MYCONCRETE

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

Volume 13, Issue 8 | August 2022

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MyConcrete:
The Bulletin of the American Concrete Institute – Malaysia Chapter

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Published in Malaysia by
American Concrete Institute - Malaysia Chapter
70-1, Jalan PJS 5/30, Petaling Jaya Commercial City (PJCC),
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INTRODUCTION TO ACI MALAYSIA CHAPTER

American Concrete Institute - Malaysia Chapter (ACI-Malaysia) is a non-profit technical and educational society representing ACI Global in Malaysia, which is one of the world's leading authorities on concrete technology. Our members are not confined to just engineers; in fact, our invitation is extended to educators, architects, consultants, corporate, contractors, suppliers, and leading experts in concrete related field. The purpose of this Chapter is to further the chartered objectives for which the ACI was organized; to further education and technical practice, scientific investigation, and research by organizing the efforts of its members for a non-profit, public service in gathering, correlating, and disseminating information for the improvement of the design, construction, manufacture, use and maintenance of concrete products and structures. This Chapter is accordingly organized and shall be operated exclusively for educational and scientific purposes.

Objectives of ACI-Malaysia are:

❖ ACI is a non-profitable technical and educational society formed with the primary intention of providing more in-depth knowledge and information pertaining to the best possible usage of concrete.

❖ To be a leader and to be recognized as one of Malaysia's top societies specializing in the field of concrete technology by maintaining a high standard of professional and technical ability supported by committee members comprising of educators, professionals and experts.

❖ Willingness of each individual member/organization to continually share, train and impart his or her experience and knowledge acquired to the benefit of the public at large.

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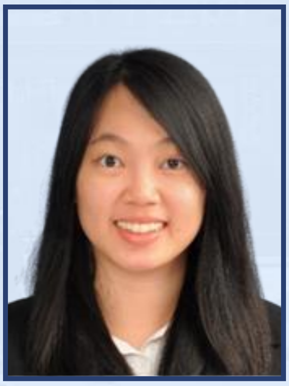
Ms. Serina Ho Chia Yu, Head of Editorial Committee

Ms. Serina holds a Chemistry degree from University of Malaya and a Master of Business Administration from University of Hull, UK. She is the Chairman of Technical Committee in Cement & Concrete Association Malaysia (C&CA) and the committee member of Technical Committee on Cement in Standard Malaysia. She is also a member of Malaysian Institute of Chemistry (IKM) and Past President of American Concrete Institute, Malaysia Chapter (ACI-Malaysia Chapter).

Serina has a vast experience in both cement and concrete industry. She started her career as QA/QC in a ready-mixed company and later ventured out to be the chemist in cement plant, in charge of quality and R&D of cement products. She also worked as Product Manager in ready-mixed company, responsible for developing and marketing of ready-mixed concrete products before she came back to the Cement Industry in 2012 as Technical & Product Development Manager. She is currently the Sustainability Manager in Hume Cement. Her journey in the construction industry is driven by her unrelenting passion for cement and concrete.

Ts. Eric LS Soong, Editorial 1

Eric obtained his Bachelor and Masters in Engineering in Australia in the year 2006 and 2012. He spent some 6 years in environmental engineering consulting in Australia where he gained significant experience in playing leading technical roles in many major complex engineering and environmental projects. On returning to Malaysia in 2013, Eric took on a leading role in managing and leading the technical advisory support to the Sales and Marketing Team in the decorative concrete industry. Over the last 8 years, Eric has worked proactively with the manufacturing arm to develop environmentally friendly concrete products and systems.



Dr. Leong Geok Wen, Editorial 2

Dr. Leong obtained her Bachelor Degree in Civil Engineering (Hons) from Universiti Malaya (UM) in 2018 and was honoured with Board of Engineers Malaysia (BEM) Excellence Award session 2017/2018. She then continued her PhD studies in the same university and graduated in 2022. Currently she is an academic staff in UM and her research area is on lightweight concrete, high strength concrete and fibre reinforced concrete.

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i) ACI Malaysia is only a platform for our members to advertise for interns.

ii) All application to be made direct to companies and would be subject to their terms and conditions.

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CERTIFICATION (level 1)

4 AUGUST 2022

8:00 AM - 5:00 PM

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Preceding Events

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BUILDING DEFECTS, REPAIR AND MAINTENANCE: FLOOR TRAP

MS. LOH POH YEE (SPEAKER)
Epsilon Enterprise

PohYee graduated with a Bachelor's degree in Electronics Engineering from Multimedia University and is currently a PhD student in Material Engineering from the Universiti Malaya. She is active in a multidiscipline career related to building such as building inspection and repair related to water leakage issues especially for residential units. She investigates water leakages from different sources inside the building and its facades.
Floor trap is one of the major components in floor design for wet areas. In this session, she will share her knowledge about the floor traps, problems related to and proper maintenance.

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25 AUGUST 2022 (THURSDAY)
8.30 PM - 9.30 PM

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Preceding Events

Reprint from CI Magazine, Volume 42, No 6, Page 30-32

Let There Be Light

Viettel Offsite Studio in Hanoi was strategically designed to let in light and nature

by Deborah R. Huso

Few materials demonstrate how simple design and construction can create startlingly beautiful places as well as concrete. This is especially true for the newly completed office space and pavilions for the Vietnamese telecommunications group Viettel on the outskirts of Hanoi, Vietnam.

Through a striking use of concrete walls placed to create a series of V-shaped rooms facing walls of glazing, the structure—known as the Viettel Offsite Studio-establishes a refuge from the city and guides the eye toward green space (Fig. 1).

Amplifying Nature

Designed by Vo Trong Nghia Architects (VT Architects), which has offices in both Hanoi and Ho Chi Minh City, Vietnam, the 1427 m_2 (15,360 ft_2) Viettel Offsite Studio uses geometry to amplify visitors' experience of the building's natural surroundings. Situated on the Viettel Academy campus in Hanoi's Thạch Thất district about 30 km (18.6 miles) from the city center, the building is mainly meeting space and retreat for the company's executives. The building's structure and façade are concrete and glass (Fig. 2), while the interior showcases tall walls of glazing and spaces designed with metal and wood (Fig. 3).

Defined by six V-shaped structures that open like books to a lake and garden view (Fig. 4), the facility contains four indoor studios, a reception space (Fig. 5) and a dining hall. Opening to the north, the interconnected rooms take advantage of soft daylighting. The terraced roof gardens serve as outdoor studio spaces. While the tall concrete walls shield the courtyards and roof gardens from the harsher sunlight of morning and afternoon, small openings in the walls provide

Fig. 1: The Viettel Offsite Studio on the Viettel Academy campus in Thạch Thâ ́ t district, Hanoi, Vietnam

gentle lighting and breezes for the occupants (Fig. 6 and 7). Vo Trong Nghia, Principal at VTN Architects, says his firm's first purpose was "to design a building with open spaces to take advantage of the site's surrounding landscape, such as hills, trees, and a big lake."

Fig. 2: Concrete and glass are the main components of the building's structure and façade

Fig. 3: One of the meeting spaces at the Viettel Offsite Studio

Fig. 4: V-shaped concrete structures open like books to a lake and garden view

Fig. 5: A reception area at the Viettel Offsite Studio

Fig. 6: One of the terraced rooftop gardens at the Viettel Offsite Studio

Fig. 7: Stepped square cutouts in the slabs allow some direct sunlight to pass onto the courtyards created by the V-shaped units

Segmenting Space

The V-shaped concrete façades pitch into one another at slight angles to create a triangular apex at the top of the connecting walls. Each apex shields the entrance to the enclosed space to the north. The overall effect is an architecture resembling the brutalist style of the mid-twentieth century. Each set of bookended walls exhibits differing heights to create what Nghia calls "a rhythm that blends into the beautiful landscape of the faraway hills and mountains." This "open-book" design compels the structure's occupants to focus their attention on the outdoors. The V-shaped blocks follow the lay of the land and are all connected by a singlestory open corridor (Fig. 8).

