Bridge Views

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Inside This Issue...

The New Shear Resistance Factor for Lightweight Concrete and Its Effects on Design

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Lightweight high performance concrete was used for the cast-in-place superstructure segments

When the first edition of the AASHTO LRFD Bridge Design Specifications was published in 1994, a special shear resistance factor, φ, for light-weight concrete of 0.70 was introduced. The shear resistance factor for normal weight concrete was 0.90. The lower shear resistance factor for lightweight concrete was introduced because of a lack of available data to evaluate the statistical variability of lightweight concrete (Mertz, 2012). However, a special shear resistance factor for lightweight concrete had never appeared in the AASHTO Standard Specifications or the ACI Building Code Requirements for Structural Concrete (ACI 318). Both of these other documents had used the same resistance factor for normal weight and lightweight concrete since strength design was introduced in the early 1960s. Even the most recent edition of ACI 318, which was released in 2008 and incorporated significant revisions to the treatment of light-weight concrete throughout the code (including shear provisions), did not include a special shear resistance factor for lightweight concrete. Both of these codes did use a factor (λ in ACI 318) to reduce the concrete contribution, Vc, to the shear capacity of lightweight concrete members. This reduction factor is also used in the LRFD Specifications.

The difference in the shear resistance factors for lightweight and normal weight concrete may seem minor. However, it was found that the use of a special shear resistance factor for lightweight concrete combined with the reduction factor applied to the concrete contribution for shear design of lightweight concrete, has resulted in a significant reduction in shear resistance for lightweight concrete members designed using the AASHTO LRFD Specifications when compared to designs performed using the AASHTO Standard Specifications. The reduced shear capacity for lightweight concrete members requires an increased shear width for a cross-section and/or an increased quantity of shear reinforcement. Furthermore, the special shear resistance factor for lightweight concrete not only reduces the shear capacity of the concrete but also reduces the contribution of the shear reinforcement.

For some types of elements, such as prestressed concrete girders, the change was not very significant. However, in other elements, the increase was significant. For example, in one concrete segmental box girder bridge, the use of the special shear reduction factor eliminated lightweight concrete from consideration as a design alternate. In another case, the special shear reduction factor made the use of lightweight concrete uneconomical for precast pier caps for a bridge project in NY. In both projects, the LRFD design eliminated the benefit of the reduced density of the lightweight concrete by requiring an increase in member size or reinforcement to offset the reduced shear capacity.

Since the special shear resistance factor for lightweight concrete was introduced because of insufficient data for a statistical evaluation of the shear resistance factor, the Expanded Shale, Clay and Slate Institute (ESCSI) coordinated the collection of lightweight concrete cylinder compression test results from projects across the US. This data was submitted to Professor Andy Nowak of the University of Nebraska. Combined with a small amount of data from a previous study, a total of 8,889 data points were used by Professor Nowak and his team to perform a statistical analysis of lightweight concrete. The analysis revealed that the statistical parameters for lightweight concrete were similar to, or in some cases better than, those of normal weight concrete (Nowak & Rakoczy, 2010).

As the second step in the evaluation of the shear resistance factor for lightweight concrete, shear test results were compared to shear capacities computed using the AASHTO LRFD Specifications. While a limited quantity of shear test results for lightweight concrete was available, they were sufficient to allow the researchers to conclude that the resistance factor for shear for lightweight concrete could be increased from its current value of 0.7 to a value of 0.8 (Paczkowski & Nowak, 2010).

Based on the work by Professor Nowak, the AASHTO Subcommittee on Bridges and Structures (SCOBS), at their annual meeting in 2011, approved increasing the special shear resistance factor, φ, for lightweight concrete that appears in Section 5.5.4.2.1 of the AASHTO LRFD Specifications from 0.70 to 0.80. The shear resistance factor for normal weight concrete remained unchanged at 0.90. This change has improved the situation for shear design with lightweight concrete, allowing the wider consideration of the material as an option improving the economy of bridges. Shear test results that have recently become available from NCHRP Project 18-15 and an FHWA research project may allow a further reevaluation of the shear resistance factor for lightweight concrete.

