice for floating bridges, which have been successfully implemented on previous floating bridge applications.
4. **Issues related to the Pontoon's service life:** The repair areas should be inspected throughout the towing and assembly on Lake Washington. Special consideration and attention should be given to these areas according to normal WSDOT maintenance and operation procedures throughout the life of the SR 520 floating bridge. Under these circumstances, it is expected that the service life of the bridge can be met.

Cycle 2 and Future Pontoons:

1. **Verify that contract requirements are adhered to with special emphasis on the following elements:**
   a. Maintain the water/concrete (w/c) ratio in the range of 0.33 - 0.36. This requires control of water at the batch plant, water added at the site and moisture control of the aggregates. (The contract did not specify a lower limit for the w/c ratio).
   b. Thermal control plan must be followed per contract requirements.
   c. Concrete curing requirements must be followed per contract requirements.
   d. Released for Construction (RFC) drawings are to be issued by the contractor and reviewed by WSDOT before construction in the subject areas commences. In this regard, given the number of changes experienced in Cycle 1 and which are also expected for Cycle 2 and subsequent cycles, a reliable process to allow the required Quality Control (QC) and Quality Assurance (QA) by the contractor and the associated Quality Verification (QV) by WSDOT must be in place and enforced. The Panel was advised that a process with this intent was used but the specifics of this process have not been communicated to the Panel. This process needs to be verified in terms of adequacy and effectiveness.

e. Reduce length of wall pours to 100 feet or less. This is possible without increasing the number of construction joints and still considering the anchor chamber configuration. WSDOT construction managers have reported that a 75-foot length may be feasible. Similar considerations apply to the top and base slabs.

f. Test cement for C_3A C_4AF and Fineness. Levels of these elements can affect shrinkage.

g. The post-tensioning sequence designed by the Contractor is to be followed absolutely.

h. Increased WSDOT site and Quality Verification (QV) staff are needed to verify that Quality Control/Quality Assurance (QC/QA) is being performed correctly by the Contractor.

i. A review of batch plant and its operations, inputs and controls should be made in order to determine the adequacy of the procedures, inputs and controls that are essential for consistent production of concrete that meets the requirements of the contract. The ACME pilot program was able to achieve satisfactory results for concrete placement and limitation of cracking and should be a guideline in this and other aspects, as was specified in the contract.

Bolt beam and end wall design, Type 1 Pontoons[31]

a. Analyze the bolt-beam longitudinally and transversely (relative to the long axis of the Pontoon) using a sufficiently detailed finite element analysis and redesign if necessary.

b. Perform an independent external review of the design and recommended changes.

c. Rescind Request for Information (RFI) 111 (post-tension tendon location, tangent point and curvature inside bolt-beam).

d. Add reinforcing to resist post-tension tendon pull-out for the bolt-beam and slab (hat bars).

e. Add reinforcing to resist spalling and splitting stresses, for the bolt-beam in the transverse direction at the face of the end wall.

f. Evaluate, add and/or revise the end wall reinforcing to better resist concrete shrinkage, thermal effects and post-tensioning forces.

g. Consider deletion of deck hatch openings or other measures to reduce their influence for the future Cross-Pontoon Type 3 (A at cross walls 7T/1L).

h. Add trim bars around wall openings (if such openings must be used) but, preferably, eliminate end wall construction openings. If absolutely necessary, locate near center of the end panels (horizontal and vertical).

i. Decouple end walls from interior precast walls until after prestressing is complete.

j. Investigate moving the PT anchorages closer to the exterior perimeter (Cycle 2 and subsequent cycles, if possible).

[31] Similar considerations will in all likelihood apply to the remaining Type 3 pontoon (A).

WSDOT'S RESPONSE TO THE ERP, PHASE 1 MEMORANDUM

To: Paula Hammond, WSDOT Secretary

From: Julie Meredith, SR520 Program Director

cc: Jerry Lenzi, Pasco Bakotich, Jeff Carpenter, Jugesh Kapur, Mike Cotten, Larry Kyle, Tom Horkan, Dave Ziegler, Dave Becher
Expert Review Panel – John Reilly, Neil Hawkins, Tom Sherman, John Clark

Date: August 19, 2012

RE: Pontoon Peer Review Panel Recommendations and WSDOT Actions

Dear Secretary Hammond:

Attached are the findings and recommendations provided to WSDOT by the Expert Review Panel, which was convened in June to evaluate the concrete cracking and spalling which occurred in the first six (of 33) SR 520 pontoons being constructed in Aberdeen. The Panel has:

- visited the site;
- reviewed the design and contract documents;
- reviewed the repair procedure developed by the WSDOT Bridge and Structures office and implemented by the Contractor (Kiewit-General);
- reviewed cracking in the longitudinal and cross-pontoons and;
- summarized their findings and recommendations, which are the result of their review of conditions in Cycle 1. The recommendations are applicable to Cycle 2 and beyond.