Made of cast-in-place concrete, each wall is 450 mm (18 in.) thick. The maximum wall height is 30 m (98 ft). Nghia indicates the design team employed cast-in-place concrete

Fig. 8: A single-story open corridor connecting V-shaped buildings at the Viettel Offsite Studio

intentionally from the start. "Precast concrete would have provided a better quality of concrete," he adds, "but it cannot provide high units." Cast-in-place concrete also allowed for rigid connections between placements.

Nghia also points to the uniqueness of the concrete itself:"We used raw concrete with unfinished layers," he explains. While such finish is very popular in developed countries like Japan, Korea, and the United States, it is quite rare in Vietnam. The design team's goal was to use raw concrete to emulate the rustic, natural experience of the building while also reducing long-term maintenance needs.

While the total project construction time took about a year and a half, erection of the exterior structure took only about 10 months, according to VTN Architects. The building was opened for business in July 2019.

Acknowledgment

All photos courtesy of Vo Trong Nghia Architects.

Selected for reader interest by the editors.

Deborah R. Huso is Creative Director and Founding Partner of WWM, Charlottesville, VA. She has written for a variety of trade and consumer publications such as Precast Solutions, U.S. News and World Report, Concrete Construction, and Construction Business Owner. She has provided website development and content strategy for several Fortune 500 companies, including Norfolk Southern and GE.

TECHNICAL REPORT

Reprint from CI Magazine, Volume 42, No 9, Page 43-46

Serviceability of Concrete Elements with High-Strength Steel Reinforcement

by Alana Lund, Aishwarya Y. Puranam, Ryan T. Whelchel, and Santiago Pujol

The increased availability of steel reinforcing bars with yield stress f_y larger than 80 ksi has drawn the attention of the civil engineering profession.[1] In addition to increasing the flexibility of the design process, high-strength steel reinforcement (HSSR) can help reduce congestion in applications related to earthquakes or blast scenarios where increased strength may be required.[2] As the profession moves toward the general use of HSSR, it is necessary to implement simple constraints and let designers decide when and if HSSR may benefit their projects.

This study investigates the reliability of provisions for service-level deflection and bar spacing in ACI 318-19[3] in relation to the use of HSSR. The issue of serviceability belongs in the realm of linear response and is therefore more closely related to service stress f_s than to f_y. Nevertheless, current provisions related to serviceability enforce assumptions on the service-level stress of the elements by direct reference to yield stress. Thickness minima, for example, are set with the often-implicit assumption that $f_s = (2/3)f_y$. Although elements reinforced with smaller amounts of higher-grade steel are expected to experience larger service stresses, this may not always be the case. In applications requiring high strength for transient demands, for example, service-level bar stress can be much smaller than $2f_y/3$. To accommodate cases in which service stress caused by sustained loads may be smaller than $2f_y/3$, and to allow the use of bars with $f_y > 80$ ksi without prohibitive increases in thickness, thickness minima could instead be expressed in terms of f_s with a lower bound on allowable thickness. The minimum proposed thickness h is given by Eq. (1):

$$h / h_{ref} = 0.4 + 3f_s / 200 \quad (1)$$

where h_{ref} is total thickness of an element with Grade 60 reinforcement ($f_y = 60$ ksi) and f_s is the working stress of the higher-grade steel in units of ksi. Note that our study focuses on the application of Eq. (1) for specimens reinforced with Grade 120 reinforcement. The definition of a particular lower bound for allowable thickness is suggested as future work to support the implementation of this thickness minimum.

Equation (1) is based on experience and follows expressions for minimum thickness of slabs in previous editions of ACI 318.[4] Because of uncertainties associated with parameters that affect deflections, such as concrete modulus of elasticity, crack and curvature distribution, modulus of rupture, and the long-term effects of creep and shrinkage, the application of serviceability limits to HSSR requires vetting against test data. Although the proposed change requires the determination of service stress for the element, we do not anticipate that calculating these stresses will be a stumbling block to designers. Service stress simply refers to service level stress in the longitudinal reinforcing bars before undergoing changes caused by creep and shrinkage, and its estimation should not be outside the designer's reach. This change is also unlikely to open the door for thinner slabs with excessive deflections. Current practice favors calculation over use of the thickness minima, as calculations often lead to smaller thickness values. Nevertheless, our study shows that the alternative thickness minima (Eq. (1)) may be preferable because conventional deflection calculations are not consistently reliable.

Experiments were conducted on one-way slab elements reinforced with either conventional steel or HSSR. The results from these tests, in combination with results from four previous investigations,[5-8] are used to address whether:

- Equation (1) produces reliable results for elements with HSSR;
- Common methods for estimating deflections produce acceptable results for elements with HSSR; and

- Limiting bar spacing s using the expression $s \leq 600 / f_s - 2.5 \times cover$ would be sufficient to avoid intolerable crack widths (f_s in ksi, and s and *cover* in in.).[3]

Experimental Investigation

Three series of tests, each consisting of a pair of one-way slabs, were conducted. Properties of all specimens are listed in Table 1, and their cross sections are shown in Fig. 1. The slabs were reinforced in the longitudinal (span) direction with No. 3 deformed bars. No reinforcement was provided in the transverse direction. The main variables studied were h, which ranged from 4 to 8 in.; reinforcement ratio 〉, which ranged from 0.10 to 0.73%; and reinforcing bar grade (either ASTM A615/A615M Grade 60 or ASTM A1035/A1035M Grade 120).

Setup and instrumentation

All specimens spanned 12 ft between two roller supports and had 1 ft overhangs past each support (Fig. 2). The clear span of each specimen was supported with shoring prior to testing to reduce preliminary deflections. The specimen were tested in four-point loading with concentrated loads applied 3 ft from each support. Four hydraulic rams were used to apply loads (two at each location of concentrated load), and they were connected to a common pump. The total weight of the equipment at each load point was 270 lb.

Deflection at midspan was measured using a linear variable differential transformer and a string potentiometer. Applied load was measured using a load cell placed under one of the two hydraulic rams at each load point. Mean strains in the reinforcing bars were inferred from the average of surface deformations measured at the level of the reinforcing bars along the constant-moment region. In Series 1 tests, surface deformations were measured by tracking optical targets (Optotrak™) at gauge lengths of 6 in. in the middle 48 in. of the span. In Series 2 and 3 tests, surface deformations were measured using Whittemore gauges, with targets affixed to the specimens at gauge lengths of 5 in. in the middle 60 in. of the span. The instruments and their accuracies are detailed in the dataset by Lund et al.[9]

Table 1:
Slab properties

<table>
<tr><th colspan="2" rowspan="2">Slab ID</th><th colspan="4">Geometry</th><th colspan="3">Reinforcement</th><th colspan="3">Material properties</th></tr>
<tr><th>b, in.</th><th>h, in.</th><th>d, in.</th><th>L, in.</th><th>db, in.</th><th>nb</th><th>ρ, %</th><th>fy, ksi</th><th>fc′, psi</th><th>Ec, ksi</th></tr>
<tr><td rowspan="2">Series 1</td><td>S1-120</td><td rowspan="6">30</td><td>8</td><td>7</td><td rowspan="6">144</td><td rowspan="6">3/8</td><td>4</td><td>0.21</td><td>135</td><td rowspan="2">9300</td><td rowspan="2">5500</td></tr>
<tr><td>S1-60</td><td>5</td><td>4</td><td>8</td><td>0.73</td><td>75</td></tr>
<tr><td rowspan="2">Series 2</td><td>S2-120</td><td>6</td><td>5</td><td>3</td><td>0.22</td><td>135</td><td rowspan="4">8000</td><td rowspan="4">5600</td></tr>
<tr><td>S2-60</td><td>4</td><td>3</td><td>6</td><td>0.73</td><td>66</td></tr>
<tr><td rowspan="2">Series 3</td><td>S3-120</td><td>8</td><td>7</td><td>2</td><td>0.10</td><td>135</td></tr>
<tr><td>S3-60</td><td>5</td><td>4</td><td>5</td><td>0.46</td><td>75</td></tr>
</table>

b = section width; h = section depth; d = effective depth of section; L = clear span length; d_b = diameter of longitudinal bar; n_b = number of longitudinal bars; ρ = reinforcement ratio; f_y = yield stress of reinforcement; f_c' = concrete compressive strength; E_c = modulus of elasticity

