**CITY AND COUNTY OF DENVER, COLORADO**
**FOR AND ON BEHALF OF ITS DEPARTMENT OF AVIATION**

# VOLUNTARY EVENT NOTICE RELATING TO

# AIRPORT TERMINAL CONCRETE TESTING RESULTS

The City and County of Denver, Colorado (the "City"), through and on behalf of its Department of Aviation (the "Department"), and Denver Great Hall LLC ("DGH") entered into a Development Agreement dated as of August 24, 2017 (the "DA"). Pursuant to the DA, DGH, among other things, has agreed to design, construct, finance, operate and maintain certain areas within the Jeppesen Terminal (the "Terminal") at the Denver International Airport (referred to as the "Great Hall Project").

In the fall of 2018, DGH tested concrete samples from certain areas of Levels 3 and 5 of the Terminal (the "Phase I Construction Area") for the Great Hall Project. On November 2, 2018, DGH provided the Department a notice of relief event (the "Notice") indicating that preliminary test results of certain concrete samples from the Phase I Construction Area yielded compressive strength results lower than the design strength that was specified in the DA (the "Relief Event"). The design assumptions used in the DA for the Great Hall Project were derived from the City's historic Terminal design record documents.

In the Voluntary Event Notice Relating to Construction of Great Hall Project dated January 29, 2019, the City stated that "[a]dditional comprehensive testing and analysis will continue … to further evaluate the strength and composition of the concrete in the entire Terminal." Engineering and material testing firms engaged by DGH performed additional testing to assess compressive strength of concrete and presence of alkali-silica reaction ("ASR") in the Phase I Construction Area. The Department also engaged its own engineering and material testing firms to assess compressive strength of concrete and presence of ASR in the Phase I Construction Area and other areas throughout the Terminal, including its foundation.

In a letter dated May 28, 2019, Simpson Gumpertz & Heger ("SGH"), an engineering firm retained by the Department, analyzed the results of the foregoing testing performed by each party and provided recommendations for additional ASR testing of concrete. A copy of the SGH letter is attached as Appendix A. A brief summary of certain conclusions from the SGH letter is set forth below; such summary does not purport to be comprehensive and is qualified in its entirety by reference to the SGH letter attached as Appendix A.

- Since concrete strength in the Terminal floor essentially meets ACI 318 standards (the industry standard adopted by U.S. building codes for design and evaluation of reinforced concrete structures) and normal occupancy loads are substantially less than the original design load, the Terminal floor has adequate strength for continued safe use in its intended occupancy (for the few groups and cores that did not meet this criterion, the deviation below the acceptance criteria was very small);

- Concrete cores removed from shear walls and foundations contain concrete that meets or exceeds the strength requirements specified by the design documents and provide the strength intended by the original designer;

- Petrographic examination of cores removed from certain parts of the Terminal foundations show trace amounts of ASR have occurred, but the extent of ASR is not destructive and strength testing of concrete removed from these same foundations exhibited excellent strength in excess of the specified values; and

- The limited amount of ASR that has occurred is due to the presence of moisture from the surrounding soils but has been limited by the fly ash content of the concrete and ASR does not pose an immediate threat to the structure.

SGH recommended (1) additional strength and petrographic testing of foundations located around the perimeter of the garage structures located to the immediate east and west of the Terminal, (2) additional strength and petrographic testing of concrete in the concourse structures, parking structures, AGTS tunnels and other significant structures, and (3) in five years, a repeat of testing foundation elements previously tested to determine if significant increase in the trace indications of ASR in these elements has occurred. If no further indications of ASR are found at that time, the SGH letter stated that ASR will not be considered an issue for the Terminal and no further testing is recommended.

The Department intends to follow the recommendations of SGH set forth in its letter. It is too early to determine the impact, if any, of the results of additional testing on Airport operations, or associated costs.

Dated: May 28, 2019

# APPENDIX A

28 May 2019

Mr. Max Taylor
Assistant City Attorney
Denver International Airport
DEN Legal
Airport Office Building 9th Floor
8500 Pena Blvd.
Denver, CO 80249-6340

Project 187355 – Investigation of Low Strength Concrete, Jeppesen Terminal, Denver International Airport; Letter Report

Dear Mr. Taylor:

This letter summarizes the results of our investigation of structural concrete at the Jeppesen Terminal at Denver International Airport (DEN).

