Characteristics of Compressive Stress Distribution in High-Strength Concrete
by Halit Cenan Mertol, Sami Rizkalla, Paul Zia, and Amir Mirmiran

This paper describes fundamental characteristics of the compressive stress distribution in the compression zone of flexural members with concrete compressive strengths up to 18 ksi (124 MPa). The proposed model is based on testing of 21 plain concrete specimens subjected to combined flexure and axial compression up to failure. The main variable considered was the strength of concrete that ranged from 10.4 to 16 ksi (71.7 to 110.3 MPa). Each specimen was subjected to two independent loads with a specific configuration to induce maximum compressive strain at one face and zero strain at the opposite. The measured stress-strain curves and stress block parameters were compiled with the data found in the literature. The results were used to develop recommended revisions for the LRFD specifications to extend their current limitation of 10 ksi (69 MPa) for concrete compressive strength up to 18 ksi (124 MPa).

Keywords: compression; flexure; high-strength concrete; loading; rectangular stress block; strain; stress.

INTRODUCTION
Flexural failure of a reinforced concrete member occurs when its extreme fiber reaches the ultimate compressive strain of concrete. Concrete in the compression zone is subjected to a stress distribution, referred to as the stress block, that follows the stress-strain relationship of a concrete cylinder tested in axial compression. This paper focuses on the evaluation of the stress block in the compression zone of high-strength concrete (HSC) flexural members. The strength considered in this investigation ranged from 10 to 18 ksi (69 to 124 MPa). The authors used a similar test setup developed by Hognestad et al.1 in which an eccentric bracket specimen was subjected to an axial compression and a moment to simulate the stress profile in the compression zone of a rectangular flexural member.

The load and resistance factor design (LRFD) specifications,2 first published in 1994, include an article (5.4.2.1) limiting its applicability to a maximum concrete strength of 10 ksi (69 MPa), unless physical tests are made to establish the relationship between concrete strength and its other properties. This limitation was imposed due to the lack of sufficient research data on HSC at the time when the specifications were developed. Many design provisions stipulated in the LRFD specifications2 are still based on test results obtained from specimens with compressive strengths up to 6 ksi (41 MPa). Although such a strength limit is not explicitly imposed by ACI 318-05,3 except in its provisions for shear and development length, its applicability to HSC is not fully and explicitly addressed either. Details on other design codes are given in Mertol.4

RESEARCH SIGNIFICANCE
This study focuses on general characteristics of the stress profile in the compression zone of HSC flexural members. The proposed model is based on the test results of 21 unreinforced HSC members as well as a significant amount of data found in the literature. Stress-strain curves and stress block parameters for HSC were obtained, evaluated, and compared to test results available in the literature. Test results of this study as well as those from previous research served as the basis for recommended revisions to code provisions on stress block parameters for concrete strength up to 18 ksi (124 MPa).

EXPERIMENTAL PROGRAM
The experimental program consisted of 21 concrete specimens with a cross section of 9 x 9 in. (229 x 229 mm) and 40 in. (1 m) long. A general view of the concrete specimen is presented in Fig. 1. The main parameter considered in this study was the concrete strength. Three different target concrete compressive strengths of 10, 14, and 18 ksi (69, 97, 124 MPa).

Fig. 1—General view and steel reinforcement configuration.

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and 124 MPa) were used. Five specimens were tested for the target concrete compressive strength of 10 ksi (69 MPa) whereas six and 10 specimens were tested for the target concrete compressive strengths of 14 and 18 ksi (97 and 124 MPa), respectively. Three 4 x 8 in. (100 x 200 mm) cylinders were also tested for each specimen to evaluate the concrete strength at the time of testing. Additional cylinders were cast for each batch of concrete to establish the 28-day compressive strength of the batch.

To prevent premature localized failure, both ends of the specimens were heavily reinforced with three No. 4 U-shaped longitudinal and three No. 3 transverse reinforcements, as shown in Fig. 1. In addition, the ends of the specimens were confined with 1/2 in. (13 mm) thick, 10 in. (254 mm) long rectangular steel tubes with holes on two opposite faces to ensure proper transfer of the axial load and moment to the middle 16 in. (406 mm) plain concrete test section.