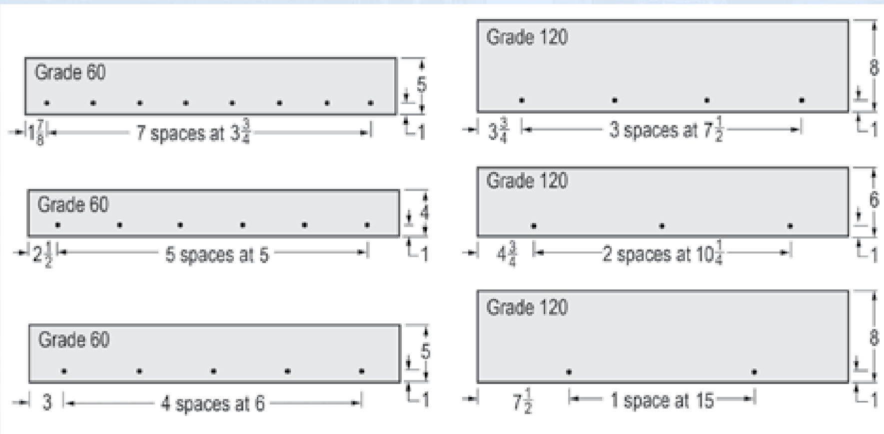

Fig. 1: Slab specimens were 30 in. wide and reinforced with No. 3 bars: (a) Series 1; (b) Series 2; and (c) Series 3 (all dimensions in in.)

Testing

Each slab was loaded to an initial mean service strain in the reinforcing bars, as estimated from the average of the concrete surface deformations measured at the level of the bars, of $2f_y / 3E_s$, where f_y is either 60 or 120 ksi and E_s is the modulus of elasticity of steel, taken as 29,000 ksi. The load corresponding to this strain was held constant (±100 lb) for at least 150 days. During this period, surface strain, midspan deflection, and load were monitored and recorded regularly. Measured load-deflection relationships for both initial and long-term loading are shown in Fig. 3. All measurements were made relative to the initial deflection caused by self- and equipment weight. Reported deflections are therefore increases in deflection caused by the applied load only. All data from these tests, including measurements and recorded media, are available in Lund et al.[9]

Analysis and Discussion of Results

Elements reinforced with conventional steel or HSSR were tested to investigate the use of Eq. (1) for setting minimum thickness of flexural elements, to evaluate expressions for predicting service-level deflections, and

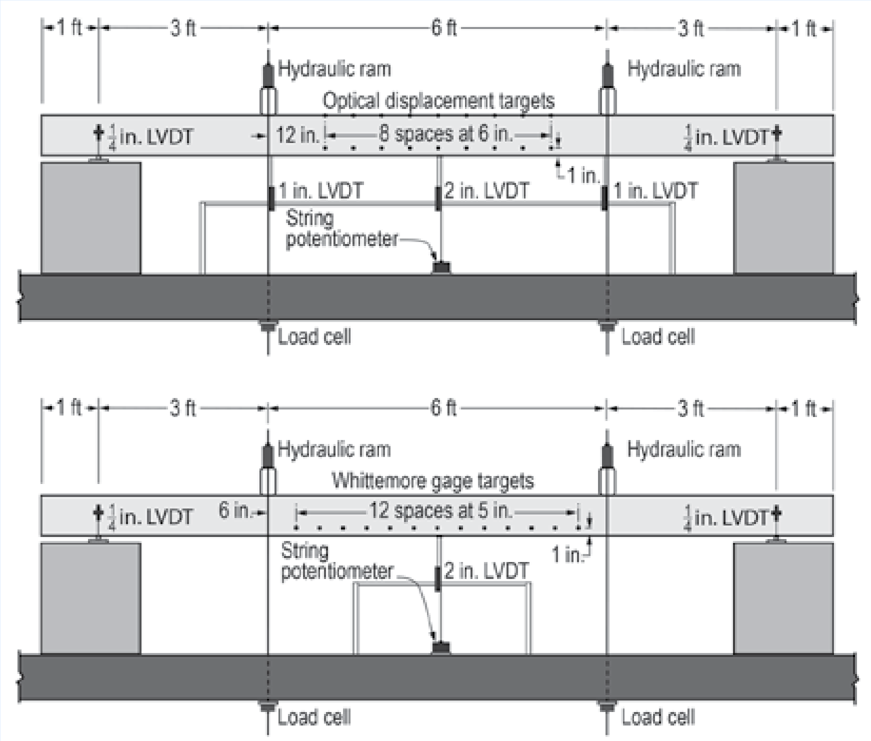

Fig. 2: Experimental setup: (a) Series 1; and (b) Series 2 and 3

to determine the applicability of provisions for bar spacing.

Minimum thickness

Results of the experiments described in the previous section are first used to investigate the applicability of the thickness minima represented by Eq. (1). Two sets of comparisons are drawn based on estimates of reinforcing bar stress from either specimen properties or observed surface strains at the level of the reinforcement.

For the first set of comparisons, deflection measurements for each specimen were recorded at loads corresponding to values of bar stress selected between 30 and 100 ksi. Loads associated with these stresses are calculated using measured material properties and assuming cracked sections and linear response. Deflections for each calculated load are

Relative deflections are defined herein as the ratio of the deflections measured at the calculated target loads to the deflection of a reference specimen with conventional steel operating at *fs* of 40 ksi, where specimen S3-60 is chosen as the reference. This ratio is plotted against the ratio of required-to provided depth in Fig. 4, with corresponding values listed in Table 2. Required depth is taken as the product of 0.4 + 3*fs* / 200 (Eq. (1)) and the depth of Specimen S3-60. The figure shows that when the provided depth exceeded the required depth—that is, the value on the x-axis is smaller than 1—then the measured deflection did not exceed the deflection of the reference specimen at 40 ksi. It also shows the converse relationship. The comparison supports the use of Eq. (1) for *fs* up to about 100 ksi (in the case of HSSR). Figure 5 is analogous to Fig. 4 but

