Final
Report of
Inspection

Project Management Deficiencies in Constructing the Paul S. Sarbanes Silver Spring Transit Center

Report # OIG-14-007

April 15, 2014

Except as to the *Subsequent Event* information on pages 10 and 54, and the Chief Administrative Officer's Statement on page 163, which are as of May 8, 2014

Montgomery County Maryland
Office of the Inspector General

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Stakeholders in the SSTC Covered in this Report

Within this report, numerous stakeholders are mentioned. The following is a short overview of those interested parties that are mentioned throughout this report.

Owners:

Montgomery County Maryland, represented by Department of General Services (DGS)

Washington Metropolitan Area Transit Authority (WMATA)

Governmental Project Funding:

Federal Transit Administration (FTA)

Maryland Transit Administration (MTA)

Design Team:

Parsons Brinckerhoff, Inc. (PB) *(known as Parsons Brinkerhoff Quade and Douglas, Inc. and PB Americas, Inc. at commencement of the SSTC)*

Sub-Contractors to Parsons Brinckerhoff:

Zimmer Gunsul Frasca Architects LLP (ZGF) - architect

Construction Team:

Foulger-Pratt Contracting, LLC (FP)

Sub-Contractors to Foulger-Pratt:

Facchina Construction Company, Inc. (Facchina) - concrete project work

Sub-Contractors & suppliers to Facchina:

VStructural LLC (VSL) - post-tensioning

Gerdau Ameristeel - mild steel reinforcing design and installation supervision

R&R Reinforcing, Inc. - post-tensioning and mild steel reinforcing installation

Lafarge Concrete, and Rockville Fuel and Feed Co., Inc. (RFF) - concrete suppliers

Inspection Team:

Montgomery County Maryland under the Special Inspections Program administered by the Department of Permitting Services (DPS)

The Robert B. Balter Company (Balter)

Report in Brief

Project Management Deficiencies in Constructing the Paul S. Sarbanes Silver Spring Transit Center

April 15, 2014

Background

The Paul S. Sarbanes Silver Spring Transit Center (SSTC) is a ground transportation facility located in downtown Silver Spring, Maryland at the intersection of Colesville Road and Wayne Avenue.[1] It was designed to accommodate bus and taxi movements while loading and unloading passengers. Bus loops are located on the ground (Level 305) and second (Level 330) floors, while private vehicles and taxis use the third, smaller floor (Level 350). The Levels 330 and 350, which are the focus of this report, are made of concrete reinforced with mild steel reinforcing bars and post-tensioned tendons (a post-tensioned tendon consists of 7 high strength wires braided together to form one tendon) embedded in the floors to provide strength.

Under a formal Memorandum of Understanding (MOU) dated November 17, 2004 (amended and restated September 25, 2008) between the two owners of the land being used for this project - Montgomery County Maryland and Washington Metropolitan Area Transit Authority (WMATA) - Montgomery County, represented by its Department of General Services (DGS), is authorized to manage the development and construction of the SSTC. Upon completion of the project and WMATA's acceptance and approval, WMATA will control, operate, and maintain the facility.

Construction of the structure began in 2009 but project progress was severely delayed due to unforeseen contaminated soil and utility relocations. By June 2010, the project was already several months behind schedule. By November 2010, visible evidence of structural issues and concerns about durability had emerged, including:

- Cracks discovered in the concrete slabs, beams and girders;

[1] For additional background information about the SSTC, reference the *Silver Spring Transit Center Structural Evaluation of Superstructure* report dated March 15, 2013, prepared by KCE Structural Engineers, PC., pp. 3-4, and the *Evaluation of Silver Spring Transit Center, Silver Spring, Maryland* report dated May 2, 2013, prepared by Whitlock Dalrymple Poston & Associates, Inc., page 1.

- Concrete that broke away from the finished drive surface (spalling), revealing post-tensioned tendons and evidencing that an insufficient concrete cover had been placed over the tendons;
- Issues related to post-tensioned tendon elongations and tensioning; and
- Reinforcing bars that were incorrectly installed or partially omitted in a slab pour.

Although concerns about concrete thickness, inadequate concrete cover, and related structural deficiency and durability were continually raised in monthly project oversight meetings, potential repairs and remediation had not been resolved by the end of the major construction activities in 2012.

Project oversight was provided based on a formal Project Management Plan (PMP) by a Project Management Team (PMT) consisting of representatives of all major project stakeholders, including the property owners, Montgomery County and WMATA, and the state and federal government agencies that provided significant funding for the project (the Maryland Transit Administration [MTA] and the Federal Transit Administration [FTA]). The team held formal monthly meetings for which meeting minutes were kept. In April 2012 DGS reported to the PMT that the construction contractor would prepare a presentation regarding a remediation plan. Recommended actions, including a 2 inch Latex Modified Concrete (LMC) overlay, recommended by Parsons Brinckerhoff, Inc. (PB) and MTA in mid-2012, were proposed during the following months, but meeting minutes indicate "WMATA has not accepted this proposed fix and continues to question the root cause of the cracks."

In June 2012, Montgomery County contracted with KCE Structural Engineers, P.C. (KCE) to conduct a document review and structural evaluation of in-situ conditions at the SSTC. In July 2012, the firm of Whitlock Dalrymple Poston & Associates, P.C. (WDP) was retained by WMATA to evaluate the SSTC. Both evaluations had similar purposes - to determine the condition of the SSTC and to understand whether the structure as constructed satisfied the strength and durability requirements necessary to meet its intended use and service life. Both KCE and WDP based their findings on independent document reviews, field investigation observations, and engineering analyses.

On March 15, 2013 KCE issued its report that identified a number of serious deficiencies in the structure, and determined that the SSTC required strengthening and repairs to meet Building Code and WMATA requirements. On May 2, 2013, WDP released its report which documented construction deficiencies consistent with those identified in the KCE report.

As of March 2013, when the KCE report was issued, information we were provided by FTA indicated that total project cost stood at $104,618,000. However, approximately $7,000,000 in change orders were pending. FTA had provided $53,957,000. The balance had been provided by the MTA and Montgomery County. The initial estimate in 2004 was $35 million.

Why We Did This Inspection

The objective of our Inspection was to identify and document any project management deficiencies during the construction of the Silver Spring Transit Center. In achieving our objectives, we attempted to determine which project management controls failed, how these controls should have functioned, why they failed, and what measures should be taken to ensure controls will be effective in future projects undertaken by Montgomery County.

A report on the Silver Spring Transit Center entitled "Analysis of Project Controls" was prepared at our request by the Alpha Corporation. That report, which includes both recommendations and lessons learned, is included in its entirety as Exhibit I. The **objectives**, **scope, and methodology** of our report are provided in Exhibit II.

What We Found

The significant structural strength and structural durability concerns identified in both the KCE and WDP Reports resulted from deficiencies in construction, design issues cited in the KCE report, and failure to effectively address these issues when they were first identified. Each of these issues contributed to widespread cracking in the slabs, beams, and girders that is now evident in the Silver Spring Transit Center.

Project Controls ***(see page 11)***

Fourteen of the 22 relevant construction project controls analyzed for adequacy of design, implementation, and effectiveness were either weak or ineffective.

Structural Strength ***(see page 13)***

Concrete compressive strength ***(page 13)*** as measured by KCE is weaker in some areas than required by the contract documents. Although inspectors asserted that no undocumented water was added to the concrete, forensic testing in the SSTC suggests a presence of 36% more water than was documented by the concrete provider and the inspector.

Specifically, testing for the workability of concrete via slump measurements provided an indicator of additional water. Concrete with greater workability was documented for 19% of the second slump tests taken on the deck – a result that is inconsistent with the passage of time and the asserted absence of undocumented additional water. These results raise questions about the accuracy and validity of the recorded data, as the results are inconsistent with the other data. Greater amounts of water in a concrete mix would contribute to lesser compressive strength.

We found evidence that concrete did not cure properly in some areas, further impacting the compressive strength of the concrete placed in the structure (in-situ concrete). The condition of the in-situ concrete may have been affected by the failure to observe cold weather curing procedures, potentially contributing to the early shrinkage cracking observed in the structure. The placement of thermal protection was delayed and prematurely discontinued during some cold-weather pours, and temperatures were not monitored as indicated in the specifications.

The effects of extra water and improper curing should have been detected during testing, but concrete specimen samples upon which test results relied were not representative of the in-situ concrete.

Most specimen cylinders were collected at the construction site inspection station. For three trucks during each pour, however, comparative specimens were also collected on the deck where the concrete slabs were poured. Compressive strength tests relied upon for decision-making were primarily those from specimen cylinders collected and cured at the inspection station.

We found that for 49 of the 56 comparative specimen sets, cylinders collected from the deck slab pours demonstrated lower compressive strength than that of the cylinders taken at the inspection station. However, records do not indicate that the test results from cylinders collected at the two locations were ever compared by the contractors. As a result, the differences were not identified or investigated, and the same batch performance differences relative to specifications were not detected.

Concrete placement *(page 34)* resulted in insufficient concrete cover over reinforcing steel and post-tensioned tendons, which allowed the concrete covering tendon ducts in several locations to crack away when grout was placed in the ducts. Concrete drive paths as poured do not provide the minimum concrete cover (thickness) required by the design specifications. In other areas, the concrete cover was thicker than design specification requirements.

By late 2010, design, construction, and inspection personnel were aware that proper concrete thickness was not always being achieved, yet effective corrective measures were not taken, and the problem persisted throughout the period of the major construction project activities.

The three pour strips[2] *(page 37)* on the 330 and 350 levels were each constructed in a different manner and neither of the pour strips on the 330 level was constructed in a manner that conformed to the design requirements identified in the structural drawings. The Contractor's Quality Control plan provided for resolution of construction questions through a written process, but the contractor did not use this process to seek answers to questions it may have had about design requirements. The east pour strip on the 330 level was poured without post-tensioning tendons but with mild

[2] Pour strips are areas of a slab in the deck that are left out during construction and then placed after adjacent concrete has been poured and has been allowed an opportunity to shrink. See Finding 6.

steel reinforcement, while the west pour strip on the 330 level was poured without post-tensioning tendons and without sufficient steel reinforcement in one direction.

Pour strip deficiencies resulted from the failure to prepare necessary and/or accurate shop drawings and professional errors in detecting the omission and inaccuracy of the drawings.

Durability of the Structure *(see page 42)*

Water penetrating the structure through the cracks could reach and corrode the embedded reinforcing steel, thus potentially shortening its life span significantly from the intended 50-year life. Significantly greater maintenance of the structure would be required, thus greatly increasing the cost of maintaining the structure through its projected life.

The primary causes of the reduced durability include widespread cracking of various sizes throughout the structure, which are attributable to the design of the structure that according to KCE and WDP was not prepared in accordance with applicable building codes, WMATA design criteria, or industry standards. A major issue was the lack of construction and design details to accommodate normal movement.

Although evaluation of The Robert B. Balter Company (Balter) (the project inspector) compressive strength testing of the sample cylinders led PB to determine that concrete had attained the 4,000 psi minimum strength necessary to commence post-tensioning stressing, the findings of this report conclude that in-situ concrete was likely less mature and of questionable strength at the time stressing commenced. Cracking observed during the first month following concrete placement appears consistent with drying and shrinkage resultant from improper curing, and the horizontal cracking in the beams and girders documented by KCE during its testing is likely resultant from excessive stressing force applied to immature concrete.

However, after this initial setting and curing period whose passage is approximated by the 28-day compressive strength tests, existing cracks worsened, and new cracking appeared. We have found no evidence that the cracking that persisted after the 28 day period could have resulted from any cause other than design issues.

Problems with structural design and construction were identified by late 2010, and repeatedly discussed in subsequent Project Management Team meetings, but were not effectively addressed.

In a reactive response to problems that were identified during construction, DGS contracted with an independent firm, KCE, but did not do so until 2012, when the structure was almost complete.

In hindsight, the County would have benefitted from retaining an objective third party firm to perform a "peer review" function during the design of the structure.[3] That firm could have been retained to work with the design professionals to either substantiate or modify the design.

[3] See discussion of Peer Review in Finding 7.

The County also would have benefitted from retaining an objective third party firm to perform the Construction Management function during the construction.

Structural Remediation ***(see page 53)***

As a follow-up to a meeting held on April 25, 2013, a Cooperative Remediation Working Group (CRWG)[4] was formed to develop a plan to remediate the defects at the SSTC with a resultant structure that meets the design and operational objectives and standards outlined in the project documents.

The CRWG quickly agreed upon, designed, and implemented corrective actions to strengthen both of the Level 330 pour strips. Those actions were completed by the end of 2013. The CRWG also adopted a plan to fill slab cracks and resolve the slab thickness deficiencies by topping the Level 330 and 350 slabs with a Latex Modified Concrete (LMC) overlay that will be applied once the weather and temperatures permit, and decisions about other remedial actions necessary to address durability issues have been made. As of the mid-April 2014, the CRWG had not agreed upon a remediation plan to address the latter issues.

What We Recommend

Recommendation 1: DGS should improve its controls for future projects in a manner that is consistent with the lessons learned and additional recommendations contained in Exhibit I, the report "Analysis of Project Controls," in addition to other recommendations made in this report.

Recommendation 2: DGS should ensure construction documents clearly establish responsibility for and performance of systematic analysis of data collected and recorded during construction in order to identify possible inconsistencies with specifications, project control weaknesses, and construction deficiencies that should be investigated and resolved.

Recommendation 3: In future projects, DGS should ensure that all specification requirements are reviewed and implemented unless a variance is mutually discussed and agreed upon. Temperature limits during curing should be monitored and maintained, and specification for duration of curing should be strictly observed. Confusion about where to take samples and about cold weather limits should be avoided by clearer language in

[4] The CRWG is comprised of key participants in the SSTC project, representing Montgomery County, the Federal Transit Administration, the Maryland Transit Administration, the Washington Metropolitan Area Transit Authority, Parsons Brinkerhoff, Foulger Pratt, and KCE, as well as their respective consultants and subcontractors.

specifications. Any conflicts between specifications and standards should be resolved in favor of the more conservative of those required by stakeholders (in the case of the SSTC, the stakeholders are DGS, and WMATA).

Recommendation 4: DGS should modify its contract specifications for future construction projects to ensure that concrete test specimens are made as near as possible to the actual point where concrete is placed. Where referenced standards require testing at the point of delivery, DGS should clarify in the specification that such testing is in addition to typical testing.

Recommendation 5: In future projects, DGS should ensure its construction contractors utilize a construction method that allows direct measurement of floor thickness so that inspectors can help the Contractor by identifying problems before the concrete is placed. Alternatively, a second, independent survey should be performed. Survey equipment could be utilized by inspectors to continuously monitor concrete thickness during placement, and submit a report of survey results for Owner and Structural Engineer of Record (SEOR) approval.

DGS should hold construction contractors accountable for any remediation and increased maintenance costs that will likely result from the contractor's failure to ensure specified concrete slab thickness was attained during placement.

Recommendation 6: Those professionals whose lack of diligence resulted in the pour strip construction deficiencies should be held accountable.

DGS should consider implementation of changes to guard against occurrence of such errors in future projects, for example:

- All shop drawings could be required to be submitted before the pre-installation conference occurs, or
- A pre-installation conference could occur with each new area covered by a recently approved shop drawing, or
- A Submittal Registry should project the number and identity of proposed shop drawings anticipated for all phases. (For example, if only one pre-installation conference occurs at the beginning of the Definable Feature of Work, part of the conference should identify the number of submittals that will be generated for Designer review for the phased construction. Then as construction proceeds discussion should occur whether each of those proposed submittals have been approved during the progress meetings.)

Recommendation 7: DGS should develop procedures to identify circumstances under which an independent peer reviewer should be employed to review and improve the design of unique and challenging construction projects. The trigger for a peer review could be the nature and complexity of the project design.

Recommendation 8: DGS should develop procedures to identify circumstances under which an independent third party should be employed to serve as Construction Manager on an atypical construction project. The trigger could be a dollar value or uniqueness of the project.

DGS should develop protocols to ensure that controversial issues encountered/problems experienced by or with the construction contractors are promptly and effectively addressed. As an example, DGS could develop and incorporate into its contracts a systematic process that identifies deficiencies and withholds payments pending resolution. Once an item is identified as deficient, it would be added to a "rolling punch list" which is tied to payments. Therefore, the Contractor is motivated to correct issues in a timely manner. Foulger-Pratt Contracting (FP) generated their own internal contract compliance list, which was included and discussed at progress meetings, but evidently was not tied to payments.

Subsequent Event

On May 8, 2014, the County Chief Administrative Officer advised members of the County Council that the County Executive had directed County contractors to move ahead on remediation work at the Silver Spring Transit Center. That work would address the shear and torsion recommendations contained in the April 21, 2014 report commissioned by the County Executive entitled Report of the Independent Advisory Committee Regarding the Status of the Silver Spring Transit Center.

Summary of Chief Administrative Officer's Response

The response of the Chief Administrative Officer (CAO) to the final draft report is included in its entirety on page 55 of this report. The CAO addressed each recommendation individually in his response. The responses did not cause us to alter our findings or recommendations.

Analysis

Analysis of Project Controls

Project Controls

Finding 1: Fourteen of the 22 relevant construction project controls analyzed for adequacy of design, implementation, and effectiveness were either weak or ineffective.

We engaged the Alpha Corporation to evaluate those project controls used during the construction of the SSTC that should have directly controlled the construction activities related to the deficiencies identified by KCE and WDP in their reports. We asked that in their analysis, they first determine whether a control, if properly implemented, should have been effective as designed, and second, whether the control was in fact implemented as designed.

If a control was not properly designed but correctly implemented, the expected outcome would be that the control was ineffective and a negative result, such as an error or construction deficiency, could have gone undetected and uncorrected. Alternatively, if a control was properly designed but not correctly implemented that control would also be ineffective and a negative result, such as an error or construction deficiency, could also be have gone undetected and uncorrected. If all construction project controls were appropriately designed and implemented, the deficiencies identified by KCE and WDP at the SSTC should not have existed, with the possible exception of deficiencies that could have resulted from flawed design elements.

