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American Concrete Institute
Malaysia Chapter

MYCONCRETE

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Table of Content

<u>Contents of Bulletin</u> **<u>Page</u>**

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ARTICLE

Reprint from CI Magazine, Volume 37, No 6, Page 26-30

Modeling and Forming the Turkish American Community Center

3-D models facilitate the construction of a complex reinforced concrete structure

by Kenneth Graff and Scott Hover

Comprising a large underground parking structure, a mosque, a Turkish bath, and additional assembly spaces, the Turkish American Community Center (TACC) in Lanham, MD, posed many challenges for the Center's concrete construction team. Besides the sheer size of the project, its arched beams, large and small domes, battered walls transitioning into chorded radial walls, and crescent-shaped columns made it a clear candidate for three-dimensional (3-D) modeling. In fact, although it was not contractually required, the construction team assembled by the concrete contractor, Facchina Construction Company, started modeling the project as soon as the contract was awarded. The modeling investment paid dividends from the outset, as it:

- Facilitated mockup design and construction;
- Allowed Facchina to generate requests for information(RFIs) with professional detail and speed;
- Provided a visual tool for on-site meetings with the design professionals; and
- Provided field personnel with detailed views of areas with reinforcing bar congestion.

The utility of the 3-D model of the reinforced concrete structure was extended even further, as it was provided to the construction manager and formwork vendors (after signing a disclaimer) as a courtesy and to assist with their coordination efforts. Concrete construction started in January 2013 and was substantially completed 13 months later. Largely because the 3-D model allowed detailed coordination and resolution of potential issues, no remedial work was required.

A Massive Project

The footprint of the TACC is dominated by its underground parking structure (Fig. 1). Because this portion of the project has a relatively conventional design, the project managers for Facchina directed that initial modeling efforts would be focused on the most complex structure above the parking level—the mosque (Fig. 2). The resulting model provided a multi-month look ahead so that formwork systems and the placement sequencing could be selected. The remainder of the project was then modeled, including the Turkish bath (Fig. 3) and the community center (Fig. 4).

The model allowed the construction team to identify, communicate, and resolve potential problems early (Fig. 5), and it facilitated the selection and implementation of the five forming systems that were ultimately required to construct the project. These formwork solutions are discussed in more detail in the following sections.

Formwork Systems

Decks and Platforms

Multiple platform systems were used to construct elevated beams, slabs, and domes (Fig. 6). In the Turkish bath, elevated floors were formed using MULTIPROP shoring, formwork girders, and SKYDECK panel slab formwork supplied by PERI Formwork Systems, Inc., along with HICO beams provided by Form Service, Inc. The formwork for the dome framing and domes surrounding the mosque courtyard were supported on Harsco 20K heavy-duty steel shoring. Finally, the formwork systems for the main arch platform and main dome of the mosque were supported by plywood installed on Harsco GASS® aluminum shoring and joists.

Small domes

The intermediate and small domes
on the Turkish bath and the mosque were cast on expanded polystyrene (EPS) foam forms (Fig. 7). MEVA Formwork Systems, Inc., was contracted to supply these unique forms, which were fabricated by a third-party vendor. The blocks were shipped as half domes and were assembled on-site using straps to hold them together. Because the textured surface of EPS foam bonds well to concrete, it was necessary to tape polyethylene sheets to the casting surface prior to placing the reinforcement and concrete. Shipping straps were also taped over the EPS domes so that the forms could be pulled free of the hardened concrete.

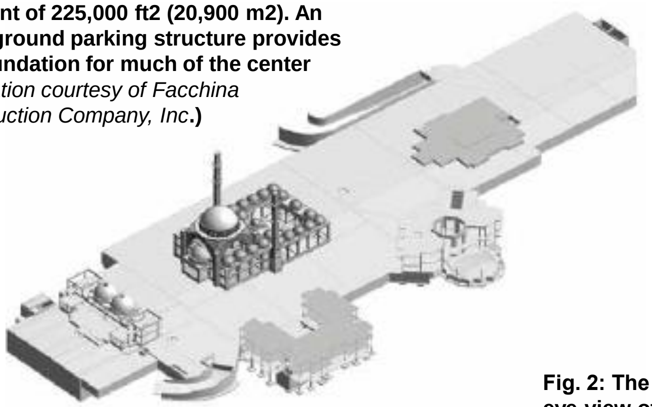
Main dome

The upper reaches of the mosque created two major forming challenges: the arched beams supporting the dome (Fig. 8) and the dome itself (Fig. 9). The arched beams were constructed using laminated lumber and shoring combined with PERI TRIO panel formwork to form the sides. The dome was formed using Radius Track crimped cold-formed steel channels and plywood sheathing. The Radius Track framing and the lower portion of the sheathing were assembled on the ground and placed on the dome frame using a tower crane. As with the smaller domes, the concrete mixture was placed with a bucket (Fig. 10). The slump ranged from 2 to 5 in. (50 to 125 mm) and the thickness was set using depth gauges.

Minarets

The TACC minarets (Fig. 11) had to transition from a square base to a polygonal tower structure. Dimension lumber

Fig. 1: The TACC facilities have a footprint of 225,000 ft2 (20,900 m2). An underground parking structure provides the foundation for much of the center (*illustration courtesy of Facchina Construction Company, Inc***.)**



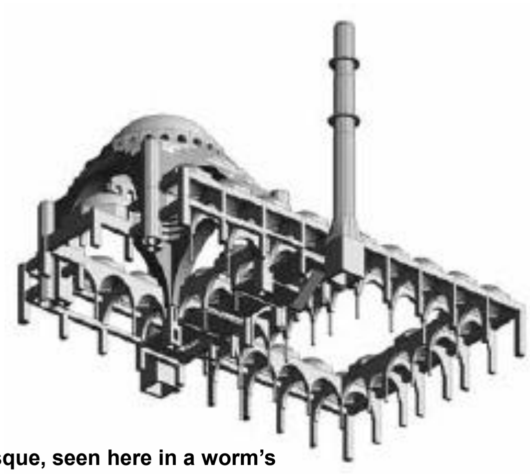


Fig. 2: The mosque, seen here in a worm's eye view of the BIM model, includes 66 arched beams, 27 small domes (radius of 6 ft 7 in. [2.00 m]), one main dome with a radius of 22 ft 1 in. (6.72 m), and two, 120 ft 6.6 m) tall minarets *(illustration courtesy of acchina Construction Company, Inc.)*