After the repairs were made to Cycle 1, including repairs of the post-tensioning related spalling and cracking, concrete shrinkage and other "structural"[32] cracks (according to WSDOT standard procedures), the six pontoons were successfully floated out of the Aberdeen casting basin on July 30. The pontoons then were inspected by the contractor as part of their quality assurance responsibilities and by WSDOT as part of their quality verification responsibilities. Towing of the first pontoons began on August 8 and the first pontoon traveled through the Ballard Locks on August 11.

Staff from the SR 520 Program, Aberdeen construction site office, WSDOT Bridge & Structures office and WSDOT Construction office have reviewed the panel findings and recommendations and are addressing all the recommendations. The following summarizes the recommendations and the steps being taken by WSDOT to address them. The urgent nature of the work is driven by the need to define changes that can be implemented in time for construction of the Cycle 2 pontoons.

WSDOT and the contractor have been working to address the issues encountered, minimize any impact to the overall project schedule and budget, and deliver pontoons that will meet the needs of the people of Washington for decades. WSDOT and the Contractor will begin discussions soon to explore opportunities for schedule recovery, as well as an exchange of information related to any entitlement that may be due under the terms of the contract. We are now entering the entitlement phase, and both parties will be taking due diligence to assess responsibility for schedule and cost.

[32] The contract specifies all cracks which are 0.006" wide and greater as "structural". Cracks smaller than this are generally caused by concrete shrinkage due to thermal effects or concrete curing.

Specifics of findings, recommendations and actions being taken follow.

Cycle 1 Pontoons, per Expert Panel observations and recommendations:

1. Panel Observation: Repairs have been made to all of the areas affected by spalling and cracking caused by post tensioning forces. Based on site visits, review of repair details and other available data, the Panel concurs with the WSDOT Bridge and Structures office that repairs made to the bolt-beam/slab spalling and pulled-out post-tensioning ducts are adequate for structural capacity. Pontoons T, U and V were successfully re-tensioned longitudinally after this repair.
2. Panel Observation: All other cracks have been repaired "per Contract procedures" which are understood to be similar to the procedures successfully used on the other WSDOT floating bridges.

   WSDOT Response/Action (to Observations #1 and #2): The Cycle 1 pontoons have been inspected by the contractor (per their quality assurance responsibility) and WSDOT (per our quality verification responsibility) to determine conformance with contract requirements following float out. Final correction items will be completed in Aberdeen or on Lake Washington as needed and where possible. These items, when completed, will result in completed pontoons that will fulfill the intended requirements for bridge support and design service life.
3. Panel Observation/Recommendation: The Panel stated "the repair areas should be inspected throughout the construction, towing and assembly on the lake. Special consideration and attention should be given to these areas according to normal WSDOT maintenance and operations procedures".

   WSDOT Response/Action: In response the Peer Review Panel's recommendation, additional inspections will be scheduled to verify pontoon performance during transport, bridge construction and service use. These inspections will be conducted in accordance with WSDOT normal procedures to verify safety and early detection of potential problems so that they can be corrected.

Cycle 2 and subsequent Cycles per Expert Panel recommendations:

1. Panel Observation/Recommendation: Verify that contract requirements are adhered to. The Panel reiterated that the Contractor's QC and QA staff, with oversight from WSDOT QV site staff must verify that contract requirements (in general) are followed – and specifically that the concrete thermal control plan, concrete curing requirements, concrete mixing, batching procedures and moisture control are performed as required by the contract and that the necessary construction documentation procedures are in place.

   WSDOT Response/Actions: The Headquarters Construction Office and the Aberdeen Project Office are reviewing all contract requirements to verify that they have been followed. If not, the contactor will be directed to correct their procedures. Project site and HQ construction staff are working with the contractor to define changes to the technical requirements which could improve results. This may include contractual changes which could result in cost and schedule adjustments.
2. Panel Observation/Recommendation: "Many of the observed cracks are more extensive than anticipated" (compared to the extent of cracking in the ACME concrete pilot program). Accordingly, the panel has stated "the cracking needs to be understood and measures taken" to minimize or eliminate such cracking in future cycles.

   The Panel recommended changes to the contract requirements to improve the concrete's resistance to cracking, such as a lower limit for allowable water/cement ratio and maximum length of wall and slab pours.

   WSDOT Response/Action: The WSDOT materials section regularly verifies cement properties, batch plant procedures and practices for construction projects. The cement in this application is different than the ACME demonstration project and this may be a contributing factor to cracking. The WSDOT materials section has been asked to look into this with oversight by SR 520 program staff and the Expert Panel.

Additionally, concrete batching, moisture control, water/cement ratios and thermal control and concrete curing procedures have a significant effect on cracking. These are being considered by the project site and SR 520 program and will be reviewed by the Expert Panel in terms of actions and procedures which are beneficial for Cycle 2 and subsequent cycles.

