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MYCONCRETE

THE BULLETIN OF THE AMERICAN CONCRETE INSTITUTE - MALAYSIA CHAPTER
(E-bulletin)

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MyConcrete:
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Table of Content

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ARTICLE

Reprint from CI Magazine, Volume 37, No 1, Page 45-50

The Sky's the Limit

Evolution in construction of high-rise buildings

by Pierre-Claude Aïtcin and William Wilson

During the last 50 years, concrete technology has made great progress, largely due to the control of concrete rheology through the use of high-range water-reducing admixtures (HRWRAs) and viscosity-modifying admixtures (VMAs). Thus, concrete rheology no longer depends only on water, but rather on a judicious balance between water and dosages of HRWRA and VMA.

Reducing the water-cement ratio (w/c) or water-binder ratio (w/b) means getting cementitious particles closer to each other in the hardened cement paste so that compressive strength can increase up to over 100 MPa (14,500 psi)—in spite of the fact that such concretes do not contain enough water to fully hydrate all of the cementitious particles. Compressive strength continues to increase as the w/c or w/b decreases because concrete compressive strength is related more to the closeness of the cement particles in the cement paste rather than to the amount of hydrated cement. Feret's law for cement paste and Abrams' law for concrete continue to be valid even when not all of the cement particles are hydrated.

Before the 1970s, it was impossible to produce concretes having both a w/c lower than 0.40 and a slump of 100 mm (4 in.). The lignosulfonate-based water-reducing admixtures (WRAs) that were the only dispersing admixtures then available on the market were not capable of sufficient dispersal to provide such performance. But as soon as the very efficient dispersing properties of polymelamine sulfonates and polynaphthalene sulfonates were discovered in Germany1 and Japan,2 respectively, it became possible to produce mixtures having both a w/c lower than 0.40 and a slump of up to 200 mm (8 in.).

These two innovations resulted in a significant advantage for concrete over steel for the construction of high-rise buildings. It is no longer necessary to use cranes to transport and place concrete—with the help of VMAs, concrete can be pumped from the first to the highest floor.

Thus, concrete was pumped up to 586 m (1922 ft) using a single pump during the construction of Dubai's Burj Khalifa. In the near future, concrete pumping is expected to reach up to 1000 m (3280 ft), yet steel beams and columns will still have to be raised with cranes.

This article shows how the construction of high-rise buildings has evolved from entirely structural steel structures to almost exclusively reinforced concrete structures, by discussing the construction of some landmark structures built from 1968 to the present time.

The article comprises two parts. The first part discusses the high-rise buildings—Water Tower Place (1968) and the CN Tower (1973)—constructed with lignosulfonate WRAs to illustrate the state-of-the-art technology available before 1975 (before the arrival of HRWRAs on the market). The second part briefly reviews the construction of high-rise buildings with HRWRAs—Scotia Plaza (1983), Two Union Square (1989), the Petronas Towers (1998), Burj Khalifa (2010), the Worli Project (presently under construction), and the anticipated Kingdom Tower.

The Era of Lignosulfonate WRAs

Water Tower Place

In the 1960s, the highest compressive strength available in the Chicago, IL, market was 30 MPa (4350 psi) for concrete having a 100 mm (4 in.) slump. This was the concrete used to build columns in high-rise buildings until John Albinger found that, by carefully selecting his cement, fly ash, and WRA, he was able to double that compressive strength.3

To introduce his high-strength concrete to the market, he used the following stratagem. During the construction of a high-rise building, he asked the engineer and the architect for permission to cast a 40 MPa (5800 psi) concrete column at no extra cost. His proposal was accepted. The next day, nobody noticed any difference between this 40 MPa (5800 psi) concrete and the 30 MPa (4350 psi) concrete that was used for the construction of the other columns on the project. The architect was very pleased to see that it was possible to decrease the size of the columns, so he asked the engineer to design the next high-rise building with a 40 MPa (5800 psi) concrete. The engineer was also happy to decrease the dead load of the building. During the construction of the 40 MPa (5800 psi) high-rise building, John Albinger repeated his tactics and asked for permission to cast one 50 MPa (7250 psi) concrete column, again at no extra cost.

The same approach was employed with 60 MPa (8700 psi) concrete, which opened the door for the architect and the engineers to design the next high-rise building—Water Tower Place (Fig. 1(a))—using 60 MPa (8700 psi) concrete for the columns of the lower floors. The compressive strength was then decreased progressively down to 30 MPa (5800 psi) for the columns of the top floors. By adjusting the amount of reinforcing steel, the engineer was able to maintain the same cross-sectional area for all the columns of the building so that the same set of prefabricated steel forms could be used from the first to the last floor. Moreover, as all the floors had exactly the same geometrical pattern, interior finishing became a repetitive operation resulting in significant economies.

The CN Tower

Until the recent construction of Burj Kalifa, the CN Tower, Toronto, ON, Canada, was the highest freestanding concrete structure in the world at 482 m (1581 ft), as shown in Fig. 1(b). The tower was built using slipforms and an air-entrained concrete having an average compressive strength of 55 MPa (7980 psi).4 The concrete had to be air-entrained to be resistant to freezing and thawing. The tower was built continuously from the beginning of May to the end of November, with ambient temperatures as high as 35°C (95°F) and as low as −10°C (14°F). Because the walls were 2.1 m (6 ft 10 in.) thick at the base of the tower, cement with a low heat of hydration had to be used to limit thermal gradients. Normal portland cement was then progressively substituted for the initial low-heat-of-hydration cement as the weather was getting colder and colder while the structure was rising. Accelerated tests were also used to control concrete compressive strength during construction.

Fig. 1: High-rise buildings constructed before 1975: (a) Water Tower Place (photo credit: © Jeremy Atherton); and (b) the CN Tower (photo courtesy of Benson Kua)

The HRWRAs Era

Scotia Plaza

Scotia Plaza, Toronto, ON, Canada, is an attractive office building 68 stories (275 m [900 ft]) high, clad with red granite. Its concrete structure was built using climbing forms (Fig. 2(a)) and concrete without entrained air having a maximum design compressive strength of 70 MPa (10,150 psi)—the maximum strength permitted by the Canadian Building Code at that time.5 A ternary blend of portland cement, slag cement, and silica fume was used to obtain such compressive strength. The concrete had a w/b of 0.30 and a 175 mm (7 in.) slump, and it was pumped from the first floor up to the last.

To meet the project's 18°C (64°F) maximum temperature requirement for the fresh concrete, the concrete had to be cooled with liquid nitrogen during the warm days of July. Accelerated testing and pullout tests were used to monitor concrete compressive strength as the construction progressed.

Two Union Square

Seattle, WA, is a very windy city. To minimize the lateral deflections, the building's designers decided to build a rigid composite structure with steel tubes confining concrete having a modulus of elasticity of 50 GPa (7.25 × 106 psi) (refer to Fig. 2(b)).6 To make such concrete, the concrete producer was forced to import aggregate from nearby Canada—a glacial granitic pea gravel having a maximum size of 10 mm (0.4 in.). This glacial gravel had several advantages: it was hard and had been crushed smoothly by the glacier and not brutally by crushers, it was composed of rounded particles particularly useful from a rheological point of view, and the particles had a rough surface which exhibited very good bonding properties with the cement paste (in contrast to river gravels polished by very fine sediments). To meet the 50 GPa (7.25 × 106 psi) modulus of elasticity requirement, the w/b had to be limited to 0.22. The resulting compressive strength was 130 MPa (18,850 psi), in spite of the fact that the design strength used by the engineer was only 90 MPa (13,050 psi).7

Delivering a 0.22 w/b concrete was not (and still is not) an easy task because timing is critical. Therefore, it was decided to transport concrete on weekend nights to minimize traffic problems. To mitigate ill will in the community surrounding the concrete plant, the contractor and concrete producer offered to build, free of charge, a playground for children of that community—a playground that the City of Seattle had been refusing to build for several years. Also, earplugs were provided free for those who needed them to have a quiet sleep during the weekend nights when the concrete had to be delivered. The community was happy to make such a deal.

