

Report on Structural Design and Detailing for High-Strength Concrete in Moderate to High Seismic Applications

Reported by ACI Innovation Task Group 4
and Other Contributors



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Report on Structural Design and Detailing for High-Strength Concrete in Moderate to High Seismic Applications

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ACI ITG-4.3R presents a literature review on seismic design using high-strength concrete. The document is organized in chapters addressing the structural design of columns, beams, beam-column joints, and structural walls made with high-strength concrete, and focuses on aspects most relevant for seismic design. Each chapter concludes with a series of recommended modifications to ACI 318-05 based on the findings of the literature review.

The recommendations include proposals for the modification of the equivalent rectangular stress block, equations to calculate the axial strength of columns subjected to concentric loading, column confinement requirements, limits on the specified yield strength of confinement reinforcement, strut factors, and provisions for the development of straight bars and hooks.

An accompanying standard, ITG-4.1, is written in mandatory language in a format that can be adopted by local jurisdictions, and will allow building officials to approve the use of high-strength concrete on projects that are being constructed under the provisions of ACI 301, "Specifications for Structural Concrete," and ACI 318, "Building Code Requirements for Structural Concrete."

ITG 4 has also developed another nonmandatory language document: ITG-4.2R. It addresses materials and quality considerations and is the supporting document for ITG-4.1.

Keywords: bond; confinement; drift; flexure; high-strength concrete; high-yield-strength reinforcement; seismic application; shear; stress block; strut-and-tie.

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Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/Engineer.

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CHAPTER 1—INTRODUCTION

1.1—Background

The origin of ACI Innovation Task Group (ITG) 4, High-Strength Concrete for Seismic Applications, can be traced back to the International Conference of Building Officials (ICBO) (now International Code Council [ICC]) *Evaluation Report* ER-5536, “Seismic Design Utilizing High-Strength Concrete” (ICBO 2001). Evaluation Reports (ER) are issued by Evaluation Service subsidiaries of model code groups. An ER essentially states that although a particular method, process, or product is not specifically addressed by a particular edition of a certain model code, it is in compliance with the requirements of that particular edition of that model code.

ER-5536 (ICBO 2001), first issued in April 2001, was generated by Englekirk Systems Development Inc. for the seismic design of moment-resisting frame elements using high-strength concrete. High-strength concrete was defined as “normalweight concrete with a design compressive strength greater than 6000 psi (41 MPa) and up to a maximum of 12,000 psi (83 MPa).” It was based on research carried out at the University of Southern California and the University of California at San Diego to support building construction in Southern California using concrete with compressive strengths greater than 6000 psi (41 MPa).

The Portland Cement Association performed a review* of ER-5536 and brought up several concerns that focused on inconsistencies between the evaluation report and existing industry documents in two primary areas: material and structural. Despite those concerns, it was evident that the evaluation report had been created because quality assurance and design provisions were needed by local jurisdictions, such as the City of Los Angeles, to allow the use of high-strength concrete without undue restrictions. ACI has assumed a proactive role in the development of such provisions with the goal of creating a document that can be adopted nationwide.

ACI considered its own Committee 363, High Strength Concrete, to be the best choice to address the materials and quality aspects of the document, while ACI Subcommittee 318-H, Structural Concrete Building Code—Seismic Provisions, was considered the best choice to address the seismic detailing aspects. Because 318-H is a subcommittee of a code-writing body, the development of a technical document of this kind is not part of its intended mission. In addition, producing a document through a technical committee can be a lengthy process. Based on these limitations, a request was made to form an ITG that would have the advantage of following a shorter timeline to completion. In

*Unpublished report available from PCA, Skokie, Ill., Aug. 2001.

response to the request, the Technical Activities Committee (TAC) of ACI approved the formation of ITG 4 and established its mission. The mission was to develop an ACI document that addressed the application of high-strength concrete in structures located in areas of moderate and high seismicity. The document was intended to cover structural design, material properties, construction procedures, and quality-control measures. It was to contain language in a format that allowed building officials to approve the use of high-strength concrete in projects being constructed under the provisions of ACI 301-05, “Specifications for Structural Concrete,” and ACI 318, “Building Code Requirements for Structural Concrete.”