3. Panel Observation/Recommendation: Bolt Beam. The bolt-beam is the structure that holds the bolts which connect the pontoons in service on Lake Washington. It is a highly stressed area containing the end anchorages for the post-tensioning tendons and is the location of the major concrete spalling and cracking which occurred on May 11.

   In order to more clearly understand and eliminate the post-tensioning cracking observed in the Cycle 1 pontoons, the Panel recommended additional analysis so that changes can be implemented for Cycle 2 and subsequently. Specifically they recommended a detailed finite element model analysis of the end wall/bolt-beam area, and revisions to the design if indicated by that analysis.

   Additional potential changes include modifying the as-constructed post tensioning profiles, changing some deck hatch configurations, changing construction sequencing and adding reinforcing steel in specific areas to minimize the potential for spalling and cracking in future pontoons.

   WSDOT Response/Action: The Bridge & Structures office is reviewing the design of the bolt-beam using a finite element model developed by SC Solutions. They have modified the duct head locations and defined additional steel reinforcement to more securely locate the post-tensioning tendons in position per the contract M-11 drawings. The Expert Panel will review the results of this analysis regarding the need and configuration of additional steel reinforcement. It will need to be verified that this analysis and design review is sufficient in terms of necessary changes for Cycle 2 and beyond.

   Independently, WSDOT has engaged Parsons Brinckerhoff to analyze the as-designed M-11 bolt-beam for structural adequacy and potential changes for Cycle 2 and beyond. The Expert Panel will also review the results of this analysis.

4. Panel Observation/Recommendation: Pontoon end walls. The post-tensioning also stresses the pontoon's end walls, which has resulted in significant and extensive cracking due to this post-tensioning, plus thermal and autogenous[33] concrete shrinkage and the resistance of the interior precast panel walls. The Panel recommended that the interior precast panel walls be decoupled from the end walls until after post-tensioning and that the steel reinforcement in the end walls be reviewed and modified if necessary.

   WSDOT Response/Action: The Bridge & Structures office is modeling the pontoons with the end walls decoupled from the interior precast panel walls to verify that this will resolve cracking from post-tensioning. It will need to be verified that they are reviewing steel reinforcement in the end walls and will modify if necessary.

5. Panel Observation/Recommendation: Quality Control (QC - Contractor), Quality Assurance (QA - Contractor) and Quality Verification (QV – WSDOT). The Panel suggested WSDOT assign additional quality verification personnel to the project, over what is normally provided for a design/build contract, because of the magnitude and complexity of this particular project and the complex, integrated design and construction requirements.

   WSDOT Response/Action: The contractor has been directed to review their QC/QA staff and procedures to see where improvements can be made. WSDOT will review our QV procedures including staffing levels and areas of focus.

[33] Autogenous shrinkage is a volume change when there is no moisture transfer to the external environment. It is therefore different than drying shrinkage and most prevalent in high performance concrete where the water-cement ratio (w/c) is under approximately 0.32. The w/c ratio for the Aberdeen pontoons was as low as 0.28.

APPENDIX E

BACKGROUND AND EXPERIENCE OF THE EXPERT PANEL

JOHN REILLY P.E., C. P. Eng. – Chair.

John Reilly consults on management, strategy, organization, technical review/oversight, team-alignment/partnering, contacting and delivery strategies & methods, risk management and probalistic cost/schedule (WSDOT CEVP® process[34]) for large, complex transportation programs.

John graduated in Civil/Structural Engineering from the University of Sydney (B.E. Hons. 1963) and University of California Berkeley (M.S. 1964). First Registered in B.C. Canada, he is a Registered Professional Engineer in the US and Australia with 50 years experience and has worked in Australia, Canada, US, UK, Europe and the Mid-East. President of the American Underground Construction Association (1999-2001) and Chair of 2 International Tunneling Association Committees.

Relevant practice areas include:

- Management & strategic consulting - organization, delivery, cost and risk, team-alignment (London Underground; Alaskan Railroad; WSDOT; TTC Toronto; Dubai; LA Metro, MBTA)
- Advanced highway bridge designs (Sydney 1967, Massachusetts 1988, Los Angeles 1995)
- Transit programs design, construction, technical reviews and management oversight: – Washington D.C. 1968-1972; Boston MBTA 1978-85; Los Angeles 1991-1997; San Francisco 1995-1998; Philadelphia 1997-99; Atlanta 2003; Los Angeles Metro 2012.
- Advanced contracting & delivery, risk, risk mitigation, probabilistic cost estimating (2001-present)
- Underground structures (Washington DC 1969, Boston 1978, Los Angeles 1991, Seattle 2001-)
- Building structural design/historic restoration (Vancouver B.C. 1964-65, Washington DC 1972-78)
- Partnering, team-alignment, organization (San Francisco, London, Boston, Philadelphia, Toronto.)
- Transit vehicle manufacturing & systems (San Francisco 1995, Philadelphia 1997)

For the last 30 years John has focused on the management of very large, complex infrastructure projects, Internationally and in North America, including Highways, Airports, Tunnels, Road and Transit Systems. He has chaired strategic, advisory and expert review panels for several projects.