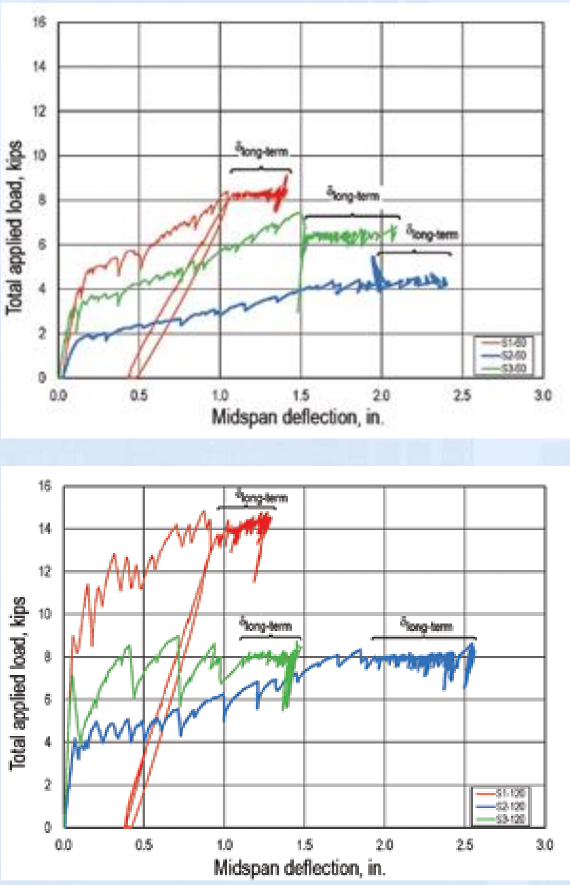

Fig. 3: Load-displacement curves for full history of serviceability loading: (a) specimens with Grade 60 reinforcing bars; and (b) specimens with Grade 120 reinforcing bars

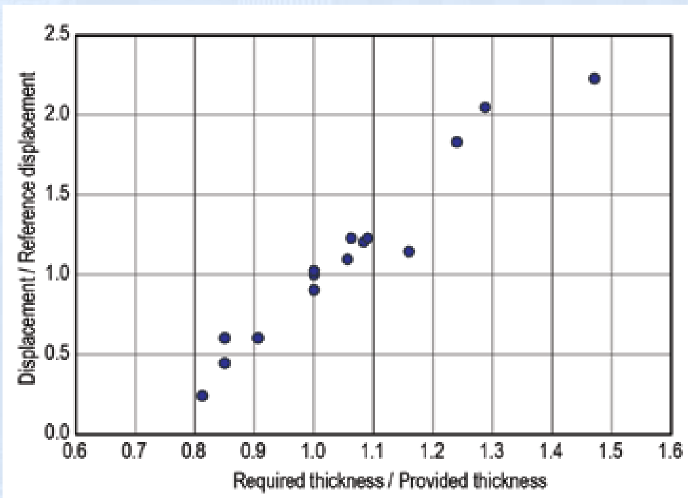

Fig. 4: Short-term displacement (calculated stresses) versus thickness

examines long-term (5 months) rather than initial deflections— corresponding values listed in Table 3. The data suggest that Eq. (1) can be used even when the long-term effects of creep and shrinkage are considered. In relation to the data presented in Table 2:

- In the assessment of Eq. (1) using this first comparison, self-weight is not included in the calculation of reinforcing steel stresses; and

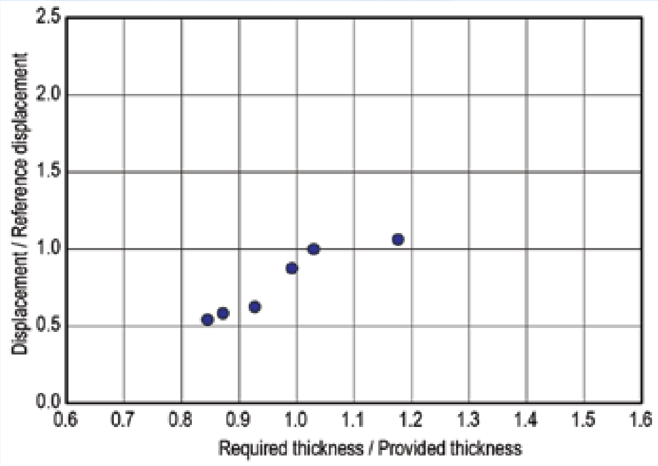

Fig. 5: Long-term deflections (calculated stresses) versus thickness

- During initial testing, loading was stopped to measure cracks and document the state of the specimen. Loads decreased during these stops, so a single value of load is not always associated with a single value of deflection. For load drops not exceeding 30% of maximum loads, the maximum deflection associated with a given load is reported in Table 2. Load drops exceeding 30% coincided with crack initiation.

To confirm the results generated from calculated stresses with those observed experimentally, a second comparison is made in which Eq. (1) is evaluated using stresses estimated from measured mean strains. Bar stress is estimated at each measured displacement increment as mean strain times an elastic modulus of 29,000 ksi, where mean surface strain is calculated from optical or Whittemore gauge measurements. For each mean stress-strain pair, relative deflections are determined as the ratio of deflection occurring at the instant the strain was measured to the deflection of a reference specimen (S3-60) at a mean bar stress of 40 ksi (also inferred from measured mean strain). The remaining steps are the same as in the first comparison. The results of this procedure, shown in Fig. 6, suggest a similar relationship between the deflection ratio and depth ratio as in the previous comparison, though in this case the specimens in which the provided depth was smaller than the required depth (from Eq. (1)) exceeded the displacement of the reference specimen to a lesser degree.

Table 2:
Instantaneous deflection data (used to produce Fig. 4)

Specimen	f_s, ksi	Total applied load, kip	h, in.	d, in.	Meas. δ, in.	δ_{rel}	h_{rel}	$0.4 + 3f_s / 200$	h_{req} / h
S3-120	97	8.0	8	7	1.0	1.14	1.60	1.86	1.16
S3-120	80	6.6	8	7	0.8	0.90	1.60	1.60	1.00
S3-60	56	6.4	5	4	1.5	1.83	1.00	1.24	1.24
S3-60 REF	40	4.6	5	4	0.8	1.00	1.00	1.00	1.00
S3-60	30	3.4	5	4	0.4	0.45	1.00	0.85	0.85
S2-120	91	8.0	6	5	1.9	2.23	1.20	1.77	1.47
S2-120	60	5.2	6	5	1.0	1.20	1.20	1.30	1.08
S2-60	42	4.2	4	3	1.7	2.05	0.80	1.03	1.29
S2-60	30	3.0	4	3	1.0	1.23	0.80	0.85	1.06
S1-120	86	14	8	7	0.9	1.10	1.60	1.69	1.06
S1-120	70	11.4	8	7	0.5	0.60	1.60	1.45	0.91
S1-120	60	9.8	8	7	0.2	0.24	1.60	1.30	0.81
S1-60	46	8.3	5	4	1.0	1.23	1.00	1.09	1.09
S1-60	40	7.2	5	4	0.9	1.02	1.00	1.00	1.00
S1-60	30	5.4	5	4	0.5	0.60	1.00	0.85	0.85

f_s = stress in longitudinal reinforcing bars (calculated for the listed applied load); h = thickness; d = effective depth; δ = displacement at midspan; δ_{rel} = relative displacement (δ / δ_{ref}); h_{rel} = relative thickness (h / h_{ref}); h_{req} / h = ratio of required-to-provided thickness

Note: Total load refers to the sum of loads applied at both loading locations. Self-weight was neglected in the calculation of reinforcing steel stresses

For load drops not exceeding 30% of maximum loads, the maximum deflection associated with a given load was chosen for this table. Including selfweight in the calculation of f_s does not change the trend in Fig. 4.