The concept of “moderate to high seismic applications,” stated in the mission of the document, dates back to when U.S. seismic codes divided the country into seismic zones. These seismic zones were defined as regions in which seismic ground motion on rock, corresponding to a certain probability of occurrence, remained within certain ranges. Present-day seismic codes (ASCE/SEI 2006) follow a different approach to characterizing a seismic hazard. Given that public safety is a primary code objective, and that not all buildings in a given seismic zone are equally crucial to public safety, a new mechanism for triggering seismic design requirements and restrictions, called the seismic performance category (SPC), was developed. The SPC classification includes not only the seismicity at the site, but also the occupancy of the structure.

Recognizing that building performance during a seismic event depends not only on the severity of bedrock acceleration, but also on the type of soil that a structure is founded on, seismic design criteria in more recent seismic codes are based on seismic design categories (SDC). The SDC is a function of location, building occupancy, and soil type. The TAC Technology Transfer Committee (TTTC)-established mission of ITG 4 was interpreted to mean that the Task Group was to address the application of high-strength concrete in structures that are:

- Located in Seismic Zones 2, 3, or 4 of the “Uniform Building Code” (ICBO 1997); or
- Assigned to SDC C, D, or E of “The BOCA National Building Code” (BOCA 1993 and subsequent editions) or the “Standard Building Code” (SBCCI 1994); or
- SDC C, D, E, or F of the “International Building Code” (IBC 2003) or the National Fire Protection Association (NFPA) NFPA 5000 “Building Construction and Safety Code” (2003).

SPC or SDC C is also referred to as the “intermediate” category. Similarly, SPC D and E or SDC D, E, and F are referred to as “high” categories. The terminology “moderate to high seismic applications,” however, is used throughout this document.

1.2—Scope

This document addresses the material and design considerations when using normalweight concretes having specified compressive strengths of 6000 psi (41 MPa) or greater in structures designed for moderate to high seismic applications.

Irrespective of seismic zone, SPC, or SDC, this document is also applicable to normalweight high-strength concrete in intermediate or special moment frames and intermediate or special structural walls as defined in ACI 318-05 (ACI Committee 318 2005).

The term “high-strength concrete,” as defined by ACI 363R-92 (ACI Committee 363 1992), refers to concrete having a specified compressive strength for design of 6000 psi (41 MPa) or greater. The 6000 psi (41 MPa) threshold that was chosen for this document is similar to that adopted by ACI Committee 363.

Even though high-strength concrete is defined based on a threshold compressive strength, the concept of high strength is relative. The limit at which concrete is considered to be high strength depends largely on the location in which it is being used. In some regions, structures are routinely designed with concrete having specified compressive strengths of 12,000 psi (83 MPa) or higher, whereas in other regions, concrete with a much lower specified compressive strength is considered high strength. Essentially, the strength threshold at which concrete is considered high strength depends on regional factors, such as the characteristics and availability of raw materials, production capabilities, testing capabilities, and experience of the ready mixed concrete supplier.

ITG-4 produced three documents: ITG-4.1 is a reference specification that can be cited in the project specifications; ITG-4.2R addresses materials and quality considerations that are the basis for the ITG-4.1 specification; and ITG-4.3R, this document, addresses structural design and detailing. Certain modifications of ACI 318 requirements are proposed in **Chapter 10** of ITG-4.3R.

From a materials perspective, there are few differences between the properties of high-strength concrete used in seismic applications and those of high-strength concrete used in nonseismic applications; therefore, the information presented in ITG-4.1 and ITG-4.2R is generally applicable to all high-strength concrete. When special considerations are warranted due to seismic applications, they are addressed specifically. Unlike ITG-4.1 and ITG-4.2R, most of the material contained in ITG-4.3R is specific to seismic applications of high-strength concrete structural members.