The specimens and the cylinders were demolded 24 hours after casting and were then covered with wet burlap and plastic sheets for a week. The specimens were then stored in the laboratory where the temperature was maintained at approximately 72 °F (22 °C) with 50% relative humidity until the time of testing. The ends of the cylinders were ground before testing.

Materials

Concrete mixture designs for the three different target strengths were developed by Logan5 and are summarized in Table 1.

The coarse aggregate was crushed stone with a nominal maximum size of 3/8 in. (10 mm). Two types of fine aggregate were used depending on the target compressive strength. The first was natural sand and the second was manufactured sand, generically known as 2MS Concrete Sand. The cement was Type I/II. Fly ash, silica fume, high-range water-reducing admixture (HRWRA), and retarding admixture were used to obtain the appropriate strength.

Test method and test setup

A schematic view of the test setup is shown in Fig. 2. The two axial loads of $P_1$ and $P_2$ were adjusted during the test to maintain the location of the neutral axis, that is, zero strain at the exterior face of the specimen. On the opposite side of the cross section, the extreme fiber was subjected to a monotonically increasing compressive strain. In each loading step, the main axial load from the test machine, $P_1$, was applied first to a predetermined level to produce a uniform axial strain in the section. Then, the secondary load $P_2$ was applied by a jack to develop the strain gradient, maintaining zero strain at the exterior face, and a maximum compressive strain at the opposite face.

Two steel moment arms were connected to the specimen using six threaded rods through the holes in the rectangular steel tube at each end. Each steel arm consisted of two 24 in. (610 mm) long C8 x 11.5 channel sections welded to two 9 x 1 x 24 in. (229 x 25 x 610 mm) steel plates at the top and the bottom. Half-inch (13 mm) stiffeners were used for the steel arms. Two specially designed roller connections were used to eliminate the end restrictions due to the applied axial load from the machine. Each roller connection consisted of six 1 in. (25 mm) diameter rollers and two curved plates, tapering through inside and outside, respectively. The roller connection assembly was fixed to side plates that were released at the time

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**Table 1—Three mixture designs for target concrete compressive strengths**

<table>
<thead>
<tr>
<th>Material</th>
<th>10 ksi (69 MPa)</th>
<th>14 ksi (97 MPa)</th>
<th>18 ksi (124 MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement, lb/ft³ (kg/m³)</td>
<td>703 (417)</td>
<td>703 (417)</td>
<td>935 (555)</td>
</tr>
<tr>
<td>Silica fume, lb/ft³ (kg/m³)</td>
<td>75 (44)</td>
<td>75 (44)</td>
<td>75 (44)</td>
</tr>
<tr>
<td>Fly ash, lb/ft³ (kg/m³)</td>
<td>192 (114)</td>
<td>192 (114)</td>
<td>50 (30)</td>
</tr>
<tr>
<td>Sand, lb/ft³ (kg/m³)</td>
<td>1055 (625)†</td>
<td>1315 (780)†</td>
<td>1240 (736)†</td>
</tr>
<tr>
<td>Rock, lb/ft³ (kg/m³)</td>
<td>1830 (1085)</td>
<td>1830 (1085)</td>
<td>1830 (1085)</td>
</tr>
<tr>
<td>Water, lb/ft³ (kg/m³)</td>
<td>292 (173)</td>
<td>250 (148)</td>
<td>267 (158)</td>
</tr>
<tr>
<td>High-range water-reducing admixture, fl oz/100 lb (mL/100 kg)</td>
<td>17 (1110)</td>
<td>24 (1565)</td>
<td>36 (2345)</td>
</tr>
<tr>
<td>Retard ing agent, fl oz/100 lb (mL/100 kg)†</td>
<td>3 (195)</td>
<td>3 (195)</td>
<td>3 (195)</td>
</tr>
<tr>
<td>w/cm</td>
<td>0.30</td>
<td>0.26</td>
<td>0.25</td>
</tr>
<tr>
<td>28-day compressive strength, ksi (MPa)</td>
<td>11.5 (78.9)</td>
<td>14.4 (99.1)</td>
<td>17.1 (117.8)</td>
</tr>
</tbody>
</table>

†Ounces per 100 lb of cementitious material.
of testing. Both the steel arms and the roller connections were designed with a factor of safety of at least two against yielding to ensure failure in the test region.