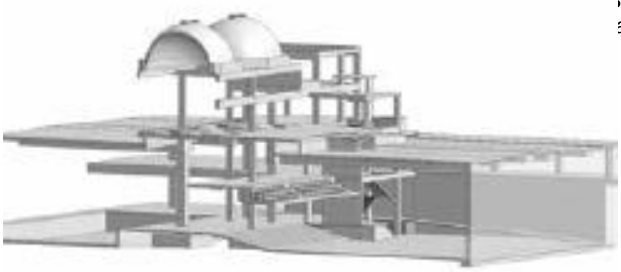


Fig. 3: The Turkish bath includes multi-story walls, a swimming pool, cast-in-place concrete seating areas, ramps, and domes (*illustration courtesy of Facchina Construction Company, Inc.)*

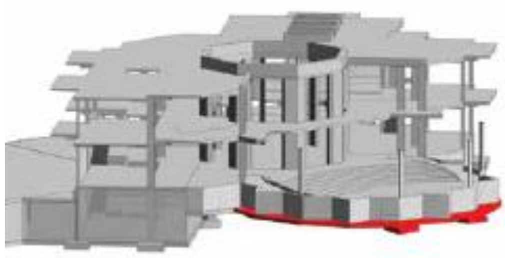


Fig. 4: The cultural center includes radial stepped slab-on-ground, C-shaped columns, and skylights *(illustration courtesy of Facchina Construction Company, Inc.)*

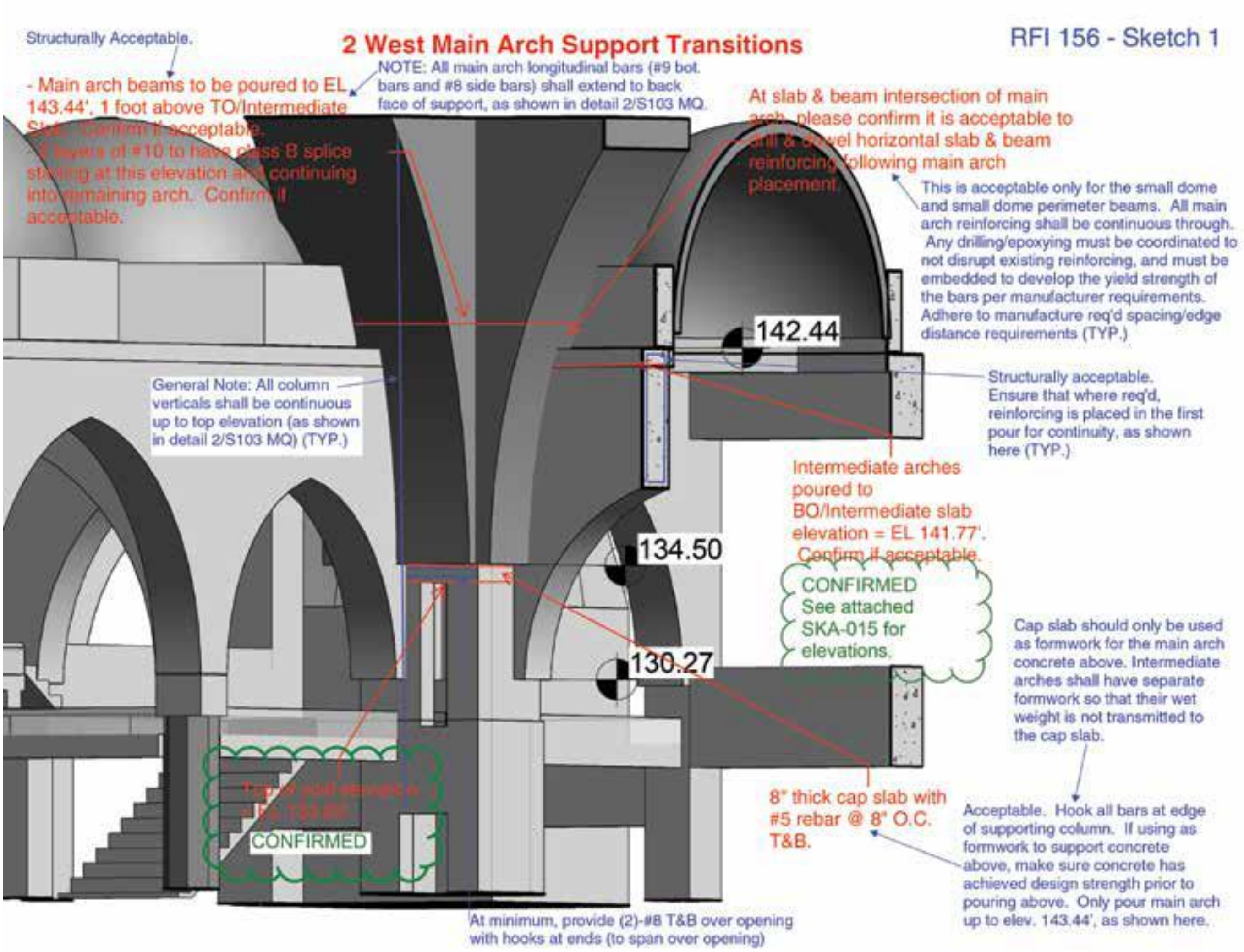


Fig. 5: The BIM model not only provided means for identifying potential constructibility issues but it also served as the communication tool for the resulting RFIs *(illustration courtesy of Facchina Construction Company, Inc.)*

Fig. 6: Multiple shoring and decking solutions were used for construction of the project: (a) elevated floors in the Turkish bath were formed using PERI MULTIPROP shoring, formwork girders, SKYDECK panel slab formwork supplied by PERI Formwork Systems, Inc., along with HICO beams provided by Form Service , Inc. *(photo courtesy of Mostafa Fahimi, EyeConstruction, Inc.)*; **(b) formwork for the small domes surrounding the mosque courtyard were supported on Harsco 20K steel shoring** *(photo courtesy of Facchina Construction Company, Inc.)*; **and (c) support for the main dome arch formwork and the work platform below the main dome formwork were provided by Harsco GASS aluminum shoring and joists** *(photo courtesy of Mostafa Fahimi, EyeConstruction, Inc.)*

and plywood sheathing were used to produce the custom formwork for the transition. Each polygonal tower was formed using a hybrid system compris-ing plywood and dimension lumber backed by MevaLite wall formwork.