In their report to the OIG, "Analysis of Project Controls", the Alpha Corporation found that the design of nine construction project controls was either weak or inconsistent with contract requirements. They also found that implementation of ten controls was either weak or deficient. Overall, eleven controls were determined to be either weak or ineffective, and the effectiveness of four other controls could not be determined from the data available.[5] Their detailed analysis of these controls is presented in Exhibit I, page 10, and summarized in Chart 1.

Chart 1: Deficient Project Controls

Control	Deficiency Observed in Control			Effectiveness	No
	Design	Implementation	Effectiveness	Unknown	Deficiency
Pour Strips					
RFIs & Meetings			✓		
Submittal Review		✓	✓		
Pre-Installation Conference	✓		✓		
Daily Reports			✓		
Concrete Composition					
Pumped Concrete Samples	✓		✓		
Batch Plant Inspections	✓	✓	✓		
Concrete Mix Design					✓
Water Added at Site			✓		
Slump Measurements	✓				
Cold Weather Curing	✓	✓	✓		
Surface Curing	✓	✓			
Entrapped Air					✓
Entrained Air		✓		✓	
Concrete Placement					
PT Tendon Placement			✓		
Steel Rebar Placement		✓		✓	
Floor Thickness	✓	✓	✓		
Post Tensioning					
Stressing Records					✓
Concrete Stresses	✓	✓	✓		
Grout Strength		✓		✓	
Time to Grouting		✓		✓	
Strength at Stressing	✓				
Age at Stressing					✓

Source: Alpha Corporation

Although each project control may have operated in isolation, many of these controls operated collectively during the construction period as systems intended to control time, cost, scope, and quality. This OIG staff analysis of the body of narrative and statistical information available to us, combined with the Alpha Corporation's analysis of individual project controls, identified specific areas of concern that are presented in findings 2 through 8 and the related recommendations that follow. The recommendations are consistent with those presented in "Analysis of Project Controls" even though only some of the recommendations are drawn directly from that report.

The "Analysis of Project Controls" contains "lessons learned" and additional recommendations in the Considerations and Conclusions sections of the Alpha Corporation report.

Recommendation 1

DGS should improve its controls for future projects in a manner that is consistent with the lessons learned and additional recommendations contained in Exhibit I, the report "Analysis of Project Controls," in addition to other recommendations made in this report.

[5] Some controls deficiencies met more than one criteria of finding.

Deficiencies in structural strength resulted from work that did not meet contract requirements and was undetected by inspectors

Concrete compressive strength in some areas as measured by KCE is less than that required by the contract documents. All data we reviewed indicates that water was added to concrete after testing specimens were collected. In addition, concrete did not cure properly in many areas. Samples taken at the inspection station produced compressive strength test results that were not representative of the strength of the in-situ concrete. As a result, areas were identified in which concrete strength is weaker than required by the design. These results were based on an analysis that primarily focused on the concrete slabs on Levels 330 and 350 of the structure.

Structural Strength

Finding 2: Analysis of data collected during construction indicates that addition of water to concrete after collection of primary testing specimens but before placement of the **concrete in the structure accounts for the lesser strength of the in-situ concrete.**

Compressive strength was to be tested by collecting specimen cylinders of fresh concrete and measuring the force needed to break the concrete cylinders at prescribed intervals as they hardened.[6] Design and construction quality control specifications required that a set of test cylinders be made for each 50 cubic yards (yd^3) of concrete poured in order to confirm whether concrete in post-tensioned members had reached required design strengths. Controls were designed and observed to capture the adequate number of compressive strength test cylinders.

In order to achieve sufficient strength for designed loading requirements, SSTC Construction Documents[7] required that the concrete achieve a minimum compressive strength[8] of 4,000

[6] For an explanation of this test, reference the Portland Cement Association, Washington, DC 20001, “What are the most common tests for fresh concrete?” Web. 20 January 2014. <http://www.cement.org/cement-concrete-basics/faqs>.

[7] Construction Contract between Montgomery County and Foulger-Pratt Contracting dated September 3, 2008, Attachment A – Schedule of Documents, List of Specifications, § 03300-Cast-In-Place Concrete, Part 2.16(E)

pounds per square inch (psi) before commencing tendon stressing[9] and 8,000 psi 28 days after the concrete was poured.

During this project, the concrete was primarily collected for testing at the end of the *concrete truck chute* at the inspection station. For most testing sets made during a pour, twelve[10] specimen cylinders were collected from tested trucks at the inspection station and before discharge into the hopper of the pump used to deliver concrete to the point of placement on the deck. For three additional testing sets, The Robert B. Balter Company (Balter) was directed to cast another six comparison cylinders on the deck at the end of the *concrete pump hose*. This casting of comparison sets (which was directed by DGS) was fortuitous as it provided the evidence of differences between inspection station and in-situ concrete.

The majority of Balter-reported laboratory test results indicated that compressive strength of the collected specimens exceeded minimum required values. Much later, KCE Structural Engineers, P.C. (KCE) excised sample cores from slabs to test for the compressive strength of the in-situ concrete, and determined the samples "exhibited significantly lower compressive strengths when compared to [the Balter-reported compressive strengths]." Based on this structural analysis, KCE concluded that the concrete strength for all deck pours was 6,970 psi.[11]

KCE's report states "Our analysis of the as-built post-tensioned slabs indicates slab areas with thicknesses below approximately 9 inches and with compressive strengths at or below 6,970 psi do not have adequate shear capacity in certain locations to support the design loads (the areas less than 9 inches thickness are limited in extent and therefore do not limit overall load-carrying capacity)." (See our discussion of concrete thickness in Finding 5 of this report.)

Chart 2 displays a comparison of a sample of KCE test results (conducted on core samples excised from the deck of the SSTC) to results of testing conducted by Balter that had been taken from the same location in the SSTC (we identified nine sets of KCE and Balter test results that had been taken from concrete for the same location in the SSTC).[12] (A comparison of all results may be found in Exhibit IV of this report.) With the exception of comparison samples that were taken at the point of placement, Balter's primary test results were based on samples taken at the inspection station. As indicated in Chart 2, the compressive strength determined from KCE-tested, in-situ specimens ranged from 5,330 psi to 11,040 psi, while the Balter-tested, lab-cured

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8 The measured resistance of a concrete or mortar specimen to axial loading; expressed as pounds per square inch (psi) of cross-sectional area. Source: http://www.allmetalssupply.com/concrete_terms.htm @ 17:50 on 1 August 2013

9 The Construction Contract between Montgomery County and Foulger-Pratt Contracting dated September 3, 2008, Attachment A – Schedule of Documents, List of Specifications, § 03381-Bonded Post-Tensioned Concrete, Part 3.7(C) required that stressing operations not begin until concrete strength had reached 4,000 psi as indicated by compression tests of field-cured cylinders, and that stressing be limited to 50 percent of the total tendons until the concrete had achieved 6,000 psi strength. Part 3.7(D) required that stressing of 50 percent of the total tendons be completed within 96 hours of concrete placement.

10 During the course of the project, 14 primary cylinders were collected for later pours.

11 KCE strength values have been converted to "equivalent specified strengths", and are based on formulas recommended by ACI 214.4R-10 in an attempt to approximate equivalency with Balter values reported under the AASHTO T22. The reader should be aware of the professional judgments that are required when interpreting core sample strengths. Comparisons between test results obtained through application of different standards should be accompanied with an understanding that there is some uncertainty in the comparison. See Exhibit III: Standards

12 The location of the Balter sample was determined from the "Location of Sample" documented by the Balter inspector on the "Compressive Strength Test Specimen Data" report. The location of the KCE sample was determined by reference to the KCE exhibit that mapped the location of each extracted core.

specimens at 56 days after the pour dates ranged from 12,480 psi to 14,400 psi.[13] KCE's highest result was less than Balter's lowest result. For the four pours in this sample, the average compressive strength of the KCE samples was only 62% of the compressive strength of the Balter samples (with a minimum of 37% and a maximum of 86%).

Chart 2: Comparison of KCE In-Situ Compressive Strength & Petrographic Test Pairings to Balter Construction Inspection Tests

Pour Information Date	#	Testing Information Core #	KCE TestType	Strength (psi)	w/c	unhydrated
20-Dec-10	1D	72	Compressive Strength	7,100		
		71	Petrographic		.35 - .40	
KCE-Reported average compressive strength - Pour 1D				6,780		
56-day RBB Test Cylinder Batch91832: Test Report # 522*				14,400	.26	
30-Dec-10	1F	105	Petrographic		.35 - .40	
		106	Compressive Strength	9,350		
		107	Compressive Strength	9,000		
		108	Petrographic		.35 - .45	8% - 13%
KCE-Reported average compressive strength - Pour 1F				6,990		
56-day RBB Test Cylinder Batch 92297: Test Report # 551				13,495	.26	
7-Dec-10	2B	121	Compressive Strength	**11,040**		
		122	Petrographic		.35 - .40	
		123	Petrographic		.35 - .45	7% - 11%
		124	Compressive Strength	10,060		
KCE-Reported average compressive strength - Pour 2B				8,810		
56-day RBB Test Cylinder Batch 91111: Test Report # 486				13,575	.26	
56-day RBB Test Cylinder Batch 91160: Test Report # 495				13,740	.24	
14-Jan-11	2C	127	Petrographic		< .38	
		128	Compressive Strength	10,710		
		131	Petrographic		.35 - .45	7% - 12%
		132	Compressive Strength	5,330		
KCE-Reported average compressive strength - Pour 2C				6,870		
56-day RBB Test Cylinder Batch 93009: Test Report # 590				12,480	.26	
56-day RBB Test Cylinder Batch 93019: Test Report # 591				14,390	.26	

The factor that most influences concrete strength is the ratio of water to the cement that binds the aggregates together. The higher the ratio of water to cement, the weaker the concrete will be and vice versa. The Portland Cement Association opines that every desirable physical property that can be measured will be adversely affected by adding water.[14] Alternatively, by reducing water, the resulting higher-strength concrete can carry loads more efficiently than normal-strength concrete, possibly reducing the total amount of material placed, and lowering the overall cost of the structure.[15]

To achieve a high degree of durability and strength, the American Concrete Institute (ACI) recommends the use of a concrete mixture with a low *w/c*, but notes an insufficient amount of water can inhibit the complete hydration of the concrete mixture.[16] Too much water and the concrete does not achieve the pore density it needs for optimum strength. Water not absorbed during hydration remains as free water, which can bleed to the surface or evaporate. Excessive loss of water due to evaporation can promote the development of the plastic shrinkage cracking that was observed in late 2010 (see Finding 7).[17] Too much water produces a weaker, less

[13] All testing facilities reported observation of the same industry testing standards.

[14] Portland Cement Association, Washington, DC 20001, "What are the most common tests for fresh concrete?" Web. 20 January 2014. <http://www.cement.org/cement-concrete-basics/faqs>.

[15] Portland Cement Association, Washington, DC 20001, "High-Strength Concrete." Web. 20 January 2014. <http://www.cement.org/cement-concrete-basics/concrete-products/high-strength-concrete>.

[16] *American Concrete Institute (ACI)*, Farmington Hills, MI 48333-9094, Guide to Curing Concrete (ACI 308R-01), Chapter 1.3.1

[17] *Bonini, Julius PE, M and P Labs,* Schenectady, NY, and *Smith, Andrew, PhD, CERAM Research, Stoke-on-Trent, UK. "Forensic Investigation of Hardened Concrete: Water/Cement Ratio." Presentation at the Celebration of Capital District's National Engineers Week I7 February 2011. Albany, NY. Web. 20 January 2014.* <http://www.mandplabs.com/interface/uploads/files/pdfs/Forensic_Investigation_Hardened_Concrete.pdf>

durable concrete that will contribute to early spalling of the surface. Too little water, and the cement does not complete the chemical reaction required to achieve optimum strength.[18]

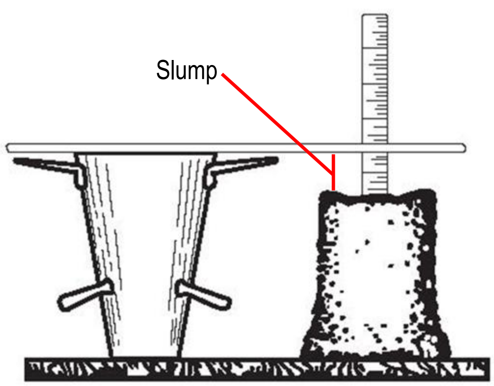
KCE's petrographic testing results presented in Chart 2 indicate each of those pours contained a water to cement ratio (*w/c*) ranging from 0.35 to 0.45. Balter reported *w/c* ranged from 0.24 to 0.26.

Slump Test Results were routinely inconsistent with other support data recorded by Inspectors

As previously stated, concrete matures and hardens as it cures via hydration. Concrete that has a low *w/c* ratio may be more difficult to work with due to its higher viscosity.[19] To overcome this problem, special additives (admixtures) may be added to the concrete in place of small amounts of water. Those admixtures improve the workability of the concrete.

The *w/c* ratio of fresh concrete cannot be directly tested, so the quantity of water added is controlled via records from both the batch plant and the project site. The Statement of Special Inspections[20] required that Balter provide project-site verification of the design mix in use.

Image 1: Illustration of Slump Test Measurement



Source: Gates Concrete Forming Systems

Absent a field test for *w/c*, an indicator of the amount of water in a mix may be its workability, and workability can be field measured by means of a slump test.[21] There is no established direct relationship between *w/c* and slump, but slump should be less (less workable) after a period of time than it was when it arrived on site unless water has been added. As an example, concrete with slump of 7" would be expected to contain less water than would concrete with a slump of 8" but an otherwise equal amount of admixture, cementitious material, and age.

As represented in Image 1, a standardized conical shape is filled with fresh concrete. When the mold is removed, the fresh concrete subsides and is measured against the original conical shape. Fresh concrete that is more viscous would contain a greater amount of water or admixture, and would have a greater tendency to collapse in height. Conversely, concrete would tend to stay close to its conical shape if lesser water and admixture are present or if the concrete has had an opportunity to age and commence the curing process.

For the high strength concrete mix to be used on levels 330 and 350, Parsons Brinckerhoff (PB) had approved a slump of "*4" or 8" for concrete with a verified slump [a test made at the*

[18] Portland Cement Association, Washington, DC 20001, "What are the most common tests for fresh concrete?" Web. 20 January 2014. <http://www.cement.org/cement-concrete-basics/faqs>.

[19] Viscosity is a measure of the friction between neighboring particles in a fluid. For liquids, it corresponds to the informal notion of "thickness". For example, honey has a higher viscosity than water. Source: Wikipedia, accessed 8 April 2014

[20] See the discussion of the Montgomery County Special Inspections Program in Exhibit I, page 57.

[21] A conical form is filled with concrete, inverted, and the form removed. The amount of height in inches the cone loses during a set period of time measures slump. Refer to "Slump Measurement" Alpha Corporations SME report.

batching plant and verified by an inspector that the concrete had a slump] of 2"-4" before a high-range water reducing admixture is added." WMATA provided only for a 2 to 4 inch slump. Slump tests conducted by Balter routinely measured at 7-8 inches. With the passage of time and absent the addition of water or admixtures to extend the workable life, concrete can be expected to begin setting up, which, in turn, would result in a smaller slump measure.

Data collected as a part of the normal construction process was analyzed by the OIG. Chart 3 includes data from a sample of four Rockville Fuel and Feed Co (RFF) batch tickets and Balter Reports of Concrete Cylinder Tests. This data compares Balter compressive strength test results of cylinders that their inspectors collected at the inspection station to the results for cylinders they collected at the point of placement, and catalogs for each test the slump measurement, the air content, the amount of water added on site, if any, the number of times the truck's mixing drum revolved, the water to cement ratio identified by RFF on its batch ticket, and the time that elapsed in minutes between batching the concrete at the plant and the collection of the specimen cylinder at the project site.

Chart 3: Comparison of Same Batch, Inspection Station to Surface Deck Field Cured Strength Results

Concrete Batch			Sample	RBB Strength Test		Slump	Air Content	Added H_2O (gal)	Revs	W/C ratio	Time Lapse	3-Day Strength		28-Day Strength	
Pour	Truck #	Ticket #		#	Location							Sample 1	Sample 2	Sample 1	Sample 2
1 D	67	91818	Set 1	518	Inspection Station	6.5	5.1%	20.0	71	0.25	53	10,480	10,220	13,100	13,440
				519	Deck	6.5	5.3%	20.0	71	0.25	74	5,140	5,020	10,620	10,890
	77	91837	Set 2	523	Inspection Station	7.0	6.2%	0.0	112	0.26	45	9,190	9,580	12,100	11,820
				524	Deck	8.0	5.7%	0.0	112	0.26	65	3,820	3,930	7,550	7,410
	79	91883	Set 3	530	Inspection Station	7.0	5.7%	0.0	250	0.26	53	9,910	10,190	11,470	11,460
				531	Deck	7.5	7.5%	0.0	250	0.26	73	4,460	4,130	9,120	9,510
1 F	77	92269	Set 1	543	Inspection Station	7.5	5.0%	0.0	116	0.26	19	6,560	6,730	12,220	11,700
				544	Deck	7.0	4.7%	0.0	150	0.26	44	6,910	6,960	8,780	9,340
	62	92282	Set 2	547	Inspection Station	8.0	6.3%	0.0	120	0.26	45	7,930	7,810	12,690	12,660
				548	Deck	7.5	5.8%	0.0	153	0.26	75	6,120	6,670	9,160	9,250
	32	92316	Set 3	554	Inspection Station	8.0	6.1%	0.0	128	0.25	52	5,700	5,310	12,040	11,910
				555	Deck	7.5	5.9%	15.0	160	0.27	101	7,190	7,550	8,680	8,730
2 B	67	91088	Set 1	481	Inspection Station	8.0	6.3%	0.0	195	0.25	41	4,080	4,150	11,150	10,670
				482	Deck	8.0	5.1%	0.0	195	0.25	62	4,270	4,590	9,280	8,840
	69	91152	Set 2	493	Inspection Station	7.5	5.1%	0.0	119	0.26	77	6,840	6,910	12,680	12,790
				494	Deck	8.0	4.6%	0.0	119	0.26	101	5,990	6,060	11,180	11,310
	37	91251	Set 3	507	Inspection Station	7.0	4.7%	0.0	88	DNA	78	4,300	3,960	11,240	10,130
				508	Deck	7.0	4.2%	0.0	88	DNA	94	5,750	5,740	10,100	10,260
2 C	67	92950	Set 1	578	Inspection Station	7.0	4.5%	0.0	176	0.26	57	7,060	6,490	11,400	11,600
				579	Deck	8.0	4.3%	20.0	195	0.28	67	7,080	7,170	11,200	11,140
	81	92978	Set 2	585	Inspection Station	8.0	5.6%	0.0	110	0.26	60	5,380	5,300	12,890	13,120
				586	Deck	8.0	5.4%	0.0	110	0.26	75	8,030	8,060	12,830	12,700
	61	93053	Set 3	594	Inspection Station	7.0	4.8%	0.0	250	0.26	95	6,380	6,590	13,170	12,650
				595	Deck	8.0	5.1%	0.0	250	0.26	109	5,390	5,160	9,620	9,110

DNA = Data Not Available. Source: Robert B. Balter Company Report of Concrete Cylinder Test and Rockville Fuel and Feed Company, Inc. job batching and delivery tickets.
3-Day Strength results for Pour 1 F were actually tested on Day 4.