The Petronas Towers

Just before 2000, the United States lost its supremacy in matters of the height of high-rise buildings when the Petronas Corporation from Malaysia decided to build 451.9 m (1482 ft) high—1.5 times the height of the Eiffel Tower—twin towers in Kuala Lumpur, Malaysia. The towers shown in Fig. 2(c) are essentially made of concrete with varying strengths, the strongest having a compressive strength of 80 MPa (11,600 psi) for the columns of the lower floors.8

During the construction of this building, Samsung engineers found that it was not practical to raise buckets of concrete up to the top floors because of the limited number of cranes available. Therefore, they started a major research and development program on the pumping of high-performance concrete. This research is still ongoing.9,10

Burj Khalifa

Burj Khalifa, Dubai, the United Arab Emirates (Fig. 3(a)), at 828 m (2717 ft) high, is currently the tallest building in the world. It is a reinforced concrete structure up to 586 m (1923 ft). The concrete was transported by a single pump up to this height.11 During the construction of the steel structure on top of the concrete structure, Samsung engineers found that erecting the last 242 m (794 ft) was very painful, time-consuming, and very costly because they had only two cranes to do so.

These two cranes operated both night and day to raise pieces of steel, and it was impossible to complete the installation of finishes during this period. Therefore, as steel cannot be pumped, the engineers were asked by Samsung to design all future high-rise buildings entirely with concrete.

Worli Project

In Mumbai, India, the Samsung Corporation is now supervising the construction of an 83-story concrete structure where all the concrete is pumped from the first floor up to the top. The concrete pump is directly fed by the concrete mixer. The columns in the lowest floors are being built with an 80 MPa (11,600 psi) concrete having a 200 mm (8 in.) slump, and all the floor slabs with a 30 MPa (4350 psi) self-leveling concrete having a 650 mm (26 in.) slump flow.12
This is the present state of the art for the construction of high-rise buildings (refer to Fig. 3(b)). We are quite far from the 60 MPa (8700 psi) concrete having a 100 mm (4 in.) slump and placed using buckets, as was done for the construction of Water Tower Place in 1968.

Kingdom Tower

The Saudi Arabian Binladin Group has been considering the construction of a 1.6 km (about 1 mile) high tower— the Kingdom Tower in Jeddah, Saudi Arabia. Their technical team has determined, however, that the transition from the Burj Khalifa's 800 m (about 0.5 mile) to 1.6 km (about 1 mile) is too ambitious. Nevertheless, a 1000 m (3280 ft) building is feasible. Two pumping scenarios are presently under study. The first one is the use of a concrete pump able to transport concrete up to 1000 m (3280 ft). If this scenario does not work, the alternative would be to use two pumps in series, with each capable of transporting concrete 500 m (1640 ft). Self-consolidating concrete will be used for the majority of the elements, and compressive strengths of up to 100 MPa (14,500 psi) are expected for the columns. The firm Advanced

Fig. 2: High-rise building constructed with HRWRAs: (a) construction of Scotia Plaza (photo courtesy of John Bickley); (b) construction of Two Union Square (photo courtesy of Weston Hester); and (c) the Petronas Towers (photo courtesy of Morio and Wikimedia Commons)

Fig. 3: Tallest high-rise buildings: (a) Burj Khalifa; and (b) construction of the Worli Project (photo courtesy of Pierre-Claude Aïtcin)

Construction Technology Services has been hired to provide quality control with an on-site laboratory.13

Why So High?

Why build super-tall structures? One reason is the prestige of saying: "We have built the tallest structure in the world!" But this prestige is ephemeral. Eventually, even taller buildings will be constructed somewhere else. A longer-lasting reason to challenge the limits of structure height would be to contribute to the evolution of standard construction practices. Thanks to pioneering builders, suppliers, architects, engineers, and materials scientists, we presently master concrete technology at a level that was unthinkable 50 years ago. And concrete is starting to largely replace steel as the best-suited material for construction of high-rise structures.

With today's concrete technology, it is possible to increase the strength of concretes at the same level as the strongest rocks found in nature, to pump these concretes up to 600 m (1970 ft)—and very soon up to 1000 m (3280 ft)— and to improve the workability of these concretes so that vibration is no longer necessary during casting.

Moreover, when one succeeds at pumping a high-strength concrete up to 600 m (1970 ft), building structures 200 to 300 m (655 to 985 ft) high with 50 to 80 stories becomes a relatively trivial exercise. According to Clark,14 the real great market for high-rise buildings in the coming years are these 200 to 300 m (655 to 985 ft) tall towers. When building such structures, the lateral forces acting on the building can be taken into account relatively easily during the design process. Beyond this height, the design stage becomes more complicated and costly.

Conclusions

Due to the development of powerful HRWRAs and VMAs, it is now possible to very efficiently and economically build high-rise concrete structures. Thanks to the entrepreneurs and inventors who challenged the limits of the use of concrete, the industry has progressively learned to pump high-strength concretes higher and higher. Perhaps the sky is the only limit.

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Selected for reader interest by the editors.

ACI Honorary Member Pierre-Claude Aïtcin is Professor Emeritus at the Université de Sherbrooke, Sherbrooke, QC, Canada. He was the Scientific Director of Concrete Canada, the Network of Centres of Excel-lence on High-Performance Concrete for 8 years. He also held an Industrial Chair on Concrete technology for 9 years in col-laboration with 13 industrial partners.

ACI member William Wilson is pursu-ing advanced graduate studies at the Université de Sherbrooke. His research interests include microstructure charac-terization and properties engineering of highly durable concrete incorporating alternative supplementary cementitious materials. He received his BEng from the Université de Sherbrooke and his MS from Massachusetts Institute of Technology, Cambridge, MA.

TECHNICAL REPORT

Reprint from CI Magazine, Volume 40, No 4, Page 57-63

Internal Imaging of Concrete Elements

Ultrasonic technology is developing as a practical nondestructive inspection tool

by James A. Bittner, Agustin Spalvier, and John S. Popovics

Concrete is the most widely used construction material because of its relatively low cost and overall robust mechanical features. After placement, however, inspection of concrete elements remains a challenging task. The verification of proper material properties or geometric characteristics often requires destructive testing, which may degrade the quality of the recently constructed concrete element. Nondestructive testing (NDT) tools enable verification of quality without compromising the integrity of the structure. An overview of various NDT techniques for concrete is available in ACI 228.2R-13.1

NDT methods for concrete can be roughly divided into two principal groups: electrical- and mechanical-based methods. Electrical-based methods include resistivity and pulsed electromagnetic radar, also known as ground-penetrating radar or GPR. Mechanical-based methods include the rebound hammer, ultrasonic wave propagation, and impact-echo. Both principal NDT groups have advantages and limitations that align with various inspection requirements. While electrical methods are excellent for detailing electrochemical effects such as conductivity potentials and active corrosion currents, they are significantly influenced by material moisture conditions that can be uncontrolled or unknown. Mechanical methods are less influenced by moisture and have found broad use for detecting internal defects, evaluating uniformity of stiffness, and measuring element geometries. However, mechanical methods require the application of precise forces, which may be difficult to apply consistently.

This article provides an overview of one emerging mechanical NDT technology that uses multi-element ultrasonic shear wave arrays to provide internal image reconstructions of concrete. The general operation of the ultrasonic shear wave array method is reviewed, and an example NDT field application is demonstrated through estimation of the depth of an in-service concrete bridge deck. Lastly, improvements to the method’s existing data analysis procedure for concrete deck thickness measurement are proposed and are shown to result in a 57% reduction in measurement error. The objective of this article is to provide an overview of this emerging technology and highlight recent advances in processing algorithms to improve the overall device performance for one specific NDT task.