Table 3:
Long-term deflection data (used to produce Fig. 5)

Specimen	f_s, ksi	Total applied load, kip	*h*, in.	*d*, in.	Meas. δ, in.	δ_{rel}	h_{rel}	$0.4 + 3f_s / 200$	h_{req}/h
S3-120	97	8.0	8	7	1.5	0.63	2.00	1.86	0.93
S3-60	56	6.4	5	4	2.1	0.88	1.25	1.24	0.99
S2-120	91	8.0	6	5	2.6	1.06	1.50	1.77	1.18
S2-60 REF	42	4.2	4	3	2.4	1.00	1.00	1.03	1.03
S1-120	86	14	8	7	1.3	0.54	2.00	1.69	0.85
S1-60	46	8.3	5	4	1.4	0.58	1.25	1.09	0.87

Note: Total load refers to the sum of loads applied at both loading locations. Self-weight was neglected in the calculation of reinforcing steel stresses
For load drops not exceeding 30% of maximum loads, the maximum deflection associated with a given load was chosen for this table

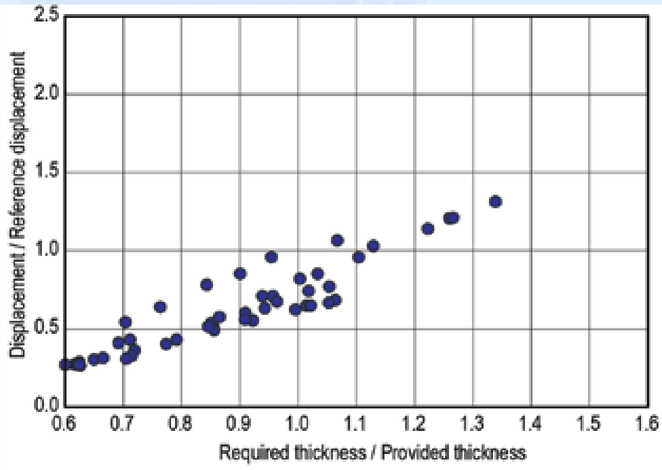

Fig. 6: Short-term displacements (measured average strain) versus thickness

Figures 4 through 6 show that Eq. (1) leads to acceptable immediate and long-term deflections for one-way slabs. This conclusion is supported by the analytical work of Desalegne and Lubell,[10] which likewise suggests that previous (ACI 318-14) and current (ACI 318-19) limits suffice to control deflections in elements with HSSR.

Estimating service-level deflection

Service-level deflections in flexural members can be estimated through the geometric relationship between deflection and unit curvature captured by the moment-area theorems that are a common subject of undergraduate courses in mechanics. Two methods for estimating curvature are evaluated herein, and estimated deflections resulting from these methods are compared with the experimental deflections of specimens from the current study and those reported by Puranam[8] (refer to Table 4). In each case, deflections are estimated by calculating the moment of the area under the resulting curvature diagrams. Because deflections were measured relative to the initial deflection caused by self- and equipment weight, deflections are calculated as:

$$\delta_{net} = \delta_{total} - \delta_0$$

where δ_{net} is deflection calculated as the response to applied loading, which is comparable to the experimentally reported deflection; δ_{total} is deflection calculated from all loads, including self-weight, applied, and equipment loading; and δ_0 is deflection calculated from self- and equipment loads using assumed values of E_c of 57,000 (psi) $c\ f\ 2$, with f_c' being measured compressive strength, and the nominal moments of inertia of gross cross sections.

Method 1: Equations for effective moment of inertia by Branson11 and Bischoff12

In the first method, unit curvature ϕ_a is estimated as the ratio of applied moment M_a to the product of E_c and effective moment of inertia I_e. Two common expressions for I_e that account for the effect of cracking and reinforcement on element stiffness are considered: Eq. (3) by Branson,11 used in ACI 318-14; and Eq. (4) by Bischoff,12 used in ACI 318-19, proposed as an alternative to the expression by Branson to obtain reasonable estimates of deflections for elements with small reinforcement ratios.

$$I_e = (M_{cr}/M_a)^3 I_g + \left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \qquad (3)$$

$$I_e = \frac{I_{cr}}{1-\left(\frac{M_{cr}}{M_a}\right)^2\left(1-\frac{I_{cr}}{I_g}\right)} \le I_g \qquad (4)$$

where I_e is effective moment of inertia, M_{cr} is cracking moment and may be estimated as $f_r I_g / 0.5h$, M_a is applied moment, including effects of self- and equipment loads, for the section at which deflections are to be estimated, I_g is gross moment of inertia, I_{cr} is moment of inertia of the fully cracked cross section, f_r is concrete modulus of rupture taken as 7.5 (psi) $c\ f\ 2$, and h is section depth.

Method 2: Idealized moment-curvature relationships

In the second method, unit curvature for a given bending moment, defined as the ratio of strain to neutral axis depth, is obtained by using two idealized moment-curvature relationships (Eq. (5) and (6)). Gross section properties are used up to cracking and, if the applied moment exceeds the

Table 4:

Parameters and measured deflections from current study* and Puranam[8]† (30 in. section width b)

Specimen	N	Shear span, in.	A_s, in.^{2}	s, in.	h, in.	d, in.	Measured		I_{cr}, in.^{4}	M_{cr}, k-in.		Calc. δ_0, in.	Total applied load, kip‖	Meas. δ, in.‖
							f_y, ksi	f'_c psi		calc.‡	meas.§			
S1-60*	4	36	0.88	3.75	5	4	75	9300	53	90	116	0.05	8.3	1.0
	4	36	0.88	3.75	5	4	75	9300	53	90	116	0.05	7.2	0.9
	4	36	0.88	3.75	5	4	75	9300	53	90	116	0.05	5.4	0.5
S1-120*	4	36	0.44	7.5	8	7	135	9300	94	231	224	0.02	14.0	0.9
	4	36	0.44	7.5	8	7	135	9300	94	231	224	0.02	11.4	0.5
	4	36	0.44	7.5	8	7	135	9300	94	231	224	0.02	9.8	0.2
S2-60*	4	36	0.66	5	4	3	66	8000	25	54	70	0.10	4.2	1.7
	4	36	0.66	5	4	3	66	8000	25	54	70	0.10	3.0	1.0
S2-120*	4	36	0.33	10	6	5	135	8000	38	121	121	0.04	7.9	1.9
	4	36	0.33	10	6	5	135	8000	38	121	121	0.04	5.2	1.0
S3-60*	4	36	0.55	6	5	4	75	8000	38	84	96	0.06	6.4	1.5
	4	36	0.55	6	5	4	75	8000	38	84	96	0.06	4.6	0.8
	4	36	0.55	6	5	4	75	8000	38	84	96	0.06	3.4	0.4
S3-120*	4	36	0.22	15	8	7	135	8000	53	215	189	0.02	8.0	1.0
	4	36	0.22	15	8	7	135	8000	53	215	189	0.02	6.6	0.8
S1-60-18-A†	4	36	0.44	7.5	8	7	75	8940	95	227	224	0.02	10.7	0.9
S2-120-18-A†	4	36	0.44	7.5	8	7	135	8230	99	218	215	0.02	18.5	1.6
S1-120-14-A†	4	36	0.33	10	8	7	135	8920	73	227	215	0.02	13.2	1.4
S1-120-09-A†	4	36	0.22	15	8	7	135	8520	52	222	182	0.02	7.5	1.2
S1-60-18-B†	3	72	0.44	7.5	8	7	75	8810	96	225	229	0.02	5.5	0.4
S1-120-14-B†	3	72	0.33	10	8	7	135	8580	75	222	236	0.02	6.7	0.8
S1-120-09-B†	3	72	0.22	15	8	7	135	8550	51	222	236	0.02	4.1	0.4

N = number of load points in flexural bending; A_s = area of steel in section; s = center-to-center reinforcement spacing; I_{cr} = cracked section moment of inertia; δ = measured midspan deflection; δ_0 = calculated initial deflection caused by self- and equipment weight; M_{cr} = cracking moment
‡For a modulus of rupture equal to $7.5\sqrt{f'_c}$ (psi), §Includes effects of self- and equipment weight, ‖Excludes the effect of self- and equipment weight