The information in **Chapters 4** through **9** of this document is presented in a report format. **Chapter 10** contains suggested modifications to design and detailing requirements in ACI 318-05.

Some topics, such as compressive stress block and confinement of beam-columns, are more developed than others because there is significantly more literature available on these topics. For all topics, an attempt was made to be as thorough as possible in summarizing the most relevant information pertaining to the design of members with high-strength concrete. For topics with limited information in the literature, however, recommendations were made with the intent of preventing potentially unsafe design.

CHAPTER 2—NOTATION

$A_{b,max}$	= cross-sectional area of largest bar being developed or spliced, in. ² (mm ²)	c_1	= dimension of rectangular or equivalent rectangular column, capital, or bracket measured in direction of span for which moments are being determined, in. (mm)
A_{cc}	= cross-sectional area of structural member measured center-to-center of transverse reinforcement, in. ² (mm ²)	c_2	= dimension of rectangular or equivalent rectangular column, capital, or bracket measured in direction perpendicular to c_1 , in. (mm)
A_{ch}	= cross-sectional area of structural member measured out-to-out of transverse reinforcement, in. ² (mm ²)	c_b	= smaller of: a) distance from center of bar or wire to nearest concrete surface; or b) one-half center-to-center spacing of bars or wires being developed, in. (mm)
A_{cv}	= gross area of concrete section bounded by web thickness and length of section in direction of shear force considered, in. ² (mm ²)	c_c	= clear cover of reinforcement, in. (mm)
A_g	= gross area of concrete section, in. ² (mm ²). For hollow section, A_g is area of concrete only and does not include area of void(s)	c_{cb}	= least distance from surface or reinforcement to tension face, in. (mm)
A_{sh}	= total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension b_c , in. ² (mm ²)	c_{max}	= maximum of c_{cb} and c_s , in. (mm)
A_{sp}	= cross-sectional area of transverse reinforcement crossing potential plane of splitting of bars being developed or spliced, in. ² (mm ²)	c_{min}	= minimum cover used in expressions for bond strength of bars not confined by transverse reinforcement. Smaller of c_{cb} and c_s , in. (mm)
A_{st}	= total area of nonprestressed longitudinal reinforcement (bars or steel shapes), in. ² (mm ²)	c_p	= $\rho_{vr} \cdot f_{yt}/f'_c$ = volumetric confinement index
A_{sv}	= total area of vertical reinforcement in structural wall, in. ² (mm ²)	c_s	= minimum of c_{so} and $(c_{si} + 0.25)$ in. [$(c_{si} + 6.35)$ mm], in. (mm)