Selected Experience, Management, Technical Assistance & Reviews

<u>Management and Technical Assistance, WSDOT Urban corridors program</u>

Strategic assistance to the Washington State Department of Transportation (WSDOT), mega-projects. Organization, strategy, team-alignment, technical studies and recommendations for contracting and delivery of the large, complex transportation projects of the Puget Sound Region and Columbia River Crossing. Chair, Strategic/Technical Advisory Teams (STAT) for the SR520 Lake Washington Floating Bridge replacement and the Alaskan Way Tunnel (currently the largest TBM in production).

<u>Mass Highway Department – advanced Bridge Design Program</u>

Definition and implementation of innovative bridge design applications, State-wide workshop and management of the advanced bridge design implementation program for MassHighway (1986-89).

[34] John Reilly was the principal developer of the CEVP process with colleagues and WSDOT Managers.

Contracting and Delivery Methods

Risk Management, Cost-Risk Estimating, Cost Validation Process - CEVP®

In 2002, with the Washington State Department of Transportation and a colleague, John developed the WSDOT Cost Estimate Validation Process (CEVP®), a structured approach to cost estimating which combines base cost with defined risk and opportunity events to estimate the "range of probable cost and schedule". The defined risks are then included in explicit risk management plans. CEVP® has been implemented as a normal business process by WSDOT and is being used by FHWA and other US and Canadian transportation and infrastructure agencies.

Alternative and Innovative Project Delivery systems

Working with the Washington State Department of Transportation on mega-project delivery options, John was the Principal Investigator and consultant lead for a report to the State Legislature and Secretary of Transportation on "Alternative Contracting and Innovative Project Management". This report considered traditional delivery systems including Design-Bid-Build (DBB), Design-build (DB), Design-Build-Operate-Maintain (DBOM) and other similar methods. It contrasted these with other promising methods such as General Contractor/Construction Manager (aka GCCM, CM/GC or Contractor at risk), incentive options (A+B bidding) and Alliancing (Australia, New Zealand, UK). (2005-2008)

Transit Programs

London Underground

Organization, partnering/team–alignment/management/implementation planning: new signal systems for Bakerloo, Central, Victoria, District, Circle and Metropolitan lines. London Underground (*2005-2006*).

Los Angeles Metro System

Project management oversight and technical assistance for the Los Angeles Metro heavy and light rail projects - tunnels, underground stations, at-grade sections and bridges. Management oversight, cost-to-complete, design reviews, technical reports, agency and consultant costs, risk workshops (1991–1997).

Readiness review and management oversight, LACMTA Crenshaw/LAX light Rail Program (2011); Report on contracting and delivery options for the Westside heavy rail extension project (2012)

Toronto Rapid Transit Expansion Program

Management, organization, processes and implementation of team-alignment for the fully integrated TTC/consultant team on the CN $3 billion Rapid Transit Expansion Program. Assistance with value engineering, configuration management, design and construction interfacing (1994-1996).

Boston – MBTA, SouthWest Corridor Transit program

Program Director for program management, final design, construction management assistance to MBTA for Boston's $1 billion (1980 $) Southwest Corridor Transit, High-speed and Commuter Rail, Highway and Urban Design Project. Delivered under budget/close to schedule. Winner of the President's Design Award and named the ASCE Outstanding Civil Engineering Achievement of 1987 (1978-1987).

EXPERT PANEL MEMBERS

NEIL M. HAWKINS, Ph.D., Dist. M. ASCE, Hon. M. ACI, FPCI, FSEI.

Dr. Hawkins is Professor Emeritus, Civil and Environmental Engineering, University of Illinois and Adjunct Professor, Civil and Environmental Engineering, University of Washington.

He has authored over 250 articles and supervised the research of 22 Ph.D. recipients. Throughout his career Dr. Hawkins concentrated his research on issues directly related to the practice of structural engineering. His findings on reinforced and prestressed concrete, and particularly shear in those materials

and their response to earthquake motions, are extensively cited in the American Concrete Institute's Building Code (ACI 318), the Loading Standard for Buildings of the American Society of Civil Engineers (ASCE/SEI 7), the Bridge Design Specifications of the American Association of State Highway and Transportation Officials, and the National Earthquake Hazards Reduction Program Provisions of the Federal Emergency Management Agency (NEHRP Provisions). He has been a member of the governing committees for ACI 318 since 1992, for ASCE/SEI 7 since 2000, and for the NEHRP Provisions since 2000, and continues to be active on all three committees. He has served on the Boards of Direction of the American Concrete Institute, the Earthquake Engineering Research Institute, the Post-Tensioning Institute, and the Structural Engineering Institute of ASCE. He is Chair of ASCE's Board Committee on Codes and Standards and a member of the Research Council of the Precast/Prestressed Concrete Institute (PCI).