Overview of Ultrasonic Shear Wave Array Technology

Recent advances in ultrasonic transducer design have produced small and lightweight sensors that can be housed in a handheld array. The array devices enable quick and repeatable collection of multiple ultrasonic datasets over the footprint of the device. The transducers generate shear waves that propagate into the concrete; shear waves offer smaller wavelength (better resolution) and less mode conversion (less complicated wave parameters) than compressional waves, which for example are used in the standard ultrasonic pulse velocity method.

The whole measurement process consists of three steps:

- Generation of ultrasonic waves and detection of direct and reflected waves;
- Preprocessing of data, using a predetermined wave propagation velocity; and
- Construction of an image based on the preprocessed dataset.

In the first step, ultrasonic waves are generated by the transducers in the array and propagate into the concrete. The waves are produced by a transducer array sequence, where some of the transducers within the array generate waves while the others detect the direct and reflected waves. Waves travel both directly between transducers along the surface, and outward through the concrete mass. Internally propagating waves are observed when they are reflected from internal defects, bars, or surfaces.

In the second step, the data collected by the sensors are filtered and processed to extract wave propagation parameters, such as wave velocity needed for later imaging. The response output signal from each individual sensor represents a surface motion as a function of time. However, the output image is represented by a reflector's position in space. To convert time signals to a space (distance traveled) representation, the wave propagation velocity is needed. In a globally inhomogeneous material like concrete, the apparent wave velocity can be variable when calculated between different measurement points, even within an area composed of the same concrete batch. To account for this velocity variation, most NDT practices require an operator either to assume a reasonable average value of wave velocity or to measure it prior to carrying out an investigation; in either case, the velocity value is considered to have a constant value in subsequent calculations for analysis procedures within that experimental campaign. The use of a constant velocity value however can result in systematic error in the imaging results. Because of this inherent variability and the importance of wave velocity within the imaging scheme, accurate and representative values of wave velocity must be determined at each testing location to obtain the most accurate and reliable images.

Using ultrasonic shear wave array technology, wave velocity is determined by considering only those waves that travel directly between transducers along the concrete's surface. Velocity is calculated as the ratio of the distance between transducer-arrays and time needed for the wave pulse to travel that distance. The standard wave velocity algorithm that is carried out by the commercial device as a default process is based on a typical linear array velocity measurement method. This standard method consists of measuring the arrival times of the waves that travel directly between sending and receiving transducers and plotting those time measurements with respect to the corresponding distances between transducers. A least-squares line of best fit is then computed from the time versus distance data set, and the inverse of its slope is the estimated velocity.

The third step consists of constructing a representative spatial image from the preprocessed dataset. To do so, intricate data processing algorithms, such as the Synthetic Aperture Focusing Technique (SAFT), must be deployed. SAFT was originally developed to process radar data under the name of Synthetic Aperture Radar (SAR).2 The technique is based on the principles of signal time shifting, summing, and superposition, where multiple received summed signals indicate enhanced amplitude of some internal wave reflector, while the noise component amplitude remains at a lower uniform intensity.3 In other words, SAFT uses space-averaged signals to provide an image that represents a cross-sectional slice through a material perpendicular to the surface, where the location of reflections within that cross section are indicated through enhanced amplitude indications above the background noise. For example, when scanning a concrete slab-on-ground to determine the thickness, the reflections from the bottom surface of the slab will result in a nearly monotonic constructed image with a zone of high amplitude (in the device we used, this zone appears red) representing the best estimate for the slab thickness.

Several different commercial ultrasonic array devices for concrete are currently produced by different manufacturers, although the fundamental array operation and data processing algorithms used by them are largely similar. In our research, we used the "MIRA" array device manufactured by Acoustic Control Systems (ACSYS). However, the improvements and methods discussed in this article are device-independent and can be applied generally to all shear wave array devices regardless of manufacturer.

An illustration of basic operation and output SAFT image produced by a commercial device are shown in Fig. 1. In this illustration, the device was placed on the surface of a 500 mm (19.7 in.) deep steel-reinforced structural concrete column. The operator triggers the device, which causes the array to generate and receive ultrasonic waves in a sequence that includes every combination of the 12 sets of transducers, resulting in 66 total combinations of sensor pairs and time signals for each measurement location. The fact that many unique signals are collected from one measurement location is an asset that can be exploited when using such devices.

The resulting SAFT image that is built up from many signals arising from the array set, seen in Fig. 1(b), represents an interior cross-sectional slice into the column, perpendicular to the surface. The bright red colors in the image represent the position (depth) of an internal object that reflects wave energy, while blue colors represent areas that do not contain significant internal reflectors. Circular reflectors such as the cross section of internal reinforcing bars are represented by circles in the SAFT image while elongated and flat reflectors, such as the back surface of the column, are represented by linear features. It is important to note the image clearly indicates the location of reflectors within cross-sectional space, but the apparent size of the reflector in the image is representative of the magnitude of the reflection process and not necessarily the true size of the reflector. Ultrasonic array imaging using SAFT has been applied for a wide array of inspection tasks such as geometric sizing of concrete elements,4 internal duct localization,5 tunnel lining condition,6 and pavement characterization.7,8

Even though ultrasonic array imaging has found broad application for inspecting concrete structures, several limitations of the technology have been noted, as described:

- Restricted depth of penetration—Large internal reflectors can set up a “shadow zone” behind them in the concrete within which very little wave energy penetrates4;
- The polarized nature of the transducer array—The performance of the device may depend on the orientation of the array unit.6 For the SAFT algorithm to effectively represent reflectors in the constructed image, the reflectors must be oriented so that a complete set of four transducers record identical reflections. Thus, a slender reflector like a reinforcing bar requires that the principal axis of the device be positioned perpendicular to the bar axis to be well detected; and
- Dependence on accurate measurement of mechanical wave velocity—The use of inaccurate or nonrepresentative wave velocity values negatively influences the ability of the device to reconstruct an accurate image.8

This article addresses the last listed limitation, inaccurate wave velocity measurement, and suggests approaches to minimize its influence by using the multi-element array data provided by the measurement device. In other words, we take advantage of the increased volume of data available from ultrasonic arrays so that measurement of wave velocity can be carried out automatically at every measurement location, thus minimizing the influence of systematic velocity error.

Accuracy of Ultrasonic Shear Wave Imaging

As with any assessment technology, it is important to understand the accuracy of the ultrasonic shear wave imaging device as operated under ideal conditions. To establish such a performance baseline with respect to estimating concrete slab thickness, a test series of 12 concrete slab samples, each with a nominal depth of 228 mm (9 in.), were measured under laboratory conditions. We used the standard velocity algorithm on the commercial device described in the previous section. Because there was full access to both surfaces of each slab, accurate thickness measurements could be directly obtained with a mechanical caliper. The slab sample set comprised two concrete mixture designs with average 14-day companion cylinder compressive strengths of 36 and 54 MPa (5200 and 7800 psi). The mixtures were typical of those used in reinforced concrete highway structures in Illinois. On each slab, three test locations were identified and measurements were applied three separate times at each test location; refer to Fig. 2(a) for image of typical test slab after cores (to monitor global thickness and compressive strength) were removed and before the nondestructive tests were performed.

The nondestructive tests were carried at locations between cores, and thickness at each test location was determined directly with calipers. The three measurements at each location were used to calculate one average thickness value for each test location. The measurement error was defined as the difference between the average caliper and ultrasonic prediction of length at each test location.