Miscellaneous challenges

The project also required custom formwork fabricated on-site by carpen-ters. Stepped seating areas in the Turkish bath, for example, were formed using dimension lumber and plywood sheathing (Fig. 12).

(a) (b)

Fig. 7: The multiple small domes on the mosque and Turkish bath were formed using EPS foam blocks cut to the required radius: (a) after the dome forms were set on a shoring and decking platform, plastic sheets and nylon straps were taped over the casting surface (to ease stripping), and a pre-assembled reinforcing cage was placed; and (b) a 2 to 5 in. (50 to 125 mm) slump concrete mixture was applied using a template and depth gauge to set the thickness *(photos courtesy of Facchina Construction Company, Inc.)*

Fig. 8: The mosque structure included a platform for the main dome supported on four arched beams. The arched beams were formed using a combination of shored structural laminated timber to form the arch and PERI TRIO panel formwork to form the sides *(photos courtesy of Facchina Construction Company, Inc.)*

Constructibility

The scope and complexity of the Turkish American Community Center created many unique challenges for the concrete construction team. The large footprint and complex structures made it essential that problems were identified and resolved well before the formwork was erected and the concrete was cast. The early creation of a 3-D model was key to minimizing project risk and ensuring success. Finally, because very few details were repeated throughout the multiple buildings, numerous formwork systems had to be employed to complete the project on schedule. Early visualiza-tion allowed the planners to consider options and ensure that the right systems were in place at the right time.

Selected for reader interest by the editors.

(a) (b) (c)

Fig. 9: The primary dome on the mosque was too large to be formed using EPS foam, so custom formwork was fabricated from crimped cold-formed steel shapes and plywood sheathing: (a) the Radius Track cold-formed shapes were assembled at the ground level; (b) the partially completed assembly was installed using a tower crane; and (c) the remainder of the dome formwork sheathing was fastened to the framing and the window formwork was installed *(photos courtesy of Facchina Construction Company, Inc.)*

Fig. 10: Workers place concrete on the main dome of the mosque. The final depth and shape of the dome were achieved using a depth gauge and a curved screed *(photo courtesy of Mostafa Fahimi, EyeConstruction, Inc.)*

Fig. 12: Seating areas in the Turkish bath were formed using dimension lumber and plywood sheathing *(photo courtesy of Facchina Construction Company, Inc.)*

(a)

(b)

Fig. 11: The minarets comprise rectangular bases and polygonal towers: (a) the transition between base and tower was formed using dimension lumber and plywood sheathing; and (b) the tower was formed using a hybrid system comprising MevaLite wall formwork, dimension lumber, and plywood sheathing lifted into place using the site's tower crane *(photo courtesy of Mostafa Fahimi, EyeConstruction, Inc.)*

Project Credits

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Concrete Contractor – Facchina Construction Company, Inc.
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Reinforcing Bar and Post-Tensioning System Installer – R&R Reinforcing

Kenneth Graff is a Project Manager with Facchina Construction Company, Inc. His responsibilities have included coordination of subcontractor and vendor work, analysis of contract drawings and specifications, reporting quantities and tracking costs, scheduling, jobsite documentation and submittals, and change order pricing and negotiations as well as troubleshooting bonded and unbonded post-tensioning systems. He received his BS in business administration and management information systems from High Point University, High Point, NC, in 2004.

Scott Hover is a BIM Manager with Facchina Construction Company, Inc. He developed and managed the building information model for the TACC, among other projects. A LEED Green Associate, he received his BS in graphic design from Towson University, Towson, MD, in 2001.

TECHNICAL REPORT

Reprint from CI Magazine, Volume 37, No 3, Page 59-62

Controlling Early-Age Cracking in Mass Concrete

Thermal control plans may include near-surface reinforcement to supplement insulation and other measures

by Ronald L. Kozikowski and Bruce A. Suprenant

A worker places concrete in a mat foundation with a heavy grid of reinforcing steel that could have been included in a thermal control plan to limit early-age thermal cracking. In this case, however, the limiting temperature differential was selected on the basis of no cracking. This made it necessary to keep insulating blankets in place for 3 weeks, thereby increasing costs and the construction schedule (photo courtesy of Baker Concrete)

Mass concrete placements are conducted using thermal control plans that provide limits on maximum concrete temperature—to prevent concrete damage due to delayed ettringite formation (DEF)—and on maximum temperature differential—to minimize the potential for early-age thermal cracking. Recommendations for temperature limits and contractor measures for thermal control plans can be readily found in the literature.1,2,3 These measures can include reducing the cementitious material content in the concrete (particu-larly portland cement), precooling the fresh concrete, providing internal cooling, or providing external insula-tion of the placed concrete. Such measures can add significantly to the construction cost and schedule.

This article discusses an additional measure that can be included in thermal control plans—using near-surface reinforcing steel to help control the width of cracks that do occur as the result of temperature differentials. While this approach will not eliminate the need to limit the maximum temperature differential, the allowable differential that can be accommodated for a given crack width will increase with the amount of reinforcement. As a result, the construction cost associated with the use of insulating blankets, including labor for placement and removal as well as project delays, may be reduced for some mass concrete placements.