Data for pour 1F in Chart 3 is typical for expected results. Over time and in the absence of additional water, the slump demonstrates less workable concrete resulting in smaller measures. Even though water is added in set three, it does not compensate for the additional setting time.

By referencing slump test results in Chart 3 for comparison specimen Set 2 of Pour 2B, the conical slump made at the inspection station dropped 7½" from the height of the shape during the test. Twenty-four minutes later, when the test was repeated on the deck, the shaped concrete "slumped" 8" - another ½" - demonstrating a lesser viscosity and more workability despite the passage of time and the absence of recorded additional water or admixture. Slump, as a measure of consistency and workability of wet concrete, is a standardized test that should yield consistent results when comparing as-mixed to as-placed slump values. An increase in the slump measurement could be an indicator of added water.[22]

In five of the twelve comparison sets, the slump measures presented in Chart 3 do not appear to be supported by the other data in the chart. The second slump test taken on the deck for Pour 1D comparison Sets 2 and 3, and Pour 2B comparison Set 2 demonstrated a slump with more workable concrete even though Balter did not record the addition of any water and there was a passage of 15 minutes or more since the first test at the inspection station. More workable concrete was also observed at Pour 2B comparison Sets 1 and 3and Pour 2C Set 2, although those results indicate no change in the slump measurement. Exhibit V (from which the sample in Chart 3 was extracted) illustrates that of all 37 comparison sets, there were seven occurrences when the second specimen collected at the deck presented a slump measure of concrete that was equal to or more workable than the slump tested at the inspection station despite no recorded addition of water or passage of 15 minutes or more between tests. It is possible that undocumented water could have been added in other instances without manifesting itself in the slump test. A slump test indicating a more workable concrete despite the passage of time with no addition of water raises questions about the accuracy and validity of the recorded data, as the results appear to be inconsistent with the other data.

Relationship between compressive strength and addition of water

Compressive strength test results indicated in Chart 3 show that concrete samples taken at the inspection station demonstrated different 28-day strength than the comparison tests of specimens collected at the deck.

Chart 3 uses Balter and RFF data to provide a comparison of field cured compressive strength test results on specimen cylinders made from the same batch of concrete, with one set collected at the inspection station and the other set on the deck. Of the 56 total field-cured specimens compared in Exhibit V, 49 deck specimens[23] demonstrated a lower compressive strength that was, on average, just 83% of the strength of its inspection station counterpart (with a minimum of 48%, and a maximum of 99%). For example, Set 2 of Pour 1D shows three-day compressive strength of 3,820 and 3,930 psi[24] for the specimens collected at the deck, while the specimens collected at the inspection station indicated strength of 9,190 and 9,580 psi. At 28 days, the strength disparity continued with deck specimens with 7,550 and 7,410 psi compared to inspection station specimens

[22] *Bonini, Julius PE, M and P Labs,* Schenectady, NY, and *Smith, Andrew, PhD, CERAM Research, Stoke-on-Trent, UK. "Forensic Investigation of Hardened Concrete: Water/Cement Ratio." Presentation at the Celebration of Capital District's National Engineers Week I7 February 2011. Albany, NY. Web. 20 January 2014.* <http://www.mandplabs.com/interface/uploads/files/pdfs/Forensic_Investigation_Hardened_Concrete.pdf>
[23] After three days of curing, 52% of the inspection station samples exhibited greater compressive strength than the deck samples.
[24] Note well that both of these compressive strength test results were below the 4,000 psi acceptance limit for 3-day test results.

with 12,100 and 11,820 psi. For most testing sets made during a pour, 12 specimen cylinders were collected at the inspection station. Foulger-Pratt Contracting (FP) primarily relied upon test results of the inspection station specimens. Three additional testing sets were made during the pour, from which 6 additional specimen cylinders were collected on the deck surface after the concrete had been pumped to the point of placement. Although data for all 18 specimens was available for these three comparison testing sets, records do not indicate a comparison was made by FP, nor do records indicate that Balter highlighted the matter as a possible concern in communications with DGS. These documented inconsistences in the compressive strength of cylinders could have been compared and the differences investigated.

The data in Chart 3 further indicate, for example, in Pour 1D Set 1, a difference in 28-day strength of specimens where records do indicate water was added to some concrete compared to those specimens collected at the inspection station before the addition of water. This added water could have acted to diminish the strength of the structure.

Estimation of additional water

Prior to commencement of construction activities, the Structural Engineer of Record (SEOR) approved the proportions of the ingredients to be used for concrete in the structure. Proportions of ingredients in the concrete varied depending upon on the strength of the concrete required for use. For the 8,000 psi concrete mix used for the slabs, beams, and girders, the SEOR approved mixture called for 32 gallons of water per cubic yard of concrete to obtain a 0.29 *w/c* ratio. Approval was granted for the use of optional admixtures, with a requirement to decrease water in an amount necessary to offset the moisture content represented by the admixture in order to maintain the 0.29 *w/c*. The admixtures used required that water be reduced by approximately one gallon for each gallon of admixture used in the mix. For each 10 cubic yard batch, the batch tickets noted that the total water content, including the admixtures, was not to exceed 310 gallons.

Balter asserts it noted on specimen cylinder data sheets the number of gallons of water its inspectors observed being added to the concrete, and further asserts that its sampling and testing was performed after any water was added to the concrete load at the project site. In every case in which added water is documented, the amounts of additional water that Balter reported were not in excess of RFF indicated amount of water allowed at the jobsite.

Of the 37 comparison sets analyzed in Exhibit V, Balter inspectors documented twelve sets (32%) where water was added to the concrete. In seven sets (19%) where the addition of water was documented, that addition occurred between the inspection station and point of placement, and after superplasticizer[25] and other admixtures had been added.

[25] A chemical added to concrete in lieu of water to improve the viscosity and flow of concrete.

In Chart 3, Pour 2B documents record no evidence of added water. However, KCE's petrographic results (presented in Chart 2 on page 15) indicate this pour presented *w/c* ranging from 0.35 to 0.45. The minimum *w/c* ratio in KCE's petrographic-tested cores was 0.35. The maximum *w/c* (before the addition of any on-site water) reported by RFF and documented by Balter was 0.26 for any location proximate to the KCE-excised core.

In Chart 4, the weight of the cementitious material is reported under the heading "C+P lbs", and the weight of the water content (water plus moisture content of admixtures) is reported under the heading "Water lbs". The *w/c* was then calculated by dividing the total "Water lbs" by the "C+P lbs". During the production of concrete, the sand and stone components would have contained some amount of moisture, an offset for which should have been quantified by RFF and held out of the water added to the mix. Based on OIG calculations in Chart 4, the KCE-excised core would have contained water content of 38 gallons per yd^3 of concrete. This core contained 36% more water ($\frac{(38-28)}{28}$) than the Balter specimen cylinder with 28 gallons of water per yd^3 and a 0.26 *w/c*. In Exhibit IV (data from which is represented in Chart 2), the average Balter reported *w/c* was 0.26 while the minimum *w/c* KCE reported was 0.35.

Chart 4: Difference in KCE-Excised Core and Balter Specimen Cylinder *w/c*

	KCE Core Minimum *w/c*		C+P *lbs*	Water Content lbs	Water Content gal	RFF Cylinder Maximum *w/c*		C+P *lbs*	Water Content lbs	Water Content gal
Sand	1,000	lbs			0.0	1,000	lbs			0.0
Stone	1,850	lbs			0.0	1,850	lbs			0.0
Cement	550		550			550		550		
Slag	360	lbs	360			360	lbs	360		
AEA	7.8	oz		0.5	0.1	7.8	oz		0.5	0.1
WRDA	41.0	oz				41.0	oz			
Super	54.6	oz		3.4	0.4	54.6	oz		3.4	0.4
DCI	256.0	oz		16.0	1.9	256.0	oz		16.0	1.9
Water	36	gal		298.6	35.8	26	gal		216.7	26.0
Water Added										
			910	318.5	$\boxed{38.2}$			910	236.6	$\boxed{28.4}$
w/c ratio			$\frac{319}{910}$ =	**0.35**				$\frac{237}{910}$ =	**0.26**	

weight of water in lbs/gallon ***8.32967***

Source: OIG original work paper.

Inspection inconsistencies

As indicated below, Balter did not fulfill all of the requirements set out for it under the Construction Documents, the Statement of Special Inspection, and its contract with the County.

The Statement of Special Inspections required Balter to periodically inspect RFF's plant operation to verify materials identified for the approved concrete mix were being provided.[26] Balter asserted that it had "requested [to inspect RFF's] plant several times, but authorization was never granted,"[27] even though PB advised Balter that the "concrete plant inspection can

[26] Statement of Special Inspections, Concrete Element, part 4 – Verifying use of required design mix.
[27] RBB Letter regarding Response to KCE Report Dated March 15, 2013, April 22, 2013, page 8. "[RBB] requested inspection of Rockville Fuel and Feed's plant several times, but authorization was never granted."

occur anytime, [DGS] and [Balter] to coordinate a time."[28] Based on available documentation, the requisite Balter inspections were not performed at any time during the period of major construction activities.

Concrete was transported from the plant to the site in trucks that carried a load of 10 yd^3 of concrete. The primary batch plant was located 11 miles from the SSTC.[29] Data from delivery documents revealed some concrete placed in the structure had reached or exceeded 90 minutes from the time it was batched. Construction Documents and industry standards[30] require that concrete be completely discharged within 90 minutes of mixing, or before 300 revolutions of the truck's mixing drum, to prevent concrete from setting up before placement. Seven of the test specimen sets evaluated in Chart 3 and Exhibit V reached or exceeded 90 minutes in age prior to discharge (and 4 of these more than 100 minutes in age).[31] We noted only one Balter daily report that documented a load of concrete had been rejected due to excessive age.

Conclusions

During concrete placement, three sets of specimen cylinders were collected on the deck for comparison to other specimen cylinders collected from the same batch at the inspection station. Inspection data records that between the two tests water was added to the concrete in seven of the 37 total comparison specimens (19%). These same records indicate that the water added did not exceed the hold back amount designated by RFF.

However, 888 concrete trucks loads would have delivered all the concrete used to construct levels 330 and 350. Only 233 (26%) of these were tested. Testing of concrete specimens collected from the deck occurred for only 37 (4%) of all truck loads.

KCE petrographic testing of extracted specimens suggests in-situ concrete contained 36% more water than RFF calculated and Balter reports document. Slump testing also suggests an addition of water. Nineteen percent (7 of 37) of the comparison sets record slump tests demonstrating an equal or more workable concrete (indicating thinner concrete) was placed on the deck despite the passage of 15 or more minutes and no documented addition of water.

If water was added, it could have been done so before placement the deck after arrival at the construction site or at the concrete plant when the concrete was batched.[32] While there may have been economic gain by substituting water in lieu of admixtures (admixtures are relatively

28 PB Construction Progress Meeting #43, July 15, 2010 minutes. Item 3.1 of FP Preinstallation Conference minutes dated April 28, 2010 is similar and reads, "Mike Bailey indicated there is a requirement for [RBB] to inspect the concrete plant. John Hershey indicated any of us could call and come by anytime."

29 Internet mapping systems approximate as a 16 minute journey.

30 *ASTM International* (formerly American Society for Testing and Materials), West Conshohocken, Pennsylvania, Standard Specifications for Ready Mixed Concrete (ASTM C94/C94M), Section 12.7

31 ASTM C94/C94M, Section 12.7– allows a waiver by the purchaser to the time and revolution limitation if the slump was reached without addition of water. Water was added to the concrete in all documented instances where more than 100 minutes elapsed. RBB asserted that Montgomery County and WMATA allowed a deviation of 10 to 15 min (not in hot weather) provided the concrete did not appear to change consistency

32 Although it was possible a driver could have stopped in route to add water, that scenario is not probable and is inconsistent with documented transit times.

expensive), and the resultant *w/c* and petrographic analysis would have been consistent with KCE's test result, the 8,000 psi high strength mix would not be achievable without the admixtures.

From the data we examined, we found no single reason for the lower compressive strengths found by KCE. Evidence does exist that Balter samples were not representative of the concrete at the pump end, providing opportunity for undocumented water to be added at the construction site. The remaining difference between the Balter and KCE reported *w/*c may be found in the petrographic results themselves. The results are labeled as estimates, and the methodology in assigning a *w/c* value is not exact. However, it is likely that extra water was added at the construction site.

For the three comparison testing sets collected during the pour, twelve of 18 concrete specimen cylinders from each truck were collected before discharge into the hopper for the pump used to deliver concrete to the point of placement on the deck. If, as in-situ testing results suggest, and as Balter comparison specimen data indicate, water was added to the concrete mixture, controls as designed were inadequate for ensuring that water additions adhered to specifications and variances. In light of the recurring instances of shrinkage cracking documented throughout this construction project, analysis of the data collected, tested, and available to FP, Balter, PB, and DGS and their subcontractors could have identified inconsistences whose cause could have been investigated and remedied.

Recommendation 2

DGS should ensure construction documents clearly establish responsibility for and performance of systematic analysis of data collected and recorded during construction in order to identify possible inconsistencies with specifications, project control weaknesses, and construction deficiencies that should be investigated and resolved.

Structural Strength

Finding 3: Records collected during construction demonstrated that 1.) construction specifications for cold weather curing were not implemented correctly, and 2.) surface temperatures were not maintained or monitored as required by specifications.

Concrete is a composite material in which Portland cement, water, aggregates, and admixtures are bound together through a chemical and physical reaction of cement with water (hydration).[33] Concrete construction requires proper curing to increase concrete strength and durability. Concrete curing is defined as "the process by which concrete matures and develops hardened properties over time as a result of the continued hydration[34] of the cement in the presence of sufficient water and heat."[35] Diminishment of these hardened properties leaves the concrete susceptible to abnormal cracking which in turn can lessen the long-term durability of a concrete structure.

Controls relating to cold weather curing were not correctly implemented.

In normal conditions, cement absorbs 0.21 - 0.28 of its weight in water during complete hydration.[36] At an approved concrete mix *w/c* target of 0.29 and a Rockville Fuel and Feed concrete batch ticket documented *w/c* of 0.26, concrete would have been expected to achieve hydration. Yet, on average, the KCE Structural Engineers, P.C. (KCE) tests observed unhydrated cementitious material from 7% to 13%.[37] A possible reason for that level of unhydrated cementitious material would be inadequate or improperly observed curing procedures that would have allowed the concrete surface to dry before hydration had completed.[38] In KCE-tested locations, the presence of unhydrated cementitious material evidences that in-situ concrete was not properly cured, further slowing, or possibly arresting development of compressive strength of the concrete in the structure.

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33 *American Concrete Institute (ACI)*, Farmington Hills, MI 48333-9094, Guide to Curing Concrete (ACI 308R-01), Chapter 1.3.1

34 Hydration refers to the chemical and physical changes that take place when Portland cement reacts with water or participates in a pozzolanic reaction. *American Concrete Institute,* Guide to Curing Concrete (ACI 308R-01), *Chapter 1.2 – Definition of Curing.* See Exhibit III: Standards.

35 *American Concrete Institute (ACI)*, Farmington Hills, MI 48333-9094, Guide to Curing Concrete (ACI 308R-01), *Chapter 1.2 – Definition of Curing.* See Exhibit III: Standards.

36 *American Concrete Institute (ACI)*, Farmington Hills, MI 48333-9094, Guide to Curing Concrete (ACI 308R-01), Chapter 1.3.1 (cross-citing Powers and Brownyard 1947; Copeland, Kantro, and Verbeck 1960; Mills 1966). The *KCE report contradicts this value, stating that 0.28 as the theoretical minimum water/cement ratio that would be required for 100% cementitious material hydration.*

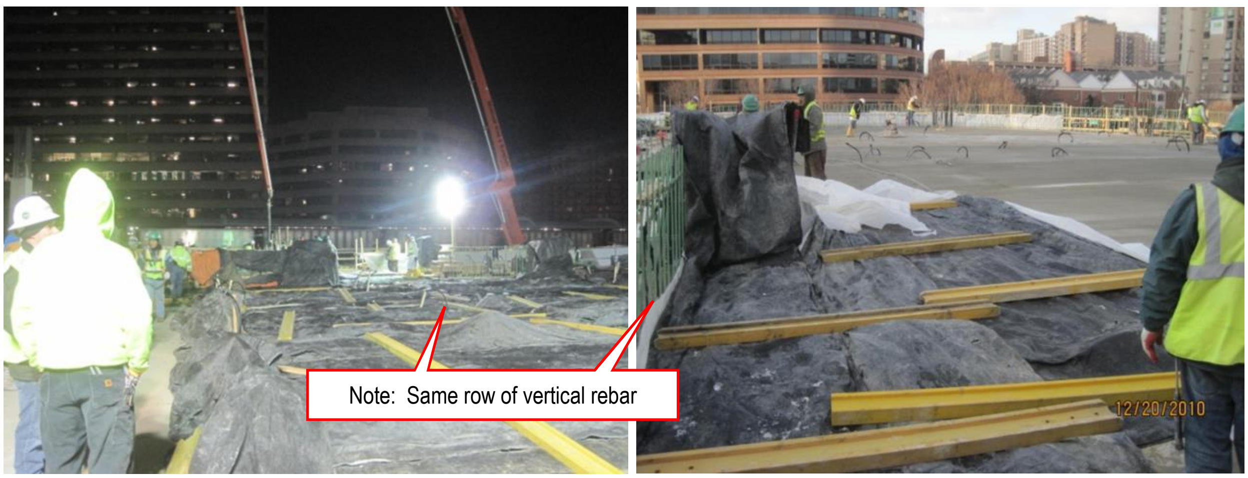
37 See Chart 2, page 15

38 A potential source of unhydrated cementitious material is a type of drying called self-desiccation. Self-desiccation can arise with mixtures having w/c ratios around 0.40 or less, when the water initially incorporated into the concrete is insufficient to completely hydrate all the cementitious materials. Self-desiccation can be prevented by using saturated, porous aggregate to provide internal curing.