## 1. INTRODUCTION

### 1.1 Background

Denver International Airport was constructed in the period 1991-1994 and officially opened to receive airline traffic in February 1995. The airport comprises a main passenger terminal (the Jeppesen terminal) a series of garages located immediately to the east and west of the terminal, three passenger concourses (A, B and C) a light rail system (AGTS) that runs north to south through the terminal connecting to the three concourses, a runway complex, and a number of maintenance and administration buildings.

The Jeppesen terminal itself is a six-level structure with airline ticketing counters on the 6th level, baggage claim and a "Great Hall" on the 5th level, a passenger station for the AGTS on the 4th level, baggage handling on the 3rd level, and back of house functions and mechanical rooms on the remaining levels. The distinctive architectural feature of the terminal is a tensioned fabric superstructure, designed to evoke the image of the front range of the Rocky Mountains. The central portion of the 5th level, which houses the Great Hall, is open to the fabric roof above. The Great Hall currently houses TSA security screening at the north and south ends and a transit hall with access to baggage claim on the east and west sides. Figure 1 is an overall aerial view of the terminal complex area showing the locations of the major structures. Figure 2 is a key plan of the main terminal showing column line numbering and Figure 3 is a cross section through the terminal, looking to the north.

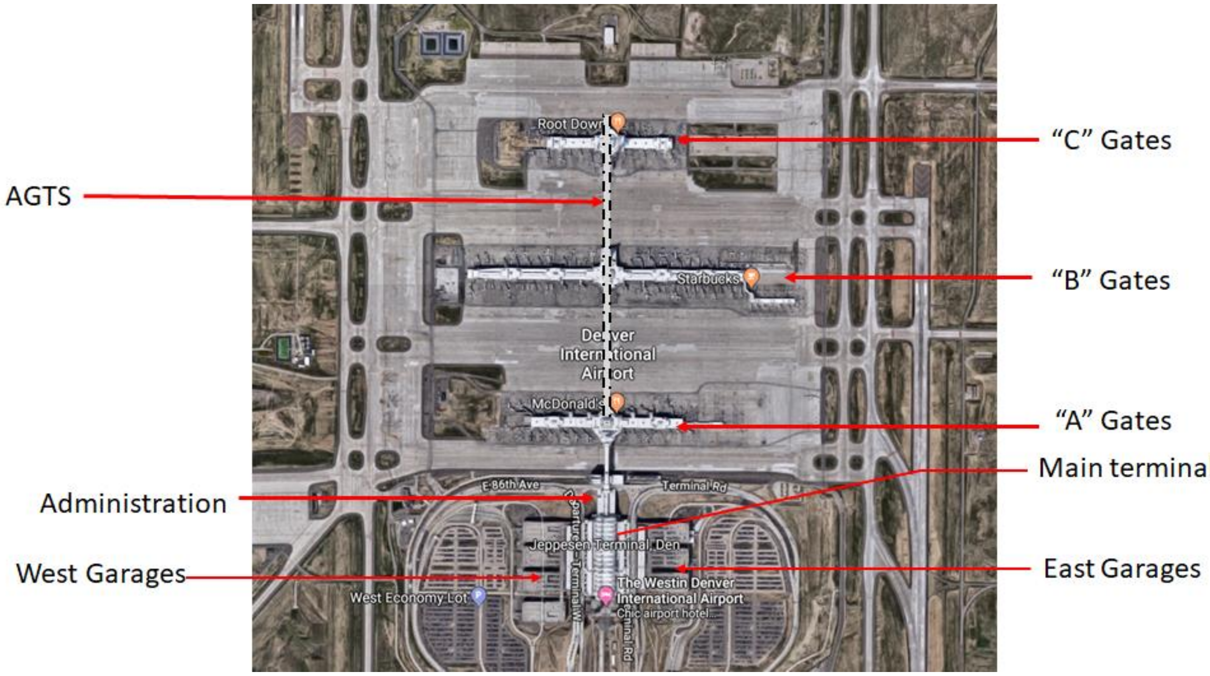


**Figure 1. Aerial view of terminal complex**

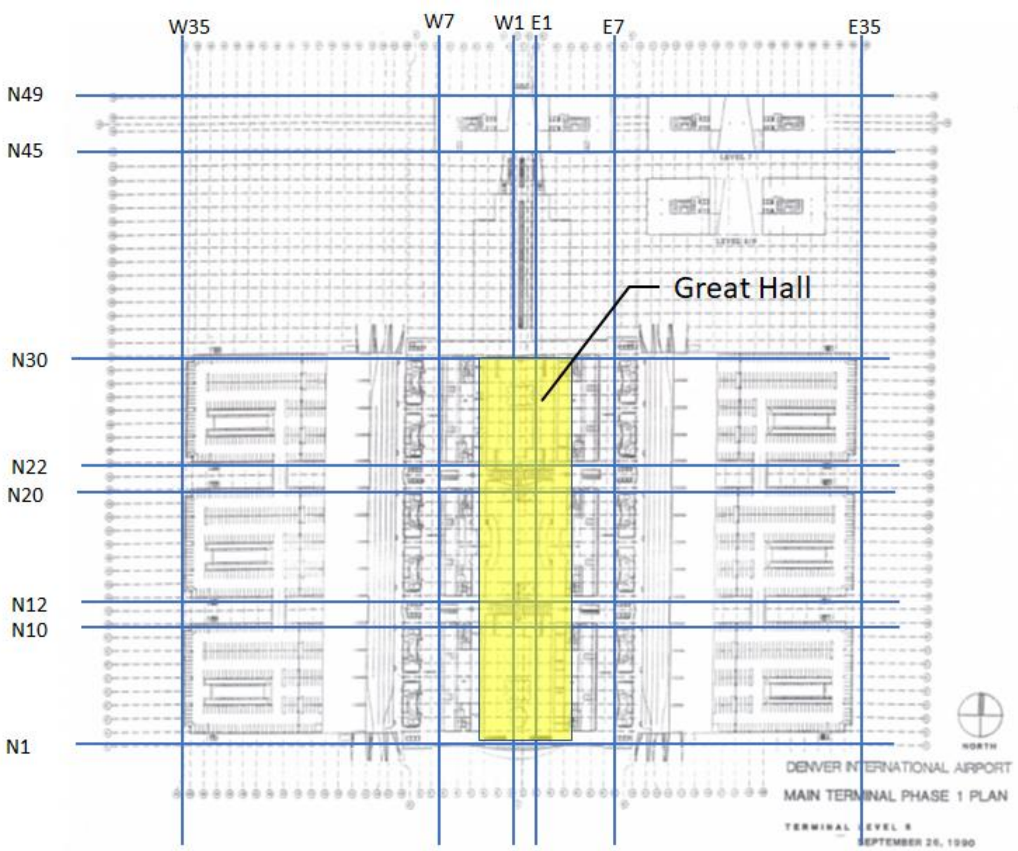


**Figure 2. Key plan of main terminal complex showing location of Great Hall**

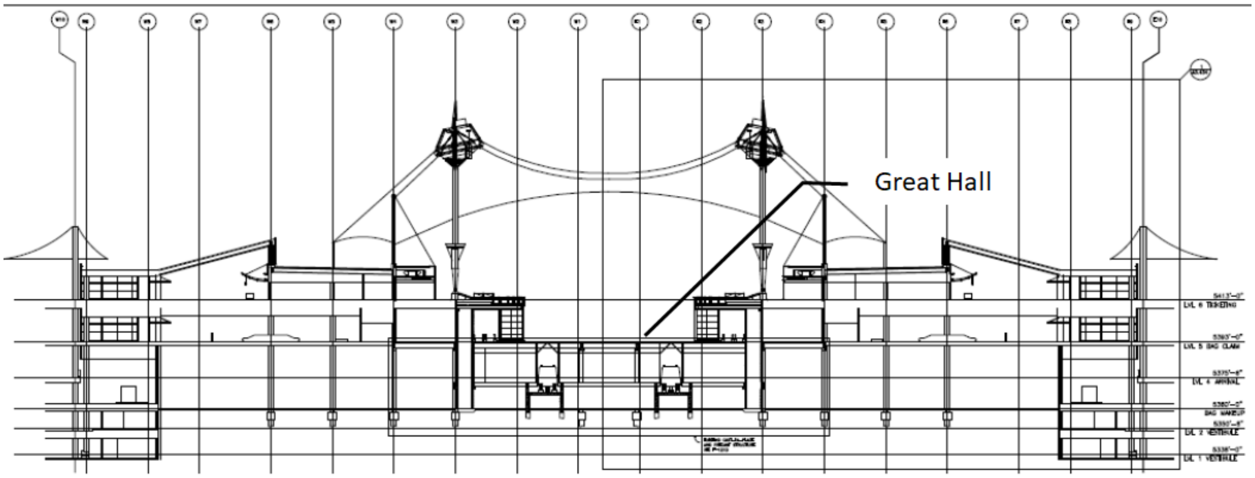


**Figure 3. Cross-section through main terminal complex**

DEN has entered into a public-private partnership (P3) with Denver Great Hall LLC ("Developer") to redevelop and repurpose the Great Hall. Under this program, the 6th level floor will be extended out over the sides of the Great Hall with cantilevered steel framed sections. Airline ticketing counters on the 6th level will be re-configured and TSA screening will move from the Great Hall to the expanded 6th level, making the Great Hall available for passenger serving retail spaces, which will be operated by the Developer under a long-term lease.