For durability, the maximum temperature must be limited within any mass concrete placement. For safety and utility, cracking must be minimized in structures such as nuclear containment vessels, liquid natural gas storage tanks, and water reservoirs. Although it may be unnecessary or uneco-nomical to take extensive measures to avoid cracking in many other structures, any cracking that does occur can be a source of dispute. That’s why it’s important that owners understand that early-age thermal cracking can be consistent with good construction practices, provided that it has been properly considered as part of a thermal plan in the design and construction process.

Precedents for Accommodating Cracksin U.S. Practice

In regard to the design of mat foundations, the ACI 318 Building Code and Commentary encourages designers to consider reinforcement to control thermal cracking. The current Code, ACI 318-14,4 has clarified some confusion over the minimum reinforcement requirements that existed in previous versions of ACI 318. While the amount of reinforce-ment required by the current Code is the same as in previous versions, the Code now includes different placement strate-gies. The change in placement strategy is beneficial to mass concrete design. The ACI 318-14 Commentary provides insight into the minimum reinforcement requirements by stating: "To improve crack control due to thermal gradients and to intercept potential punching shear cracks with tension reinforcement, the licensed design professional should consider specifying continuous reinforcement in each direc-tion near both faces of mat foundations."

Also in the United States, some transportation departments have mass concrete specifications that allow early-age thermal cracks. The Virginia Department of Transportation, for example, issues special provisions that supplement and modify the department's specifications for hydraulic cement concrete and operations.5 The special provisions include a requirement that: "The maximum allowable thermal gradient between the core and skin temperatures of a pour is limited to 35F, unless the analysis submitted under section C.3.b.(d) demonstrates that the element is sufficiently reinforced to prevent crack widths in excess of those listed on the plans or in Table 4.1 of ACI 224." The referenced analysis is required to include:

- A table of calculated peak temperatures for the range of expected air and concrete temperatures at the time of placement;
- A calculation of maximum temperature gradients within the element during the curing period;
- A calculation of time to peak temperature; and
- A calculated curve of maximum allowable temperature differential as a function of concrete strength for each element under consideration.

Figure 1 shows Table 4.1 from ACI 224R-01(08),6 which provides reasonable crack widths versus exposure conditions based on a historical perspective, and Fig. 2 illustrates examples of calculated curves of maximum allowable temperature differential.7

In addition to ACI 224R-01(08),6 the noted calculations are to be based on recommendations outlined in ACI 207.1R-958 and ACI 207.2R-95.9

The most recent specification issued by the Florida Department of Transportation provides various dispositions for cracked concrete, depending on the crack width, the cracking significance (based on the total cracked surface area relative to the total surface area), and the aggressiveness of the ambient environment.10 In mild environments, the specifica-tion indicates that cracks as wide as 0.008 to 0.012 in. (0.20 to 0.30 mm) may require no repair.

Table 4.1—Guide to reasonable* crack widths, reinforced concrete under service loads

Exposure condition	Crack width	
	in.	mm
Dry air or protective membrane	0.016	0.41
Humidity, moist air, soil	0.012	0.30
Deicing chemicals	0.007	0.18
Seawater and seawater spray, wetting and drying	0.006	0.15
Water-retaining structures†	0.004	0.10

*It should be expected that a portion of the cracks in the structure will exceed these values. With time, a significant portion can exceed these values. These are general guidelines for design to be used in conjunction with sound engineering judgment.

†Excluding nonpressure pipes.

Fig. 1: Table 4.1 in ACI 224R-01[6] provides guidelines for reasonable crack widths to be expected in reinforced concrete structures

In European Practice

In Europe, engineers have long advocated using reinforc-ing bars to control early-age thermal cracking.11,12 "Early-age thermal crack control in concrete (CIRIA C660)"13 provides a simplified design procedure to assist engineers in checking the amount of steel required to control early-age thermal cracking. The indicated document also provides guidance on selecting an appropriate crack width and an equation for calculation of crack widths. The calculation approach has been made more manageable with the inclusion of a compact disc with Excel spreadsheets.

The minimum amount of steel As,min is determined using the assumption that the steel force after cracking will be equal to the tensile force in the concrete just prior to cracking. The area of concrete in tension is estimated as 20% of the thickness of the member (Fig. 3). The steel is assumed to be at yield stress fky, and the concrete tensile zone is assumed to have area Act and a triangular stress distribution with a maximum stress of fct, eff :

$$A_{s,min} = 0.5A_{ct}\frac{f_{ct,\ eff}}{f_{ky}}$$

CIRIA C660 provides guidance for setting fct,eff based on the compressive strength class and age at which cracking is a concern (3 days for early-age cracking). Finally, in addition to providing a method for estimating crack widths, the document lists considerations for selecting the target crack width based on three categories:

- Cracking that leads to durability problems and consequent-ly a reduction in structural capacity (limit crack width to 0.012 in. [0.3 mm]);
- Cracking that leads to a loss of serviceability of the structure such as water or radiation leaks or damages to finishes (limit crack width to 0.002 to 0.008 in. [0.05 to 0.20 mm]); and
- Cracking that is aesthetically unacceptable (crack width should be limited to 0.012 in. [0.3 mm] but could be wider based on requirements for appearance).

It notes that achieving small crack widths requires considerably more reinforcement than As,min—4 to 5 times that amount will be needed to keep the stress in the steel in the elastic range and limit crack widths to about 0.012 in. (0.3 mm). In effect, this measure will reduce the stress induced by thermal differential to 20 to 25% of the yield strength.

In Japanese Practice

For over 25 years, the Japan Concrete Institute (JCI) has provided guidelines for control of cracking of mass concrete.14 In addition to recommendations for contractor measures for thermal control plans, the guidelines provide estimates for the probability of cracking and crack width. These estimates are largely based on a thermal cracking index Icr developed from a study of cracking (or noncracking) of 728 members from 65 structures. Icr is the ratio of the splitting tensile strength of the concrete and the tensile stress in the member. While tensile stresses are generally determined using a three- dimensional finite element analysis, the guidelines also provide simple equations for Icr for wall-type, layer-type, and column-type structures.