Thermal protection was not maintained.

The unhydrated cementitious material was attributed by KCE to delayed placement and/or early removal of thermal protection during cold weather.[39] We reviewed photographic evidence from pour 1D on December 20, 2010, when temperatures required cold weather curing measures, that supports this conclusion. In the photographs in Image 2, below, 11 hours elapse[40] from the beginning to the end of the pour when requisite protective covers were placed. NOAA records reflect a mean temperature of 31° F for the day of this pour, and temperatures ranging between 26° and 41° F over the ensuing 7 days. The mid-afternoon photograph from the end of the pour

Image 2: Photos taken near beginning & end of Pour 1D on December 20, 2010



4:35 am on 20 Dec 2010 **(img_3357.jpg)** **3:22 pm on 20 Dec 2010** **(img _3395.jpg)**

The first photograph shows commencement of Pour 1D at 4:35 am (orange pump arm to rear right, discharge pipe in rear center). Almost 11 hours later, the second photograph shows the construction crew beginning to cover the area of work in the first photograph, with the majority of the slab remaining uncovered (at picture rear). Balter inspection logs for the day record a temperature range of 25° – 32° F. NOAA weather archives report the day's low temperature at 22°, the high at 35°, and a mean of 28.5°. Both temperature ranges would evoke cold weather curing per contract specifications.

Photographs source and courtesy of the Montgomery County Maryland Department of General Services.

depicts workers who were beginning to place the moisture-retaining plastic sheeting and blankets on the area where the pour had initiated in the early morning image. Industry standards indicate that covering "should follow closely the finishing of concrete."[41]

Construction specifications required Foulger-Pratt Contracting (FP) to "protect freshly placed concrete from premature drying and excessive cold or hot temperatures"[42] with "Moisture-Retaining-Cover Curing: [a process that covers] concrete surfaces with moisture-retaining cover for curing concrete, placed in widest practicable width, with sides and ends lapped at least 12

[39] KCE Structural Engineers, PC, Walter P Moore, and Wiss, Janney, Elstner Associates, Inc., Silver *Spring Transit Center Structural Evaluation of Superstructure*, March 15, 2013.

[40] The date, time, and other metadata were digitally recorded on the photographs provided by the Department of General Services.

[41] *American Concrete Institute (ACI)*, Farmington Hills, Michigan, Cold Weather Concreting (ACI 306R), Section 7.6 *Covering after placement.*

[42] Construction Contract between Montgomery County and Foulger-Pratt Contracting dated September 3, 2008, Attachment A – Schedule of Documents, List of Specifications, § 03300-Cast-In-Place Concrete, Part 3.13(A)

inches, and sealed by waterproof tape or adhesive" for "not less than seven days."[43] WMATA Specifications also required that curing protection should last 7 days.[44]

Standard ACI 306.1 Section 3.4.4[45] required a three-day minimal period of thermal protection during cold weather. Construction Meeting minutes document that there was confusion among participants about how long curing protection should last: "Facchina believed the cold weather protection requirement to be 3 days. Subsequent research of ACI leads the group to believe that 3 days cold weather cure time is proper."[46] As a result of this interpretation, controls for cold weather concrete as designed and implemented were less restrictive than contract documents and WMATA Specifications. Balter inspection reports only contain information about cold weather curing, when applicable, for the first three days following the pour. One report indicated that on the third day "Heat turned off under deck, stopped monitoring temps,"[47] suggesting that some of the cold weather curing activities ceased after 3 days whether or not blankets were removed.

Surface temperatures were not maintained or monitored as required by specifications.

The Statement of Special Inspection required the monitoring of fresh concrete temperature with one test hourly when air temperature is 40° F and below or when 80° F and above, and one test for each composite sample.[48] ACI standards call for concrete and the outdoor air temperatures to be recorded at regular time intervals but not less than twice per 24-hr period.[49] Balter inspection records reflect that inspectors used a high/low thermometer read twice a day.

The Standard Specification for Cold Weather Concreting ACI 306.1 establishes 55° F as the minimum, and 75° F as the maximum surface temperature for concrete immediately following a pour.[50] ACI 306.1 also sets 55° F as the minimum surface temperature for concrete during the period of curing protection, and sets 50° F as the maximum decrease in surface temperature over a 24-hour period. Contract specifications[51] referenced this standard which also required curing protection to be maintained until the concrete surface temperature was within 20° F of the ambient or surrounding temperature.[52] WMATA Specifications provide for a minimum surface temperature of 55° F, with no upper limit. The Contractor's Quality Control program required procedures for correcting any temperatures that were outside of these limits.

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43 Construction Contract between Montgomery County and Foulger-Pratt Contracting dated September 3, 2008, Attachment A – Schedule of Documents, List of Specifications, § 03300-Cast-In-Place Concrete, Part 3.13(E)(2)
44 WMATA specification 03300 section 3.06 B.1.c.
45 *American Concrete Institute (ACI)*, Farmington Hills, MI 48333-9094, Standard Specification for Cold Weather Concreting (ACI 306.1), Section 3.4.4 *Protection against freezing*.
46 Item 4.1 of FP preparatory meeting 03300 Cold Weather Concrete minutes dated 11/4/2010.
47 Balter's 12/14/10 Daily Report, Concrete Slab Temperature Report monitoring the 12/10 Pour 1Eb.
48 Statement of Special Inspections, Concrete, 5 – Sampling Fresh Concrete
49 *American Concrete Institute (ACI)*, Farmington Hills, Michigan, Cold Weather Concreting (ACI 306R), Section 2.4.2 *Temperature Records.*
50 *American Concrete Institute (ACI)*, Farmington Hills, Michigan, Standard Specification for Cold Weather Concreting (ACI 306.1), Section 3.2.1 *Placement temperature.*
51 Construction Contract between Montgomery County and Foulger-Pratt Contracting dated September 3, 2008, Attachment A – Schedule of Documents, List of Specifications, § 03300-Cast-In-Place Concrete, Part 3.9(E)
52 ACI 306.1, §3.2.3.

Balter inspection reports document that concrete surface high and low temperatures were monitored at several locations, for pours meeting the ACI cold weather definition. Balter inspection reports were found to provide daily temperature monitoring reports for the three days following a pour.[53] Temperature monitoring reports were not prepared for some weekend days and holidays that fell within the three days post-pour. For one Friday pour (2C), only the third day report from the following Monday was available.

Chart 5a: How to Read the Comparison of Cold Weather Curing Temperatures (Degrees Fahrenheit)

Pour Location Pour Date	Average Ambient Temp	Mix Temp Day of Pour	Test Period			Concrete Inspection Temperatures								
						On Slab Surface		@ Inspection Station		@ Deck Cure Box		In Cure Shed		Under Deck
			Day	Time	Temp	Min	Max	Min	Max	Min	Max	Min	Max	
Pour 1 D 20 Dec 2010	31	59	Day 1 12/21/10	AM	28	75	96			45	60	68	108	**83**
				PM	36	82	108			60	64	76	80	84
			Day 2 12/22/10	AM	30	102	108			54	58	62	80	90
				PM	36	98	102			52	60	68	80	90
			Day 3 12/23/10	AM	25	84	94			48	50	62	82	80
				PM	30	76	86			43	44	68	75	88

Concrete for levels 330 and 350 of the SSTC was placed in 19 separate pours. These were designated by a number and letter: 1 for level 330 or 2 for level 350, with the letter further designating the location on the level. Mapping of the pours can be found on pages 74 and 75 of KCE Exhibit I. "Pour 1D", above, was the fourth section of level 330. Concrete for that pour was placed on December 20, 2010. The air temperature that day was 31 degrees, but the temperature of the concrete being poured was 59. Balter monitored temperatures at various locations for three days after the pour, typically taking a reading early in the morning, and another late in the afternoon, noting the air temperature at the time they took the readings. Inspectors used a thermometer that recorded the lowest and highest temperature since the last time the thermometer was reset - typically the last reading. One reading was taken from a thermometer laid on the concrete slab surface under the curing blankets. On occasion, a reading was taken for the field cured specimen cylinders stored at the inspection station. Another reading was taken for the comparison set of specimen cylinders that was cast on and cured in a curing box on the deck, and another set taken inside the curing shed where specimen cylinders to be laboratory cured were stored until transit to the lab. The last reading was taken under the deck where workers screened off the area and used space heaters to warm the area.

Chart 5b presents cold weather curing temperatures recorded by Balter inspectors. Chart 5a, which does not cite any specific deficiencies, serves as a "How to Read" orientation to Chart 5b. As indicated in Chart 5b, for three of the six cold weather pour dates examined in detail by the OIG, Balter inspectors recorded surface temperatures below the 55° F minimum specified by the ACI standard.[54] On three occasions, concrete temperatures below the ACI minimum were recorded on the last day the inspectors documented cold weather curing. Balter did not raise the occurrence of a temperature below the ACI 306.1 standard, nor did Balter Daily Reports or FP Daily Contractor Quality Control Reports document the quality control failure to observe the referenced standard. The reports also failed to note if any effort was made to alert the Contractor to the need to implement temperature correcting procedures.

[53] The OIG evaluated records for five of the seven ACI-defined cold weather concrete pours: , Pour 1D on December 20, 2010, Pour 1Eb on December 10, 2010, Pour 1G on February 8, 2011, Pour 2B on December 7, and Pour 2D on January 31, 2011.

[54] *American Concrete Institute (ACI)*, Farmington Hills, Michigan, Standard Specification for Cold Weather Concreting (ACI 306.1), Section 3.2.2 - *Protection temperature*

In five of these six cold weather curing periods, the last recorded low surface temperature was more than 20° above the ambient temperature recorded by either the Balter inspector or NOAA for that date. Balter Daily Reports did not document whether there was a gradual decrease in surface temperature since Balter did not monitor concrete temperatures [55] after area heat was discontinued, which typically occurred after 3 days. Inspection reports also failed to document when protective plastic and insulating blankets were removed by the contractor.

Chart 5b: Comparison of Cold Weather Curing Temperatures (Degrees Fahrenheit)

Pour Location / Pour Date	NOAA Average Ambient Temp	Mix Temp Day of Pour	Test Period: Day	Test Period: Time	Test Period: Temp	On Slab Surface: Min	On Slab Surface: Max	@ Inspection Station: Min	@ Inspection Station: Max	@ Deck Cure Box: Min	@ Deck Cure Box: Max	In Cure Shed: Min	In Cure Shed: Max	Under Deck
Pour 1 D 20 Dec 2010	31	59	Day 1 12/21/10	AM	28	75	96			45	60	68	108	**83**
				PM	36	82	108			60	64	76	80	84
			Day 2 12/22/10	AM	30	102	108			54	58	62	80	90
				PM	36	98	102			52	60	68	80	90
			Day 3 12/23/10	AM	25	84	94	> 20°		48	50	62	82	80
				PM	30	76	86			43	44	68	75	88

Observations for Pour 1D: Except for the maximum temperature recorded in the curing shed, each measure, minimum or maximum, for each location increased in temperature by day 2 and then started to cool. Inspectors did not record temperatures for the specimen cylinders that were field-curing at the inspection station. On the last day (Day 3) that inspectors recorded temperatures, the minimum temperature on concrete on the slab surface - 76 degrees - was not within 20 degrees of the ambient air temperature, 30 degrees, thus cold weather curing, including monitoring, should have continued.

Pour Location / Pour Date	NOAA Average Ambient Temp	Mix Temp Day of Pour	Test Period: Day	Test Period: Time	Test Period: Temp	On Slab Surface: Min	On Slab Surface: Max	@ Inspection Station: Min	@ Inspection Station: Max	@ Deck Cure Box: Min	@ Deck Cure Box: Max	In Cure Shed: Min	In Cure Shed: Max	Under Deck
Pour 1 E(b) 10 Dec 2010	26	60	Day 3 12/13/10	AM		66	78					80	84	
				PM		56	70	> 20°				62	82	
			Day 4 12/14/10	AM	22	54	71					72	74	64
				PM	29			< 55°				70	84	

Observations for Pour 1E(b): This pour occurred on a Friday, so the first recorded data is for Monday, December 13 - 3 days following the pour. Inspectors were very inconsistent at recording data. In the morning of day four, the difference between the minimum slab temperature, 54, and the ambient air temperature is more than twenty degrees, thus cold weather curing should have continued, and at 54 degrees, the temperature was one degree below the minimum slab temperature allowed during cold weather curing.

Pour Location / Pour Date	NOAA Average Ambient Temp	Mix Temp Day of Pour	Test Period: Day	Test Period: Time	Test Period: Temp	On Slab Surface: Min	On Slab Surface: Max	@ Inspection Station: Min	@ Inspection Station: Max	@ Deck Cure Box: Min	@ Deck Cure Box: Max	In Cure Shed: Min	In Cure Shed: Max	Under Deck
Pour 1 G 8 Feb 2011	27	64	Day 1 2/9/11	AM	20	77	106					70	84	71
				PM	33	78	108					73	78	73
			Day 2 2/10/11	AM	20	72	106					70	77	83
				PM	33	70	104					69		81
			Day 3 2/11/11	AM	23	96	109	> 20°				70	80	78
				PM	33	77	92					65	75	49

Observations for Pour 1G: In the afternoon of the last day that data was recorded, the difference between the minimum slab temperature and the ambient temperature was 44 degrees, yet it appears the heater under the slab was stopped, as the under deck temperature fell to 49 after hovering in the 70's and 80's during the preceding two days. Inspectors were inconsistent at recording data.

- - continues next page - -

55 Balter's 12/14/10 Daily Report, Concrete Slab Temperature Report monitoring the 12/10 Pour 1Eb notes "Heat turned off under deck, stopped monitoring temps." The low surface temperature was 54° F (below ACI minimum), and the ambient temperature was 28° F (a difference between ambient and surface temperatures that was greater than the ACI 20° difference required to cease sold weather protection)

Chart 5b: Comparison of Cold Weather Curing Temperatures (Degrees Fahrenheit) - ***continued***

Pour Location Pour Date	NOAA Average Ambient Temp	Mix Temp Day of Pour	Test Period			Concrete Inspection Temperatures								
						On Slab Surface		@ Inspection Station		@ Deck Cure Box		In Cure Shed		Under Deck
			Day	Time	Temp	Min	Max	Min	Max	Min	Max	Min	Max	
Pour 2 B 7 Dec 2010	29	62	Day 1 12/8/10	AM	25	89		44	64					
				Noon	32	114		42	64					77
				PM	33	116		42	64					76
			Day 2 12/9/10	AM	24	106		40	64			70	88	74
				PM	34	Thermometer Broken		50	80	52		74	94	77
			Day 3 12/10/10	AM	19	76	94	46	78	42	56	82	88	76
				PM	35	72	90			43	55	72	75	75
			Day 6 12/13/10	AM		62	96			42	54	80	84	70
				PM		60	76	> 20°		38	52	82	62	46
			Day 7 12/14/10	AM	22	61	74	58	78	24	40	72	74	38
				PM	29			67				70	84	

Observations for Pour 2B: This was the longest recorded data set of any cold weather pour. The heater under the slab appears to have been turned off during the morning of Day 6 - temperatures dropped from the 70's to 46 and then 38. Yet, there was a 39 degree difference between ambient and slab temperatures, so cold weather curing should have continued. Although inspectors recorded full data on the mornings of Day 3 and Day 7, inspectors were still inconsistent at recording data.

Pour 2 C 14 Jan 2011	34	58	Day 3 1/17/11	AM	25	44	104							
				PM				< 55°						

Observations for Pour 2C: January 14 was a Friday pour - the only data recorded was on the following Monday. Between the time of the pour and the recording of data, a minimum temperature of 44 degrees was recorded - 11 degrees colder than allowed by standards and specifications.

Pour 2 D 31 Jan 2011	28	57	Day 1 2/1/11	AM	30	46	104	< 55°				70	77	91
				PM	32	90	90							62
			Day 2 2/2/11	AM	35	78	87					70	72	92
				PM	40	80	84					65	75	88
			Day 3 2/3/11	AM	30	72	79	> 20°				68	74	88
				PM	30	82	89					69	79	57

Observations for Pour 2D: On the last entry of recorded data, there was a 52 degree difference between ambient and slab temperatures, yet some cold weather curing was stopped as evidenced by the drop in "under deck" temperature from 88 to 57 consistent with turning off the heater under the slab .

Sources: The Robert B. Balter Company Concrete Slab Temperature Reports and 56-Day Reports of Concrete Cylinder Tests; KCE Report Table 12 C Alternative ACI 214R-10 PSI. Two cold weather pours - Pour 2 C and the Level 330 East Pour Strips - are not included in the evaluation due to a lack of statistically significant data.

Controls were in place during concrete curing to record temperatures at least during the first three days following a concrete pour. There was no evidence that the Balter Inspector alerted the Contractor and the Quality Control System Manager when measured temperatures exceeded project limits. Inspection records documented the difference between concrete surface and ambient temperatures great enough to have required a continuation of cold weather protection, yet daily reports evidence that supplemental cold weather curing heat was stopped after three days.