The Developer planned to place construction cranes on the Great Hall floor slab to facilitate erection of structural steel for the expanded 6th level. Prior to proceeding with the placement of cranes, the Developer retained Olsen Engineering of Wheat Ridge, Colorado to obtain cores from the existing concrete and perform testing to evaluate its strength and other properties. Olsen determined that several cores extracted from the 5th level floor slab and 3rd level shear walls tested at lower indicated compressive strengths than specified by the original construction documents. Reported shear wall strengths were substantially lower than the 6,000 psi believed to be the original design strength. DEN retained Simpson Gumpertz & Heger to evaluate this reported condition and to advise the airport of potential impacts on continued operations and effect on the Great Hall project.

## 1.2 Objectives

The objectives of our investigation are to:

1. Evaluate the compressive strength of concrete elements throughout the complex.

2. Determine if low-strength concrete represents a safety issue that would impair continued use and development of the terminal.

3. Recommend programs of additional testing and/or remediation as appropriate.

### 1.3 Scope

Our scope of investigation included:

1. Review concrete testing reports prepared by Olsen Engineering.

2. Review test reports, design documentation and supporting calculations prepared by the Developer's consultants.

3. Review construction documents and records for the original terminal construction.

4. Develop a program of supplementary concrete testing.

5. Visit the site to observe the structures, select test locations, and observe removal of specimens.

6. Review applicable standards.

7. Perform independent calculations.

8. Form an opinion as to the adequacy of concrete in the complex and the impact of any deficient concrete on airport operations and the construction project.