A_{swb}	= total area of vertical reinforcement in boundary element of structural wall, in. ² (mm ²)	c_{sfw}	= flexural stress index for structural wall that represents measure of ratio of neutral axis depth to length of wall, in. (mm)
A_{sww}	= total area of vertical reinforcement in web of structural wall, excluding the boundary elements, in. ² (mm ²)	c_{si}	= one-half of clear spacing between bars, in. (mm)
A_{te}	= sum of areas of tie legs used to provide lateral support against buckling for longitudinal bars of column, in. ² (mm ²)	c_{so}	= clear side concrete cover for reinforcing bar, in. (mm)
A_{tr}	= total cross-sectional area of all transverse reinforcement within spacing s that crosses potential plane of splitting through reinforcement being developed, in. ² (mm ²)	DR_{lim}	= (Δ_{lim}/h_{col}) = limiting drift ratio
A_v	= area of shear reinforcement with spacing s , in. ² (mm ²)	d	= distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in. (mm)
A_w	= gross cross-sectional area of structural wall, in. ² (mm ²)	d_b	= nominal diameter of bar, wire, or prestressing strand, in. (mm)
a_v	= shear span, equal to distance from center of concentrated load to either: a) face of support for continuous or cantilever members; or b) center of support for simply supported members, in. (mm)	d_s	= nominal diameter of bar used as transverse reinforcement, in. (mm)
b	= width of compression face of member, in. (mm)	E	= load effects of earthquake or related internal moments and forces
b_c	= cross-sectional dimension of column core measured center-to-center of outer legs of transverse reinforcement comprising area A_{sh} , in. (mm)	EE_p	= $[(M_{calc} - M_{exp})/M_{exp}] \times 100$ = parameter used to characterize accuracy of nominal moment strength of column
b_w	= web width or diameter of circular section, in. (mm)	E_s	= modulus of elasticity of reinforcement and structural steel, psi (MPa)
c	= distance from extreme compression fiber to neutral axis, in. (mm)	f'_c	= specified compressive strength of concrete, psi (MPa)
c'	= $c_{min} + d_b/2$ = spacing or cover dimension, in. (mm)	f'_{co}	= in-place strength of unconfined concrete in columns, psi (MPa) (often assumed as $0.85f'_c$)
		f_p	= $P/A_g f'_c$ = axial load ratio
		f_{pc}	= $P/A_{ch} f'_c$ = axial load ratio based on area of confined core
		f_s	= calculated tensile stress in reinforcement at service loads, psi (MPa)
		$f_{t,l}$	= stress imposed on concrete by compression field associated with reinforcement oriented in direction parallel to flexural reinforcement located at edge of compression field, psi (MPa)
		$f_{t,t}$	= stress imposed on concrete by compression field associated with reinforcement oriented in