KCE reported that "[p]etrographic examinations of the concrete cores from the slabs indicate that unacceptable percentages of the Portland cement and slag were unhydrated. This observation is consistent with concrete experiencing a temperature [recorded in Balter Daily Reports that were]

Image 3a: Inspection Station Orientation
9:33 am on 20 Dec 2010 – Pour 1D (img_3387.jpg)

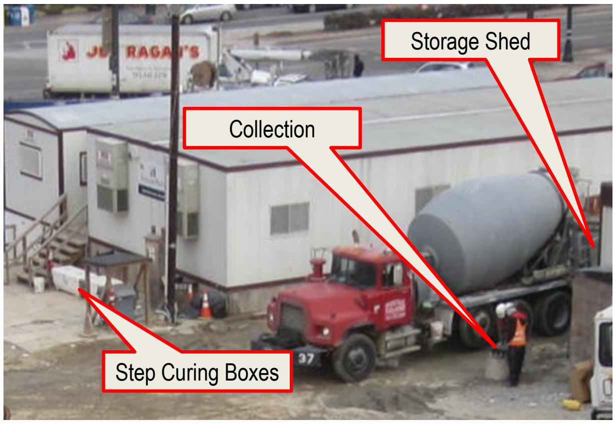


Image 3b: Field Cured Inspection Station Collection & Curing
4:29 am on 18 Feb 2011 – Pour 1H (img_4292.jpg)

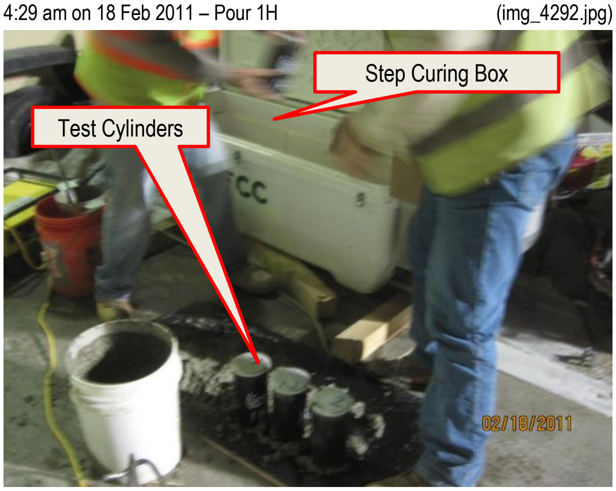
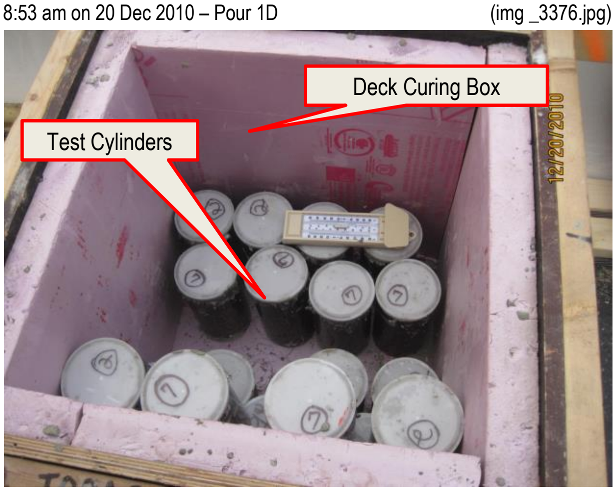


Image 3c: Field Cured Deck Collection & Curing
8:53 am on 20 Dec 2010 – Pour 1D (img _3376.jpg)



Photographs source and courtesy of the Montgomery County Maryland Department of General Services.

low enough to slow hydration to the point that the available water dried out before the cement could hydrate."[56]

By removing protection early, hydration of the concrete would have been slowed or stopped, which would explain the presence of unhydrated cement and slag. Controls for this project should have clearly conveyed temperature limits during cold weather curing, and the duration of these limits should have been coordinated with those set by WMATA.

Concrete testing specimens were cured in an environment not representative of in-situ concrete.

In Chart 5 (pages 26-27), inspection documents evaluated by the OIG confirm that the temperatures of the Balter compressive strength test cylinders stored in the curing box near the inspection station (field cured - see images 3a and 3b) and the cure box on the deck were not representative of the temperature of the concrete in the poured slabs. Six of the 18 cylinders were cast and cured on the slab deck (see image 3c). For three sample sets collect during each pour, six of the primary test cylinders were cast and cured at the inspection station, and six[57] cylinders were transported to a laboratory for the balance of the curing period (lab cured).

ASTM C31/C31M requires protection of the field-cured cylinders from the elements in as near as possible the same way as the formed work, and that cylinders should be provided with the same temperature and

[56] KCE Report, page 76.
[57] In later stage of the project, 14 primary cylinders were cast, with 7 remaining on site, and 7 transported to the lab.

moisture environment as the structural work. This standard was not observed for the field cured concrete compressive strength cylinders.

Cylinders were to be made and stored in or on the structure as near as possible to the point of deposit of the concrete represented by the sample (discharge end of the pump hose and stored under the poly and insulated blanket protective cover). The cylinders were to be cured either under ideal conditions in a laboratory or in the field experiencing the same condition as the concrete in the structure.

Documents prepared by Balter inspectors recorded that temperatures in the on field curing box on the deck and in the field curing boxes at the storage station were different from the temperatures on the deck slab under the curing blankets. Our analysis of these records indicates that while the temperatures differed, the effect cannot be determined from the limited data recorded.

Conclusions

Records collected by Balter and FP indicate that the details of curing concrete were not addressed in accordance with specification. Analysis of the records collected should have identified inconsistences between specification requirements and procedures implemented that could have been investigated and remedied.

Thermal protection was not placed early following the placement of the concrete in accordance with established specifications. Thermal protection was not continued in accordance with cold weather curing specifications. Surface temperature monitoring was not observed in accordance with specifications. As a result, the condition of the in-situ concrete may have been impacted by the failure to observe cold weather curing procedures, and potential contributing to plastic shrinkage cracking observed in the structure.

Recommendation 3

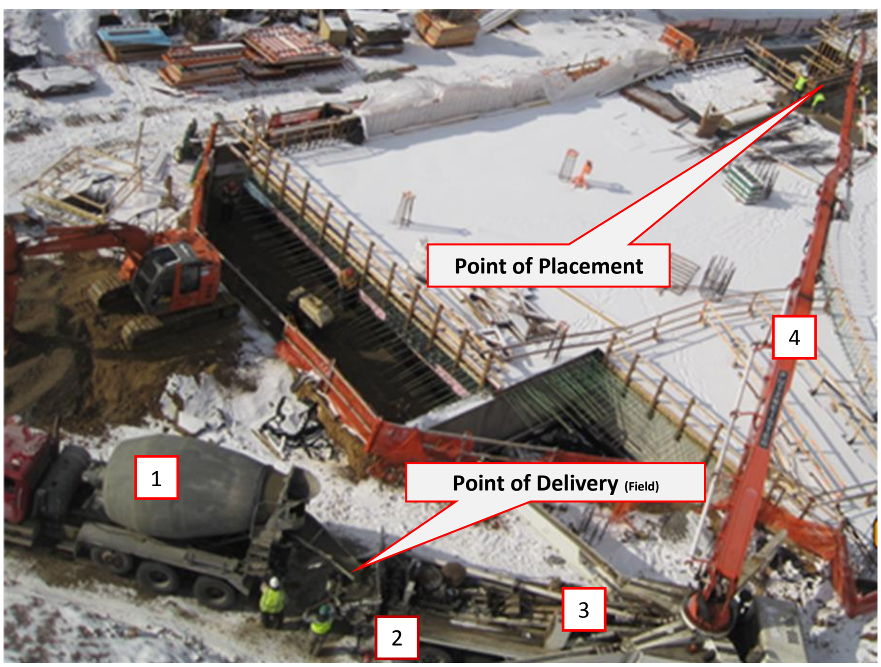
In future projects, DGS should ensure that all specification requirements are reviewed and implemented unless a variance is mutually discussed and agreed upon. Temperature limits during curing should be monitored and maintained, and specifications for duration of curing should be strictly observed. Confusion about where to take samples and about cold weather limits should be avoided by clearer language in specifications. Any conflicts between specifications and standards should be resolved in favor of the more conservative of those required by stakeholders (in the case of the SSTC, DGS, and WMATA).

Structural Strength

Finding 4: Construction documents referenced specifications and standards that differed as to where concrete testing samples should be taken. Reliance upon samples taken at the inspection station produced compressive strength test results that were not representative of the strength of the in-situ concrete.

Ambiguity existed over where to collect the concrete samples to be used to test for compressive strength. Construction Documents referenced specifications and applicable standards that differed as to where the specimen cylinders should be taken. The Statement of Special Inspections[58] that establishes the inspection criteria for the SSTC, and the Balter contract references ASTM International's (ASTM) standard C31/C31M, which indicates that cylinders should be *made and stored* in or on the structure *as near as possible to the point of deposit* (placement) of the concrete represented by the sample[59], which because of pumping operations during this project, was at the discharge end of *the pump hose*. This standard was not strictly observed.

Image 4: Level 330 East Pour Strip Concrete Placement – January 12, 2011



Legend: 1 – Concrete Truck 3 – Concrete Pump Truck
2 – Pump Hopper 4 – Pump Discharge Pipe, Hose, and Crane assembly

Source: Montgomery County Maryland Department of General Services

[58] Statement of Special Inspections, Concrete, 5 – Sampling fresh concrete and performing slump, air content and determining the temperature of fresh concrete at the time of making specimens for strength tests: Compression Test Specimens

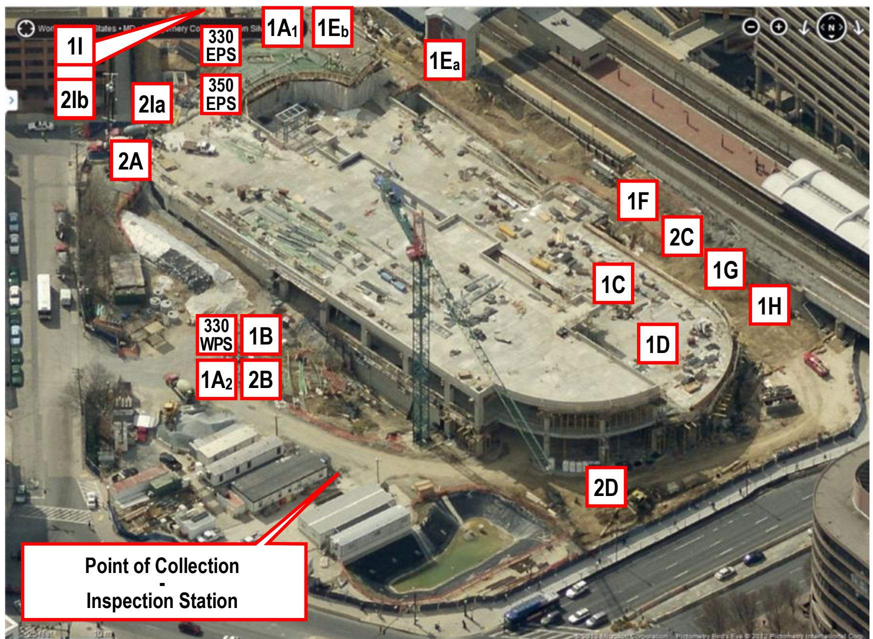
[59] *ASTM International* (formerly American Society for Testing and Materials), West Conshohocken, Pennsylvania, Standard Practice for Making and Curing Concrete Test Specimens in the Field (ASTM C31/C31M), Section 9.1 Place of Molding and 10.2.1 Field Curing - Cylinders

The construction contract's specifications section 03300.1.5.B references ASTM C 94[60], requiring concrete compressive strength testing to be in conformance with this international standard. The international standard states samples should be made and stored *as near as possible to the Point of Delivery* (see Image 4).

During this project, the concrete was primarily collected for testing at end of the *concrete truck chute* at the inspection station.

One truck out of every five was directed to the site's inspection station where concrete was drawn from the truck's load for use in on-site testing, and for casting the cylinders to be used for compressive strength testing. After the testing concrete was drawn, the truck was directed to a pumping location (Point of Delivery (Field)) located at numerous work areas throughout the site (see Image 5). The next 4 trucks delivering the remaining 40 cubic yards (yd^3) were sent directly to the pumping locations.[61]

Image 5: Transit Center Site Plan with Locations of Inspection Station & Concrete Points of Delivery



Key: 1A Location of Pump Trucks and Concrete Points of Delivery (Field) – in this example, the location for Pour 1A See Footnote 63. Source: Pictometry International Corp. © 2012

Entrained air content[63] and other properties can change during pumping. Additionally, low viscosity and high cohesion are needed for concrete to move easily through the pump - adding water can improve these properties when needed.

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[60] *ASTM International* (formerly American Society for Testing and Materials), West Conshohocken, Pennsylvania, Standard Specification for Ready-Mixed Concrete (ASTM C94/C94M), Section 17.2 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimen

[61] For concrete pours on levels 330 and 350, concrete pump trucks were used for most of the pours. Although this report focuses on pumping operations, some concrete was discharged directly from the truck chute, and other concrete was discharged from a bucket that was hoisted to the Point of Placement via tower crane.

[62] Location of pumping trucks approximated from site photographs taken on the days of pour: 1A1 – 2010-09-13 Pour 1A Pump 1 Location.jpg; 1A2 – 2010-09-13 Pour 1A Pump 2 Location.jpg; 1B – xx; 1C – 2010-18 Pour 1C Pump Location.jpg; 1D – IMG_3386.jpg; 1Ea – 2010-11-12 Pour 1Ea Pump Location.jpg; 1Eb – 2010-12-10 Pour 1Eb Pump Location.jpg; 1F – 2010-12-30 Pour 1F Pump Location (2 of 2).jpg; 1G – IMG_1539.jpg; 1H – IMG_4294.jpg; 1I – 2011-05-03 Pour 1I SOG Pump Location.jpg; East Pour Strip Level 330 – 2011-01-12 Pour Strip Level 330 East Pump Location (2 of 2).jpg; West Pour Strip Level 330 – IMG_1658.jpg (placed by bucket); 2A – 2010-11-02 Pour 2A Pump Location (2 of 3).jpg; 2B – 2010-12-07 Pour 2B Pump Location.jpg; 2C – 2011-01-14 Pour 2C Pump Location.jpg; 2D – 2011-01-31 Pour 2D Pump Location.jpg; 2Ia – 2011-03-29 Pour 2Ia pump Location (2 of 4).jpg; 2Ib – 2011-03-29 Pour 2Ia pump Location (2 of 4).jpg; East Pour Strip Level 350 – 2011-06-01 Pour Strip 350 Level Discharge Location (2 of 2).jpg; Inspection Station – 2010-10-02 Pour 1B Pump Location & Truck at Insp Stn by Trailers.jpg. Locations that appear to be on a transit center deck were pours completed before the pour of the indicated deck. Photographs source, and courtesy of Montgomery County Maryland Department of General Services.

[63] Entrained air is microscopic cells of air distributed throughout the concrete paste that are beneficial because they improve concrete's resistance to damage caused by freezing.

The ambiguity over where samples should be collected was discussed during the July 2010 pre-installation meeting. Minutes indicate that whether collection of samples should occur at the end of the truck chute at the inspection station or at the end of the pump was "left open for later resolution."[64] In a meeting one month later, minutes record that RFF's representative indicated that concrete samples should be collected from the truck and not at the end of the pump hose. WMATA's representative disagreed.[65] Eventually, Balter was directed to cast a limited number of comparison cylinders at the end of the concrete pump hose while conducting the primary testing at the truck chute, although meeting minutes do not specify who directed the change. Balter Daily Inspection reports, however, note "(6) extra cyl[inder]s made [at] end of concrete pump on deck as per Montg[omery] Co[unty] Tim H[erbold]."[66] This DGS directed casting of comparison sets provided the opportunity to identify differences between inspection station and in-situ concrete. However there is no indication those comparisons were made during the period of major construction activity.

In its report, KCE Structural Engineers, P.C. (KCE) observed that in-situ sample cylinders of concrete it extracted from "pours 1A, 1B, 1E, 1H, and 2C [had] unacceptable concrete strength based on the ACI 318-02 [compressive strength] requirements."[67] Based on records maintained by DGS, the average size for each the KCE-identified pours with unacceptable concrete strength[68] was 729 yd^3 of concrete which would have been delivered to the site in 73 concrete trucks[69], fifteen of these trucks would have been tested at the inspection station, with the remaining 58 trucks being sent directly to the remote pumping station.[70] While Balter asserts that an inspector observed each of the other 58 truckloads, no records were found that document the Point of Delivery (Field) inspections other than a general notation on the Balter inspector's daily report.

Taking most of the samples at the inspection station as opposed to at the end of the pump hose increased the risk that the concrete samples would not be representative of the in-situ concrete, and thus that tests conducted on such samples might present compressive strength results that were not representative of the in-situ concrete. Appendix C demonstrates that samples from the end of the pump were, in fact, significantly weaker than those taken at the inspection station.

Recommendation 4

DGS should modify its contract specifications for future construction projects to ensure that concrete test specimens are made as near as possible to the actual point where concrete is placed. Where referenced standards require testing at the point of delivery, clarify in the specification that such testing is in addition to typical testing.

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64 Minutes of the 7/13/10 SSTC Preparatory Meeting and Preinstallation Conference conducted by Foulger Pratt.
65 0300 Concrete Placement Methods, Logistics and Testing Meeting Agenda and notes, dated August 25, 2010.
66 Balter Compressive Strength Test Specimen Date ticket number 2 dated September 13, 2010 for concrete batch ticket 85320, et.al.
67 KCE Structural Engineers, PC, Walter P Moore, and Wiss, Janney, Elstner Associates, Inc., *Silver Spring Transit Center Structural Evaluation of Superstructure*, March 15, 2013.
68 KCE Structural Engineers, PC, Walter P Moore, and Wiss, Janney, Elstner Associates, Inc., *Silver Spring Transit Center Structural Evaluation of Superstructure*, March 15, 2013.
69 Each truck was loaded with 10 yd^3 of concrete.
70 OIG Work Paper - Establishing Average Size in yd^3 of Unacceptable Concrete Pours and Calculations Based Thereupon

Concrete placement resulted in insufficient concrete cover over reinforcing steel and post-tensioned tendons, which allowed the concrete covering tendon ducts in several locations to crack away when grout was placed to the ducts. In some areas concrete drive paths as poured do not provide the minimum concrete cover (thickness) required by the design specifications. In some areas, the concrete cover was thicker than design specification requirements.