9. Prepare this report.

## 2. DOCUMENT REVIEW

### 2.1 Original Construction Documents

#### 2.1.1 Structural Drawings – Contract F121D

We reviewed a series of structural drawings prepared by S. A. Miro, Inc., entitled "Denver International Airport Terminal Complex, DIA Contract No F-121D, Construction Documents, Terminal Complex, AGTS Platform & Stations", dated 26 September 1990. We noted:

1. These documents include the requirements for construction of reinforced concrete structural elements at the 5$^{th}$ level and below in the area bounded by column lines N1 to N30 and W7 to E7.

2. Sheet S0.001, Note 4, "Concrete" specifies the following required compressive strengths for structural concrete:

    a. Caissons, grade beams and foundations, 4,500 psi.
    b. Cast in place walls and pilasters, 4,000 psi.
    c. Toppings, 4,000 psi.
    d. All other concrete, 4,000 psi.

3. The drawings show that the Great Hall floor consists of precast concrete double-T elements supporting a 7 in. thick concrete topping slab.

4. Sheet S0.001 shows that the 5th level Great Hall floor slab was designed for an operating live load of 100 psf and a construction live load of 250 psf.

#### 2.1.2 Structural Drawings – Contract F121B

We reviewed a series of structural drawings prepared by S. A. Miro, Inc. entitled “Denver International Airport Terminal Complex, DIA Contract No. F-121B, Construction Documents, Terminal Complex Structure and Finishes, Structural Drawings, Volume 6”, dated 31 January 1995. We noted:

1. The scope of these documents includes construction of concrete structure to the west of line W7 and east of line E7, between column lines N1 and N31, structural steel construction, including concrete-filled steel deck, the tension fabric roof, as well as all terminal construction north of line N31.

2. Sheet S0.010, Note 4 “Concrete” specifies the following required compressive strengths for structural concrete:

   a. Caissons, 4,500 psi.
   b. Foundation walls, Grade Beams and Caisson caps, 4,500 psi.
   c. Columns and shear walls, 6,000 psi.
   d. Beams slabs and miscellaneous concrete, 4,000 psi.