Instrumentation
The primary axial load $P_1$, applied by the 2000 kip (8900 kN) load-controlled hydraulic compression machine, was measured by an internal load cell. The secondary load $P_2$ was applied using a 120 kip (530 kN) manual hydraulic jack and was measured using a 100 kip (440 kN) flat load cell.

Each specimen was instrumented with 2.4 in. (60 mm) electrical resistance strain gauges. A total of nine strain gauges were mounted on each test specimen. Two gauges were applied on the zero strain face. Four gauges were mounted on the two sides of the specimen. The remaining three were placed on the maximum compression side of the specimen, one of which was used to measure the transverse strain of concrete. Three 1 in. (25 mm) linear variable displacement transducers (LVDTs) were placed at the top, bottom, and midsection of the specimen to measure its deflected shape and to incorporate the secondary moment effect. The instrumentation layout is illustrated in Fig. 2.

Test procedure
The bottom roller connection was placed first into the compression machine. The specimen was then positioned, aligned, and leveled on the roller connection. The bottom and top steel arms were then connected to the specimen using threaded rods. The load cell and the jack, used to apply the secondary load, were placed on the top arm. The bottom arm and the top arm assembly were connected to each other using a threaded rod. The top roller connection was then positioned and leveled.

The specimen was leveled using a thin layer of hydrostone. As the primary axial load was increased incrementally, the secondary load was applied to maintain the neutral axis at the exterior face. The loading rate was kept at 2 microstrains per second on the compression face of the specimen. Each test lasted for about 25 minutes, until concrete was crushed.

Three companion cylinders were tested on the same day in accordance with ASTM C39.6

**TEST RESULTS AND DISCUSSIONS**

The test-day average cylinder strengths for the three target strengths were 11.1, 14.9, and 15.4 ksi (76.4, 102.5, and 106 MPa), respectively. The highest test-day average cylinder strength achieved in this research was 16.0 ksi (110 MPa). All test specimens had a similar explosive failure mode with no visible crack up to failure. Typical failure mode for the eccentric bracket tests is shown in Fig. 3. The cylinder strength, the age at testing, the loading rate, and the ultimate compressive strain achieved by the specimens are summarized in Table 2.

**Stress block parameters**

The approach presented by Hognestad et al.1 was used to determine the stress-strain relationship for each specimen. This approach was used to calculate the concrete stress $f_c$ as

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### Table 2—Tabulated test results