Structural Strength

Finding 5: Design, construction, and inspection contractors had early knowledge that proper concrete thickness was not being achieved, but they took no effective steps to fix the problem.

By November 2010, visible evidence of structural and durability issues had raised concerns including:

- Cracks discovered in the concrete slabs, beams and girders;
- Concrete that broke away from the finished drive surface (spalling), revealing post-tensioned tendons and evidencing that an insufficient concrete cover had been placed over the tendons;

Although concerns about concrete thickness, inadequate concrete cover, spalled concrete above post-tensioned tendons, and related concerns regarding structural deficiency and durability were raised by WMATA soon after the commencement of Level 330 pours and in subsequent monthly meeting, potential repairs and remediation plans were not resolved.

In its WMATA-commissioned report, Whitlock Dalrymple Poston & Associates (WDP) opined that the "long-term durability of a structure is a function of initial construction quality, the extent of routine maintenance performed on [the] structure, and the extent of durability enhancement measures that should be installed on the structure to achieve its design service life."[71]

Durability is the ability of concrete to remain unchanged while in service, including its resistance to weathering action, chemical attack, and abrasion.[72] KCE Structural Engineers, P.C. (KCE) determined that "the durability of the in-situ concrete decks of [the] SSTC [will] not meet the 50-year useful life criteria as per WMATA requirements", and that the excessive cracking "would leave the structure vulnerable to water and chloride-ion intrusion, which reduces the time to initiation of corrosion" to occur well before design specifications. WMATA's consultant, WDP,

[71] Whitlock Dalrymple Poston & Associates, *Evaluation of Silver Spring Transit Center*, May 2, 2013. Page 69.
[72] Source: http://www.allmetalssupply.com/concrete_terms.htm @ 17:50 on 1 August 2013

concurred that the SSTC evaluations identified "initial construction quality issues that may compromise the long-term durability of the structure."

The depth of concrete cover over reinforcing steel and tensioning tendons affects concrete durability. Lesser amounts of concrete cover result in smaller distances through which water and chlorides must penetrate to reach the depth of the reinforcing steel to initiate corrosion. During the construction of the SSTC, concrete broke away from the finished drive surface, revealing post-tensioning tendons, and evidencing that an insufficient concrete cover had been placed over the tendons. Ground Penetrating Radar testing conducted for KCE indicated that "numerous tendons and reinforcing bars did not have the minimum specified concrete cover."[73]

In order for concrete slabs to have met ACI standards and Construction Document specifications[74], slab thickness should have ranged between 9 ¾" and 10 ⅜". Testing indicated that in-situ concrete slab thickness ranged between 7" and 12 ¼", with only 44 % of the level 330 and 38% of the level 350 concrete slabs in compliance with ACI and Construction Document requirements.[75]

Deficiencies with the concrete cover of completed work were identified as early as October 2010,[76] during a construction progress meeting, with an evaluation of the issue discussed during the next meeting.[77] In his October 30, 2010 site inspection report, the Structural Engineer of Record (SEOR) "observed three locations in the Pour 1A area where small portions of concrete directly over the high points of slab tendon ducts popped off during tendon grouting. It is clear that the cover over the duct in these locations was as little as 1/4 [inch]"

Thickness issues continued in concrete that was placed following this discovery,[78] with a WMATA-commissioned report indicating the "preliminary reports show that the deck thickness may be as much as 2 inches thinner than designed in certain areas," with "spalled concrete [present] above the tendons [at] 9 locations around the deck."[79] The result of WMATA's survey was confirmed by both DGS and Facchina in later meetings.[80]

Checklists used before each pour demonstrate that Balter checked to assure reinforcing steel and post tension tendons were properly situated within the formwork to allow for correct elevations with sufficient cover.[81] Efforts to control alignment did prevent some cover deficiencies.

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73 KCE Report, page 92.

74 In Table 6 on page 41 of its report, KCE illustrates that ACI 318-02 required top and bottom covers of 2" as a Minimum Concrete Cover For # 6 bars or greater Mild Steel Reinforcement while the Construction documents require a minimum 2" top and 1"bottom cover, while Table 7 indicates an ACI 318-02 Minimum Concrete Cover over Post-Tensioning Conduit as 1", top and bottom, for slabs, while the SSTC design call for a 2" top and. Bottom 2" – 2-1/2" bottom cover.

75 KCE Report, page 42.

76 SSTC Construction Progress Meeting, October 28, 2010 "popped concrete cover in three locations at slab tendons when grouting. Possibly did not have the proper coverage over the tendon."

77 PB Construction Progress Meeting #51, November 16, 2010, minutes. "Area around popped tendons was surveyed for slab thickness. Slab came in thin in some areas."

78 Greenhorne Thickness Survey. .

79 In minutes from the SSTC Project Management Team Meeting # 12 held on 8/11/11, "WMATA indicated they received the results of the survey effort to check slab thickness."

80 In minutes from the SSTC Project Management Team Meeting # 14 held on 10/18/11, it was reported that "WMATA's survey was confirmed by both MC's surveyor and Facchina's surveyor. The main issues discussed were: 1) is there a structural deficiency; 2) what is the effect on durability if the steel is less than 2 inches from the surface."

81 Balter Daily Report by Tony Lord, 12/03/11.

However, insufficient cover of reinforcing steel and tendons was more likely attributable to insufficient concrete thickness.

Records document that the Contractor established floor thickness by establishing top surface with the desired shape using survey equipment operated while concrete was being placed.[82] The inspector did not (according to the response from Balter to the KCE report, the inspector could not) independently check thickness except at the perimeter.[83] In Exhibit I, the OIG's subject matter expert noted that wet depth checks using a simple rod inserted vertically into fresh concrete would have been a practical thickness check.

The Contractor and Inspector assert that thickness of concrete floors was not directly measured during concrete pours. Despite reminders from the SEOR to "all parties" during construction to maintain thickness,[84] no independent method to check thickness was developed.

The required discussions regarding reinforcement and tendon placement occurred during the pre-installation conference and several subsequent discussions occurred during progress meetings after the discovery that adequate cover was not being maintained. Nonetheless, the deficiencies persisted. If Foulger-Pratt Contracting (FP) was unable to provide the required cover due to congestion of many elements within the slab, a Request for Information (RFI) should have been generated. The lack of cover should have been flagged as a construction deficiency by Balter and corrected prior to continuation of subsequent pours.

Recommendation 5

In future projects, DGS should ensure its construction contractors utilize a construction method that allows direct measurement of floor thickness so that inspectors can help the Contractor by identifying problems as the concrete is placed.

DGS should hold construction contractors accountable for any remediation and increased maintenance costs that will likely result from the contractor's failure to ensure specified concrete slab thickness was attained during placement.

[82] Entry 1.13 of FP minutes from meeting held 8/25/2010 regarding 03300 Concrete Placement Methods, Logistics, and Testing: "How will grades and elevations be established on finished concrete surface? Facchina's surveyor/ layout man will shoot all elevations of top of concrete as placed during the pour for use by W concrete to rake out and screed to established top of concrete elevations."

[83] "Thickness of the slab at points away from the perimeter could not be measured without survey equipment." Balter Letter regarding Response to KCE Report Dated March 15, 2013, April 22, 2013, page 5.

[84] PB Field Observation Comments, 10/15/10, 10/30/10, 11/11/10. "Elevations of formwork, system for maintaining required design elevations at the top of the concrete, and system for maintaining typical concrete thickness at 10 inches should be verified by all parties."

The three pour strips on the 330 and 350 levels were each constructed in a different manner and neither of the pour strips on the 330 level was constructed in manner that conformed to the design requirements identified in the structural drawings. The Level 350 pour strip was constructed in conformance with design requirements. The east pour strip on the 330 level was poured without post-tensioning tendons but with mild steel reinforcement, while the west pour strip on the 330 level was poured without post-tensioning tendons and without sufficient steel reinforcement in one direction.

Structural Strength

Finding 6: Despite reasonably designed controls, identified pour strip deficiencies resulted from weaknesses in the submittal process and professional error.

Image 6: West Level 330 Pour Strip Concrete Pour – April 19, 2011

7:54 am on 19 Apr 2011 **(img_1658.jpg)**

Source and courtesy of the Montgomery County Maryland Department of General Services.

Pour strips are areas of a slab in the deck that are left out during construction and then placed after adjacent concrete has been poured and has been allowed an opportunity to shrink. Specifications required two pour strips on the 330 level, one each at the east and the west end. One pour strip was required on the smaller 350 level at the east end. Each pour strip was purposely installed at least 60 days later than the rest of the adjoining floor. The east 330 level strip was poured in January 2011while the west strip was poured in April. The level 350 strip was poured in June 2011. The SSTC Pour Strips are substantially wider than the normal industry practice of 3-4 feet. Both pour strips at the 330 level are 760 square foot

Image 7: Cracking Evident on Underside of Level 330 East Pour Strip – April 9, 2013

Source and courtesy of the Montgomery County Maryland Department of General Services.

rectangles approximately 10' wide and up to 76' in length while the 350 level pour strip is slightly larger at 800 square feet, 20' wide, and 40' Long

Drawings in the Construction Documents appear to require mild steel and post-tensioning tendons within the three pour strips on the 330 and 350 levels. A photograph taken by the County (see Image 6) captures workers pouring the concrete at the West 330 level pour strip without the presence of post-tensioning tendons and without most of the mild steel reinforcing in the North-South direction (although there is some at 51 inches on center).[85]

Ground Penetrating Radar (GPR) scans conducted by KCE Structural Engineers, P.C. (KCE) confirmed that neither the east nor the west pour strip on Level 330 was constructed with post-tensioning tendons, and that the west pour strip on Level 330 was missing the required mild steel reinforcing in the North-South direction. The pour strip constructed on the east end of Level 350, the last of the three to be placed, was constructed with both the mild steel reinforcing and post-tensioning tendons. KCE found that one of the Level 330 pour strips was constructed with mild steel reinforcing spaced at 51 inches on center, while the Contract Documents require mild steel reinforcing at 12 inches on center.

Further, the pour strips contain the severe cracks (see Image 7) and unacceptable concrete that are present in many other slabs. The KCE report states: "Results of an analytical 4.8-foot wide strip indicate that the slabs at these locations, as built, do not have sufficient shear or flexural capacity to support the design loads." The project control deficiencies associated with the concrete, as discussed in separate sections of this report, also apply to the concrete used in the pour strips.

[85] Post tensioning ducts would appear as wide, white, ribbed tubes draped under the green reinforcing steel bars that are present in the picture.

Shop Drawings

The failure to install post-tensioned tendons and some of the reinforcement steel in level 330 pour strips resulted from failures to ensure that shop drawings for the pour strips were received and conformed to the design requirements identified in the structural drawings.

According to the General Terms and Conditions of the construction contract, "Shop Drawings generally consist of those drawings, diagrams, schedules and other data specially prepared for the Work by the Contractor or a Subcontractor, Sub-subcontractor, manufacturer, Supplier or distributor detailing the fabrication or assembly of some portion of the Work, copies of which are submitted by the Contractor to the [Architect/Engineer] for approval to indicate the details of execution of that portion of the Work."

As the Construction Contractor, Foulger-Pratt Contracting (FP) was required to interpret the Construction Documents and prepare (or cause to be prepared) trade-specific shop drawings that communicate FP's understanding of the proposed construction. The designer of record, Parsons Brinckerhoff (PB), was to review and approve the shop drawings and submittals to ensure FP's intended construction was in conformance with the design intent.

Shop drawings from VStructural LLC (VSL) were submitted in phases, and each drawing included a "key plan" to indicate the scope of the shop drawing. PB's shop drawing reviewer would have reasonably expected that shop drawings for all phases of work would be submitted, and that pour strip drawings would have followed submission of other shop drawings since the pour strips would have been poured last. None of the key plans in shop drawings submitted by VSL included the two Level 330 pour strips.

The process for submission and review of shop drawings (part of the project control system) should have, but did not, detect the omission of the post-tensioned tendons shop drawings for the pour strips. The phased submission of drawings increased the vulnerability that PB would not have identified omission of a required shop drawing. The absence of these shop drawings should have been detected if the Design team had ensured that all required shop drawings were identified and contained in the submittal control system, and their preparation scheduled and tracked.

Request for Information and Meetings

In a response to the KCE report, Facchina stated that VSL shop drawings were intentionally prepared without post-tensioning tendons, and asserted that the level 330 drawings did not require such tendons.[86] VSL shop drawings were not submitted for the design and layout of the

[86] Facchina letter dated August 30, 2012, item 4, page 2.

post-tensioning tendons in Level 330 pour strips.[87] A VSL shop drawing, approved by PB, correctly indicating post-tensioning cables, does exist for the pour strip on the 350 level.

The required post-tensioning is indicated in the Construction Documents using "callout notations." VSL and FP claimed the variability in callout locations created ambiguity (for detailed explanation see Exhibit I, page 27-29).

The Contractor Quality Control Plan provided for the resolution of questions regarding interpretation or ambiguity of the Construction Documents through discussion at meetings or written answer via the RFI process.[88] FP and their subcontractors had multiple opportunities to ask for clarification of any ambiguity regarding callout notation for locations of post-tensioning tendons in/near pour strips. Adequate channels of communication, including regularly scheduled meetings, were available to the Contractor. The RFI process, available to address and clarify any such issues, was heavily used in the SSTC project. However, FP and VSL did not use these channels in this case, relying instead upon their judgment.

Due to phased shop drawing submittal, the pre-installation conference occurred before all shop drawings were reviewed. While this approach was not prohibited in the Specifications, it allowed for ambiguity regarding anticipated and outstanding submittals. Since shop drawings were prepared as construction progressed, it was critical that a strong document control system be in place to ensure that all submittals that needed to be prepared by the construction contractor and reviewed by Architect/Engineer were known and tracked. The failure of reviewers to detect the absence of specified post-tensioning shop drawings for two of the pour strips suggests not only a weakness of the submittal control system, but also a lack of diligence with regard to this work.

Professional Error

The mild steel reinforcement was omitted from shop drawings for the level 330 west pour strip, despite performance of the required review and approval process. That control provided for a review that should have been effective had all parties adequately exercised their professional responsibilities with respect to that shop drawing. Independent review by the Quality Control manager failed to highlight differences from the contract drawings that should have been identified as variances.

In the case of the mild steel reinforcement for the west Level 330 pour strip, diligent review by the Architect/Engineer of all shop drawings was not performed, thus the A/E did not ensure that submittals depicted Contractor interpretations and methodologies of the proposed work that were in accordance with design intent.

[87] "Based on a review of our shop drawing files, no post-tensioning shop drawing submittals were provided for the Level 330 delayed pour strip areas." PB letter dated August 24, 2012, page 3.
[88] Item 1.8.A of Specification 01310 reads, "Immediately on discovery of the need for interpretation of the Contract Documents, and if not possible to request interpretation at Project meeting, prepare and submit an RFI in the form specified."

Recommendation 6

Those professionals whose lack of diligence resulted in the pour strip construction deficiencies should be held accountable.

DGS should consider implementation of changes to guard against occurrence of such errors in future projects, for example:

- All shop drawings could be required to be submitted before the pre-installation conference occurs, or
- A pre-installation conference could occur with each new area covered by a recently approved shop drawing, or
- A Submittal Registry could project the number and identity of proposed shop drawings anticipated for all phases. (For example, if only one pre-installation conference occurs at the beginning of the Definable Feature of Work, part of the conference should cover how many submittals will be generated for Designer review for the phased construction. Then as construction proceeds discussion should occur whether each of those proposed submittals have been approved during the progress meetings.)

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Concerns regarding the durability of the structure are attributed to suspected design deficiencies

Water penetrating the structure through the cracks could reach and corrode reinforcing metal, thus potentially shortening its life span significantly from the intended 50-year life. Significantly greater maintenance of the structure would likely be required, greatly increasing the cost of maintaining the structure through its projected life. Some cracking is attributable, in part, to over tensioning of tendons in concrete that was inadequately cured. The primary causes of the reduced durability include widespread cracking of various sizes throughout the structure, which are attributable to the design of the structure that according to KCE and WDP was not prepared in accordance with applicable building codes, WMATA design criteria, or industry standards. A major issue was the lack of details in the structure to accommodate normal movement.

Structural Durability

Finding 7: Stakeholder concerns related to thermal and flexural design issues were raised in early 2010 to the Structural Engineer of Record for resolution, but cracking persisted throughout later stages of construction.

In an email sent on April 7, 2010, approximately five months before the level 330 (the first elevated level) slabs were poured, DGS asked Parsons Brinckerhoff (PB) to contact the Montgomery County Department of Permitting Services (DPS) in order to resolve "a structural issue" - potential cracks of the concrete slabs as a result of stressing the post-tensioned tendon cables. Notes from a May 11, 2010 discussion among representatives from PB, Foulger-Pratt Contracting (FP), Facchina, DGS and the DPS inspector that PB had been asked to contact indicate the DPS inspector's concerns that post tensioning of the slabs and girders with the built in wall would create a zone of cracking in the slabs along certain points. The notes further state: 1.) that the inspector identified the design as an "unusual application" and expressed his opinion that the slab would crack at stressing locations since it is the weakest point; 2.) that the DPS inspector expressed his understanding that his comments were only observations and that PB was the "Engineer of Record" who did (and would therefore be responsible for) the analysis; and 3.)

PB's responses that defended the design, indicating it was consistent with 2003 revision of the building code. It was agreed, however, that Facchina and VStructural LLC (VSL) (the post-tensioned tendons subcontractor) "should" evaluate and discuss the conditions and concerns with PB to respond to the DPS concerns.

The Department of Permitting Services inspector entered a note regarding the meeting in DPS' inspections system stating that: "The DPS position is that the joints shall be designed, detailed, and constructed to permit limited movement of the slab relative to its support in order to prevent cracking of the structure during stressing operations." (DPS does not require design calculation data, as part of the permit submission requirements. DPS asserted that if, during plan review, the DPS reviewer needed more information, the reviewer could ask for whatever information is needed.)