The histogram in Fig. 2(b) shows the statistical distribution of slab thickness measurement error obtained for all 36 measurement locations (three locations for each 12 slabs). The error shows a mean value of approximately 2.6 mm (0.1 in.). The fact that the mean value of error is not zero suggests that the thicknesses are systematically under predicted by approximately 2.6 mm on average. Assuming a standard Gaussian

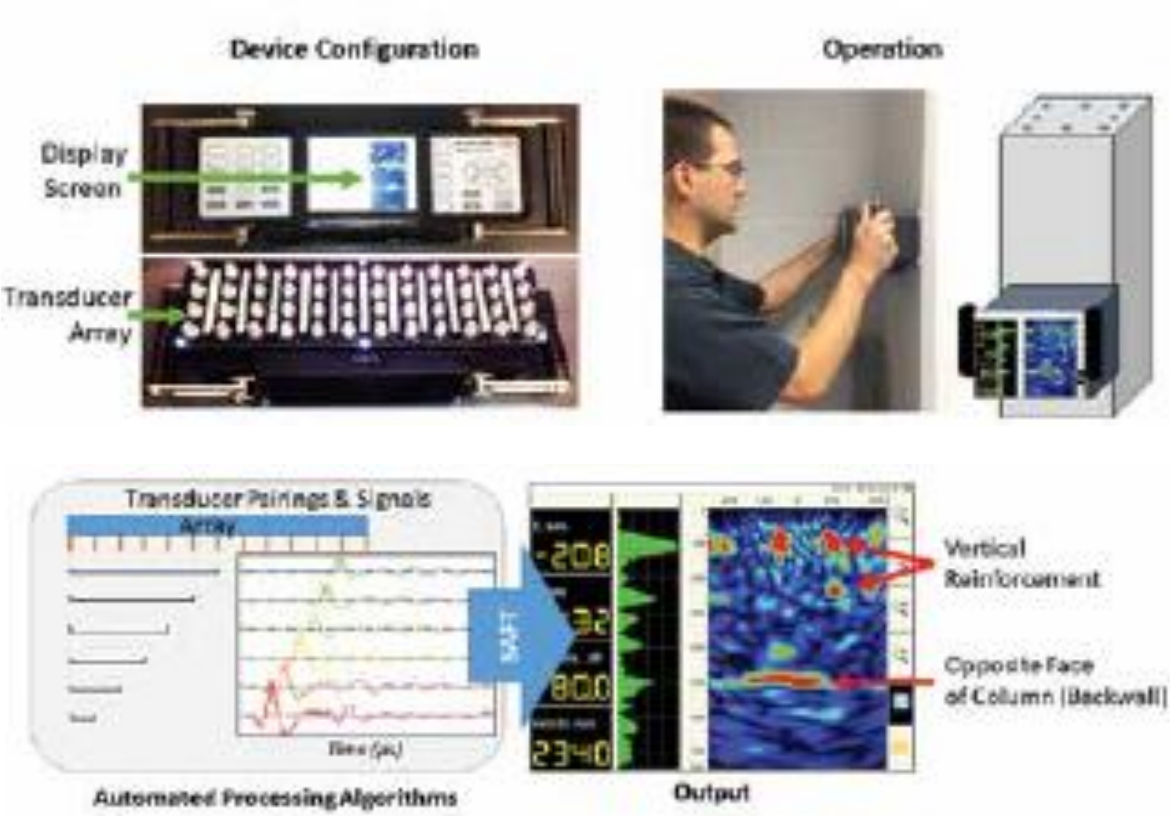

Fig. 1: Application of a commercial ultrasonic shear wave array device to evaluate a structural concrete column: (a) configuration and operation—green arrows denote the device display screen on the device top and the 12 sets of four contact transducers on the device bottom; and (b) processing schematic and SAFT image output—red arrows indicate high reflection amplitude (red colors) within the constructed cross-sectional image

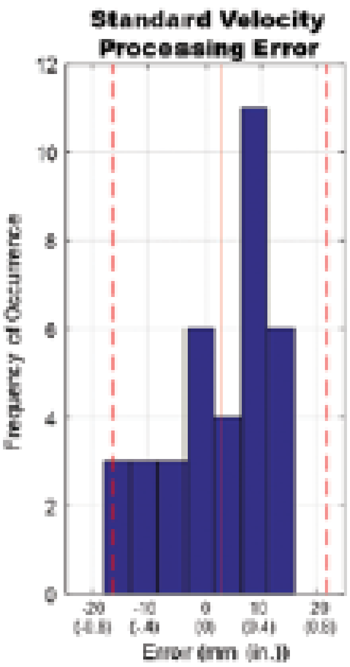

Fig. 2: Establishment of baseline performance of ultrasonic array unit for slab thickness estimation: (a) a typical concrete slab sample after removal of cores; and (b) statistical distribution of the measurement error from all tests, where the dashed red lines represent the 95% confidence intervals and the solid red line indicates the mean of the error

distribution of the error data and accepting the average caliper measurement as the true thickness, the 95% confidence interval of the error was calculated as the interval within two standard deviations away from the error mean value. For the data shown in Fig. 2(b), the 95% confidence interval was 16.5 to 21.8 mm (−0.65 to 0.86 in.). Thus, a thickness measurement collected in well-controlled experimental conditions is expected to be within approximately 20 mm(0.79 in.) or 9% of the true slab thickness of 228 mm, 95% of the time. We can expect that measurements carried out in a less-controlled environment, such as that associated with field operations, would not achieve higher accuracy and may achieve lower accuracy.

Example of Concrete Element Imaging

A common NDT task is verification of installed geometries of structures and elements. In the example shown here, a recently constructed rural concrete bridge deck exhibited regions of insufficient thickness based on field measurements during construction. The bridge has a concrete deck placed on a steel girder superstructure. At the time of construction, the inspector on site directly measured deck thickness on the still-fresh (plastic) concrete with a ruler three times across the width of the lane. However, complete understanding of the situation requires additional confirmation of deck thickness throughout the width of the lane, at more locations than would be feasible to be measured with core samples. Thus, the bridge management agency sought a nondestructive method to estimate deck thickness at multiple locations. The bridge deck construction documents call for a minimum thickness of 203 mm (8 in.), while the inspector recorded three measurements of 191 mm (7.5 in.). The inspector also recorded several other ruler measurements over a series of stations to confirm his observation; however, in this example, we will focus on a single station’s profile.

Several months after the deck was cast, the ultrasonic array device was used to collect data from the deck at several positions across one lane from the centerline to the parapet wall, perpendicular to the direction of traffic. A series of 30 measurements, with a step size of 170 mm (6.7 in.) between measurement positions, was carried out across the deck width, and a SAFT image was created for each measurement location. An example of a single image reconstruction built up from the data from one measurement is shown in Fig. 3(a). Near the center of the SAFT image, a flat horizontal region of high reflection can be seen, which is presumed to represent the reflection from the bottom surface of the bridge deck. As noted in the figure, the location of maximum reflection value of the feature was used to estimate the depth of the deck at this location.