3. Sheet 5.002F shows an elevation of walls termed “Shear Walls” located at Gridlines 32.1 and 44.9, located in the Administration Building north of the Jeppesen terminal.

4. Sheet 5.253 shows elevations of walls termed “Shear Walls” at Gridlines N32.1, N36, N40, N41, N42, N43, N44, and N44.9 all in the Administration Building area north of the Jeppesen terminal.

5. The drawings show numerous other elevations and cross sections of walls that are simply labeled “C.I.P. walls.”

### 2.2 Specifications

We reviewed project specifications dated June 1991. Section 3300 “Cast in Place Concrete” specifies concrete compressive strengths as follows:

1. Columns and Shear walls, 6,000 psi.

2. Slabs on grade and walls, 4,500 psi.

3. Normal weight topping slabs, 4,000 psi.

### 2.3 Submittals

We reviewed a submittal by Denver Ready Mix dated 18 February 1991 noted “re: D.I.A. Terminal Foundation and AGTS Structures, Concrete submittal No. 2.” This submittal documents the proposed concrete mix design for use in Contract F-121D. It specifies the following:

1. Foundations, Mix No. 105027, design compressive strength 4,500 psi, maximum slump 4-1/2 in., Air Entrainment 4%-7%.

2. Walls, slabs and beams, Mix No. 104522, design compressive strength 4,000 psi, maximum slump 4-1/2 in., Air Entrainment 4% - 7%.

3. Topping slabs with fibers, Mix No. 104322, design compressive strength 4,000 psi, maximum slump 4-1/2 in., Air Entrainment 4% - 7%.

The submittal bears an approval stamp from S.A. Miro Inc., dated 19 February 1991 indicating "No exceptions taken."

## 2.4 Reports by Others

### 2.4.1 Phase 1 Olsen Engineering Report

We reviewed a report by Olsen Engineering entitled: Phase 1 Area Coring Assessment, Denver International Airport Terminal Building, Denver, Colorado, dated 18 February 2019. The report presents the results of concrete compressive testing conducted on 99 cores extracted from walls, columns, and topping slabs located throughout the structure generally bounded by Gridlines N12 to N20 and W4 to E4. The report also presents the results of a petrographic examination conducted on 6 cores similarly removed from the structure. The report notes:

1. Olsen tested 19 cores removed from the level 5 topping slab in the Great Hall area. Cores were removed in clusters from 6 areas on the slab. The average strength of these cores was 3,344 psi and the coefficient of variation, 11%. The lowest strength of any core was 2,581 psi.

2. Olsen tested a total of 20 cores extracted from reinforced concrete shear walls at level 3, generally within the same area as the 5th level floor slab cores. The average strength of these cores was 3894 psi with a coefficient of variation of 9%.

3. Olsen tested 34 cores removed from concrete fill on metal deck at levels 5 and 6. The average strength of these cores was 5,615 psi with a coefficient of variation of 17%,

4. Olsen tested 7 cores removed from curbside slabs at levels 5 and 6. Average compressive strengths for the cores was 5,751 psi with a coefficient of variation of 19%.

5. Olsen removed cores of concrete from two cast-in-place columns and two precast concrete columns at the 4th level. The average compressive strength for the cores extracted from the cast-in-place columns were 3,683 psi and 4,493 psi, respectively. Average compressive strengths for the precast columns were 5,813 and 6,007 psi, respectively.

6. Olsen removed cores of concrete from precast double tees supporting the 5th level. One group of cores exhibited an average compressive strength of 4,830 psi. The other two groups had average strengths in excess of 6,000 psi.

### 2.4.2 DRP Report

We reviewed a report prepared by DRP Petrographic and Materials Investigations of Boulder, Colorado entitled: Investigation of Concrete Cores Extracted from the Great Hall Building at Denver International Airport, located in Denver, CO, dated 7 February 2018. This report appeared as Appendix D to the Olson Engineering report discussed above. The report notes:

1. The cause of low strength concrete in slabs and walls is a result of elevated air content and excessive water cement ratio in the concrete mix.

2. None of the cores show evidence of alkali silica reaction (ASR) or any other progressive deterioration mechanisms.

### 2.4.3 Olson Phase 2 Coring

We reviewed an excel spreadsheet purported to summarize the results of a second phase of core testing undertaken by Olson Engineering. The spreadsheet shows the test results for 23 cores in 7 groups removed from the 5th floor topping slab between Gridlines N2 and N6, W3 and E3. The average strength of these cores was 3,672 psi with a coefficient of variation of 15%. The spreadsheet also shows compressive strength test results obtained from 22 cores, in 7 groups removed from 3rd level shear walls in the same area. The average compressive strength for these cores was 4,574 psi with a coefficient of variation of 14%.