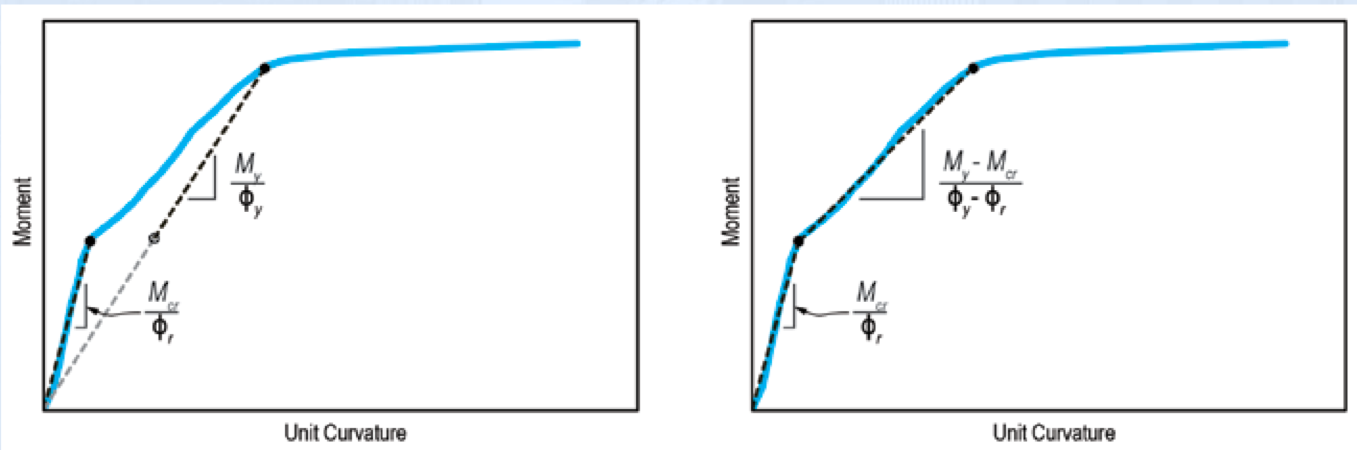

Fig. 7: Moment-curvature relationships for estimating service-level deflections: (a) graphical representation of Eq. (5); and (b) graphical representation of Eq. (6)

cracking moment, unit curvature is estimated assuming: (1) the section is fully cracked in case of Eq. (5), Fig. 7(a); and (2) a gradual transition between initial cracking and yielding in case of Eq. (6), Fig. 7(b):

$\phi a = \phi r\ (Ma / Mcr)$ if $Ma \leq$ Mcr

or

$\phi a = \phi y\ (Ma / My)$ if $Ma > Mcr$ (5)

$\phi a = \phi r\ (Ma / Mcr)$ if $Ma \leq$ Mcr

or

$\phi a = \phi r + [(\phi y - \phi r) / (My - Mcr)]\ (Ma - Mcr)$ if $Ma > Mcr$ (6)

where ϕa is curvature at applied moment Ma, ϕr is curvature at cracking and may be estimated as $Mcr / EcIg$, and ϕy is curvature at yielding and may be estimated as $\varepsilon y / [(1 - k) \cdot d]$. Yield strain ε_y is equal to f_y / 29,000 ksi, k is ratio of depth to neutral axis to effective depth d.

A fundamental difference between Methods 1 and 2 is the determination of ϕ_a. Method 2 is based on calculating M_a at each section along the entire span, as opposed to the determination of ϕ_a in Method 1, which is based on the moment M_a at the section for which deflection is to be calculated. Though this change in approach increases the complexity of the deflection calculation, we found the process accessible and simple to automate as it follows directly from the moment-curvature relationships described in Fig. 7.

Deflections observed in the experiments were compared with those calculated according to Method 1 (Fig. 8) and Method 2 (Fig. 9). The points close to the vertical axis

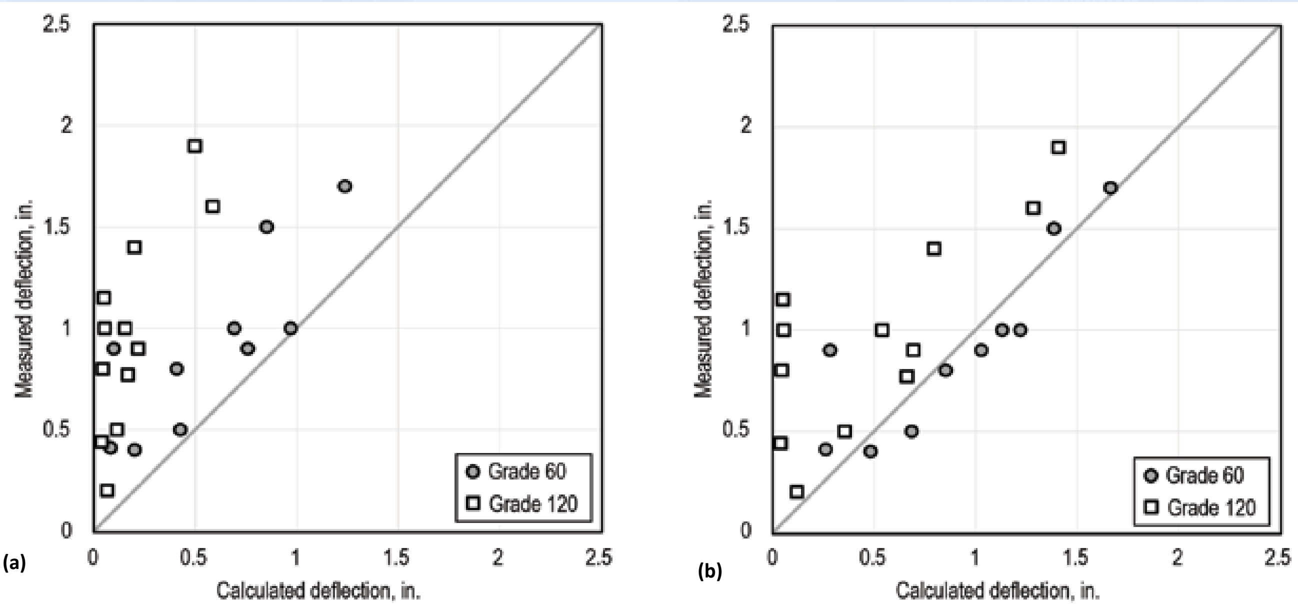

Fig. 8: Estimates of service-level deflections in slabs: (a) using Branson,[11] Eq. (3); and (b) using Bischoff,[12] Eq. (4)