In the area of transportation Dr. Hawkins developed in 1982, in conjunction with WSDOT, the Washington State Transportation Research Center and became one of their principal researchers. In the 1980s he served extensively as a consultant to the City of Seattle on the design and construction of the West Seattle Bridges for both the high and low level crossings of the Duwamish River. In the 1980s he worked with faculty at the University of California, Berkeley to develop and extension course and notebook on the design of concrete ships and off-shore structures. Following the loss of the Hood Canal floating bridge he worked with state officials in attempting to better define the reasons for that loss and was a member of Arvid Grant and Associates team to design a replacement bridge. In 1991 he was appointed one of five members of Governor Gardner's Blue Ribbon Panel to investigate the safety of floating bridges in Washington State following the loss of the I-90 floating bridge. He was the primary engineer on that panel and wrote extensive reports for the other panel members to help them understand the issues involved in the loss of the floating bridge and the implications of that loss. Subsequently he worked with the State and Traylor Brothers on the arbitration of legal, technical and fiscal issues related to the loss of that bridge. In Illinois he served on Illinois' Transportation Research Center Board, developed for the University of Illinois an Advanced Transportation Research Center for testing airfield pavements and transportation structures, and developed for IDOT methods for the visual and forensic assessment of side-by-side prestressed concrete girder bridges following the collapse of two such bridges in service in Illinois in 1998. He has been a principal author for five National Cooperative Highway Research Program Reports dealing with the design, repair and performance of concrete bridges with the latest report appearing in 2008. He is the author of several publications on the design of floating concrete bridges and barges and has worked with senior FHWA bridge officials on related subjects. He was a member of FHWA's advisory team for the review of the FHWA sponsored bridge research work of the Multidisciplinary Center for Earthquake Engineering Research at the University of Buffalo from 1995 through 2005.

Throughout his career Dr. Hawkins has sought to improve strategies for the transfer of results from research into practice. In 1990 he was appointed to the Advisory Board of ASCE's Civil Engineering Research Foundation (CERF) and in 1991 was principal author for CERF's study of policies affecting the transfer of research into practice in construction in Japan. In 1993 he was the principal author for CERF's studies of similar practices in France and Italy and in 1995 for CERF's studies of similar practices in the People's Republic of China. Through those studies he developed, in conjunction with Pankow Builders, strategies for evaluation and implementation of new building technologies that are now utilized by the American Concrete Institute through its use of Innovation Task Groups, by the Precast/Prestressed Concrete Institute through its research and development committee, and by the Pankow Foundation through its requirements for a plan to transfer the results of any research it supports into practice. He has been recognized for his leadership in the transfer of research results into practice by awards from ASCE (Howard, 2004, Tewksbury, 2007, Distinguished Member 2011) from ACI (Kelly 1996; Boase 2005, Turner 2005 and Honorary Member 2012), from PCI (Distinguished Educator 2001, Titan 2004; Fellow 2005), and FIB(Honorary 2002).

JOHN H. CLARK, P.E., Ph.D.

Dr. Clark has over fifty years of experience in the field of bridges and other heavy structures. His experience includes employment by an international construction firm, private engineering consultants, and public agencies. He is recognized for special expertise in the design of long span bridges and seismic design of bridges. Many of his projects have required evaluation of existing bridges for remaining life and load rating.

He is active in professional society affairs including the Prestressed Concrete Institute, International Society for Bridge and Structural Engineers, and Structural Engineers Association of Washington. He has served on the Committee on Concrete Bridges of the Transportation Research Board. He was a member of the Concrete Task Group for NCHRP Project 12-33. This project resulted in the development of the current AASHTO LRFD specifications. He participated in the ATC-18 review of current seismic design specifications for bridge design.

Dr. Clark is a registered professional engineer in the states of California and Washington.

Project experience includes participation in the following projects:

- Columbia River Crossing I5
- Columbia River Pipeline Bridge, Wenatchee Reclamation District
- Seattle Department of Transportation Bridge Seismic Retrofit Program
- Canyon B Bridge, Douglas County, WA
- Gerald Desmond Bridge, Long Beach, CA
- Seattle Monorail Study for Inclusion on West Seattle Bridge
- Hoover Dam Bypass Bridge
- Alaskan Way Viaduct, Seattle, WA Seismic Sufficiency
- Bandra-Worli Sea Link Bridge, Mumbai, India
- SR509 Thea Foss Waterway Bridge, Tacoma, Washington
- FHWA Seismic Design Course
- SR5, Lake Washington Ship Canal Bridge, Seattle, WA, Seismic Retrofit
- University Bridge Seismic Retrofit, Seattle, WA
- WSDOT Special Bridges, Seismic Vulnerability Report
- 23rd Street Viaduct, Denver, CO.
- West Seattle Freeway, High Level Bridge, Seattle, WA
- West Seattle Freeway, Low Level Swing Bridge, Seattle, WA
- Navajo Bridge, Colorado River, Marble Canyon, AZ
- Goff Bridge, Salmon River near Riggins, ID
- ALRT Bridge, Fraser River, Vancouver, British Columbia
- Pasco Kennewick Intercity Bridge, Columbia River, Pasco, WA
- SR90, Lacey V Murrow Bridge, Seattle, WA
- Boston Central Artery, Charles River Bridge, Boston, MA
- SR90, Bridge Load Rating, WA
- ATC (Applied Technology Council)-18 Review of Highway Bridge Seismic Design Specifications
- AASHTO LFRD Specifications