In a June 3, 2010 letter to Facchina regarding the potential cracking at the junction of the slab and wall, VSL opined that PB was taking the right approach to understanding the issue, but that VSL ***did not have access to the design data and assumptions used by PB to substantiate the design, and they could offer no further comments without having performed a full independent review of the design of those areas***

The appearance of cracking had been documented early in the process of constructing the 330 and 350 levels. The first three level 330 slabs (1A, 1B, and 1C) were poured between September 13, and October 18, 2010. Problems related to concrete cracks became evident within 24 hours after placement (see image 8). Three ducts became exposed to view through the surface of Pour 1A shortly after being grouted.[89] Significant cracks were observed in pours 1B and 1C prior to the post-tensioned tendon stressing operations. In a September 20, 2010 meeting to review Pour 1A - the first pour of these levels – shrinkage cracks were discussed, noting that the Structural Engineer of Record (SEOR) would visit the site to inspect.

Image 8: Slab Cracking Evident 2 Days Following Placement of Pour 1B

12:32 pm on 4 Oct 2010 **(img_2236.jpg)**

12:41 pm on 4 Oct 2010 **(img_2239.jpg)**

Source and courtesy of the Montgomery County Maryland Department of General Services.

[89] RBB Daily Report by Tony Lord, 10/28/10.

The issue of cracking in the concrete was again raised by WMATA in an email to the DGS Project Team Leader that had been relayed to him by the SEOR (with copies to the other DGS team members and Contract Administrator) on October 28 2010. Cracks had occurred in some concrete slab pours within 24 hours of placement, and WMATA asserted that field observations indicated the cracking was not consistent with shrinkage. WMATA requested that an evaluation of the cracking be made to determine the cause and proper corrective measures, and that "preliminary findings should be presented prior to further concrete deck placements." WMATA added that while the cracks may not have presented a structural concern, they would require additional long term maintenance and could result in structural issues.

The DGS Project Manager returned a copy of the WMATA e-mail to the SEOR, in which he requested the SEOR to look at the cracks and provide the DGS team with his assessment. The DGS Project Manager noted that one of the DGS team had seen some hairline cracks that did not appear out of the ordinary and that Balter had not raised that issue either. A more senior DGS manager sent a follow-up email to the SEOR stating: "The County will be looking to you as the SER to provide us the guidance in this issue. We all are sensitive to keeping with schedule, but that should not keep us from doing what is right for the long term of the facility."

An email response from the SEOR to the DGS team noted that "While much of the area is used for storage of materials, I was able to find two cracks to review." His message then quotes from a documented account of the subsequent on-site observations and discussions held by the SEOR on October 30, 2010 with FP and Facchina.

In his October 30, 2010 Site Observations report, the SEOR indicated he had "reviewed Pour 1B and 1C slab top surface to find and observe cracks noted in recent WMATA correspondence" which the SEOR indicated may have been caused by "the superstructure system [experiencing] some loading or movement at an early age" although he stated it was his opinion that the cracks in the concrete were "from surface drying and minor shrinkage of that near-surface concrete."

The "Field Observation Comments" report noted three locations in the pour 1A area where concrete popped off over the slab tendon ducts and that it was clear that cover over tendon ducts was as little as ¼ inch. The SEOR reiterated that the construction contractors were to verify the slab thickness. The document also noted surface cracks in slab pours 1B and 1C, (identified as having been the subject of WMATA correspondence), and offered an opinion that the very narrow cracks observed would have been from surface drying and shrinkage of near surface concrete. The document further states: "Typical for the project, continued and increased effort to eliminate potential causes for cracks should be made including verification that formwork/shoring is undisturbed and making every effort to keep slab surface "wet" and curing measures placed as early as possible."

A "Post Tensioning Summit" meeting was held at the construction trailer on November 30, 2010 to discuss issues stemming from post tensioning operations at the SSTC. The meeting resulted in a list of more than 15 "action items" (procedures) apparently intended to confirm that human error was not causing the problems that had been observed. VSL brought an additional level of supervision on the site and MTA later observed that the new procedures had been followed.

During 2012, KCE Structural Engineers, P.C. (KCE), and Whitlock Dalrymple Poston & Associates, P.C. (WDP) each separately conducted extensive testing and modeling of the structure's design to evaluate whether it had adequate strength to bear intended loads, and whether the structure had sufficient flexibility to withstand torsion and shearing forces. They both determined that restraint was present within the post-tensioned slab system due to omission of measures to deal with stresses and forces in the design of the slabs – slabs that had been poured without a bond breaker at the intersection of the slab and the concrete wall, and by integration of those walls with the columns that supported the stiff girders. Both KCE and WDP concluded that cracking was due to these design elements.

Although evaluation of the Balter comprehensive strength testing of the sample cylinders led PB to determine that concrete had attained the 4,000 psi minimum strength necessary to commence post-tensioning stressing, findings discussed earlier in this report conclude that in-situ concrete was likely less mature and of questionable strength at the time stressing commenced. Unlike the cracking observed during the first month following concrete placement, which does appear consistent with drying and shrinkage resultant from improper curing, the horizontal cracking in the beams and girders documented by KCE during its testing is likely resultant from excessive stressing force applied to immature concrete. After the initial setting and curing period, whose passage is approximated by the 28-day compressive strength tests, existing cracks worsened, and new cracking appeared. We have found no evidence that the cracking that persisted after the 28 days could have resulted from any cause other than the design issues identified in the KCE and WDP reports.

Conclusions

Despite the Department of Permitting Services' concerns about the design of the SSTC structure in early 2010, DPS lacked the authority under the County's Special Inspections Program to override the SEOR's professional judgment.[90] DGS relied upon the SEOR's assurance that the design would prove not to result in any of the problems DPS suspected. Construction contractors and certain subcontractors were consulted by the SEOR; however, they lacked the detailed design information necessary to perform a sufficient review of the design issues. Even though it was unclear whether the deficiencies identified during PMT meetings were related to the SSTC's "unique geometr[ic]"[91] design or to construction methods employed, DGS relied on its design

[90] Issues raised by DPS were about durability, not safety. Since the Engineer for PB was responsible for the durability issues, DPS didn't have the authority to make the decision or overrule the PB engineer. DPS would have had the authority to make the decision had the issues been about safety. The pouring strip issues, for example, are safety issues.

[91] VSL June 3, 2010 letter to Facchina Construction re: Silver Spring Transit Center Potential Wall/Slab Interface Cracking. KCE Report, Attachments Vol. II, pdf page 77.

and construction contractors to reach agreement among them regarding how to correct the deficiencies observed.

Ultimately, DGS also contracted with an independent firm, KCE, to provide objective advice on the design and construction of the SSTC structure; however, it did not do so until 2012, when the structure was almost complete. This was a reactive response to problems that arose during construction. Among the difficulties this situation presents is the requirement that DGS make decisions based on information provided by professional firms that disagree on significant aspects of the design.

DGS would have benefitted from retaining KCE or another objective third party firm at the beginning of the design process to perform a "peer review" function during the design of this unusual and challenging structure.[92] That firm could have been retained to work with PB to either substantiate or modify the design. A peer review would not only be performed in occurrence of a problem - it could also be a preventive control. However, it could also be utilized if during a project there is doubt with the Designer of Record's performance.[93]

Recommendation 7

DGS should develop procedures to identify circumstances under which an independent peer reviewer should be employed to review and improve the design of unique construction projects. The trigger for a peer review could be the nature and complexity of the project design.

[92] In 2009, after project modification was necessitated by large scale underground utility relocation and other unforeseen conditions that resulted in significant delay, DGS tasked PB with providing Construction Project Management Services. Within this contract's scope of services, PB was to provide a full-time, on-site project engineer to work under the direction of the DGS Project Manager. The scope of services in the Construction Project Management Services Contract included coordination of project design activities and issues with various outside agencies, production of required progress reports to outside agencies, coordination of document reviews, and documentation and assistance to the County staff in negotiating Construction Contract changes. The project engineer had no decision making authority.
The Construction Project Management Services provided by PB were handled separately from the company's other roles in this project as Designer of Record and SEOR. A different PB engineer sealed the Construction Documents, reviewed shop drawings, and provided site observations as the designer's representative.

[93] The OLO reports, and our SME's experience indicates, that the use of peer reviews on the County level is not widespread. The SME reports they regularly perform peer and constructability reviews for federal agencies (Veteran's Administration) and state level agencies (Maryland DGS, UMD, and VDOT).

Structural Durability

Finding 8: Problems with structural design and construction were identified during 2010, and repeatedly discussed in subsequent Project Management Team meetings, but were not effectively addressed.

In the Contractor's Quality Control Plan, the County is referred to as the Construction Manager (CM). Although the term implies broad responsibilities and authority over the construction project, in practice the role of a Construction Manager can vary between construction projects. DGS personnel had primary responsibility for continuous review of all operations and audit of all test reports, evaluation of payment requests, change order management, and interaction with contractors and outside stakeholders including MTA, FTA and WMATA as well as document control activities related to those entities.

However, as Chart 6 illustrates, the roles and responsibilities of the Construction Manager were shared among many entities, prompting WMATA to opine that it seems unclear who is responsible, allowing lapses and mistakes that potentially arise due to this troubling lack of clarity.[94]

Chart 6: SSTC "Construction Management" Responsibilities as Performed

Construction Management Element	Foulger Pratt	Parsons Brinckerhoff	Balter	MontCo DGS
Conduct & Document Periodic Progress Meetings		✓		✓
Document Control	✓			
Cost Tracking & Management				✓
Evaluation of Payment Requests,				✓
Change Order Management,	✓			✓
Quality Management	✓			
Review Daily Quality Control (QC) reports				✓
Complete Daily CM Log	✓			
Schedule Control	✓			
Review and verify contractor's project record drawings are updated		✓		
Monitoring Contractor Safety	✓			
Conduct inspections			✓	
Issue inspection deficiency letter to the contractor		✓		

Source: OIG Staff Analysis

As previously stated, oversight of the project was provided by a Project Management Team (PMT), consisting of representatives of DGS, WMATA, MTA and FTA. The Project Management Team meetings were a requirement of the Project Management Plan (PMP). The team held formal monthly meetings on a continuing basis. Meeting minutes were kept by an employee of Parsons Brinckerhoff (PB).

[94] Foulger Pratt 4/17/09 Quality Control Plan, Revised Submission dated 9/1/2009.

The DGS Contract Administrator reports to the Director of DGS. The Contract Administrator assigned six permanent staff members to work full-time overseeing the project on behalf of the County, and to serve as the County's principal representatives for the SSTC project. The specific duties of each staff member, as described by DGS, are identified in Exhibit I, Appendix B. The duties include reviews of schedules and Notices of Delay proposed by the contractor, reviews of Balter daily and monthly inspection reports, reviews of RFIs, Architect's Supplemental Instructions[95], and other change instruments on the project, attending subcontractor meetings and safety meetings, attending weekly SSTC project meetings, and attending weekly meetings with the design team (PB/ZGF). Biweekly and quarterly meetings were held with MTA.

Each month, Foulger-Pratt Contracting (FP) provided DGS with a Monthly Report in the form of a detailed notebook containing hundreds of pages of documents, including construction photographs from the month, a Critical Path method schedule update, various tracking and control logs and summary reports. The DGS project management staff summarized information provided by the contractor and provided its own monthly reports to MTA and FTA. The DGS Project Team Leader was, on a daily basis, responsible for keeping DGS Division management personnel informed of all issues that would affect the success of the project.

In 2009, after redesign was necessitated by large scale underground utility relocation and other unforeseen conditions that resulted in significant delay, DGS tasked PB with providing Construction Project Management Services. Within the Scope of Services, PB was to provide a full-time on-site project engineer to work under the direction of the DGS Contract Administrator. The scope of services in the Construction Management Services Contract also included coordination of project design activities and issues with various outside agencies, production of required progress reports to outside agencies, coordination of document reviews, and assistance to the County staff in negotiating Construction Contract changes. The responsibilities of the project engineer, who in some documents is referred to as Construction Manager, do not correlate to the role of a typical industry Construction Manager.[96] The project engineer was under the direct supervision of the County project manager and had no decision making authority.[97]

The Construction Project Management Services provided by PB were handled separately from the company's other roles in this project as Designer of Record (DOR) and Structural Engineer of Record (SEOR). A different PB engineer sealed the Construction Documents, reviewed shop drawings and provided site observations as designer's representative.

The construction contract between the county and FP uses the term "Project Manager" to refer to the person designated by FP as having authority to act on behalf of the contractor with respect to all aspects of the project and to whom the Superintendent reports. As defined by the general terms and conditions of the construction contract with FP, construction activities are performed

[95] Architect's Supplemental Instructions (ASI) are used when the Architect/Engineers proposes a modification to the Construction Documents.
[96] An Owner's Guide to Project Delivery Methods by the Construction Management Association of America, August 2012, page 15
[97] Memorandum dated June 16, 2009 attached to Construction Manager Contract

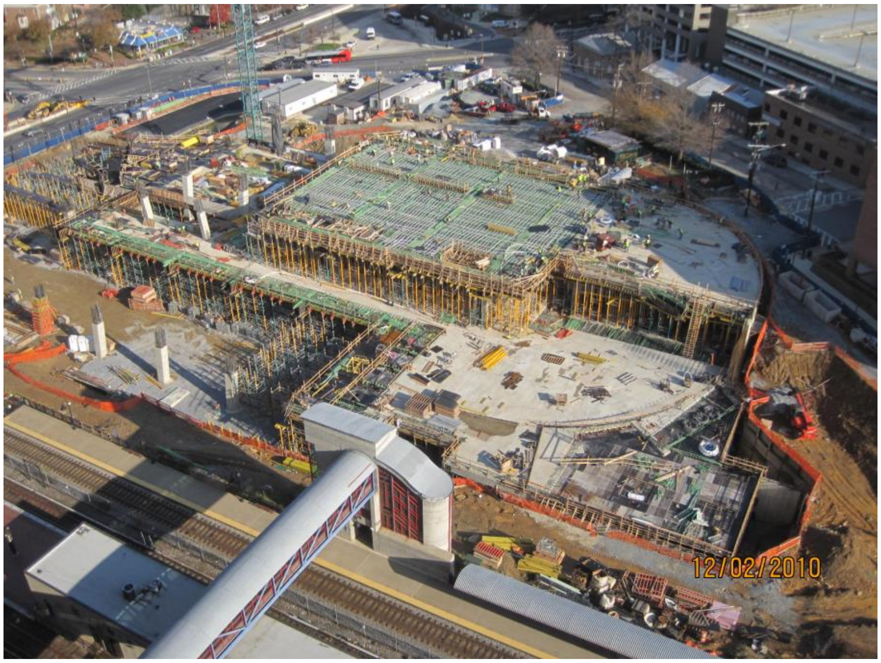
under the direction of the FP Project Manager. Responsibilities such as document control, quality management, and schedule control were performed by FP.

Quality control responsibilities, including inspecting, testing and checking the products of construction activity, were the responsibility of FP. However, responsibilities for inspections and testing were performed by Balter. In an April 17, 2009 letter transmitting a revised Quality Control plan to the County, FP states: "the independent testing agency provided by the Owner [Balter] is a major component in the QC for the project and the reviewers will note the inclusion of the testing agency and its forms in the QC Plan." [98]

PMT minutes from mid-November 2010reflect discussion of the concrete problems in the SSTC structure that were later discussed in the 2013 KCE Structural Engineers, P.C. (KCE) and Whitlock Dalrymple Poston & Associates, P.C. (WDP) reports. During that meeting, WMATA reportedly raised the issue of having the contractor perform a complete survey of deck thickness to identify thin slab locations. Other issues were to be addressed by the SEOR.

At the point in time of the November PMT meeting, less than half the slab concrete had been placed, and the structure was less than 50 percent complete (see Image 9).[99] The meeting minutes indicate that the issues were not unusual or unexpected in a complex structure like the SSTC and that the SEOR was working to address each one. Although the construction schedule and completion date were discussed during the meeting, there was no suggestion that these issues might further delay the completion of the SSTC.

Image 9: Silver Spring Transit Center Construction Progress – December 2, 2010



9:56 am on 2 Dec 2010 **(img_3024.jpg)**

Source and courtesy of the Montgomery County Maryland Department of General Services.

The meeting minutes suggested that participants might have, at that time, expected that remedial actions would be identified and applied to correct the problems, both in the constructed and unconstructed sections of the structure. These concrete issues were discussed in subsequent

[98] Foulger Pratt Quality Control Plan Revised Submission April 17, 2009 cover letter
[99] SSTC Report #49, MTA-Monthly Report for December 2010.

meeting but remained unresolved as work on the SSTC continued. Almost a full year later, October 18, 2011 meeting minutes indicated the PMT was still unable to determine the effect of and a resolution for the concrete cover and thickness and spalling issues on the potential project completion and acceptance delays. PMT meeting minutes reflect that structural strength and durability were recurring concerns in the context of actions to be pursued by WMATA and MTA.

The proposed actions included a complete building survey and Ground Penetrating Radar (GPR) survey to determine the extent of the thickness issues, petrographic testing, and spall repair. Slab laser scanning and GPR by MTA began in November 2011. Preliminary results provided to DGS indicated some remediation may be needed. WMATA's call in February 2012 for a comprehensive review that would include looking at cracking, post-tensioned tendon elongations, and thin slabs was reportedly taken under advisement by DGS. In the March 2012 PMT meeting, WMATA asserted that any remediation plan must be based on an analysis of the entire SSTC building structure to determine deficiencies. During the same meeting, FTA reportedly asked for a review of the PMP, indicating there appeared to be a requirement for a higher level meeting than the management team meeting. DGS representatives stated there were several reoccurring meetings that satisfied the requirement.

In April 2012 DGS reported to the PMT that the construction contractor would prepare a presentation regarding a remediation plan. It was also reported that PB had completed their evaluation of the SSTC structural integrity, identifying several deficiencies, and that PB would evaluate the FP remediation plan once the full plan had been submitted.

Recommended actions, including a 2 inch Latex Modified Concrete (LMC) overlay, recommended by Parsons Brinckerhoff, Inc. (PB) and MTA in mid-2012, were proposed during the following months, but meeting minutes indicate "WMATA has not accepted this proposed fix and continues to question the root cause of the cracks."

As stated earlier in this report, Montgomery County contracted with KCE in June 2012, to conduct an evaluation of the in situ conditions of the structural frame of the SSTC based on their independent document review, field investigation observations, and engineering analyses, and WMATA contracted for the services of WDP to determine the condition of the SSTC and to understand whether it satisfies the required strength and durability to meet its intended uses and service life. Those efforts resulted in a March 15, 2013 report by KCE and a May 2, 2013 by WDP, both of which identified similar deficiencies that require remediation.