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Prediction of SCC Formwork Pressure[1]

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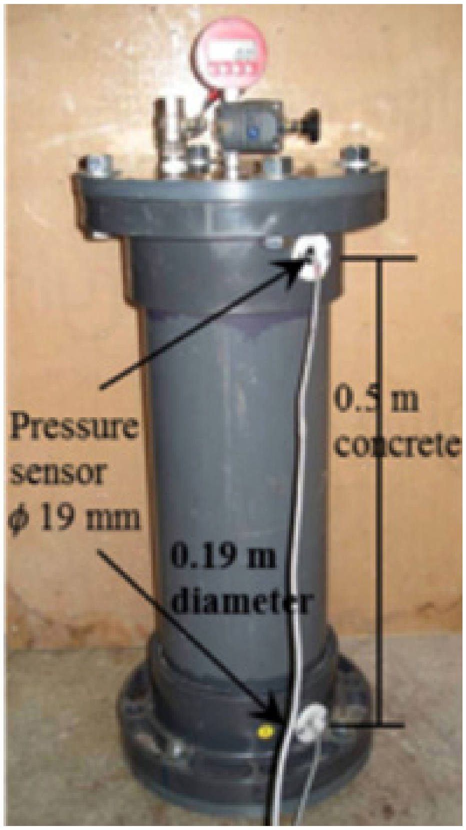


Pressure device to simulate SCC formwork pressure.

The use of SCC enables rapid placement and labor savings; however, lack of information regarding formwork pressure variations during casting and pressure decay after placement has prompted contractors and engineers, as recommended by ACI 347 (Guide to Formwork for Concrete), to design SCC formwork for full hydrostatic pressure leading to cost increase. This article summarizes an extensive project aimed at developing formulation expertise and guidelines to better understand lateral pressure of SCC. This included the development of test methods to predict lateral pressure exerted by SCC and field tests to assess rheological properties of SCC. The study investigated the role of material constituents, casting characteristics, and formwork geometry on SCC form pressure.

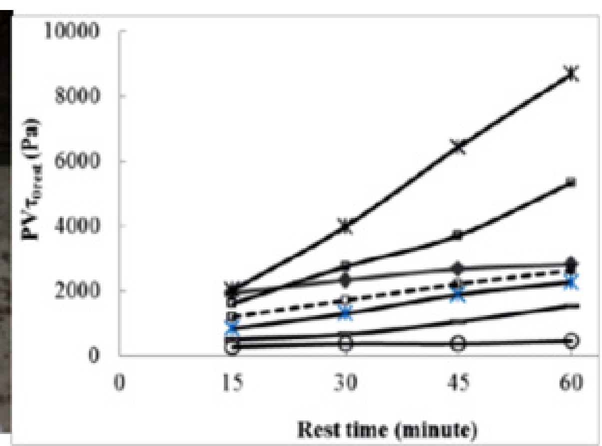
Pressure device to simulate SCC formwork pressure[2]

The device is equipped with two flush-mounted pressure sensors. It is filled with 0.5 m of concrete at given rate, and then air pressure is introduced, at the same rate, to simulate pressure increase up to 13 m. The device is used to monitor lateral pressure during casting and early pressure decay during plastic stage (2 to 3 h). The device reflects the influence of mix design and enables prediction of lateral pressure on elements cast at rates of 2 to 22 m/h.

Field-tests to evaluate structural build-up at rest of SCC[3]

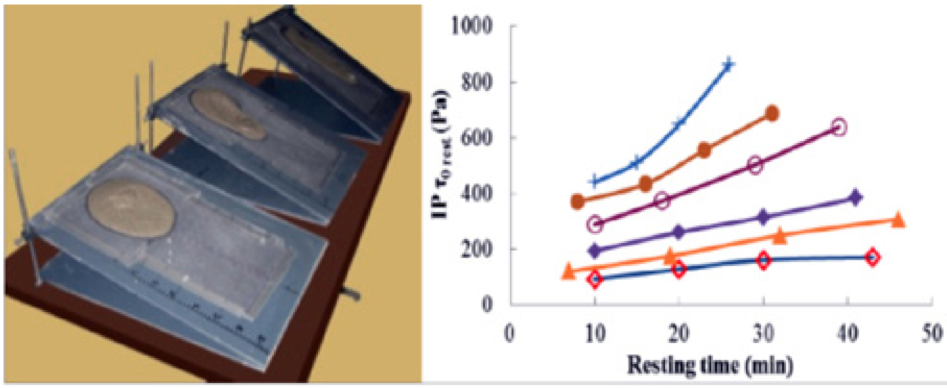
Six field-oriented tests were developed to evaluate structural build-up at rest (thixotropy) of concrete, which significantly affects lateral pressure characteristics. Among these tests, the portable vane (PV) and inclined plane (IP) tests showed good repeatability, low relative error, and comparable results to measurements obtained from concrete rheometer.