Education: BSCE (w/honors), Washington State College, Pullman, WA 1956
MSCE, University of Washington, Seattle, WA 1980
Ph.D., University of Washington, Seattle, WA 1989

Professional Society Committees

- ASCE Committee on Loads on Bridges, Chairman 1977-1978, Committee Member 1978-1994
- ASCE Technical Committee on Lifeline Earthquake Engineering Committee Member 1982-1994
- ACI Technical Activities Committee Member 1995-1998
- ACI Committee 343 Concrete Bridge Design Committee Member 1980-2000, Chairman 1989-1993
- ACI Committee 341 Earthquake Resistant Concrete Bridges Committee Member 1990-2000
- ACI Committee 342 Evaluation of Concrete Bridges and Bridge Elements Committee Member 1994-2000
- ACI Committee 348 Structural Safety, Associate Member 1983-2000
- TRB Committee AC203 Concrete Bridges, Committee Member 1983-1993
- ATC (Applied Technology Council) 12, Cooperative Research Program on Seismic Resistance of Highway Bridges, 1981, Member of US Delegation
- ATC 18, Review of Specifications for Seismic Design of Highway Bridges, 1995 Member of Project Panel
- Delegate to 7th & 9th Joint US-Japan Workshop on Bridge Design, Tskuba, Japan 1992, 1994

Honors

- Structural Engineer of the Year, Seattle Chapter, Structural Engineers of Washington, 1993
- H.T. Person Distinguished Professor, University of Wyoming, Laramie, WY, Fall 1997

MARK A. LEONARD, P.E.

Structural Engineer, Resource Center Structures Technical Services Team
Federal Highway Administration (FHWA), Lakewood, Colorado

Education
B.S. in Civil Engineering, University of Notre Dame, 1981

Registration
Professional Engineer, State of Colorado

Employment

Structural Engineer	Federal Highway Administration	2012
State Bridge Engineer	Colorado Department of Transportation	2000 – 2012
Senior Structural Engineer	Colorado Department of Transportation	1993 – 2000
Structural Engineer	Colorado Department of Transportation	1984 – 1993
Civil Engineer	Colorado Water Conservation Board	1983 – 1984
Structural Engineer	Fluor Engineers, Inc.	1981 – 1983

Professional Summary

As a member FHWA's Structures Technical Services Team Mark provides technical assistance, training, project reviews, and program reviews in the areas of structural design, construction, asset management, and inspections. From 2000 to 2012 Mark was the State Bridge Engineer for the Colorado Department of Transportation (CDOT) and managed the Department's structural engineering operations including design, construction assistance, inspection, asset management, policy and standards, rating and overloads, and fabrication inspection. The inspection and asset management operations included CDOT non-bridge structures and local agency bridges.

As a structural engineer from 1981 to 2000 Mark performed a wide variety of structural design, inspection, load rating, policy, and standards functions. From 1993 to 2000 mark supervised a CDOT bridge squad that provided structure designs, project reviews, construction assistance, standards, and specifications. Over his career Mark has provided technical assistance and guidance to numerous initiatives and reviews to address project design and construction issues, and to identify and implement operational improvements in the areas of structural design, construction, maintenance, asset management, and strategic planning.

Mark has provided guidance for the implementation of innovative structure types and components – especially in the areas of rapid bridge construction, mechanically-stabilized-earth-walls, fiber reinforced polymer repairs, and precast concrete decks, substructures, and segmental girders. Mark has also participated on research panels and technology.

Mark was recently hired by the Federal Highway Administration's Resource Center to provide technical assistance, training, project reviews, and program reviews in the areas of structural design, construction, and operations.

THOMAS E SHERMAN

Mr. Sherman has over 40 years of marine and heavy civil construction experience with General Construction Company. He has served as a Tradesman, Foreman, Project Superintendent, Project Manager and Vice President of Construction / Project Executive. He retired from General Construction in 2004. Since retiring Mr. Sherman has worked as a Construction Consultant on various issues for public and private owners.

Consulting Experience: (partial)

The Glosten Associates (2006)

Served on a Risk Assessment Panel for the Design Build and Operate group that is building the new replacement William R. Bennett Floating Bridge across Okanagan Lake in British Columbia.