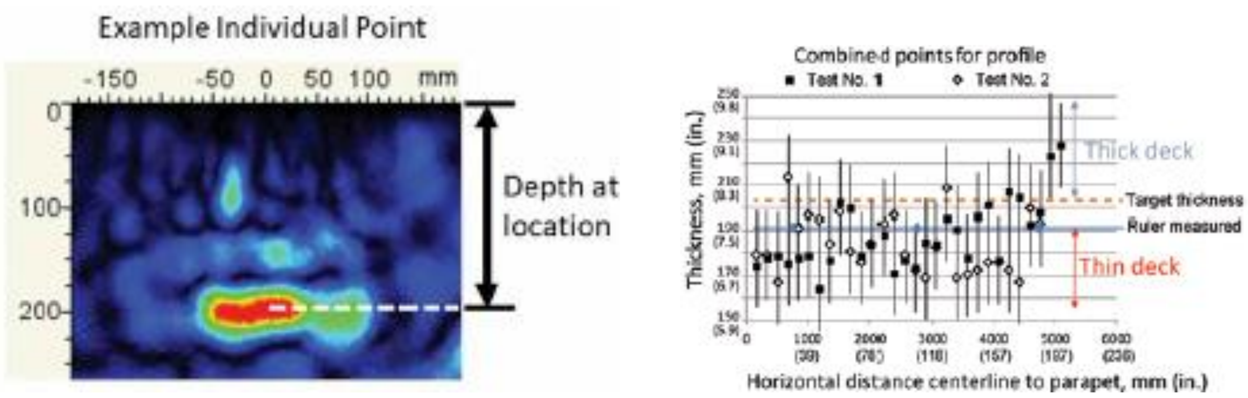

Fig. 3: Measuring thickness profile across the width of a single lane on a concrete bridge deck: (a) a two-dimensional image of the deck cross section from one measurement location; and (b) the profile from two repeated passes of ultrasonic measured thickness (using the standard velocity algorithm)—the dashed orange line represents the contract design thickness and the blue line represents ruler measurements in fresh concrete at the locations denoted with the blue diamond markers

The thickness results from each ultrasonic measurement point were assembled and plotted (refer to Fig. 3(b)). Solid square marks correspond to the thickness results of the first set of measurements (Test No. 1), obtained by applying the standard wave velocity algorithm. The target design thickness of the deck slab is shown as a dashed orange line. The three ruler measurements performed on the fresh concrete in the deck slab during construction are indicated with blue diamonds. Error bars were superimposed onto each thickness result to indicate the variability. The error bars comprise the experimental thickness measurement ±20 mm, associated with the 95% confidence intervals previously established. The error bars associated with each thickness measurement define a band that indicates significant experimental variability along the profile. The ruler measurements generally fall within that band, although the ultrasonic predictions of thickness are on average notably below those of the ruler measurements. The target deck thickness falls within that band for only about half of the measurements, and again the ultrasonic predictions of thickness are on average notably below the target thickness.

To assess the repeatability of the test data, the ultrasonic measurements were repeated along the same measurement line across the deck (Test No. 2), and those data are represented by hollow diamond points in Fig. 3(b). The average thickness difference between the repeated measurements (Test No. 1 and Test No. 2) was approximately 14 mm (0.55 in.). This difference is within the ±20 mm maximum error interval of confidence previously calculated. As in the first test series, the three ruler measurements fall within the band expected for the ultrasonic measurements, but the target thickness falls within the 95% confidence interval for only a fraction of the ultrasonic measurements. As a result, the two test series are unable to confirm with confidence the existence of a difference between the actual and target thicknesses.

From the analysis in Fig. 3(b), it can be observed that despite the powerful imaging capability of the device, the variable nature of the measurement data negatively influences the value of the output (thickness prediction). One approach to improve the performance in this case is to modify how the velocity values are obtained.

Imaging Algorithm Improvements

During this investigation, it was observed that variability in the velocity estimation method has a major influence on the generated reconstructed images. As an example, the uncertainty in estimating the bridge deck thickness (shown in Fig. 3(b)) from two different measurement sets can be directly related to variability of the measured wave velocity. The standard velocity estimation algorithm was conceived assuming that a small set of manual measurements are collected using a single sender and receiver. This procedure is schematically illustrated in Fig. 4(a). The use of multi-sensor arrays changes the nature of the collected measurement data set and opens the potential for improved velocity estimation algorithms. Several new velocity estimation schemes were considered, based on the same data sets that were collected from the bridge deck, that take advantage of the multi-sensor array data set. The most promising method was based on nearest neighbor image processing techniques.9 The same time versus distance paired dataset used by the standard method shown in Fig. 4(a) is considered. However, instead of fitting a line to the entire time versus distance dataset as the standard method does, the proposed method computes velocity measurements from each and every possible unrepeated time versus distance data pair, creating an approximately normal distribution of velocity results. After clipping the distribution for outliers outside the intentionally broad velocity range of 2000 to 4000 m/s (6500 to 13000 ft/s), the mean of the remaining distribution was used as the wave velocity estimate. An overly broad velocity range was used to avoid clipping any plausible velocity estimates. This improved method is schematically illustrated in Fig. 4(b).

The proposed distribution method of velocity estimation was applied to the previously introduced example bridge deck dataset. Because the velocity estimation stage occurs after the collection of data, the same raw collected data were processed again using the new velocities. The deck thickness profile results using the improved velocity estimation method are shown in Fig. 5. The solid square and the hollow diamond points represent the first and second measurement paths across the deck. New error bars (interval of confidence) associated to the proposed method were computed using the same procedure and dataset as previously explained. The new method's error bars were found to be ±9 mm (0.35 in.), or about 4% of the total design thickness of 228 mm; these were superimposed onto each thickness measurement in Fig. 5.

The agreement between the repeated measurements is improved over that shown in Fig. 3(b), and the predicted thicknesses are within the laboratory-measured confidence intervals. The average repeatability error between the two measurements was decreased by 57% to 6 mm (0.24 in.). The measured ultrasonic thickness predictions match those from the field ruler measurements at the three locations. The agreement with the fresh concrete measurement gives confidence in the predicted thickness of the deck at those

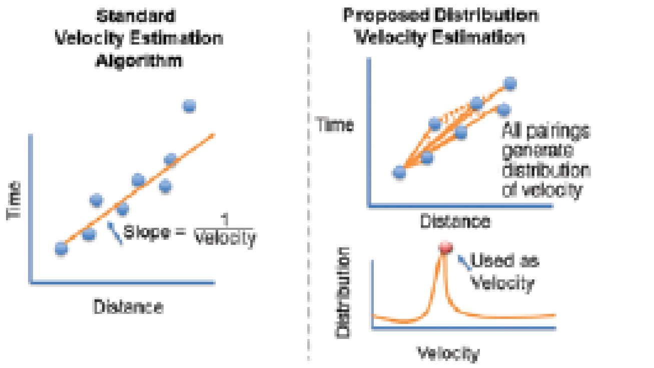

Fig. 4: Wave velocity estimation techniques: (a) the standard wave velocity algorithm computes velocity by taking the best fit line of the time versus distance data pairs and then takes the inverse of that slope; and (b) the proposed array technique computes velocity as the mean value of a velocity distribution, where this distribution is composed of velocity measurements obtained from all possible unrepeated time versus distance data pairs

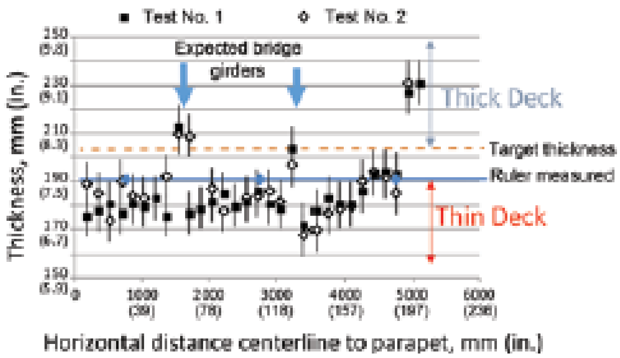

Fig. 5: Recomputed thickness profile using same dataset in Fig. 3. Test No. 1 and Test No. 2 present ultrasonic estimated thickness (using distribution velocity algorithm); the dashed orange line represents target design thickness; and the blue line represents ruler measurements in fresh concrete at the locations indicated by the blue diamond markers

locations. The target thickness value only falls within the expected ultrasonic prediction band at locations where the slab rests on the underlying girder; it is plausible that the ultrasonic results interpret the composite deck-girder system as a thicker concrete deck. With the improved agreement between the repeated measurements, the results of the investigation led to increased resolution and confidence in the deck thickness estimates and the conclusion that the deck thickness is indeed less than the minimum design value.