The PV test consists of four sets: each has cross-shaped vane that is centered vertically in square container before filling the container with concrete up to the total vane's height. The concrete in the four containers is maintained undisturbed for 15, 30, 45, and 60 min, respectively, before measuring the maximum torque (T in N.m) to shear the material. T values are converted to static shear stress (T_{0rest}) using vane's geometry G [τ_{0rest}]. The T_{0rest} at 15min [$PVT_{0rest@15min}$] or change in shear with time [$PV\tau_{0rest}$ (t)] or $PV\tau_{0rest@15min} x PV\tau_{0rest}$(t) is used to quantify structural build-up at rest. The IP method involves casting concrete in a cylindrical mold onto a horizontal plate of a given roughness, then lifting the plate to initiate flow of the material. The angle (α) necessary to initiate flow is used to determine τ_{0rest} [$IP\tau_{0rest} = \rho.g.h.sin\alpha$, where ρ: material density, g: gravitation constant, h: characteristic height of spread sample]. Four tests are



Buckets and vanes used in portable vane test. Variations of static yield stress at rest with time

Figur



Inclined plane test at different rest times. Variations of static yield stress at rest.

performed after different rest periods to evaluate shear growth at rest .

Experimental investigation for parameters affecting SCC formwork pressure[4, 5]

A comprehensive testing program was undertaken to evaluate key parameters affecting formwork pressure exerted by SCC. The investigated factors included mix design (slump flow, dosages of HRWRA and VMA, volume of coarse aggregate, sand-to-total aggregate ratio, paste volume, nominal maximum size of aggregate MSA); casting characteristics (depth, placement rate, concrete temperature, and waiting period between consecutive lifts WP); and minimum formwork dimension d. The pressure device and the PV and IP test methods were employed in the testing program.

Prediction models for SCC formwork pressure

The shear growth at rest of SCC is used to estimate the maximum lateral pressure (Pmax), as shown below.

$$P_{max} = \frac{\gamma_c H}{100}\left(98 - 3.82\,H + 0.63\,R + 11\,D_{min} - 0.021\,PV\tau_{0rest@15\,min@Ti}\right) f_{MSA} \times f_{WT} \leq \gamma_c H \qquad (R^2 = 0.94)$$

where;

P_{max} : maximum lateral pressure, KPa (0 – 350 kPa)

γ_c : concrete unit weight (18 - 26 kN/m^3)

H : height of placement, m (1 – 13 m)

R : rate of placement, oC (2 - 30 m/h)

D_{min} : Equivalent parameter to d, m

- d < 0.2 m, D_{min} = 0.2 m
- 0.2 < d < 0.5 m, D_{min} = d
- 0.5 < d < 1.0 m, D_{min} = 0.5 m

f_{MSA} : factor depending on MSA

- SCC with MSA of 10 mm and $PV\tau_{0rest@15min@22^oC} \leq 700$ Pa; $1 \leq f_{MSA} \leq 1.10$ for $4 \leq H \leq 13$ m
- SCC with MSA of 14 and 20 mm; $f_{MSA} = 1$

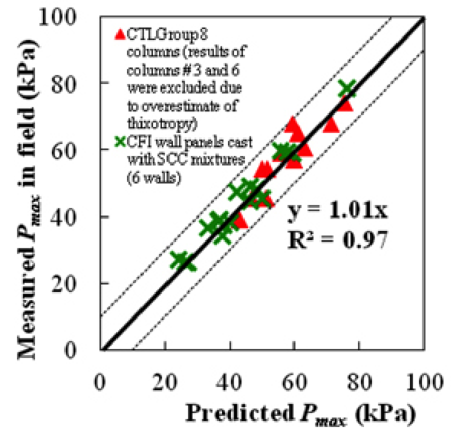
f_{wp} : factor accounting for delay between successive lifts:

- fwp = 1 for SCC with any thixotropic level cast continuously.
- wp = 1 - 0.85 for SCC with $PV\tau_{0rest@15min@22^oC}$ = 50 - 1000 Pa, respectively, when placement interrupted with 30-min waiting period in the middle of casting period