### 2.4.4 Olson Phase 3 Coring

We reviewed an excel spreadsheet purported to summarize the results of a third phase of core testing undertaken by Olson Engineering. The spreadsheet shows the test results for 22 cores in 7 groups removed from the 5th floor topping slab between Gridlines N24 and N28, W3 and E3. These cores had an average strength of 3,236 psi and a coefficient of variation of 10%. The spreadsheet also shows compressive strength test results obtained from 22 cores, in 7 groups removed from 3rd level shear walls in the same area. The average compressive strength of these cores was 4,072 psi with a coefficient of variation of 8%.

### 2.4.5 Martin/Martin Letter Reports

We reviewed a series of letter reports from Martin/Martin Consulting Engineers to Luis Camacho Camara of Great Hall Builders dated 11 March 2019; 20 March 2019; and, 20 March 2019; addressing respectively the Olsen Engineering testing conducted for Phases 1, 2 and 3, and its impacts on the ability of the Developer to utilize planned construction equipment on the 5th level floor slab. Martin/Martin is the structural engineer of record for the P3. All three reports conclude as follows:

> "*We have determined that all equipment planned to be used on the existing structure, and previously evaluated, can still be used as intended*."

## 3. LABORATORY TESTING

We directed the extraction and testing of a series of cores from the 5th floor topping slab, 3rd level shear walls, 1st level shear walls and foundations for the main terminal complex. Vivid Engineering Group, Inc. of Denver, Colorado extracted the cores and performed compressive testing of the cores in accordance with ASTM C42 and ASTM C39. Vivid extracted 33 cores, in 10 groups from the 5th level topping slab in the area bounded by Lines N1 and N30, W7 and E7; 36 cores in 12 groups from the third level shear walls in the area bounded by N1 and N30 and from walls along Lines W1.8 and E1.8; 18 cores in 6 groups from 1st level shear walls located immediately west of Line W8 and east of Line E8; and 24 cores in 8 groups from foundations between Lines W8 and W9; E8 and E9. Eight additional cores from the foundations were sent to National Petrographic Services Inc. of College Station, Texas for petrographic examination in accordance with ASTM C856.

### 3.1 Strength Testing

The average compressive strength of all cores we removed from the 5th floor topping slab is 3,870 psi with a coefficient of variation of 17%. The average strength of the cores removed from the 3rd level shear walls was 4,512 psi with a coefficient of variation of 8.5%. No core tested had a strength less than the specified strength. The strength of all cores extracted from the 1st level shear walls was 6,262 psi with a coefficient of variation of 11%. No core had a strength less than the specified value. The average strength of all cores extracted from the foundations was 7,995 psi with a coefficient of variation of 10%. No core had a strength less than the specified value.

### 3.2 Petrographic Examination

As described by ASTM C876, petrographic analysis is a process in which a petrographer (typically a trained geologist) microscopically examines samples of concrete to identify characteristic, properties, features, and components. The examination can provide information regarding the materials present in aggregate, any ongoing deterioration mechanisms, mechanical features such as voids and micro-cracks, and properties of the as-mixed concrete such as cementitious material type and the likely ratio of water to cementitious materials.

In their 3 May 2018 report National Petrographic Services (NPS) states that all 5 cores from foundations they tested indicated a mixture of volcanic and granitic rock with, strained quartz and feldspar minerals in the granitic components in both the fine and coarse aggregate. Some aggregates display rounded edges, consistent with river rock. The strained quartz found both in fine and coarse aggregates contains potentially-reactive siliceous components. NPS also reported the presence of fly ash particles.

NPS observed trace indications of alkali-silica reaction, including the presence of gel within micro cracks and cracking of aggregate in cores obtained from two foundations at the south end of the terminal, between Lines W8 and W9; from one footing at the north end of the terminal, between Lines W8 and W9; from one footing at the south end of the terminal between Lines E8 and E9; and one footing at the north end of the terminal between Lines E8 and E9.