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$f'_{c}$ at testing, ksi</th>
<th>$f'_{c}$ at testing, MPa</th>
<th>Age at testing, days</th>
<th>Loading rate, με/second</th>
<th>Ultimate strain, με</th>
<th>$k_1$</th>
<th>$k_2$</th>
<th>$k_3$</th>
<th>$\alpha_1$</th>
<th>$\beta_1$</th>
<th>$\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10EB1</td>
<td>11.0</td>
<td>76.2</td>
<td>63</td>
<td>12.2</td>
<td>3738</td>
<td>0.65</td>
<td>0.38</td>
<td>1.03</td>
<td>0.90</td>
<td>0.75</td>
<td>—</td>
</tr>
<tr>
<td>10EB2</td>
<td>11.4</td>
<td>78.7</td>
<td>109</td>
<td>2.0</td>
<td>3138</td>
<td>0.62</td>
<td>0.36</td>
<td>1.12</td>
<td>0.95</td>
<td>0.72</td>
<td>0.25</td>
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<tr>
<td>10EB3</td>
<td>11.7</td>
<td>80.7</td>
<td>111</td>
<td>2.4</td>
<td>3407</td>
<td>0.65</td>
<td>0.36</td>
<td>1.14</td>
<td>1.02</td>
<td>0.73</td>
<td>0.21</td>
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<tr>
<td>10EB4</td>
<td>10.4</td>
<td>71.4</td>
<td>63</td>
<td>2.1</td>
<td>3102</td>
<td>0.64</td>
<td>0.36</td>
<td>1.20</td>
<td>1.06</td>
<td>0.73</td>
<td>0.19</td>
</tr>
<tr>
<td>10EB5</td>
<td>10.9</td>
<td>75.2</td>
<td>62</td>
<td>2.2</td>
<td>3023</td>
<td>0.62</td>
<td>0.36</td>
<td>1.16</td>
<td>1.01</td>
<td>0.72</td>
<td>0.20</td>
</tr>
<tr>
<td>14EB1</td>
<td>14.6</td>
<td>100.9</td>
<td>49</td>
<td>2.3</td>
<td>3316</td>
<td>0.63</td>
<td>0.37</td>
<td>1.00</td>
<td>0.85</td>
<td>0.74</td>
<td>0.22</td>
</tr>
<tr>
<td>14EB2</td>
<td>14.3</td>
<td>98.7</td>
<td>51</td>
<td>1.8</td>
<td>3162</td>
<td>0.60</td>
<td>0.36</td>
<td>1.08</td>
<td>0.85</td>
<td>0.72</td>
<td>0.20</td>
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<tr>
<td>14EB3</td>
<td>14.7</td>
<td>101.2</td>
<td>52</td>
<td>2.2</td>
<td>3177</td>
<td>0.61</td>
<td>0.36</td>
<td>1.09</td>
<td>0.93</td>
<td>0.71</td>
<td>0.23</td>
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<tr>
<td>14EB4</td>
<td>15.0</td>
<td>103.7</td>
<td>57</td>
<td>2.3</td>
<td>3032</td>
<td>0.58</td>
<td>0.35</td>
<td>1.10</td>
<td>0.92</td>
<td>0.70</td>
<td>0.23</td>
</tr>
<tr>
<td>14EB5</td>
<td>15.4</td>
<td>105.9</td>
<td>57</td>
<td>5.3</td>
<td>2868</td>
<td>0.57</td>
<td>0.34</td>
<td>1.10</td>
<td>0.92</td>
<td>0.68</td>
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<tr>
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<td>15.2</td>
<td>104.5</td>
<td>101</td>
<td>4.1</td>
<td>2954</td>
<td>0.60</td>
<td>0.35</td>
<td>1.06</td>
<td>0.91</td>
<td>0.69</td>
<td>0.23</td>
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<td>15.8</td>
<td>109.1</td>
<td>76</td>
<td>2.2</td>
<td>3684</td>
<td>0.69</td>
<td>0.38</td>
<td>0.82</td>
<td>0.74</td>
<td>0.77</td>
<td>0.23</td>
</tr>
<tr>
<td>18EB2</td>
<td>16.0</td>
<td>110.2</td>
<td>77</td>
<td>2.3</td>
<td>3364</td>
<td>0.67</td>
<td>0.37</td>
<td>0.85</td>
<td>0.77</td>
<td>0.74</td>
<td>0.22</td>
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<tr>
<td>18EB3</td>
<td>15.6</td>
<td>107.5</td>
<td>81</td>
<td>2.4</td>
<td>2914</td>
<td>0.63</td>
<td>0.37</td>
<td>0.81</td>
<td>0.69</td>
<td>0.73</td>
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<tr>
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<td>108.7</td>
<td>82</td>
<td>2.6</td>
<td>3306</td>
<td>0.65</td>
<td>0.36</td>
<td>0.88</td>
<td>0.78</td>
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<tr>
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<td>110.6</td>
<td>83</td>
<td>2.1</td>
<td>3144</td>
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<td>0.85</td>
<td>0.76</td>
<td>0.72</td>
<td>0.24</td>
</tr>
<tr>
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<td>106.8</td>
<td>84</td>
<td>2.1</td>
<td>3404</td>
<td>0.66</td>
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<td>0.88</td>
<td>0.78</td>
<td>0.74</td>
<td>0.22</td>
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<tr>
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<td>15.0</td>
<td>103.7</td>
<td>96</td>
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<td>3585</td>
<td>0.64</td>
<td>0.37</td>
<td>1.05</td>
<td>0.90</td>
<td>0.75</td>
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<td>18EB8</td>
<td>14.5</td>
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<td>97</td>
<td>2.7</td>
<td>3507</td>
<td>0.65</td>
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<td>0.91</td>
<td>0.74</td>
<td>0.22</td>
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<td>14.9</td>
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<td>99</td>
<td>2.2</td>
<td>3494</td>
<td>0.62</td>
<td>0.36</td>
<td>1.06</td>
<td>0.91</td>
<td>0.72</td>
<td>0.23</td>
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<tr>
<td>18EB10</td>
<td>14.6</td>
<td>100.7</td>
<td>102</td>
<td>2.0</td>
<td>3532</td>
<td>0.64</td>
<td>0.38</td>
<td>0.97</td>
<td>0.82</td>
<td>0.77</td>
<td>0.24</td>
</tr>
</tbody>
</table>