Washington State Department of Transportation (2006)

Participated in an Expert Review Panel for the SR 520 Pontoon Construction Site for the new Highway SR520 Floating Bridge across Lake Washington just east of Seattle.

Washington State Department of Transportation (2005)

Participated on an Expert Review Panel to review and to advise WSDOT on their planning to relocate the Graving Dock facility that was planned to be used for the reconstruction of the East half of the Hood Canal Floating Bridge. Part of a team that was designated to negotiate a change order to resolve the impacts of relocating the pontoon and anchor construction to the new site.

General Construction Project Experience (partial)

Tacoma Narrows Bridge, Tacoma, WA (2002-2004)

A $615M design-build project. The prime contractor is Tacoma Narrows Constructors which is a joint venture of Bechtel-Kiewit. As a Kiewit company, General Construction was responsible for constructing and setting the two bridge tower foundation caissons.

Bremerton Naval Station, Pier D, Bremerton, WA (2000-2002)

A $53M design-build project for an aircraft carrier pier. Project scope included demolition of an existing pier and the construction of a new pile supported pier 1320 ft. x 150 ft. and all associated utilities and upland support facilities.

Snohomish River Bridge, Everett, WA (1999-2001)

SR-2 bridge replacement for The Washington State Department of Transportation. The project required the construction of an 8,000 ft. bridge across the river and adjoining wetlands. It is supported on 8 ft. drilled shafts and pipe piling.

Lacy V Murrow Floating Bridge, Seattle, WA (1992-1993)

The replacement bridge for the one that sank in late 1990. This contract was with the Washington State Department of Transportation to construct a new floating bridge over 6,000 ft. long, consisting of 20 pontoons and 56 anchors of various designs. The contract allowed for 3 years but the project was completed in just 2 years.

Puget Sound Carrier Support Complex, Everett, WA (1989-1992)

Project was to provide the mooring for the Navy's new Home Port Carrier Base in Everett, Wa. Completed over 900,000 cy of clam shell dredging then drove over 1700 concrete pilings to support the two concrete piers. Also in this project was Rock Riprap slope protection, a large concrete utilidor, a sanitary pump station constructed as a gravity caisson and all the associated mechanical and electrical utilities.

Highway 520 Concrete Floating Bridge, Seattle, WA (1991-1992)

Maintenance and repairs consisting mainly of reworking the hydraulic opening system controls, crack repairs and removing concrete parapet walls.

Original Mercer Island Floating Bridge, Seattle, WA (1990)

An emergency contract with the Washington State Department of Transportation. On Sunday November 25, 1990 the old Mercer Island Concrete Floating Bridge sank and in the process of sinking it broke off most of the south side anchor wires holding the new west bound bridge in place. Responsible for rescue to remove sinking pontoons and begin a process of replacing the broken anchor wires, so the remaining bridge could be reopened to two way traffic.

Hood Canal Floating Bridge, Jefferson County, WA (1981-1983)

Contract was with the Washington State Department of Transportation to construct a new opening span to replace the one that sank in a storm in 1979. The new work required 10 pontoons mostly of various dimensions that when put together would form the new 300 ft draw span. In order to install the new draw span we had to remove 900 ft of the existing bridge and place in storage for the owner's future use. Total traffic shut down was limited to 6 days versus the 14 days that were available per the contract.

Original Mercer Island Floating Bridge, Seattle, WA (1980-1981)

Two contracts with the Washington State Department of Transportation, the first for about 600 lf of pontoons to be used for the replacement of the old opening span (The Bulge). The second contract was for the actual removal and replacement of the old draw span which had traffic shut down for 2 days.

STEPHEN B. TATRO, P.E.

Civil engineering consultant specializing in the evaluation, testing, design, and construction of concrete materials for dams and other concrete structures.