In a 9 May 2019 report, NPS indicates the results of their testing of 3 additional cores extracted from foundations at the north end of the terminal between Lines W8 and W9, the north end of the terminal between E8 and E9; and the south end of the terminal between Lines E8 and E9. NPS reported that the aggregate in these cores is like that reported in the 3 May 2018 report and that all three cores also contain fly ash. All three of the cores exhibited some minor signs of potential ASR, primarily consisting of cracking of aggregate. National Petrographic did not observe any ASR gel product in any of the sections examined from these cores.

NPS concluded that it is unlikely that the concrete represented by any of the cores experienced any measurable distress due to ASR.

## 4. DISCUSSION

### 4.1 Concrete Strength

Testing conducted by Olson and Vivid confirms that the strength of cores extracted from shear walls and foundations exceeds the specified design values. Initially, the strength of cores extracted from shear walls were thought to be low because both airport staff and the Developer mistakenly believed the required strength of these walls is 6,000 psi. Careful review of the original construction documents indicates this is not the case. However, testing conducted by both Olsen and Vivid confirm that the strength of cores extracted from the 5th floor topping slab is lower than the 4,000 psi specified value and that the strength has high variability. Table 1 below summarizes these results.

**Table 1. Summary of Concrete Strength Tests**

| Location | Specified Strength (psi) | Olsen Engineering | | | Vivid Engineering | | |
|---|---|---|---|---|---|---|---|
| | | No. Tests | Average Strength (psi) | Coefficient of Variation | No. Tests | Average Strength (psi) | Coefficient of Variation |
| 5th Floor Topping | 4,000 | 64 | 3,424 | 12% | 36 | 3,869 | 17% |
| 3rd Level Walls | 4,000 | 64 | 4,188 | 10% | 33 | 4,512 | 8.5% |
| 1st Level Walls | 4,500 | - | - | - | 24 | 6,262 | 11% |
| Foundations | 4,500 | - | - | - | 24 | 7,955 | 10% |

It is not unusual for cores extracted from a structural element to exhibit lower strength than the specified design value or those determined during construction using laboratory-cured quality-control specimens. In fact, ACI 318, the industry standard adopted by U.S. building codes for design and evaluation of reinforced concrete structures, anticipates that strengths determined from tests of cores extracted from elements will be less than the specified value and incorporates this expectation into its acceptance criteria. Specifically, if three cores are extracted from a concrete element and tested, ACI 318 accepts the concrete as meeting specification if the average of the tests does not fall below 85% of the specified value and no individual core is less than 75% of the specified value.

This expected difference between nominal strength and true in-situ strength is built into our design codes through the years of laboratory testing that form the basis of the codes. There are many reasons why the strength of cores extracted from a concrete element can be less than the specified value, yet the concrete can still have acceptable strength. Conventionally, testing of new concrete placed in structures is obtained by testing standard concrete cylinders, that are cast at the job site, then cured in a laboratory under ideal conditions of temperature and humidity. Concrete in actual structural elements are not placed and cured in this ideal fashion and often will have lower strength than the identical concrete cast in a laboratory test specimen. This is particularly true of slabs, where the broad top surface of the slab is often not adequately protected against moisture evaporation from the concrete through application of appropriate curing techniques.

Our testing of all the cores removed from shear walls and foundations confirms that the concrete in these elements meets or exceeds the strength requirements specified by the design documents. We conclude therefore that these elements provide the strength intended by the original designer.

Almost all the groups of concrete specimens tested both by Olson and Vivid met the ACI 318 criteria that average strengths exceed 85% of the specified value and no single core be weaker than 75% of the specified value. For the few groups and cores that did not meet this criterion, the deviation below the acceptance criteria was very small.

The DRP petrographic examinations provide some information regarding contributors to the observed core strengths, indicating that higher air contents[1] and higher water contents contributed to the lower than anticipated compressive strengths. Neither DRP nor NPS found any indications that any type of concrete deterioration contributed to the lower than anticipated strengths.

In our review of the construction documents we noted that 5th level floor slab in the Great Hall comprises precast concrete double tee members, with a 7 in. thick topping slab. The double tees are designed to support the self-weight of the structure and the topping slab is designed to act compositely with the double tees to support superimposed loads including the weight of flooring, suspended lighting, HVAC ducting, etc., and live load associated with building occupancy. The drawings indicate that the composite double-tee slab system was designed for a construction live load of 250 psf to facilitate the use of construction cranes on the floor during erection of the tensile fabric roof. Based on the building's occupancy, the building code only requires design for a live load of 100 psf. We conclude that since the floor essentially meets the ACI 318 requirements and that since the normal occupancy loads are substantially less than the original design load, the floor has adequate strength for continued safe use in its intended occupancy. Martin/Martin engineers, on behalf of the Developer evaluated the slab for adequacy to support construction loading and determined that it is adequate for this purpose as well.