Conclusions

Ultrasonic shear wave arrays are an emerging and promising nondestructive inspection tool to investigate the condition of concrete elements. Although promising, the technology and data analysis schemes need improvement to enable broader and more effective application. Operators and engineers can benefit from understanding the observed limitations of the technology, and the assumptions within the analysis schemes that are implemented within the system.

The accuracy of the commercial ultrasonic shear wave arrays for the estimation of concrete slabs thickness was investigated. The ultrasonic array thickness measurement compared to a mechanical caliper, assuming a 95% confidence interval, generated an error of ±20 mm for a nominal 228 mm concrete slab, or approximately 9% of the total thickness. The confidence interval is important to consider when deploying the current generation of shear wave array devices for detailed detection tasks.

Because the SAFT image processing procedure uses averaged wave signals over some area, the wave velocity used in those calculations is a vital parameter. Accurate estimates of wave velocity are needed to produce the clearest and most accurate image reconstructions. The new algorithm introduced here, distribution velocity estimation, uses a statistical distribution of measured velocity values to improve the prediction of thickness measurements on a bridge deck. Using the distribution velocity estimation method reduced the 95% confidence interval for thickness measurement to ±9 mm for a nominal 228 mm concrete slab—about 4% of the total design thickness.

Applying the distribution velocity estimation algorithm to measured bridge deck thickness reduced the average repeatability error by 57% and improved confidence in the measured data. It also demonstrated a possibility of obtaining the true thickness of a concrete bridge deck without destructive coring.

The algorithm evaluation work presented in this article was performed using an open-source framework for processing the collected ultrasonic array data. This framework was developed with the intention of encouraging other practitioners, engineers, and researchers to contribute ideas for additional robust processing techniques. These algorithms are freely available at https://github.com/Jabittner/openSAFT.

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Disclaimer

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CASE STUDY

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In-Situ Durability

Forensic engineering approach to prolong the service life of structures affected with DEF or ASR

by Alfredo (Al) E. Bustamante and Jared Wright

In hardened concrete exposed to temperatures exceeding 70°C (158°F), sulfate phases within portland cement and gypsum will be adsorbed onto C-S-H phases.1 Upon cooling, the adsorbed sulfates are released into the pore solution of the concrete and are free to form volume- increasing ettringite phases within the hardened concrete. Unfortunately, this delayed ettringite formation (DEF) creates internal pressures that cause concrete cracking. Research shows these ettringite phases will continue to form while the relative humidity (RH) within the hardened concrete exceeds 90%.2 Besides controlling the temperature of concrete during curing, special cements have been created with low sulfate amounts to help retard and prevent this and other forms of sulfate attack.

Concrete will be subject to alkali-silica reaction (ASR) when meta-stable forms of silica in aggregates dissolve in the high-pH-alkali-hydroxide-rich pore solution of concrete.3 After dissolution, the silica can form an alkali-lime-silica gel that can swell and expand upon imbibition of water. Studies show that a RH of 80% (as opposed to 90% RH for DEF) is necessary for ASR to proceed.2 Because the aggregate is well dispersed within the concrete matrix, the swelling associated with ASR is typically manifested on the concrete surface by many interconnected cracks (map-cracking) that exude gel. Typical techniques used to regulate ASR include the use of supplementary cementitious materials (SCMs) such as fly ash or slag cement to reduce the pH, restrict transport, and bind alkalis.

For new structures, adjustments during design and batching procedures focused on ensuring aggregate quality, cement oxide composition, and SCM quality can help prevent DEF or ASR from occurring. After construction, the key to controlling DEF and ASR degradation is limiting the available moisture. Without sufficient internal RH, the consumption reactions of DEF and ASR will halt.

The following case study takes the reader through an investigation into cracking of load-bearing concrete columns, along with the suggested repair and monitoring program, of a precast concrete parking structure located in Texas.

Distress and Survey

The parking structure was completed around 1983 and consists of 12 levels of parking, with two basement levels, one level at grade, and nine levels above grade. The exterior columns of the structure exhibited considerable cracking (Fig. 1). The structural system consists of 10 ft (3 m) wide, precast concrete double-tee members that span between 47 ft 6 in. and 59 ft (14.5 and 18 m) in the north-south direction.

Fig. 1: Cracked load-bearing columns

The double tees are supported by inverted-tee beams, pocketed spandrel panels, interior panels with haunches, interior load-bearing column stacks, perimeter load-bearing and nonload-bearing column stacks, perimeter boxed shape wall panels, and perimeter steel columns encased by precast concrete wall panels. The wearing surface and primary diaphragm component on the double tees comprises a bonded cast-in-place concrete topping course.

The perimeter precast concrete column stacks have an exposed aggregate finish on the exterior faces and a smooth

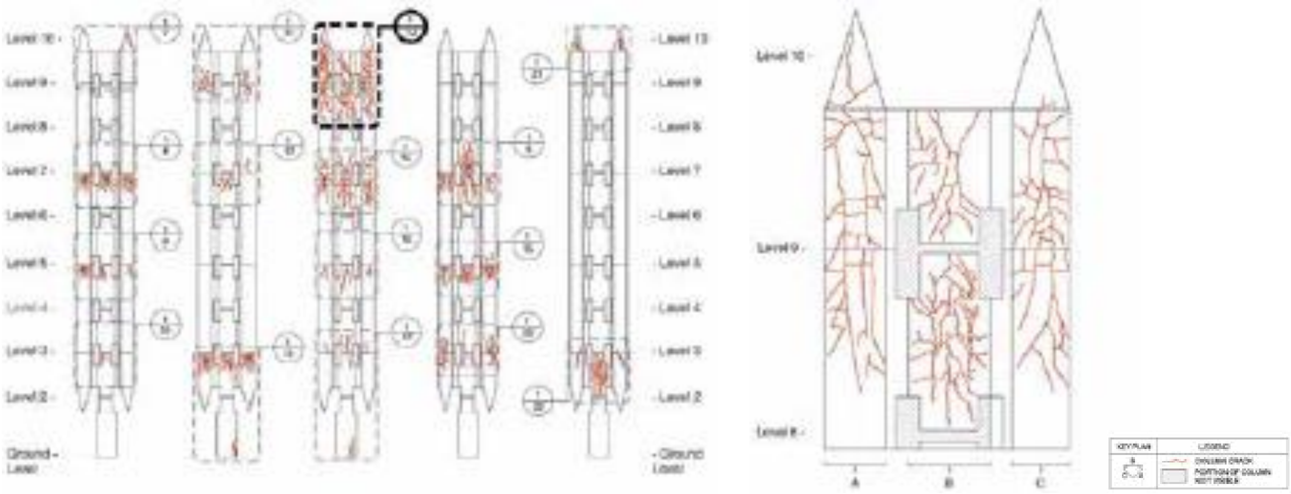

Fig. 2: North elevation column stack concrete crack map: (a) unfolded view of all sides including exterior and interior faces; and (b) an enlarged view of Detail 1/14

finish on the inside faces. The garage has 39 column stacks of pretensioned precast members with similar cross sections. These stacks are on the east, north, and south elevations of the structure. Of the 39 column stacks, 17 are load-bearing members and the remaining 22 are architectural members. After the owner of the structure contacted us to investigate cracking observed on three columns, we conducted up-close visual inspections, material sampling and testing, sounding, ground-penetrating radar (GPR) evaluations, half-cell potential testing, structural analyses, and, finally, developed a repair program.