Note: — means could not be obtained.
a function of measured strain at the most compressed fiber $\varepsilon_c$ and the applied stresses $f_o$ and $m_o$. The following equations were obtained from equilibrium of external and internal loads and moments. Note that the eccentricities due to deflection of the member were also considered in the calculation for total applied moment $M$.

$$C = P_1 + P_2 = f_o b c = \frac{b c}{\varepsilon_c} \int_0^{\varepsilon_c} f_o (\varepsilon_x) d\varepsilon_x$$  \hspace{1cm} (1)$$

$$M = P_1 a_1 + P_2 a_2 = m_o b c^2 = \frac{b c^2}{\varepsilon_c^2} \int_0^{\varepsilon_c} f_o (\varepsilon_x) c d\varepsilon_x$$  \hspace{1cm} (2)$$

where $C$ is the total applied load, $a_1$ and $a_2$ are the eccentricities with respect to the neutral surface, $b$ is the width of the section, $c$ is the depth of neutral axis, and

$$f_o = \frac{P_1 + P_2}{b c}$$  \hspace{1cm} (3)$$

$$m_o = \frac{P_1 a_1 + P_2 a_2}{b c^2}$$  \hspace{1cm} (4)$$

are the applied stresses. Some of these definitions are illustrated in Fig. 2. Differentiating the last terms of the equations for $C$ and $M$ with respect to $\varepsilon_c$ yields the following equations

$$\sigma_c = \varepsilon_c \frac{d f_o}{d \varepsilon_c} + f_o$$  \hspace{1cm} (5)$$

$$\sigma_c = \varepsilon_c \frac{d m_o}{d \varepsilon_c} + 2 m_o$$  \hspace{1cm} (6)$$

Using these equations, two similar stress-strain relationships were obtained for each eccentric bracket specimen and the average of these two was used as the stress-strain relationship of the specimen. A typical obtained stress-strain distribution for HSC is shown in Fig. 4. The numerical values of the simplified stress-strain relationships for all specimens are given in Mertol.4

In general, the stress block in the compression zone of a flexure member can be defined by three parameters: $k_1$, $k_2$, and $k_3$. The parameter $k_1$ is defined as the ratio of the average compressive stress to the maximum compressive stress in the compression zone $k_3 f_o'$. The parameter $k_2$ is the ratio of the depth of the resultant compressive force $C$ to the depth of the compression zone $c$. The parameter $k_3$ is the ratio of the maximum compressive stress in the compression zone to the compressive strength measured by concrete cylinder $f'_c$. The design values of the stress block parameters are determined when the strains at the extreme fibers reach the ultimate strain of the concrete $\varepsilon_{wu}$. The three generalized parameters of a stress block can be reduced into two parameters to establish an equivalent rectangular stress block using $\alpha_1$ and $\beta_1$, which ensures the compressive stress resultant being at the same location. The stress block parameters in this study were calculated using the compressive strengths measured by concrete cylinders, the stress-strain relationships obtained in this study, and the methodology described previously. These parameters are shown in Fig. 5. The stress block parameters for each specimen are also given in Table 2.