$PV\tau_{0rest@15min@Ti}$: at a given concrete temperature (Ti) (0 – 2000 Pa)

The predicted-to-measured P_{max} results of using this above equation lie within 90% confidence interval

Validation of model



Measured P_{max} values monitored using the pressure sensors mounted at different casting depths of six wall panels with heights of 3.7 and 4.4 m, widths of 0.2 m, and lengths of 5.5 m cast at the Université de Sherbrooke, QC, Canada as well as eight column elements with diameters of 0.61 m, heights of 3.66 m cast at CTLGroup, Skokie, Il, USA are compared to predicted values. The comparison yielded excellent correlation

Conclusions

An approach based on the measurement of the structural buildup at rest of SCC measured using field-oriented test methods can be used to estimate form pressure exerted by SCC. The study shows good correlation with field results and contributes to growing need to update form pressure prediction equations for flowable concrete that assume full hydrostatic pressure.

1. The project was sponsored by RMC Research & Education Foundation and the Strategic Development Coun-

cil of the American Concrete Institute.

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Manette Bridge 303/4A Replacement

Paul Kinderman, Washington State DOT

The Manette Bridge 303/4A replacement project replaced an 80-year-old bridge consisting of five truss main spans and eight plate girder approach spans across the Port Washington Narrows in Bremerton, Washington. The replacement bridge is 1,550 feet long, carrying two traffic lanes, two five-foot-wide shoulders, and one 12-foot-wide bike/pedestrian path.

The new bridge is a seven-span, continuous, prestressed/post-tensioned splice girder design, supported by two-column bents founded on drilled shaft foundations. The superstructure consists of five typical 250-foot spans, with end spans of 140 feet and 160 feet. The superstructure design included a unique post-tensioning layout and sequence, which is span-by-span, and staged to allow roadway deck slab placement in some spans, resulting in unique opposing tendon anchorages in the center of the hammerhead segments at the intermediate piers. In addition, the design of the prestressed segments included development of custom I-girder sections that included a parabolic haunch that varied in depth from 6 feet to 12.5 feet, with some segments weighing up to 306,000 lbs (heaviest ever produced by Conc. Technology Corp.). Other significant design features include precast shaft caps founded on 12-foot diameter drilled shafts.

The Manette Bridge in Bremerton, Washington.

The bridge is set in a small town with an historic United States Navy ship yard. The look of the new bridge was driven by architectural details provided after lengthy public input. The main architectural feature is the parabolically haunched precast girders that make up the bridge superstructure. Other details include compass rose motifs on the piers which reference seafaring navigation. The deep green colored railing is chosen to recall the old replaced steel truss. The columns are classic forms rising outboard of the superstructure to embrace pedestrian overlooks. Girder closures are highly detailed with nautical themes and traditional looking brackets.

Key Design Challenges

Bridge aesthetics were very important for this bridge because the surrounding neighborhood had a strong sense that the existing bridge defined their community. They were adamant they did not want a typical "highway" bridge and strongly resisted chorded haunches like a nearby bridge. The budget for the replacement bridge did not allow for a truly signature bridge. The spliced parabolically haunched precast girders provided an esthetically pleasing structure for a reasonable cost.

The new bridge was constructed approximately 3.0 feet from the existing bridge while the existing bridge was kept open to traffic. The exception was on the west end, where the new bridge overlapped the existing bridge. This overlap meant the existing bridge had to be closed to traffic before the new bridge was open. The use of precast girders allowed this closure to be minimized.

Innovations/Accomplishments

The precast spliced girders utilize a truly parabolic shape for the girder soffit. The top of the bottom flange also has a parabolic shape. This results in a varying web and bottom flange thickness over the entire length which adds to aesthetics of the bridge.

Each girder segment type is identical to each other. For example all of the 24 hammerhead girders are the same. Each of the drop-in girders are also the same. This allowed economy in forming the girders.

The hammerhead girders are the largest single product produced by Concrete Technology Corporation (by weight).

The hammerhead girders, have opposing P.T. anchorages at each pier. This allowed each span to be post-tension individually which reduced losses and allowed each girder to be the same. This also allowed most of the bridge to be post-tensioned and deck placed before constructing the last span which overlapped with the existing bridge. This reduced the bridge closure time for the community.