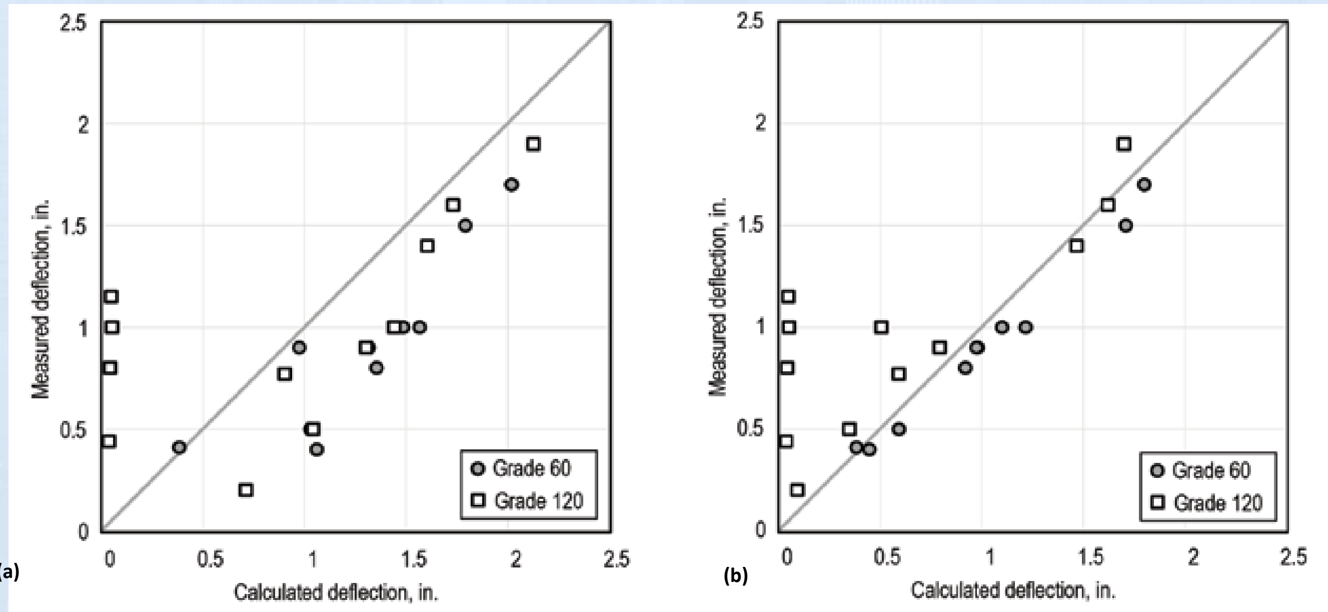

Fig. 9: Estimates of service-level deflections in slabs using mechanics and geometry: (a) using Eq. (5), Fig. 7(a); and (b) using Eq. (6), Fig. 7(b)

represent cases in which cracking occurred earlier than expected on the basis of the assumed modulus of rupture. Figure 8 shows that using the expression by Branson[11] results in underestimation of deflections for all elements within the range of variables considered here. The expression by Bischoff,[12] which is the basis for the latest design provision in ACI 318-19, produces better and safer estimates. Figure 9 shows that even more reliable control of service-level deflections can be achieved by using approximations based on simple mechanics and geometry (Method 2). Equation (5) produced the most conservative results of all the approaches studied herein. Equation (6) provided the best match between measurements and calculations.

Recent updates in ACI 318-19 suggest an alternative approach to deflection control through the use of a reduced cracking moment of $(2/3)M_{cr}$. A comparison of the results from this approach with measurements of experimental deflections is shown in Fig. 10 and 11.

Though the observed cracking moments were similar to the calculated cracking moments, as shown in Table 4,

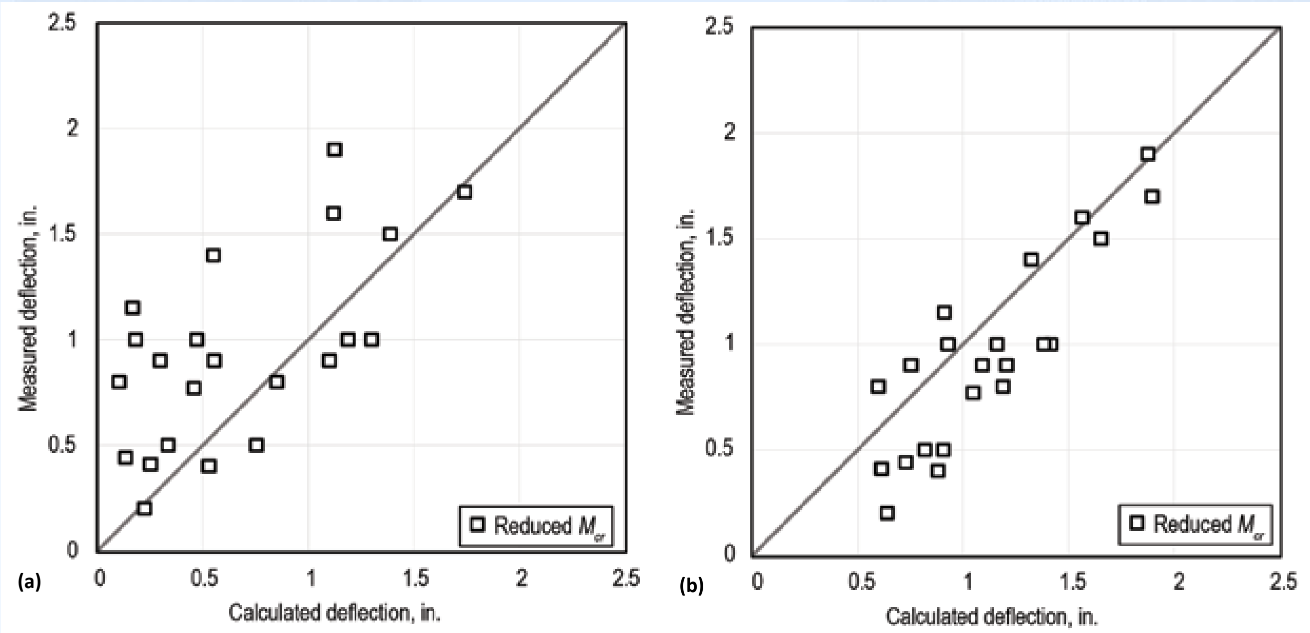

Fig. 10: Estimates of service-level deflections in slabs using a reduced cracking moment: (a) Branson,[11] Eq. (3); and (b) Bischoff,[12] Eq. (4)

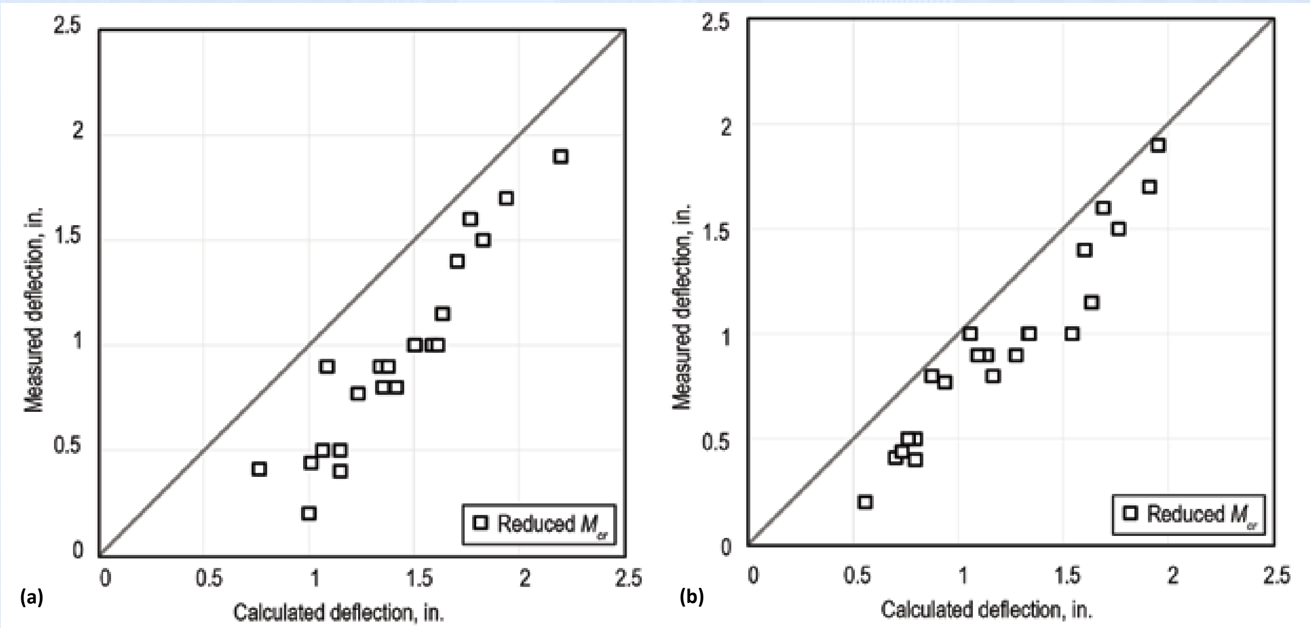

Fig. 11: Estimates of service-level deflections in slabs using mechanics and geometry and a reduced cracking moment: (a) using Eq. (5), Fig. 7(a); and (b) using Eq. (6), Fig. 7(b)

relatively small overestimations of *Mcr* can result in gross underestimations of the observed deflections, as shown in Fig. 8 and 9. The reduced cracking moment therefore acts as a reasonable lower bound on the estimated cracking moment and improves the deflection estimates for those cases in which the cracking moment was previously overestimated.