The ultimate strain of concrete member subjected to flexure is generally higher than that of concrete cylinder subjected to pure compression. The linear strain gradient in the compression zone of flexural members helps in achieving higher strain value at failure. Other reasons for higher strain are the shape and size effects of the concrete cylinder compared with the actual reinforced concrete structural member. Furthermore, the rate of loading of a structural member is usually much slower than that of a concrete cylinder. The stress distribution of concrete in flexure, however, may still be represented adequately by the stress-strain relationship of the concrete cylinder using an empirical constant $k_3$ to account for all of these differences. This constant is determined by comparing the beams tested in flexure to the companion cylinders tested under compression.1

The stress distribution of normal-strength concrete (NSC) is represented by the curved shape, as shown in Fig. 6. For this stress distribution, $k_2$ and $k_3$ are equal to 0.85 and 0.425, respectively. When converted to a rectangular distribution, $\alpha_1$ and $\beta_1$ correspond to $k_3$ and 0.85, respectively. If the stress distribution of HSC is assumed to be triangular, $k_1$ and $k_3$ would be equal to 0.50 and 0.333, respectively. Then, the rectangular stress block parameters, $\alpha_1$ and $\beta_1$, would be 0.75$k_3$ and 0.667, respectively. These parameters are also shown in Fig. 6.

The test results of this research and other research reported in the literature indicate that the majority of the collected data for the generalized stress block parameter $k_1$ for HSC is higher than 0.58 for concrete with compressive strengths between 10 and 18 ksi (69 and 124 MPa), as shown in Fig. 7. Therefore, the lower-bound value of 0.58 is suggested for $k_1$ parameter for concrete with compressive strengths higher than 0.58.
than 15 ksi (103 MPa). The collected data for stress block parameters from other researchers include test results obtained by Hognestad et al.,1 Nedderman,7 Kaar et al.,8,9 Swartz et al.,10 Pastor,11 Schade,12 Ibrahim,13 Tan and Nguyen,14 and Sargin et al.15 The tabulated values of the research data are presented in Mertol.4

The $k_2$ parameter implied in ACI 318-053 and LRFD specifications2 is already set to 0.33 for concrete with compressive strengths greater than 8 ksi (55 MPa), because the assumed $\beta_1$ parameter used in design is equal to 0.65. This provision is also confirmed by the collected data for HSC between 8 and 18 ksi (55 and 124 MPa), as shown in Fig. 8. Therefore, $\beta_1$ can be assumed to be 0.33.

The collected data also indicate that the stress block parameter $k_3$ for HSC is similar to NSC, as shown in Fig. 9. Hence, using the same value of $k_3 = 0.85$ for concrete with compressive strengths up to 18 ksi (124 MPa) is appropriate for design purposes.

Using the aforementioned values proposed for the generalized stress block parameters, the lower-bound relationships for rectangular stress block parameters $\alpha_1$ and $\beta_1$ can be obtained as follows

\[
\alpha_1 = \frac{k_1 k_3}{2 k_2} = \frac{0.58 \times 0.85}{2 \times 0.33} = 0.75
\] (7)

\[
\beta_1 = 2 k_2 = 2 \times 0.33 = 0.65
\] (8)

In light of previous discussions, the following relationships are proposed for the rectangular stress block parameters $\alpha_1$ and $\beta_1$, for concrete strengths up to 18 ksi (124 MPa)

\[
\alpha_1 = \begin{cases} 
0.85 & \text{for } f_{c'}^\prime \leq 10 \text{ ksi} \\
0.85 - 0.02 (f_{c'}^\prime - 10) & \text{for } f_{c'}^\prime > 10 \text{ ksi}
\end{cases}
\] (9)

where $f_{c'}^\prime$ is in ksi.