Our survey showed that longitudinal, diagonal, and circular pattern cracks existed on 75% of the load-bearing column stacks (Fig. 2(a)). Figure 2 shows unfolded views of the exterior and interior faces of the columns on the north elevation. Exposed faces were surveyed up-close to accurately document crack location and pattern. The hatch sections indicate the portions of the columns that are concealed behind adjacent structural elements. Each call-out depicts greater detail regarding the observed cracking (Fig. 2(b) is a representative detail depicting Detail 1/14). We found that the architectural column stacks had no cracking; however, the load-bearing column stacks were cracked and cracks were generally concentrated around joints and at the tops of column stacks (Fig. 2(a)). The crack widths varied between 0.005 and 0.250 in. (0.13 to 6.35 mm), with most exceeding 0.010 in. (0.25 mm). Petrographic analyses revealed that DEF was the primary distress mechanism. ASR was also noted in limited amounts, but the analyses indicated that it was not a significant contributor to the cracking. Typically, we have observed that transverse and longitudinal DEF-induced cracking tends to develop along structural members in locations with less restraint and low principal stresses. As observed in this study, these zones are typically located at the column segment joints and column tops. These results further confirmed the survey team’s belief that DEF was the cause of degradation. We understand that the mortar joints at the interfaces between column stack segments allow uniform vertical load transfer but do not significantly restrain expansion.

Detailed Examination

Petrographic examinations of concrete core samples obtained from three different columns were performed in general accordance with ASTM C856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Two cores (Core 1 and 2) were obtained from crack locations and one core (Core 3) from an uncracked location (control sample). The diameter and the length of each core sample were about 3.5 in. (89 mm). The petrographic studies provided additional confirmation that DEF was the primary cause of cracking as these tests revealed that the cracked and uncracked column sections had similar concrete mixtures and physical properties. This finding supports the hypothesis that a production error is the probable cause of the observed distress. While it is our opinion that the concrete cover zone was exposed to elevated temperatures during curing, we were unable to obtain data regarding the curing activities used during column fabrication.

A 5/8 in. (16 mm) thick slab was cut lengthwise from the center of each core using a water-cooled continuous-rim diamond saw blade. The sawed surfaces in Core 1 and 3 were oriented perpendicular to the full-depth crack that was present in each core. One of the remaining portions of each core was stored for 2 days at room temperature in a 100% RH environment. This procedure can sometimes mobilize silica gel that may be present in the concrete. The plane surfaces were lapped using progressively finer silicon carbide abrasives. The lapped surfaces and the remainders of the cores were then examined using methods outlined in ASTM C856.

Thin ettringite-filled voids around the perimeters of fine aggregate particles were detected in Core 1. In combination with the unusual crack pattern visible on a micro-scale, the filled voids are consistent with cementitious matrix expansion caused by DEF (Fig. 3). While confirmation of DEF is best established using high-magnification examination using a scanning electron microscope (SEM), no additional studies were completed. The advanced nature of distress in Core 1 indicated that such study was not required.

GPR testing in general accordance with ICRI Technical Guideline No. 210.44 was done at selected column locations. For concrete materials, GPR is used almost exclusively in the reflection mode, with the transmitting and receiving antennas set at a small fixed distance apart as the inspected surface is traversed. The transmitting antenna sends a diverging short pulse of energy between 1 and 3 nanosecond duration. The receiving antenna collects the energy reflected from dielectric interfaces between materials of differing dielectric properties, and the data is then processed by the radar unit and displayed on the screen. The reflected energy is recorded as a pattern. From these patterns, an experienced operator and interpreter can deduce useful information such as location and depth of steel reinforcement.

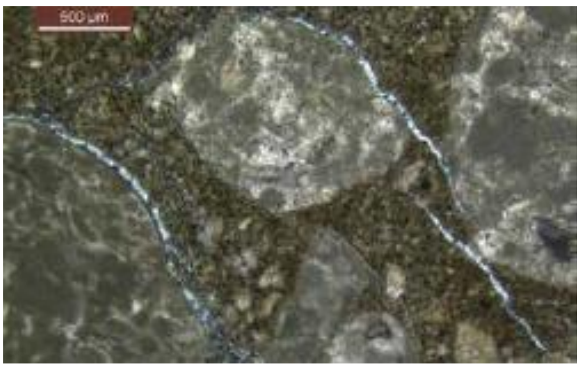

Fig. 3: Transmitted-light photomicrograph of one core sample, illustrating ettringite rims and microcracks in paste lined with ettringite

GPR testing revealed that the cracked columns had ties spaced about 10 to 18 in. (254 to 457 mm) along the column length (Fig. 4), including the area of the column stack where faulted cracks are located. In other words, wide longitudinal cracks near joints were intersected by steel reinforcement. On previous projects, we have observed that DEF-induced cracks in precast concrete columns (confined with stirrups and longitudinal reinforcing) typically are much wider at the surface than at the level of the steel reinforcement. DEF cracking therefore generally does not significantly affect the column core, which carries most of the column load.

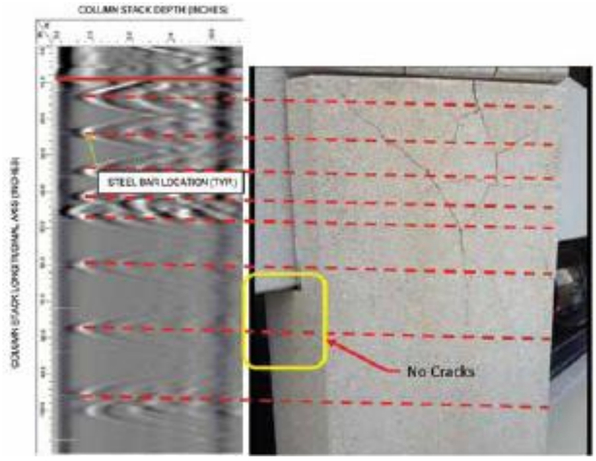

Fig. 4: Transverse reinforcing bar locations were determined using GPR scans. In this montage, bar locations are indicated by red dashed lines (Note: 1 in. = 25.4 mm)

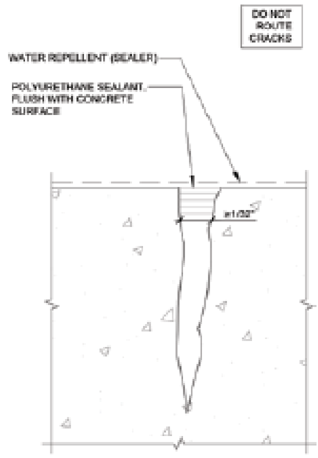

Fig. 5: Crack repair detail

For this project, the core samples taken for petrographic analyses verified that cracks were wider at the surface and tapered as they approach the reinforcing bars. The core sample with the most prominent cracking exhibited cracks that terminated about 3 in. (76 mm) from the surface. Acoustic impact testing also revealed sound concrete along the column stacks, suggesting that the steel reinforcement is engaged with the surrounding concrete. These observations show that the columns' cores remained intact and confined by the reinforcement, further indicating that the columns are not significantly compromised. However, the presence of surface cracks creates a long-term issue because they will provide conduits for water infiltration, potentially resulting in steel corrosion and additional cracking due to the formation of corrosion products and DEF- and ASR-induced expansion of the concrete.

Repair

Therefore, considering the distressed condition of the columns primarily poses durability issues and not structural concerns, we designed repairs to minimize the amount of moisture that will penetrate the precast members with the objective of maintaining an RH below 90% to stem DEF degradation. While similar, yet more robust, methods can be used to keep the RH below 80% to stem ASR degradation, we did not consider those measures to be necessary.