## 4.2 Alkali Silica Reaction

Alkali Silica Reaction (ASR) is a chemical reaction that can occur in some concrete when some specific[2] siliceous minerals in the aggregate undergo a chemical reaction with the alkali and hydroxides normally present in the concrete under moist conditions. The chemical reaction produces an absorptive gel-like material that swells in the presence of moisture. As the gel swells, it creates expansive pressures in the aggregate and surrounding concrete, causing cracking in the cement paste, between the cement paste and aggregate and in the aggregate itself. In advanced stages, this cracking is visibly evident in the form of characteristic pattern cracking and gel staining on the surface of the concrete. Ultimately, ASR can lead to spalling of concrete. When advanced, the presence of ASR reduces the shear and tensile strength of concrete. If the concrete is adequately confined, it has little effect on compressive strength.

Petrographic testing by both DRP and NPS confirm that some aggregate in all the concrete elements tested contains potentially reactive siliceous minerals, and both observe some indications of past reaction. However, the testing also indicates that the concrete contains fly ash,

[1] A typical rule-of-thumb suggests that every 1% increase in air content causes an approximately 5% drop in compressive stress

[2] Primarily non-crystalline (amorphous) or irregularly-crystalline ("strained") materials

a cementitious material commonly used in concrete as a partial replacement for portland cement. Fly ash, in sufficient quantities, is known to inhibit ASR in concrete.

Most of the concrete elements present at the Jeppesen terminal are in a controlled indoor environment and are not subject to high moisture conditions. This interior concrete presently shows no indication of active ASR and is unlikely ever to indicate such reaction given the lack of moisture.

Concrete that is exposed to moisture may be subject to ASR. Such concrete includes exterior roadways and passenger loading and unloading zones, upper decks of parking structures, and foundations (which are surrounded by soil). In our site visits to the terminal we did not observe any of the characteristic pattern cracking of concrete in exterior concrete. However, petrographic examination of cores removed from foundations generally along Lines W8 and E8 do show trace amounts of ASR have occurred. It is important to note that the petrographer concluded the extent of ASR was not destructive and that strength testing of concrete removed from these same foundations exhibited excellent strength, far in excess of the specified values. We conclude that the limited amount of ASR that has occurred is due to the presence of moisture from the surrounding soils but has been limited by the fly ash content of the concrete and that the ASR does not pose an immediate threat to the structure.

We do note however, that the foundations we tested are located within the interior of the terminal and are embedded in soil that is not normally exposed to wetting from precipitation. Soils around the perimeter of the terminal may see greater levels of moisture content due to the direct exposure of the soils surrounding the foundations to wetting from precipitation.

Given the discovery of trace evidence of ASR in the specimens tested, we recommend that additional testing be conducted to 1) confirm that at the present time, more severe conditions of ASR have not occurred at other locations in the structure and 2) that significant progression of ASR is not occurring with time.

## 5. CONCLUSIONS AND RECOMMENDATIONS

We conclude based on testing that concrete in the 5th level topping slab, shear walls, and foundations within the interior of the structure has adequate strength to resist the design loads associated with the terminal's occupancy within the parameters specified by the building code. Based on reports by Martin/Martin Consulting Engineers engaged by the Developer, we also conclude that the concrete is adequate to support proposed construction.

We recommend a program of additional testing, however, to confirm the condition of concrete elements that may be exposed to greater moisture than those tested and verification of the strength of elements in the AGTS tunnels and concourse structures. Specifically, we recommend as follows:

1. Conduct additional strength and petrographic testing of foundations located around the perimeter of the garage structures located to the immediate east and west of the terminal.

2. Conduct additional strength and petrographic testing of concrete in the concourse structures, parking structures, AGTS tunnels and other significant structures.

3. In 5 years, we recommend a repeat of testing of the foundation elements reported above, to determine if significant increase in the trace indications of ASR has occurred. If no further indications of ASR are found, we recommend that ASR not be considered an issue for the terminal and no further testing need be conducted.

Sincerely yours,

Ronald O. Hamburger, PE
Senior Principal
CO License PE.0055332
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