The expectations of DGS - that PB would ensure the design met all applicable standards, and that FP and its subcontractors would construct the SSTC in accordance with Construction Documents - were not met.

As evidenced in the comparisons of construction data presented in earlier findings in this report, Balter inspectors captured data during the course of construction that evidenced deviation from design and construction specifications, but documents do not indicate that data was ever used to find and raise major concerns to the attention of FP or DGS. Performing that type of analytical review is not a responsibility typically assigned during a construction project and there is no indication that responsibility was assigned in this case.

Conclusions

In response to problems that surfaced during the project, DGS contracted with PB to provide "construction management" services, but that individual was not independent of PB and the functions he was assigned did not allow him to serve as an effective construction manager for this project.

Rather than hiring an individual to supplement DGS staff under a "construction manager" contract, and acquiring the services of KCE after the major construction efforts had concluded, DGS would have benefitted from retaining an objective third party firm at the outset to serve as an independent construction manager. That firm could be selected on the basis of expertise in dealing with structures of unusual design similar to the SSTC.

Typical industry practice is for Construction Managers to be contracted either before or at the same time as the Contractor. Their primary role is to observe the work of the construction contractor for progress, workmanship, and conformance with Construction Documents and existing codes. The CM notifies Owners of any problems and may provide recommendations for resolution. Direction is given to the Contractor from the Owner. However, such a firm could also be utilized if during a project there are concerns about the construction contractor's performance.

Recommendation 8

DGS should develop procedures to identify circumstances under which an independent third party should be employed to serve as Construction Manager on an atypical construction project. The trigger could be a dollar value or uniqueness of the project.

DGS should develop protocols to ensure that controversial issues encountered/problems experienced by or with the construction contractors are promptly and effectively addressed. As an example, DGS could develop and incorporate into its contracts a systematic process that identifies deficiencies and withholds payments pending resolution. This "rolling punch list of deficiencies" control would address construction issues. Once an item is identified as deficient, it would be added to a rolling punch list which is tied to payments. Therefore, the Contractor is motivated to correct issues in a timely manner. FP generated their own internal contract compliance list which was included and discussed at progress meetings, but evidently was not tied to payments.

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Structural Remediation of the Paul S. Sarbanes Silver Spring Transit Center

As stated earlier in this report, Montgomery County contracted with KCE Structural Engineers, (KCE) in June 2012, to conduct an evaluation of the in-situ conditions of the structural frame of the SSTC based on its independent document review, field investigation observations, and engineering analyses. WMATA contracted for the services of Whitlock Dalrymple Poston & Associates, (WDP) to determine the condition of the SSTC and to understand whether it satisfies the strength and durability requirements necessary to meet its intended use and service life.

Those efforts resulted in a March 15, 2013 report by KCE and a May 2, 2013 by WDP, both of which identified similar deficiencies that require remediation. Following the issuance of the KCE report in March 2013, a remediation kickoff meeting was held on April 25, 2013. As a spinoff of that meeting, the Cooperative Remediation Working Group (CRWG) was formed, which consists of professional design engineers from Parsons Brinckerhoff, KCE, Wiss, Janney, Elstner Associates (WJE), Walter P. Moore, and Simpson Gumpertz & Heger (SGH); construction personnel from Foulger-Pratt Contracting (FP), VStructural, Wagman, and Facchina; and WMATA and DGS staff. The charge of that group is to agree upon design and implementation of a remediation plan to resolve all of the issues raised in the KCE and WJE reports to the satisfaction of WMATA and Montgomery County.

By late summer 2013 a remediation plan for pour strips on the 330 level had been agreed to by the CRWG and was being implemented by the contractors. Work on the pour strips consisted of adding beams under the strips and placing new reinforcing and concrete on the surface.

In early December 2013, the Project Management Team was advised that construction activities directly related to remediation of the east and west 330 level pours strips had been completed.

The CRWG also adopted a plan to fill slab cracks and resolve the slab thickness deficiencies by topping the Level 330 and 350 slabs with a Latex Modified Concrete (LMC) overlay that will be applied as a final step once the weather and temperatures permit, and after decisions regarding any remedial actions necessary to address torsion and shearing force issues have been made.

On April 8, 2014, the Director of the Department of General Services updated the County Council on the status of remediation discussions that had been ongoing among the County and its independent consultant, KCE, WMATA, and the Structural Engineer-of-Record, Parsons Brinckerhoff, to review the design calculations.

In his statement, the Director indicated that the KCE recommendation plan and engineering design requires removal of material and drilling into the structure. WMATA "…questioned whether this work needs to be performed or, if it is necessary, may be deferred until evidence of stress occurs, if at all." The Director reported that the County Executive directed DGS to engage in negotiations under which Parsons Brinckerhoff would post a bond in the amount necessary to pay for this work, should it become necessary in the future. He also reported that the County Executive had commissioned an advisory panel to provide him with advice on the final work to be done.

Subsequent Event

On May 8, 2014, the County Chief Administrative Officer advised members of the County Council that the County Executive had directed County contractors to move ahead on remediation work at the Silver Spring Transit Center. That work would address the shear and torsion recommendations contained in the April 21, 2014 report commissioned by the County Executive entitled Report of the Independent Advisory Committee Regarding the Status of the Silver Spring Transit Center.

Image 10: Additional Beams for Remediation of Shear and Torsion Deficiencies per KCE and IAC Recommendations

4:24 pm on 12 Dec 2013 **(sstc-beam10.jpg)**

Source and courtesy of the Montgomery County Maryland Department of General Services.

Chief Administrative Officer's Response

OFFICES OF THE COUNTY EXECUTIVE

Isiah Leggett
County Executive

Timothy L. Firestine
Chief Administrative Officer

MEMORANDUM

May 14, 2014

TO: Edward Blansitt, Inspector General

FROM: Timothy L. Firestine, Chief Administrative Officer

Timothy L. Firestine

SUBJECT: Final Draft Report, Project Management Deficiencies in Constructing the Paul S. Sarbanes Silver Spring Transit Center

I am in receipt of your memo and final draft report dated April 15, 2014 detailing the review conducted by your office concerning the Silver Spring Transit Center. Your assessment of this issue has been thorough and fair. Please find below specific responses to your audit recommendations.

IG Recommendation 1: DGS should improve its controls for future projects in a manner that is consistent with the lessons learned and additional recommendations contained in Exhibit I, the report "Analysis of Project Controls," in addition to other recommendations made in this report.

CAO Response: This recommendation furthers the thesis of Alpha Corporation's Analysis of Project Controls report which largely states that implementation and refinement of project controls would have prevented many if not all of the construction deficiencies in the Transit Center. The report states, "Therefore, identification of controls that were omitted, deficient or failed is necessary to avoid repeating mistakes due to misplaced confidence in deficient controls." The County set forth specific Project Controls in the Contract Documents. Many of the controls evidenced in the report, particularly those that deal with concrete composition and placement, are clearly identified and set forth in the Contract Documents and place the responsibility for quality assurance and control measures on Parsons Brinckerhoff (PB), Foulger-Pratt (FP), and Robert B. Balter Company (Balter). Those contractors should have employed appropriate quality assurance and control measures to achieve more positive results. PB, FP, and Balter failed to impose quality assurance and control measures to ensure that the concrete complies with the Project requirements. The County agrees that it should continue to improve its project controls so that the mistakes made by the contractors on the Transit Center are not repeated in future construction projects.

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May 14, 2014
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IG Recommendation 2: DGS should ensure construction documents clearly establish responsibility for and performance of systematic analysis of data collected and recorded during construction in order to identify possible inconsistencies with specifications, project control weaknesses, and construction deficiencies that should be investigated and resolved.

CAO Response: This section of the report focuses on the addition of excessive amounts of water to the concrete mixture and the subsequent lowering of the concrete compressive strength. FP was responsible for ensuring the composition of the specified and accepted concrete mix met Project requirements. Balter, as the testing agent, was required to inspect, test, and monitor the composition and placement of the concrete for the County. The Contract Documents are very clear on limiting water addition to the concrete mixture. FP and Balter were required to monitor and document the composition of the concrete. FP should have complied with the requirements of the Contract Documents and it should not have poured defective concrete. Balter should have noted the failure of FP to adequately ensure the composition of the concrete and it should immediately have alerted the County of the defective condition so that the County would have had the opportunity to stop the concrete pours until FP was prepared to place concrete that met with the requirements of the Contract Documents. On future complex construction projects, DGS will utilize the services of a Construction Management firm for greater oversight of all construction operations, thereby lessening the likelihood that similar problems will occur.

IG Recommendation 3: In future projects, DGS should ensure that all specification requirements are reviewed and implemented unless a variance is mutually discussed and agreed upon. Temperature limits during curing should be monitored and maintained, and specifications for duration of curing should be strictly observed. Confusion about where to take samples and about cold weather limits should be avoided by clearer language in specifications. Any conflicts between specifications and standards should be resolved in favor of the more conservative of those required by stakeholders (in the case of the SSTC, the stakeholders are DGS, and WMATA).

CAO Response: This section of the report addresses the requirements for cold weather curing and thermal protection as it relates to concrete placement. We agree that the controls are clearly identified and set forth in the Contract Documents. Further, we agree that the records collected by FP and Balter during the project clearly indicate that the details of curing concrete were not addressed in strict accordance with Contract Documents. The contract requirements and applicable building code requirements were clear and FP and Balter knew exactly what the cold weather curing and thermal protections were to be used for the pouring and curing of slabs. Nonetheless, both FP and Balter substantially ignored those requirements. It is clear that observations and evaluations by the County and its contractors and consultants could influence quality of future work. We agree that enforcement of the requirements of the Contract Documents

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serve to avoid or alleviate mistakes made by a general contractor and special inspector. On future complex construction projects, DGS will utilize the services of a Construction Management firm for greater oversight of all construction operations, thereby lessening the likelihood that similar problems will occur with cold weather curing and thermal protection.

IG Recommendation 4: DGS should modify its contract specifications for future construction projects to ensure that concrete test specimens are made as near as possible to the actual point where concrete is placed. Where referenced standards require testing at the point of delivery, DGS should clarify in the specification that such testing is in addition to typical testing.

CAO Response: This section of the report addresses the discrepancy of concrete sampling between the point of delivery and the point of placement. The requirements of the Contract Documents are clear in that the testing cylinders are to be made and stored as near as possible to the point of deposit. Balter failed to comply with the Statement of Special Inspections which references ASTM Standard C31/C31M that indicates that cylinders should be made and stored in or on the structure as near as possible to the point of deposit. It was Balter's responsibility as the special inspector to ensure that the test cylinders were made and stored as near as possible to the point of the concrete deposit. FP was also responsible to ensure that the cylinders were made and stored as near as possible to the point of deposit by construction contract specification section 03300.1.5.B which references ASTM C94. Therefore, we do not agree with this recommendation. The requirements are set forth in the applicable building and material codes as well as set forth in the Contract Documents. Thus, no ambiguity existed in this Project. Balter and FP ignored the applicable standards and the requirements of their respective contracts. On future complex construction projects, DGS will utilize the services of a Construction Management firm for greater oversight of all construction operations, thereby lessening the likelihood that similar problems will occur with concrete sampling.

IG Recommendation 5: In future projects, DGS should ensure its construction contractors utilize a construction method that allows direct measurement of floor thickness so that inspectors can help the Contractor by identifying problems before the concrete is placed. Alternatively, a second, independent survey should be performed. Survey equipment could be utilized by inspectors to continuously monitor concrete thickness during placement, and submit a report of survey results for Owner and SEOR approval.

DGS should hold construction contractors accountable for any remediation and increased maintenance costs that will likely result from the contractor's failure to ensure specified concrete slab thickness was attained during placement.

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CAO Response: This section of the report addresses the issue of slab thickness. The Contract Documents specified a dimension for the slab thickness. We agree that FP should have utilized a method that ensured direct measurement of the floor thickness. We further agree that we should hold FP accountable for any remediation and increased maintenance costs that will likely result from FP's failure to ensure specified concrete slab thickness. On future complex construction projects, DGS will utilize the services of a Construction Management firm for greater oversight of all construction operations, thereby lessening the likelihood that a similar problem with slab thickness would occur.

IG Recommendation 6: Those professionals whose lack of diligence resulted in the pour strip construction deficiencies should be held accountable.

DGS should consider implementation of changes to guard against occurrence of such errors in future projects, for example:

- All shop drawings could be required to be submitted before the pre-installation conference occurs, or
- A pre-installation conference could occur with each new area covered by a recently approved shop drawing, or
- A Submittal Registry should project the number and identity of proposed shop drawings anticipated for all phases. (For example, if only one pre-installation conference occurs at the beginning of the Definable Feature of Work, part of the conference should identify the number of submittals that will be generated for Designer review for the phased construction. Then as construction proceeds discussion should occur whether each of those proposed submittals have been approved during the progress meetings.)

CAO Response: This section of the report addresses the pour strips. We agree that the control measures in place should have prevented the construction deficiencies in the pour strips on Level 330. While we agree with the recommendation that we should hold FP and PB accountable for the pour strip construction deficiencies, we believe that Balter also bears responsibility for its failure to account for the omission of post-tensioning cables in that location.

IG Recommendation 7: DGS should develop procedures to identify circumstances under which an independent peer reviewer should be employed to review and improve the design of unique and challenging construction projects. The trigger for a peer review could be the nature and complexity of the project design.

CAO Response: This recommendation proposes that an independent peer reviewer be employed for unique and complex construction projects. Note that this project was designed during the period that pre-dated the formation of DGS as a department in the

Edward Blansitt, Inspector General
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County's government. Since then, the practice of independent peer review for large, complex, or unique projects has become much more commonplace. DGS frequently employs independent peer review on these types projects that feature project review by an independent team. This has had a decidedly positive effect on those projects.

IG Recommendation 8: DGS should develop procedures to identify circumstances under which an independent third party should be employed to serve as Construction Manager on an atypical construction project. The trigger could be a dollar value or uniqueness of the project.

DGS should develop protocols to ensure that controversial issues encountered/problems experienced by or with the construction contractors are promptly and effectively addressed. As an example, DGS could develop and incorporate into its contracts a systematic process that identifies deficiencies and withholds payments pending resolution. This "rolling punch list of deficiencies" control would address construction issues. Once an item is identified as deficient, it would be added to a rolling punch list which is tied to payments. Therefore, the Contractor is motivated to correct issues in a timely manner. FP generated their own internal contract compliance list which was included and discussed at progress meetings, but evidently was not tied to payments.

CAO Response: This recommendation proposes the use of a construction manager for a project like the Transit Center. Since the formation of DGS, the use of construction management expertise has been increasingly emphasized. We agree that were the Transit Center's construction begin today,DGS would use a construction management firm. DGS has currently prepared a solicitation to select construction management firms to be used on future projects.

If you have any questions, please feel free to contact me or Assistant Chief Administrative Officer Fariba Kassiri, who can be reached at (240) 777-2512 or Fariba.Kassiri@montgomerycountymd.gov.

TLF:dd

cc: Fariba Kassiri, Assistant Chief Administrative Officer
David Dise, Director, Department of General Services
Marc Hansen, County Attorney
John Markovs, Deputy County Attorney

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Acronyms & Terminology

ACI	American Concrete Institute. A non-profit technical society that has developed many of the concrete industry's design standards and recommendations.
ASTM	ASTM International (formerly the American Society for Testing and Materials). An international organization that develops and publishes voluntary consensus technical standards.
Balter	Robert B. Balter Company. The company selected as inspector of the SSTC.
Beam	In the SSTC, a secondary, horizontal structural element that withstands load by resisting bending. Loads carried by beams in the SSTC are transferred to girders.
Construction Documents	Final drawings and Specifications containing detailed requirements written in paragraph form that must be satisfied for materials, design, products, or services, that were prepared by the Design Team and approved by Montgomery County Department of Permitting Services in 2008.
DGS	Montgomery County Department of General Services, also referred to as "County"
DPS	Montgomery County Department of Permitting Services. The branch of government that issues building permits.
Facchina	Facchina Construction Company, Inc. The company selected by FP to provide all concrete construction activities for the SSTC covered in this report.
FP	Foulger-Pratt Contracting, LLC. The company selected to implement construction of the SSTC.
Girder	In the SSTC, the primary, horizontal structural element that withstands load by resisting bending. Loads carried by girders in the SSTC are transferred to vertical structural elements such as columns or walls.
KCE	KCE Structural Engineers. The company selected by the County to perform a structural evaluation of the SSTC.
PB	Parsons Brinckerhoff, Inc. and its predecessor affiliates PB Americas, Inc. and Parsons Brinckerhoff Quade & Douglas, Inc. who entered into contracts with Montgomery County. The company who, as Designer of Record, designed the SSTC. See also SEOR.
PMT	Project Management Team - The Management Team comprised of the managers responsible for the transit center project delivery.

Acronyms & Terminology

RFF	Rockville Fuel and Feed Co., Inc. A company who provided most of the ready-mixed concrete in the Level 330 & 350 slabs, beams, and girders of the SSTC.
RFI	Request for Information. Contractors generate RFIs in order to ask the Design Team a question and obtain written information regarding the project. Also, known as a Request for Interpretation.
SEOR	Structural Engineer of Record. On this project the SEOR was an employee of Parsons Brinckerhoff, Inc. Also referenced as SER.
Slab	In the SSTC, a horizontal, steel reinforced concrete structural element serving as the drive lanes and floors. On Levels 330 & 350, slabs set atop beams and girders.
Spalling	Cracking, breaking, chipping, or fraying of a concrete slab's surface, usually confined to a small area.
Specifications	See Construction Documents
SSTC	The Paul S. Sarbanes Silver Spring Transit Center, the subject of this Inspection. See the introduction for a description of the facility.
VSL	VStructural LLC. The company selected by Facchina to provide all post-tensioning for the SSTC.
w/c	Ratio of water to cement in concrete. The *w/c* ratio has a significant influence on the strength and durability of concrete.
WMATA	Washington Metropolitan Area Transit Authority. The agency that agreed to provide maintenance and operations for the SSTC.