We specified the following repair measures:

- Cleaning of concrete surfaces using water-assisted grit blasting—this method was used to minimize the amount of water entering the concrete. The concrete surfaces were then required to dry prior to the next step;
- Application of a penetrating silane sealer with 100% solids content(Fig. 5). We noted from our core samples that carbonation was very shallow, so the pH was adequate for the silane to react. We typically specify a high solids content in sealers because the material cost is generally a small portion of the total cost of a repair product;
- Application of sufficient amounts of flexible sealant (caulk) to cover cracks measuring 0.02 in. (0.05 mm) or more (Fig. 5). Penetrating sealers do not have the ability to bridge cracks wider than 0.012 in. (0.30 mm), so the sealer was supplemented with surface coatings able to bridge cracks. The sealant material was not required to completely penetrate cracks—only to seal the surface of the cracks;
- within the joints between column stack segments and joints between the column stacks and the spandrel panels. We typically prefer that sealer and sealant products are provided by the same manufacturer. If not, we require that the manufacturer of each product approves the application with the other products used; and
- Demonstration of repairs using a full-scale mockup (that is, one entire segment of a column stack). This step was used to evaluate the effectiveness and the aesthetic implications of the repairs. The mockup was a separate line item on the procurement form for the project and it was required prior to full mobilization by the contractor.

Recommendations for Future Actions

We recommended that a visual survey of the garage's precast concrete elements be performed annually for 2 years after the repairs were completed. These surveys will allow identification and sealing of additional cracking as well as document the effectiveness of the completed repairs. If additional distress is not observed during those surveys, we recommended reducing the frequency of the visual surveys to every 2 or 3 years.

While the described repair scheme will lower the internal RH of concrete, its effectiveness will not be permanent. It is our experience that penetrating sealers effectively perform for 7 to 10 years. However, there is little data available in the industry regarding the long-term performance of surface coatings to treat concrete structures affected by DEF. We therefore recommended that the penetrating sealer be reapplied after 7 to 10 years, with the timing based on inspections verifying that the sealer remains effective in keeping the internal RH in the concrete below 90%.

Furthermore, a flexible surface coating, applied after proper curing of a penetrating sealer, is a proven additional protection method. We recommended that the combination of a penetrating sealer and flexible coating should be tested for compatibility and the combined application should be approved by both product manufacturers. Flexible sealant materials are available with proven service life of 20 years or more, so removal and replacement of a properly applied flexible sealant would not be required for at least 10 years.

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Note: Additional information on the ASTM standard discussed in this article can be found at www.astm.org.

Selected for reader interest by the editors.

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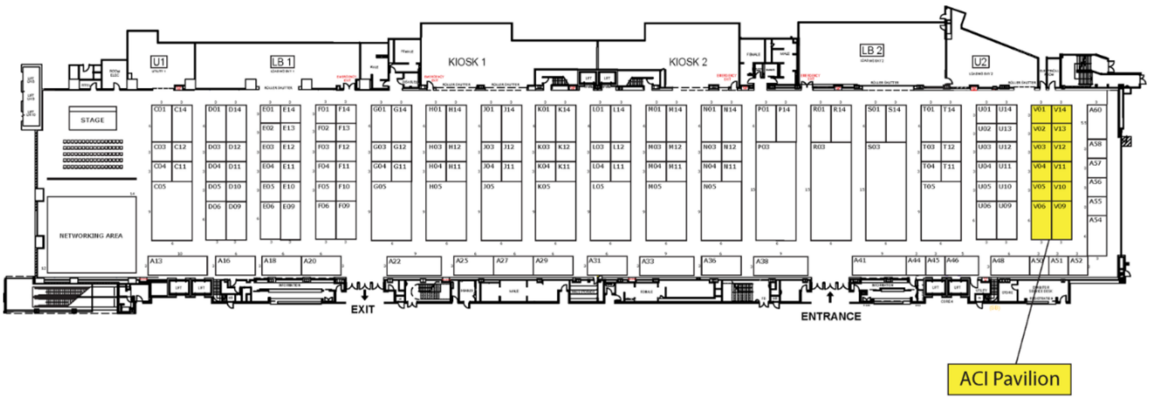

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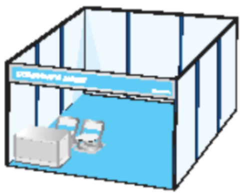

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Mr. Oscar is a specialist in concrete/cementatious decorative and facade mixture. To be specific, Mr. Oscar focuses on Glass Fibre Reinforced Concrete (GFRC) and other light weight concrete facade. Graduated from Monash University in Bach. Civil and Environmental Engineering, his goal is always to create concrete that can beautify the current dull concrete jungle of city life. With 5 years of experiences in Concrete Facade industry, he has managed to assist in designing concrete facade at not only on private buildings that include but not limited to bungalows and condominiums, but also commercial buildings like arches on KL-Selangor Boundary and also decorative item of commercial factories.
On the other hand, Mr. Oscar is also a consultant on construction contract dispute which assist contractors on claiming issues.
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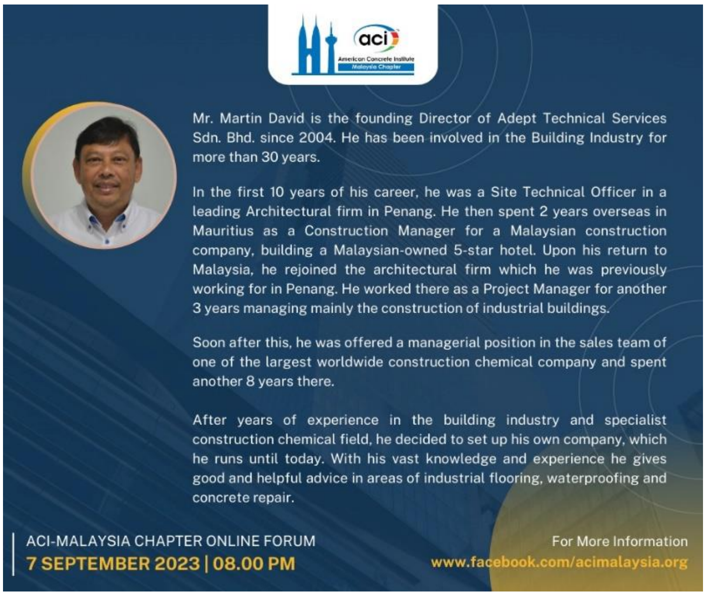
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Mr. Martin David is the founding Director of Adept Technical Services Sdn. Bhd. since 2004. He has been involved in the Building Industry for more than 30 years.
In the first 10 years of his career, he was a Site Technical Officer in a leading Architectural firm in Penang. He then spent 2 years overseas in Mauritius as a Construction Manager for a Malaysian construction company, building a Malaysian-owned 5-star hotel. Upon his return to Malaysia, he rejoined the architectural firm which he was previously working for in Penang. He worked there as a Project Manager for another 3 years managing mainly the construction of industrial buildings.
Soon after this, he was offered a managerial position in the sales team of one of the largest worldwide construction chemical company and spent another 8 years there.
After years of experience in the building industry and specialist construction chemical field, he decided to set up his own company, which he runs until today. With his vast knowledge and experience he gives good and helpful advice in areas of industrial flooring, waterproofing and concrete repair.
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James Lim is the founder and director of CRT Specialist (M) Sdn Bhd established since 2005. He specializes in concrete repair and waterproofing works and the company has been involved in various projects throughout Malaysia, Singapore and Thailand.
James Lim graduated with a degree in Civil Engineering in 1997 from the University of Auckland, New Zealand. Later in 2002, he completed his Executive MBA from the University of Bath, UK. He has accumulated over 25 years of experiences in the field of concrete repair and waterproofing works. He is also an active member of the American Concrete Institute Malaysia Chapter (ACI-MY) from 2020 to 2024 and a certified trainer by the HRDC in his field of expertise.
James has been involved in many key infrastructure projects and niche high rise development projects throughout his career. His track record includes projects like the KVMRT Line 1 & 2, St. Regis Hotel KL, Le Nouvel Residences, Lot M KLCC and others. Milestones were set when his company was chosen as the specialist contractor to undertake the complete waterproofing work of the MRT Bukit Bintang underground station by MMC Gamuda JV in 2013. The work involves waterproofing all elements of one of the deepest underground train station construction from roof slab, base slab, water tanks, slab and wall.
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