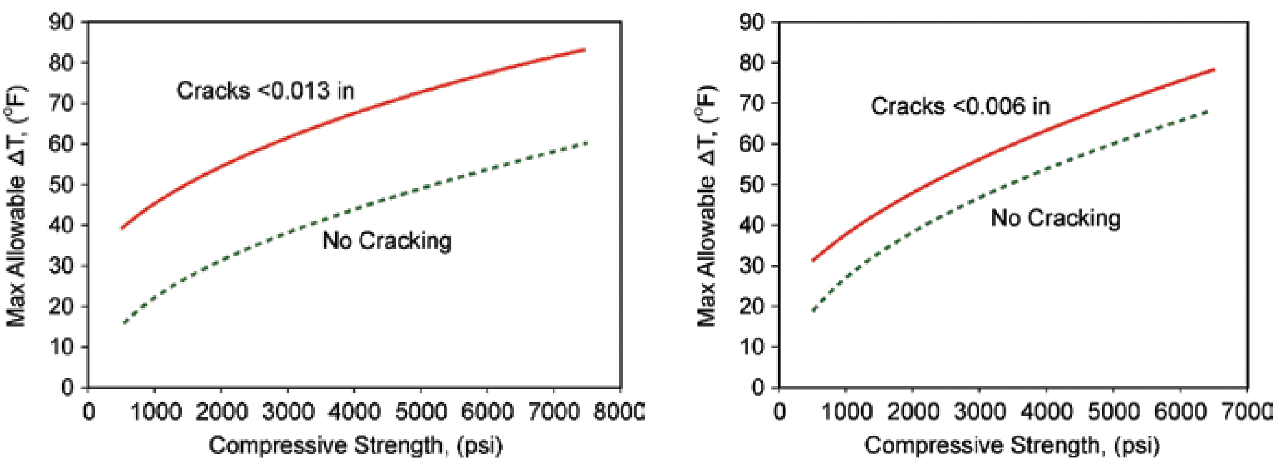

Fig. 2: Calculated maximum allowable temperature differential as a function of concrete strength for: (a) an 8 ft (2.4 m) square beam with 822 lb/yd3 (488 kg/m3) total cementitious material content (70% slag cement) and f´c of 6500 psi (45 MPa); and (b) a 6 ft (1.8 m) square pier cap with 585 lb/yd3 (347 kg/m3) total cementitious material content (40% Class F fly ash) and f´c of 3000 psi (20 MPa). The curves were submitted as part of a contractor's thermal control plan5, 6 and they allow determination of the maximum allowable temperature differential based on no cracking or on the indicated crack width limits (based on Reference 7)

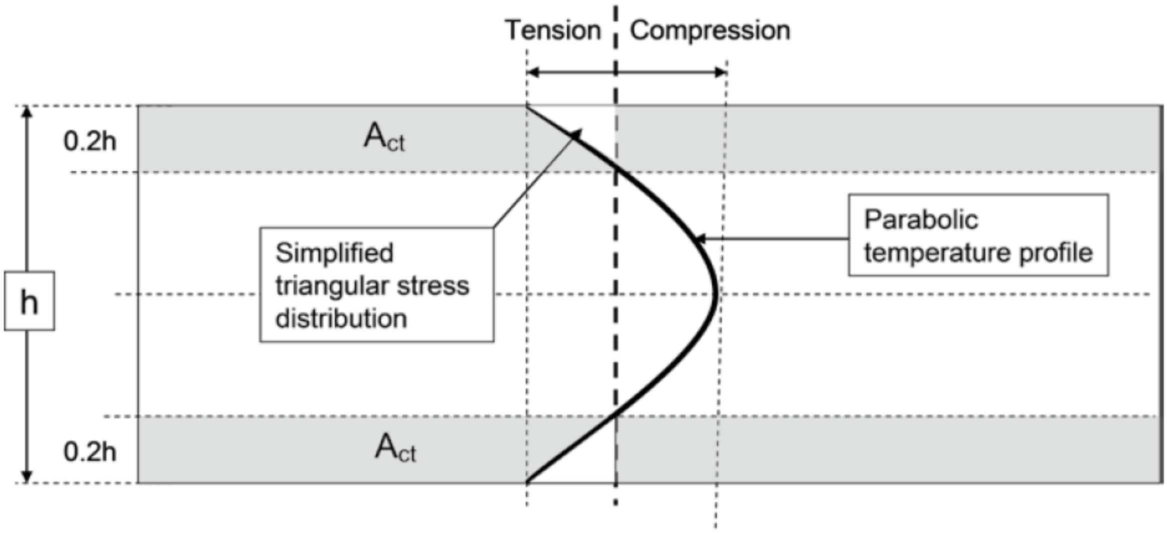

Fig. 3: Idealized structural model showing the areas A_{ct} subjected to tension as the result of internal restraint as the concrete responds to the temperature profile (based on Reference 13)

Not a Defect

Cracking in reinforced concrete is not a defect. The very basis of reinforced concrete design is that concrete has no significant tensile strength and that sufficient reinforcement is provided to control crack widths. The designer should make the owner aware that cracking is an inherent part of reinforced concrete and that, if controlled, will not be detrimental to the performance of the structure. Selecting the appropriate target crack width is an important step. And it should be remembered that crack widths in a structure are highly variable.

Both CIRIA C660 and the JCI guidelines provide equations to calculate crack widths. Designers in the United States are more familiar with, and are likely to use, the crack width prediction equations in Chapter 4 of ACI 224R-01. However, it should be pointed out that these equations apply to cracks caused by external effects (bending and tension) rather than intrinsic effects (thermal differential and shrinkage).

Concluding Remarks

Designers must understand the limitations of controlling early-age thermal cracking with reinforcement. Each project is unique (the service environment of the structure is an impor-tant consideration in choosing the most appropriate approach), and the strategy for minimizing thermal cracking (add reinforcement, rely on measures to limit temperature differen-tials, or a combination of both) must be selected by the design and construction team based on a cost analysis and in consul-tation with the owner.