	direction perpendicular to flexural reinforcement located at edge of compression field, psi (MPa)		measured from critical section to outside end of hook, in. (mm)
f_u	= maximum tensile stress that can be developed in bar with 90-degree hook, psi (MPa)	l_o	= length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, in. (mm)
f_{yl}	= specified yield strength of longitudinal reinforcement, psi (MPa)	l_w	= length of entire wall or length of segment of wall considered in direction of shear force, in. (mm)
f_{yt}	= specified yield strength of transverse reinforcement, psi (MPa)	M	= maximum unfactored moment due to service loads, including P - Δ effects, in.-lb (N-mm)
$f_{yt,l}$	= specified yield strength of transverse reinforcement oriented parallel to flexural reinforcement located at edge of uniform compression field, psi (MPa)	M_{exp}	= measured flexural strength of column, in.-lb (N-mm)
$f_{yt,t}$	= specified yield strength of transverse reinforcement oriented perpendicular to flexural reinforcement located at edge of uniform compression field, psi (MPa)	M_{ncol}	= nominal flexural strength of column, in.-lb (N-mm)
h''	= core dimension perpendicular to transverse reinforcement providing confinement measured to outside of hoops, in. (mm)	m	= $f_{yl}/0.85f'_c$ = ratio of nominal yield strength of longitudinal reinforcement to nominal strength of concrete in column
h_a	= tie depth, in. (mm)	n	= number of bars being spliced or developed in plane of splitting
h_{col}	= clear column height, in. (mm)	n_L	= number of legs of reinforcement in hoops and ties
h_w	= height of entire wall from base to top or height of segment of wall considered, in. (mm)	P	= unfactored axial load, lb (N)
h_x	= maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of column, in. (mm)	P_o	= nominal axial strength at zero eccentricity, lb (N)
j	= ratio of internal lever arm to effective depth of beam	s	= center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, prestressing tendons, wires, or anchors, in. (mm)
K_{tr}	= $(A_{tr}f_{yt}/1500sn)$ = transverse reinforcement index (refer to ACI 318-05, Section 12.2.3)	s_o	= center-to-center spacing of transverse reinforcement within length l_o , in. (mm)
K'_{tr}	= $(0.5t_dA_{tr}/sn)f'_c{}^{1/2}$ = transverse reinforcement index for Committee 408 development length expression, in. (mm)	T_b	= total bond force of developed or spliced bar, lb (N)
k_1	= ratio of average to maximum stress in compression zone of flexural member	T_s	= steel contribution to total bond force, additional bond strength provided by transverse steel, lb (N)
k_2	= ratio of distance from extreme compression fiber to location of compression reaction to distance from extreme compression fiber to location of neutral axis in flexural member	t_d	= term representing effect of bar size on T_s
k_3	= ratio of maximum stress in compression zone of flexural member to cylinder strength	V	= maximum unfactored shear force at service loads, including P - Δ effects, lb (N)
k_{cc}	= cover factor in calculation of development length of hooked bars	V_a	= nominal shear strength provided by strut spanning between load point and support in reinforced concrete members with shear span-depth ratios below 2.5, lb (N)
k_d	= development length factor in calculation of development length of hooked bars	V_{all}	= allowable shear force under service loads, lb (N)
k_j	= development length and lever arm factor in calculation of development length of hooked bars	V_c	= nominal shear strength provided by the concrete, lb (N)
k_s	= transverse reinforcement bar diameter factor for calculation of development length of hooked bars	V_n	= nominal shear strength, lb (N)
l_b	= dimension of loading plate or support in axial direction of member, in. (mm)	V_s	= nominal shear strength provided by shear reinforcement, lb (N)
l_d	= development length in tension of deformed bar, deformed wire, plain or deformed welded wire reinforcement, or pretensioned strand, in. (mm)	$V_{t,l}$	= nominal shear strength provided by uniform compression field associated with transverse reinforcement oriented parallel to flexural reinforcement located at edge of compression field, lb (N)
l_{dh}	= development length in tension of deformed bar or deformed wire with standard hook,	$V_{t,t}$	= nominal shear strength provided by uniform compression field associated with transverse reinforcement oriented perpendicular to flexural reinforcement located at edge of compression field, lb (N)
		$v_{c,all}$	= allowable shear stress in concrete
		w_{st}	= strut width, in. (mm)
		α_1	= factor relating magnitude of average stress in equivalent rectangular compressive stress