\[
\alpha_1 = \begin{cases} 
0.85 & \text{for } f_{c'}^\prime \leq 69 \text{ ksi} \\
0.85 - 0.0029(f_{c'}^\prime - 69) & \text{for } f_{c'}^\prime > 69 \text{ ksi}
\end{cases}
\] (10)

where $f_{c'}^\prime$ is in MPa.

\[
\beta_1 = \begin{cases} 
0.85 & \text{for } f_{c'}^\prime \leq 4 \text{ ksi} \\
0.85 - 0.05 (f_{c'}^\prime - 4) & \text{for } f_{c'}^\prime > 4 \text{ ksi}
\end{cases}
\] (11)

where $f_{c'}^\prime$ is in ksi.

\[
\beta_1 = \begin{cases} 
0.85 & \text{for } f_{c'}^\prime \leq 28 \text{ ksi} \\
0.85 - 0.007252(f_{c'}^\prime - 28) & \text{for } f_{c'}^\prime > 28 \text{ ksi}
\end{cases}
\] (12)

where $f_{c'}^\prime$ is in MPa.

The comparisons of the proposed relationships to all collected test results are shown in Fig. 10 and 11.

![Fig. 6—Stress block parameters for different stress distributions.](image)

![Fig. 7—Proposed relationship for the stress block parameter $k_1$.](image)

![Fig. 8—Proposed relationship for the stress block parameter $k_2$.](image)

![Fig. 9—Proposed relationship for the stress block parameter $k_3$.](image)
An analysis was performed by Mertol\textsuperscript{4} to evaluate the sensitivity of the ultimate moment capacity of a reinforced concrete member to the rectangular stress block parameters $\alpha_1$ and $\beta_1$, and the results of the sensitivity analysis are shown in Fig. 12 and 13. The analysis indicates that, for underreinforced concrete members, a reduction in the rectangular stress block parameter $\alpha_1$ by 11.8% leads to a reduction of the ultimate moment capacity by only 1.9%. For an over-reinforced concrete member, however, a reduction in $\alpha_1$ by 11.8% may lead to a reduction of the ultimate moment capacity by as much as 10.3%. Note that a reduction of $\beta_1$ by 23.5% has no effect on the ultimate moment capacity of under-reinforced concrete members, but it will cause a reduction of slightly over 12% of the ultimate moment capacity of over-reinforced concrete members.

**Concrete strain measurements**

The surface strain measurements at different load levels for Specimen 18EB#2 are shown in Fig. 14. Similar behavior was observed for other specimens. The graph validates the assumption that plane sections remain plane after deformation for HSC members.

The ultimate concrete compressive strains measured at failure on the compression face of concrete are shown in Table 2. Based on a regression analysis by Mertol\textsuperscript{4} on 188 test results available in the literature with concrete compressive strengths up to 20 ksi (138 MPa) under eccentric loading, an ultimate compressive strain of 0.003 is considered applicable for design purposes for concrete with compressive strengths up to 18 ksi (124 MPa). A comparison of the proposed ultimate compressive strain of concrete with test results of this and other research\textsuperscript{7-16} reported in the literature is shown in Fig. 15. When only the test results for concrete compressive strengths over 10 ksi (69 MPa) are considered, the 90 percentile line for $\varepsilon_{cu}$ is very close to the proposed value of 0.003.
between Poisson’s ratio using regression technique to develop the relationship between Poisson’s ratio \( \nu \). Based on all collected test results, a Poisson’s ratio of 0.2 for all concrete compressive strengths over 10 ksi (69 MPa) was found to be suitable for concrete strengths up to 18 ksi (124 MPa). If only the test results of specimens with concrete compressive strengths up to 18 ksi (124 MPa) are considered, the following conclusions can be drawn with respect to flexural design with HSC up to 18 ksi (124 MPa):