In conducting the assessments, the team must be cognizant of multiple factors, including:

- Internal restraintâ€”temperature differentials cause internal restraint. The resulting cracking is different than cracking caused by external restraint such as a thick wall restrained by its foundation. Provided that the yield stress of the steel is not exceeded, cracks that may have been caused/widened due to internal restraint will contract along with the concrete surface as the temperature differentialsthat created the crack diminish. The widths of cracks caused by external restraint do not decrease, as theyare restrained by adjacent element(s). (The methods summarized in this article are focused on cracking due to internal restraint.);
- Strains and crack widths for thermal effects and external effects are not necessarily additiveâ€”CIRIA C660 reports that it has not been common practice to add early-age strains or crack widths to strains or crack widths due to structural loading. This is another important distinction between members with internal versus external restraint. As long as the reinforcement stress is below its yield stress, it remains elastic. Thus, as the concrete cools, the steel stresses decrease and the cracks get smaller. This does not happen for members with external restraint;

- Reinforcement will not help in avoiding DEF—maximum temperatures must be controlled regardless of how thermal differentials are addressed;
- Reinforcement will not eliminate the need to consider the effects of maximum temperature differential on cracking. When properly designed, reinforcement will allow a higher differential to be accommodated for a given crack width; and
- To control crack widths, reinforcement must be distrib-uted—while the reinforcement provided to resist external loads may be sufficient to control thermally induced cracks, additional steel may be needed (for example, on the vertical sides of deep members).

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CASE STUDY

Reprint from CI Magazine, Volume 37, No 6, Page 31-35

Construction of Undulating Walls Using Dry-Mix Shotcrete

Expansive concrete surface creates the main spatial element inside the Museum of the History of Polish Jews in Warsaw, Poland

by Józef Jasiczak, Włodzimierz Majchrzak, and Włodzimierz Czajka

The impact of the rich, 1000-year history of Polish Jews on today's Poland was the basis of the decision to create the Museum of the History of Polish Jews (MHPJ) in Warsaw, Poland. In January 2005, the City of Warsaw, the Polish Ministry of Culture and National Heritage, and the Association of the Jewish Historical Institute of Poland signed an agreement establishing a joint cultural institution. An international competition for the design of the MHPJ was soon initiated. Hundreds of entries were submitted, including proposals by prestigious designers such as David Chipperfield, Peter Eisenman, Zwi Hecker, Kengo Kuma, and Daniel Liebeskind. On June 30, 2005, the Association of the Jewish Historical Institute of Poland announced that Finnish architects Rainer Mahlamäki and Ilmar Lahdelma were the winners of the competition.

In June 2009, a contract was signed between the main contractor, Polimex-Mostostal SA, and the Polish Ministry of Culture and the City of Warsaw. Eventually, SPB TORKRET Ltd., Poznań, Poland, was selected as a subcontractor, responsible for completion of a three-dimensional (3-D), curvilinear wall designed as the main spatial element of the entryway of the museum. TORKRET completed this unique shotcrete project—26 m (85 ft) high walls with almost 6000 m2 (65,000 ft2) of surface area—in 2011 and 2012.1

Building Construction

The museum building consists of two parts with an expansion joint between them. The first part (Fig. 1) is the main building with plan dimensions of 67.3 x 67.3 m (220 x 220 ft) and a total height of 26 m (85 ft). The main building has four aboveground floors and one underground level, and it was designed as a reinforced concrete structure with monolithic external walls. The façade was designed to comprise slanted narrow vertical glass panes, fixed to the reinforced concrete structure with a light steel structure made of galvanized steel square tubes. The second part of the building, visible in Fig. 2, is the services compound, which consists of a single underground floor with plan dimensions of 67.3 x 41.7 m (220 x 136 ft).

Fig. 1: The Museum of the History of Polish Jews in Warsaw, Poland. The museum stands in what was once the heart of Jewish Warsaw—an area which the Nazis turned into the Warsaw Ghetto during World War II. The building's simple form is split by a wide fracture directly opposite the nearby Monument to the Ghetto Heroes

Fig. 2: The connecting tunnel from the second part of the building, the services compound, which consists of a single underground floor

Undulating Walls

The Initial Design Concept

In the main hall of the building, two undulating walls are the most important elements shaping the interior aesthetics as well as serving as the structural support of the entry hall ceiling (Fig. 3). Both walls start from the ground floor and cover the entire height of the building from the foundation to the roof. The walls were originally designed as precast glass fiber-reinforced concrete (GFRC) shells attached to H-section rolled steel profiles using a system of adjustable fasteners. The H-section profiles were to be connected to a substructure comprising steel tubes rolled from EN 10210 Grade S355J2H steel (similar to ASTM A500 Grade C or CSA G40.21 Grade 50W).

Vertical elements in the substructure were 273 mm (10.75 in.) diameter tubes with wall thicknesses varying from 16 to 20 mm (0.62 to 0.79 in.). These vertical tubes were bent in one plane and braced with horizontal, 193.7 mm (7.63 in.) diameter tubes with 12 mm (0.47 in.) wall thickness. The substructure at the ceiling comprised 244.5 mm (9.62 in.) diameter steel tubes with 16 mm (0.62 in.) wall thickness connected to the concrete roof using rigid inserts.

The GFRC panels, which were to form the finishing layer of the curvilinear walls, were designed to be 15 mm (0.6 in.) thick. The panels were to be diamond-shaped with areas of several square meters, with four fasteners located at the corners for mounting onto the substructure. The panels had to have double curvature to comply with a digital model developed by the architect (Fig. 3), and they had to meet the following conditions:

- Color ”Stained concrete with a shade similar to pale yellow limestone, approved by the Architect on the basis of samples, and resistant to UV staining. The color was inspired by the color of the Western Wall in Jerusalem. Panels impregnated with anti-graffiti protection;
- Geometry ”15 mm (0.6 in.) thick panels with edges thickened to 40 mm (1.6 in.) to strengthen the joints and deepen the gap between panels; and
- Functional condition and in-use performance ”The contractor was obligated to develop the details of the fastening system with the provision that the system must provide load-bearing capacity and stability, meet the requirements of fire regulations, and enable the installation of the panels on both sides of the hall.