	block to specified compressive strength of concrete	ϕ	=	strength reduction factor
α_c	= coefficient defining relative contribution of concrete to nominal wall shear strength	ϕ_{lim}	=	limiting curvature of reinforced concrete wall
α_l	= angle between struts and flexural reinforcement for a compression field associated with transverse reinforcement oriented in direction parallel to flexural reinforcement	ϕ_u	=	curvature at limit state of reinforced concrete section
α_{sh}	= $1 \leq 4/[(M/Vd) + 1] \leq 2$ = factor to account for effect of shear span-depth ratio on allowable shear stress carried by concrete	ϕ_y	=	curvature at yielding of flexural reinforcement of reinforced concrete section
α_{st}	= smallest angle between strut and ties that it intersects at its nodes	γ_{vj}	=	joint shear coefficient
α_t	= angle between struts and flexural reinforcement for compression field associated with transverse reinforcement oriented in direction perpendicular to flexural reinforcement	λ_p	=	factor to account for effect of axial load ratio on strength of compression field subjected to repeated load reversals into nonlinear range of response
β_1	= factor relating depth of equivalent rectangular compressive stress block to neutral axis depth	μ_Δ	=	$(\Delta_{lim}/\Delta_{yield})$ = displacement ductility ratio
β_{fc}	= factor to account for effect of concrete compressive strength on effective compressive strength of concrete in strut	θ_p	=	expected rotation in plastic hinge region of flexural member, radians
$\beta_{nl, strut}$	= factor to account for effect of repeated load reversals into nonlinear range of response on effective compressive strength of concrete in strut	ρ_{area}	=	ratio of area of distributed transverse reinforcement A_{sh} to gross area of concrete perpendicular to that reinforcement in members with rectilinear and circular transverse reinforcement
$\beta_{nl, truss}$	= factor to account for effect of repeated load reversals into nonlinear range of response on shear strength associated with compression field	ρ_l	=	ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement
β_s	= factor to account for effect of cracking and confining reinforcement on effective compressive strength of concrete in strut	ρ_s	=	ratio of volume of spiral reinforcement to total volume of core confined by spiral (measured out-to-out of spirals)
β_{sc}	= factor to account for effect of load reversals, concrete compressive strength, confining reinforcement, and cracking on effective compressive strength of concrete in strut	ρ_t	=	ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement
β_{ta}	= factor to account for effect of interaction between truss and arch mechanisms on effective compressive strength of concrete in strut	ρ_{tc}	=	$A_{sh}/b_c s$ = ratio of area of distributed transverse reinforcement A_{sh} to area of core perpendicular to that transverse reinforcement
β_{at}	= factor to account for effect of angle of inclination of strut α_s on effective compressive strength of concrete in strut	$\rho_{t,l}$	=	ratio of area of distributed reinforcement oriented in direction parallel to flexural reinforcement of compression field to gross concrete area perpendicular to that reinforcement
χ_1	= ratio of mean concrete compressive stress corresponding to maximum axial load resisted by concentrically loaded column to specified compressive strength of concrete	$\rho_{t,t}$	=	ratio of area of distributed reinforcement oriented in direction perpendicular to flexural reinforcement of compression field to gross concrete area perpendicular to that reinforcement
Δ_{lim}	= lateral drift corresponding to 20% reduction in lateral resistance, in. (mm)	ρ_{vol}	=	ratio of volume of rectilinear or circular transverse reinforcement to volume of core confined by that transverse reinforcement
Δ_{yield}	= lateral drift corresponding to yielding of longitudinal reinforcement, in. (mm)	ρ_{vr}	=	ratio of volume of rectilinear transverse reinforcement to volume of core confined by that transverse reinforcement
δ_u	= design displacement, in. (mm)	ρ_{wt}	=	ratio of total area of vertical reinforcement to gross area of structural wall
ϵ_1	= principal tensile strain in strut	ω_c	=	$0.1(c_{max}/c_{min}) + 0.9 \leq 1.25$ = factor to account for ratio of maximum to minimum cover on development length of straight bar
ϵ_{cu}	= maximum strain at extreme compression fiber at onset of crushing of concrete	ψ_e	=	factor used to modify development length based on reinforcement coating
ϵ_{lim}	= concrete strain at extreme compression fiber corresponding to limit state being considered	ψ_s	=	factor used to modify development length based on reinforcement size
ϵ_o	= strain in concrete when it reaches peak stress	ψ_t	=	factor used to modify development length based on reinforcement location
ϵ_s	= strain demand on reinforcement			
ϵ_y	= strain in reinforcement at yield			