PRESENT	Contact: Mobile (509) 240-6422, email:steve@tatrohinds.com
TECHNICAL EXPERTISE	Specialist in the field of concrete materials design, and construction. Specialty areas include mass concrete, roller compacted concrete, fiber-reinforced concrete, specifications, concrete repairs, shotcrete, chemical grout systems for concrete, and waterstop installations. Performs laboratory and field studies, complete designs, and prepare plans and specifications for aggregates, concrete and concrete related work, specialty concretes, and serve as concrete construction advisor for many on-going contracts. Performs thermal and cracking analyses for conventional concrete and RCC structures. Inspects concrete structures and dams, to evaluate concrete condition.
EDUCATION	B.S. Civil Engineering, 1979, Walla Walla College, Walla Walla, Washington, USA M.S. Civil Engineering, 1985, Purdue University, West Lafayette, Indiana, USA Licensed professional engineer in the State of Washington, USA
CAREER HISTORY	Mr. Tatro is a concrete materials engineer. He has extensive experience in concrete materials, design, and construction. He has performed materials designs, supervised field applications, managed construction contracts, and evaluated concrete and materials problems during his career with the US Army Corps of Engineers (1979-2011). Promoted to senior engineer in 1984 and to manager of the dam safety program in 1986. Designated senior technical specialist for concrete materials in 1990. Expertise in areas of roller compacted concrete, shotcrete, contract preparation, quality control issues, and trouble-shooting field problems. Technical competence led to providing materials consulting services to other Corps of Engineers organizations in fields of concrete thermal analysis, waterstop replacements, shotcrete, roller compacted concrete design and construction. Private consultation since 1985 in same fields.
PROJECT MANAGEMENT	Effectively managed numerous programs and projects. Managed the structural instrumentation program and the dam safety assurance program for 8 major dam projects. Managed teams of designers for the design of multi-disciplined projects. Negotiated and managed contracts for design services.
CONSTRUCTION MANAGEMENT	Effectively managed construction activities by communicating with contractor staff, negotiating changes, directing activities, controlling costs, and coordinating with the design, construction, and owner team members. Projects include technical oversight for tunnel excavation project, served as construction project engineer for several roller compacted concrete dams, a directed labor drilling and grouting contract, and for expedited repairs to a damaged navigation lock structure.
PUBLICATIONS	Twenty-eight publications for technical journals, magazines, and symposiums including the *Journal of the American Concrete Institute, The Transportation Research Board, Proceedings of the American Society of Civil Engineers, Civil Engineering Magazine, and Concrete International: Design and Construction.*
PROFESSIONAL AFFILIATIONS	Fellow, ACI International, (American Concrete Institute) Member, Association of State Dam Safety Officials (ASDSO) Member, Army Engineer Regiment Member, United States Society of Dams (USSD)

Range of Concrete Experience

Mr. Tatro has been involved in many aspects of concrete evaluation, design, and construction. As a member and past chairman of the American Concrete Institute, Committee 207, Mass Concrete and Committee 210, Erosion of Concrete in Hydraulic Structures, Mr. Tatro has been the primary author of subcommittee reports "Cooling and Insulating Systems for Mass Concrete" and "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete", and "Deterioration of Concrete in Hydraulic Structures. A summary of activities on specific projects is on the attached table.

Mr. Tatro has worked on hundreds of projects during his 31-year career involving the planning, design, construction, and rehabilitation of concrete structures. A brief list of technologies and typical projects follow that illustrate the diversity of expertise and experience in concrete technology.

CONVENTIONAL CONCRETE, MASS CONCRETE, AND CONCRETE PAVEMENTS

Participated as designer and construction engineer on numerous design and construction teams for RCC dams. Served on expert panels for RCC dams nationally and internationally. Includes dams in Vietnam (2), Australia, and Lesotho. Designed and constructed numerous mass concrete structures ranging from massive spillway deflectors, stilling basin repairs, navigation lock monoliths, bridge piers and tremie seals, tunnel linings, and massive structural elements. Design and construction of conventional fixed-form, conventional slip-formed, and RCC pavements for military, industrial, and aviation applications. Most recently prepared the Corps of Engineers Guide Specification for RCC pavements.

ROLLER COMPACTED CONCRETE

Involved in approximately 50 RCC projects as lead designer, resident engineer, design team, consultant, expert panel reviewer, and troubleshooter. Most recent projects are expert panel member for Wyaralong Dam (Australia), Metolong Dam (Lesotho), Portugues Dam (Puerto Rico), and Trung Son Dam (Vietnam). Served as design team member for Gibe3 Dam (Ethiopia), Valenciano Dam (Puerto Rico), and Diamer Basha Dam (Pakistan).

MATERIALS AND LABORATORY INVESTIGATIONS

Performed numerous investigations into the availability and quality, of aggregates, cements, flyashes, admixtures, and placing conditions on concrete products. Designed laboratory investigation programs for aggregate, concrete, and forensic applications. Performed the mixture design program for several RCC dam projects and a slurry wall project. Performed laboratory investigations of chemical grout methods of waterstop repair. Currently developing testing equipment and methods for improved testing of shear, direct tension, adiabatic temperature rise, creep of concrete.

THERMAL AND CRACKING ANALYSIS

Performed numerous thermal analyses of mass concrete structures. Developed the definitive approach to non-NISA analyses utilized throughout the world. The approach is described in the Corps of Engineers ETL 1110-2-542 on Thermal Analyses.

OTHER SPECIALTY CONCRETES AND APPLICATIONS

Designed repairs and new installations utilizing a wide range of specialty concretes and specialty applications. They include shotcrete, fiber reinforced concrete, epoxy mortar, polymer impregnation, latex modified concrete, auger-cast piles, slurry walls, shrinkage compensating concrete, non-shrink grout, high performance pneumatic grout (nuclear facility application), tremie concrete, underwater concrete, self-consolidating concrete, Designed replacement waterstop systems for concrete monolith joints that included acrylamide, urethane, silicone, and other grouts. Developed the latest grout innovation for replaceable waterstops for dams and mass concrete structures. Received the Corps of Engineers Innovator of the Year Award for that development