Due to a number of difficulties, an alternative finish system shotcrete applied directly at the jobsite—was investigated.

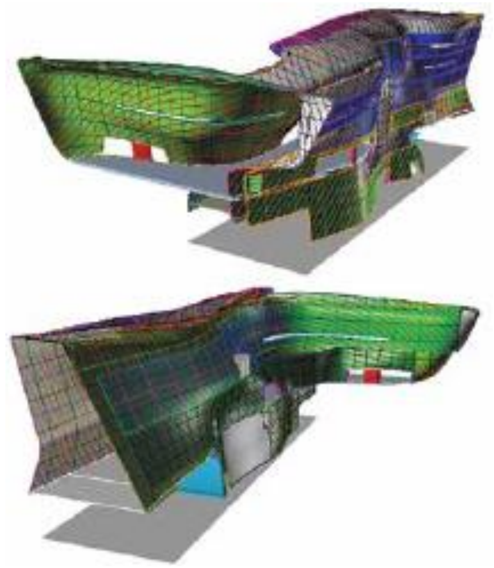

Fig. 3: A 3-D model was generated to set the geometry of the thin-wall shell of the entry hall of the museum, including the locations for the shell’s expansion and control joints

Fig. 4: Mockup of curvilinear wall divided into diamond-shaped elements. One of the designers, Rainer Mahlamäki, is on the far right

Proving the Alternative System

Before shotcrete could be used, structural research and calculations were made, taking into account the need to fulfil the color, geometric, functional, and in-use performance requirements defined in the original design concept. Knowing the possibilities of curvilinear surfaces formed using shotcrete technology, TORKRET prepared three mockups of the wall. In September 2010, TORKRET hosted a meeting with the architect and representatives of the investor and main contractor. There, the wall construction method and the mockups were presented (Fig. 4). After the visit, positive feedback was received; however, static and fire resistance tests of the models were required before final approval could be granted.

Samples of the wall elements underwent destructive structural testing as well as tests of fire resistance. It must be emphasized that the wall is not merely a decorative element and a work of art, but also serves as a partition between walking routes for visitors, as well as technical and office premises.

Laboratory studies were carried out at Poznań University of Technology (Fig. 5) on two 2.10 x 0.80 x 0.05 m (83 x 32 x 2 in.) reinforced shotcrete elements. The tests demonstrated that the shotcrete panel system and its mounting on the steel support construction was the right solution. It was determined that the full-scale structure worked as a continuous concrete shell with multiple anchors spaced at 0.8 m (32 in.) in each direction.

The 50 mm (2 in.) thick panels were reinforced with 100 x 100 mm (4 x 4 in.) welded wire reinforcement with 4.5 mm (0.18 in.) diameter deformed bars. The reinforcement was centered at the midplane of the wall section and was designed to secure the structure against complete destruction in the event of exceptional loads (for example, accidental impact or anchor failure).

This role of the welded wire reinforcement was confirmed by laboratory tests. Point loads were applied at anchor points in the test panels suspended between steel beams. The panels deflected 41 and 35 mm (1.61 and 1.37 in.) under applied forces of 4.5 and 5 kN (1010 to 1120 lb), respectively, before the concrete sections were fully cracked. Fire resistance tests were conducted at the Fire Testing Laboratory of the Building Research Institute, Warsaw. These showed that the concrete shell would achieve the required fire resistance.

Shotcrete Implementation

After completion of the basic load-bearing structure of the building (Fig. 6), the implementation of the feature walls began. The dry-mix shotcrete was prepared at TORKRET's mixing plant with a production unit exclusively dedicated to he construction of the curvilinear walls. The first layer of concrete was applied using a traditional shotcrete mixture containing 2 to 4 mm (0.08 to 0.2 in.) rounded quartz aggregates, portland cement (CEM I 42.5R), silica fume, and a non-alkaline accelerating admixture. The second layer was made of quartz aggregates up to 2 mm (0.08 in.) in size, but the binding material was white cement (CEM I 42.5R) with adequately matched dyes including oxide iron yellow and titanium white. Maintaining a uniform color to match that of the Western Wall was one of the biggest challenges. Shotcrete samples (Fig. 7) were used as references.

Fig. 5: Structural testing of a 2.10 x 0.80 x 0.05 m (83 x 32 x 2 in.) panel cut out from a mockup wall. Anchors were spaced at 0.8 m (32 in.) and load was distributed by the steel channel in the center

Fig. 6: The concrete and steel structure of the museum's entry hall defined the basic form of the undulating interior walls

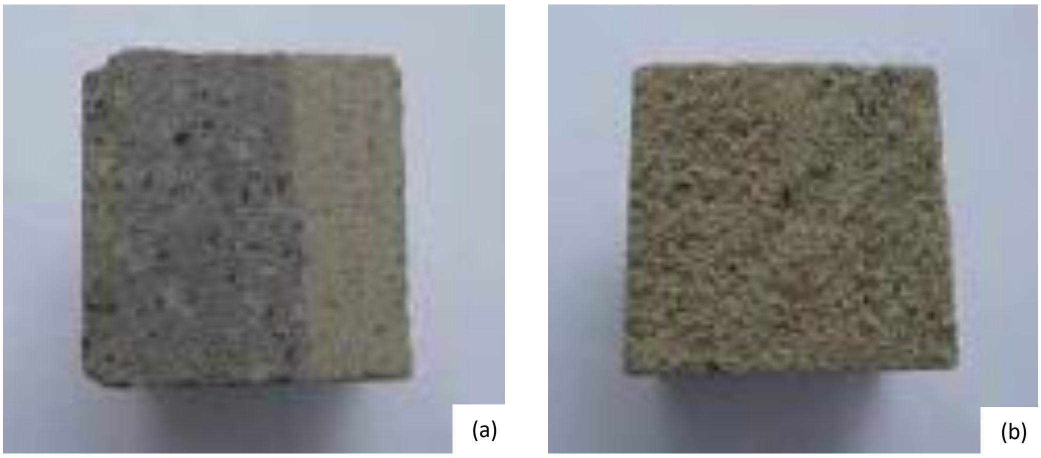

Fig. 7: Shotcrete samples—two-layer concrete with an external pale yellow architectural coat made according to the "cut" technique: (a) cross section; and (b) finished surface