It can be seen from these figures that the use of the reduced cracking moment generates conservative estimates of the deflection when using Method 2 and improves the estimation of deflection for Method 1 for all test specimens considered. Overall, the results suggest that using Eq. (6) with a reduced cracking moment produces the most reasonable estimate of service-level deflections while retaining the conservatism necessary for deflection control.

Bar spacing and crack control

Crack width is controlled by limiting longitudinal bar spacing *s*. In ACI 318-19, maximum spacing for bonded flexural reinforcing bars in one-way slabs and beams is limited to:

s = minimum of 15 (40,000 / *fs*) – 2.5*cc* and 12 (40,000 / *fs*) (7)

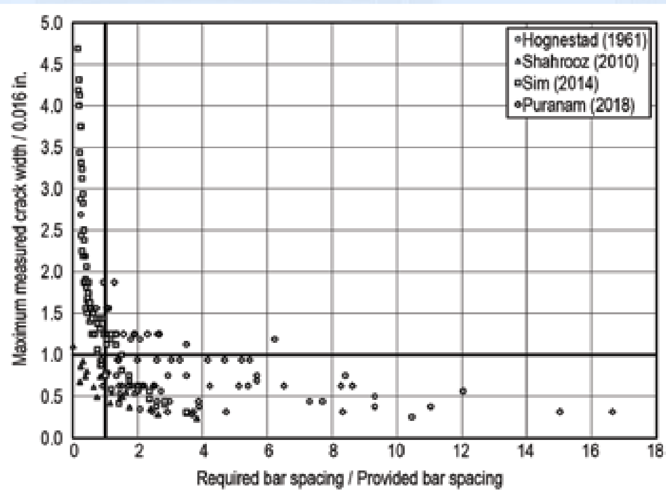

Fig. 12: Crack widths and bar spacing

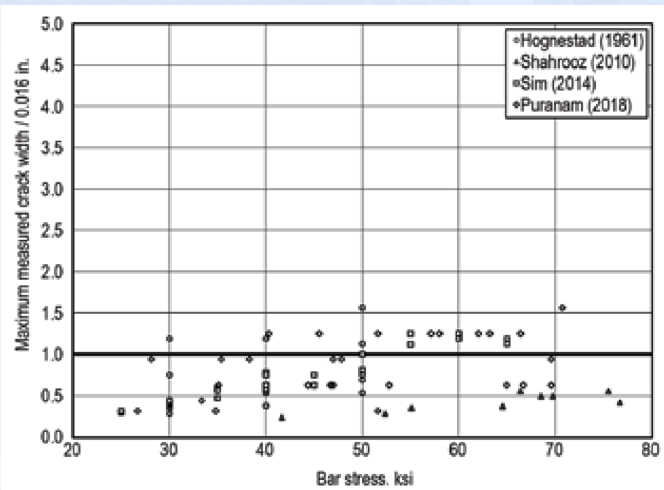

Fig. 13: Crack widths and bar stress

where *s* is maximum center-to-center bar spacing in the layer of reinforcing bars closest to the tension face, f_s is working stress in psi, and c_c is the smallest distance from the surface of bar to the tension face (with f_s in ksi, Eq. (7) simplifies to: s = minimum of 600 / f_s – 2.5c_c and 480 / f_s).

The experimental data collected by Shahrooz et al.[6] and Soltani[13] suggest that Eq. (7) may be used for elements with HSSR. The applicability of Eq. (7) at service stresses larger than 40 ksi was also investigated in experimental studies by Hognestad,[5] Sim,[7] and Puranam.[8] The tests considered included reinforced concrete beams and slabs operating at service stresses up to 80 ksi. In all cases, service stress was smaller than measured yield stress. Bar center-to-center spacing ranged from 1-3/8 to 18 in. Clear cover ranged from 3/8 to 4-3/8 in. Ratio of spacing to clear cover ranged from 0.5 to 15. Bar diameter ranged from 5/8 to 1 in. The data refer to reported crack width maxima (instead of mean crack widths).

Figure 12 shows the relationship between the ratio of maximum measured crack width to a presumed maximum allowable crack width of 0.016 in. and the ratio of required-to provided bar spacing. This comparison shows that when the provided spacing is smaller than what is required by ACI 318-19, maximum crack width does not exceed 1.5 times the traditional limit of 0.016 in. Figure 13, which contains only specimens meeting Eq. (7), suggests that it is the case even at service stresses as large as 80 ksi. The data in Fig. 13 suggest that the maximum of the reported values did not seem to be sensitive to service stress. Nevertheless, Fig. 13 also shows that the mean of the reported values increases with an increase in service stress even as spacing is reduced to meet the current provisions.

The data suggest that current provisions for cracking control that require a reduction in bar spacing with increases in steel service stress are sufficient to avoid problems with crack width. Nevertheless, for applications in which crack width is deemed critical, service stress may need to be limited to 40 ksi regardless of steel grade because of the trend observed in the mean of the reported values.

Conclusions

The data presented suggest that within the ranges of the parameters considered:

- Previous provisions for minimum thickness (ACI 318-14), expressed in terms of service stress f_s as h / h_{ref} = 0.4 + 3f_s (ksi) / 200, are sufficient and can be more efficient to control deflections in beams and one-way slabs with HSSR for f_s < 100 ksi, where h_{ref} is minimum thickness of a slab with ASTM A615/A615M Grade 60 reinforcing bars determined as a function of the span and support conditions and h is the minimum thickness of a slab with steel service stress equal to f_s;

- Deflections estimated by assuming a gradual transition between initial cracking and yielding in the momentcurvature relationship (Eq. (6)) and a reduced cracking moment corresponding to $f_r = 5\ {}_c f\ 2$ produced a reasonable and conservative estimate of measured service-level deflections (Fig. 8 through 11) for the specimensdescribed in Table 4; and
- Except in critical applications, reduction in bar spacing s with increase in service stress in the reinforcing bars, determined as $s = 600 / f_s$ (ksi) – 2.5 × *cover*, was observed to be sufficient to avoid intolerable crack widths in the elements studied that had working stresses up to 80 ksi.

Acknowledgments

The authors would like to acknowledge the generous donation of steel by MMFX. The authors also acknowledge the help from Henrik Skovgaard, Brian Rogers, and Alan Kanybek to build and test the specimens at Purdue University. This work was supported in part by the National Science Foundation Graduate Research Fellowship Program under Grant No. DGE-1333468. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the National Science Foundation.

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Received and reviewed under Institute publication policies.

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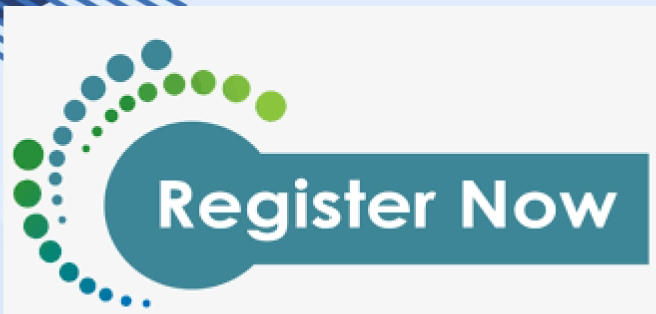

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