1. The assumption that plane sections remain plane after deformation is valid for concrete compressive strengths up to 18 ksi (124 MPa);
2. The ultimate concrete compressive strain value of 0.003 for design by the current code provision is acceptable for concrete compressive strengths up to 18 ksi (124 MPa);
3. A Poisson’s ratio of 0.2 as used in the current code provision is also acceptable for concrete compressive strengths up to 18 ksi (124 MPa); and
4. The test results, confirmed by other data in the literature, indicate that the stress block parameter \( \alpha_1 \) of 0.85 should be reduced when concrete compressive strength exceeds 10 ksi (69 MPa). A new relationship is proposed for the parameter \( \alpha_1 \) for concrete compressive strengths up to 18 ksi (124 MPa); and
5. The current value of \( \beta_1 = 0.65 \) for \( f_c' > 8 \) ksi (55 MPa) is deemed appropriate for concrete compressive strengths up to 18 ksi (124 MPa).

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**SUMMARY AND CONCLUSIONS**

A total of 21 plain HSC specimens were tested under eccentric compression to simulate the compression zone of a flexural member by varying the applied axial load and moment. The dimensions of the specimens were 9 x 9 x 40 in. (229 x 229 x 1016 mm) and the concrete cylinder strength ranged from 10.4 to 16 ksi (71.7 to 110.3 MPa). The data obtained herein were used to determine the fundamental characteristics of the stress distribution in the compression zone of a flexural member. The test results obtained in this research combined with the available data in the literature were used to develop recommended changes for code provisions. The following conclusions can be drawn with respect to flexural design with HSC up to 18 ksi (124 MPa):

1. The assumption that plane sections remain plane after deformation is valid for concrete compressive strengths up to 18 ksi (124 MPa);
2. The ultimate concrete compressive strain value of 0.003 for design by the current code provision is acceptable for concrete compressive strengths up to 18 ksi (124 MPa);
3. A Poisson’s ratio of 0.2 as used in the current code provision is also acceptable for concrete compressive strengths up to 18 ksi (124 MPa); and
4. The test results, confirmed by other data in the literature, indicate that the stress block parameter \( \alpha_1 \) of 0.85 should be reduced when concrete compressive strength exceeds 10 ksi (69 MPa). A new relationship is proposed for the parameter \( \alpha_1 \) for concrete compressive strengths up to 18 ksi (124 MPa); and
5. The current value of \( \beta_1 = 0.65 \) for \( f_c' > 8 \) ksi (55 MPa) is deemed appropriate for concrete compressive strengths up to 18 ksi (124 MPa).

**SUMMARY AND CONCLUSIONS**

A total of 246 test results with concrete compressive strengths up to 20 ksi (138 MPa) were analyzed by Mertol using regression technique to develop the relationship between Poisson’s ratio \( \nu \) and concrete compressive strength \( f_c' \). Based on all collected test results, a Poisson’s ratio of 0.2 was found to be suitable for concrete strengths up to 18 ksi (124 MPa). If only the test results of specimens with concrete compressive strengths over 10 ksi (69 MPa) are considered, there is a slight increase in the Poisson’s ratio as concrete compressive strength increases. The measurements of a horizontal strain gauge on the compression face were used to calculate the Poisson’s ratio for HSC. The calculated values of Poisson’s ratio \( \nu \) for all target concrete strength specimens are shown in Table 2. The comparison of the test results of this research and other research in the literature is shown in Table 2. There is no apparent trend for Poisson’s ratio as concrete compressive strength increases.

A total of 246 test results with concrete compressive strengths up to 20 ksi (138 MPa) were analyzed by Mertol using regression technique to develop the relationship between Poisson’s ratio \( \nu \) and concrete compressive strength \( f_c' \). Based on all collected test results, a Poisson’s ratio of 0.2 was found to be suitable for concrete strengths up to 18 ksi (124 MPa). If only the test results of specimens with concrete compressive strengths over 10 ksi (69 MPa) are considered, there is a slight increase in the Poisson’s ratio as concrete compressive strength increases. The proposed value of 0.2 for Poisson’s ratio represents the 44 percentile of the test data.