The shotcrete panels are attached to the vertical tubes in the substructure through a system of rigid anchors. Each anchorage point comprises a steel plate with a central hole for a fastener. The concentrated load applied to the anchorage is distributed into the shotcrete section via 4.5 mm (0.18 in.) diameter bars welded in a radial pattern on the plate (Fig. 8(a)). The anchorage forces were therefore distributed to the welded wire reinforcement in the wall. During placement, the welded wire reinforcement was anchored to a profiled substrate made of flexible, water-resistant, and fire-resistant plywood (Fig. 8(b)). This substrate also served as a stay-in-place form for the shotcrete.

Specially designed polymer strips were embedded in the shotcrete at expansion and control joints (Fig. 9). The strips enabled maintaining a uniform thickness of shotcrete and delineated the outer surface. They also enabled installation of plastic sheets to prevent moisture loss as well as provide protection against shotcrete overspray from subsequent placements. The expansion joint strips were later removed and replaced by a fireproof silicone material. The control joint strips were inserted in the shotcrete after placement, in a pattern specified by the architect, and they were left in place in the completed walls.

The most important issue from the wall profile shaping perspective was transferring of the 3-D design coordinates to the wall space. This was achieved by marking the intersections of joints or other typical points. The polymer strips were mounted on stay-in-place plywood formwork panels. Once the joint-defining strips were formed and fastened, two layers of shotcrete were applied. Aliva 246 dry-mix shotcreting machines were used, along with booster pumps that provided water to the nozzle.

The fresh shotcrete was cured using plastic sheets hanging from the top of the finished walls (Fig. 10). Completion of the curvilinear wall took 13 months of substructure preparation and several months of finishing work. The walls were completed in August 2012. The final results are shown in Fig. 11.

Fig. 8: Anchorage/stress distribution elements: (a) individual anchorage plate with radially welded 4.5 mm (0.18 in.) diameter bars; and (b) view of the anchorage plates and welded wire reinforcement anchored to the profiled substrate made of water-resistant plywood

Fig. 9: After plywood panels were installed as stay-in-place formwork defining the wall curvature, anchors and welded wire reinforcement were installed, polymer strips were set to form expan-sion joints and to serve as depth guides, and the first layer of shotcrete was applied

Fig. 10: Shotcreting process. In the lower section of the photo, the finished shotcrete wall is covered with plastic sheets to protect it against contamination by mortar overspray and provide curing to limit drying shrinkage

Fig. 11: Finished curvilinear shotcrete wall before and after installation of the entry hall's glazing system. The large hall divides the building, and its high, undulating, and textured walls create a dramatic space

Construction-Related Issues

Major portions of the surface area of each wall were constructed in the open space of the building. The roofing, about 600 m2 (6500 ft2) of glazing, and the entrance structure were finished at the end of the project. This forced the work to be organized so that preliminary stages could be completed during periods of low temperatures. Shotcrete was then applied during advantageous weather conditions. A major execution-related problem was accessing individual wall elements. While it would have been more convenient to apply shotcrete from scaffolding, that solution was ruled out because of the ongoing need for surveys of the spatial location of the wall surfaces.

Hydraulic boom lifts and scissor lifts were thus used instead of scaffolding. To access the highest wall elements, a temporary platform was installed. A crane track with a suspended scaffold was then mounted to the temporary platform. This solution made it much easier for the client to set, control, and verify the shotcrete surface.

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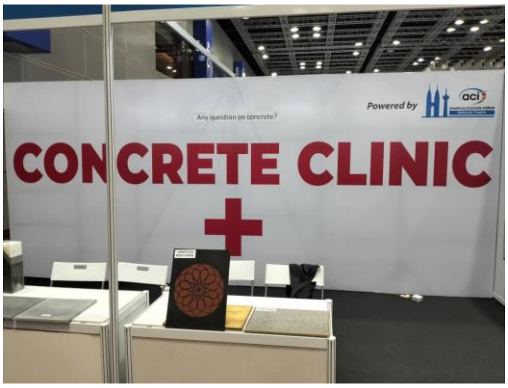

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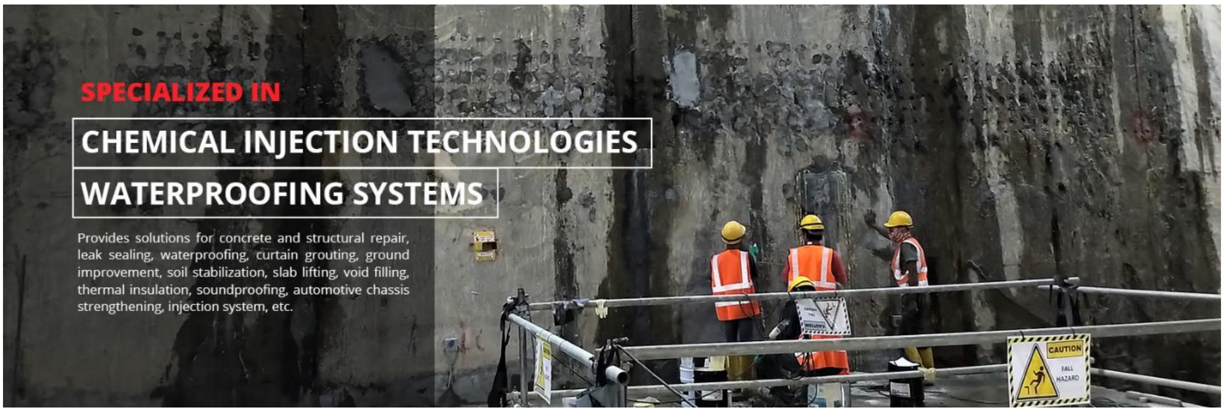

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