CHAPTER 3—DEFINITIONS

area transverse reinforcement ratio—ratio of the area of transverse reinforcement crossed by a plane perpendicular to the legs of the transverse reinforcement to the area of reinforced concrete along that plane.

axial load ratio—ratio of axial load to the product of compressive strength of concrete and the gross area of concrete cross section.

confinement index—product of transverse reinforcement ratio (either by area or by volume) and the yield strength of the transverse reinforcement, divided by the compressive strength of concrete.

curvature ductility ratio—ratio of mean curvature at failure in the plastic hinge length to curvature at the onset of section yielding. In the case of reinforced concrete columns, the majority of researchers referenced in this document define failure as a 20% reduction in lateral load resistance.

displacement ductility ratio—ratio of displacement at failure to displacement at the onset of member yielding. In the case of reinforced concrete columns, the majority of researchers referenced in this document define failure as a 20% reduction in lateral load resistance.

ductility—ability of a reinforced concrete member to maintain its strength when subjected to repeated load reversals beyond the linear range of response.

interstory drift—relative lateral displacement between two adjacent stories of a building imposed by the design earthquake.

interstory drift ratio—ratio of interstory drift to story height.

killed steel—steel made by completely removing or tying up the oxygen in the liquid steel through the addition of elements such as aluminum or silicon before the ingot solidifies, with the objective of achieving maximum uniformity in the steel.

limiting drift—drift corresponding to a 20% reduction in lateral load resistance of a reinforced concrete member subjected to load reversals with increasing maximum displacements.

limiting drift ratio—ratio of limiting drift to column height.

limiting strain—maximum strain at the extreme concrete compression fiber of a flexural member at the onset of concrete crushing, ϵ_{cu} .

volumetric transverse reinforcement ratio—ratio of the volume of transverse reinforcement confining the concrete core of a potential plastic hinge region to the volume of concrete inside the confined core.

CHAPTER 4—DESIGN FOR FLEXURAL AND AXIAL LOADS USING EQUIVALENT RECTANGULAR STRESS BLOCK

It is common practice for structures assigned to a high Seismic Design Category (SDC) to proportion the majority of the structural elements of the lateral force-resisting system so that the axial load demand remains below the balanced axial load. For these elements, variations in the shape of the stress block related to the compressive strength of the concrete do not have a significant effect on the calculated

strength. There are instances, however, in which it is difficult for engineers to avoid proportioning columns with high axial load demands, such as lower-story columns in tall buildings, lower-story columns in narrow moment-resisting frames, and columns supporting the ends of discontinuous walls. For these elements, the shape of the stress block may have a significant effect on the estimated strength. The stress block for members with high-strength concrete is also a concern in moderate seismic applications. In these cases, structures are proportioned for seismic events that impose lower force and deformation demands than high seismic applications, allowing the use of more slender columns.

The accuracy of the stress block is of concern in earthquake-resistant design because overestimating the flexural strength of columns leads to overestimating the ratios of column-to-beam moment strengths, which increases the probability of hinging in the columns due to the development of a strong beam-weak column mechanism.

Although the stress-strain characteristics of high-strength concrete are different from those of normal-strength concrete, there is no well-defined compressive strength boundary between the two; there is instead a gradual change with increasing concrete strengths (ACI Innovation Task Group 4 2006). The ascending branch of the stress-strain relationship is steeper for higher-strength concretes, indicating higher elastic modulus. It changes from approximately a second-order parabola for concretes within the normal-strength range to almost a straight line as the strength approaches 18,000 psi (124 MPa), which may be considered as the limit for high-strength concrete made with ordinary limestone aggregates. The strain at peak concrete stress, ϵ_o , increases with strength as well, varying approximately between 0.0015 and 0.0025 for 3000 to 15,000 psi (21 and 103 MPa) concrete, respectively. Failure becomes more sudden and brittle as the concrete strength increases and unloading beyond the peak becomes more rapid. In summary, concrete becomes more rigid and more brittle with increasing strength.

Several researchers developed constitutive models for the stress-strain relationship of concrete that are applicable to high-strength concrete with proper adjustments to the governing parameters (Popovics 1973; Yong et al. 1988; Hsu and Hsu 1994; Azizinamini et al. 1994; Cusson and Paultre 1995). Expressions applicable specifically to high-strength concrete have also been developed (Martinez et al. 1984; Fafitis and Shah 1985; Bjerkeli et al. 1990; Muguruma and Watanabe 1990; Li 1994).

Members subjected to uniform compression attain their maximum strength when concrete reaches a strain level corresponding to peak stress, ϵ_o . Under a strain gradient, maximum strength is attained at an extreme compressive fiber strain higher than that at peak stress, ϵ_{lim} (Hognestad 1951). This value changes with the geometric shape of the compression zone, and may also vary significantly with concrete strength and confinement. After the limiting strain has been established, the sectional strength can be computed by evaluating internal forces, including the compressive force in the concrete. The magnitude of the compressive force in the concrete can be established by relying on the

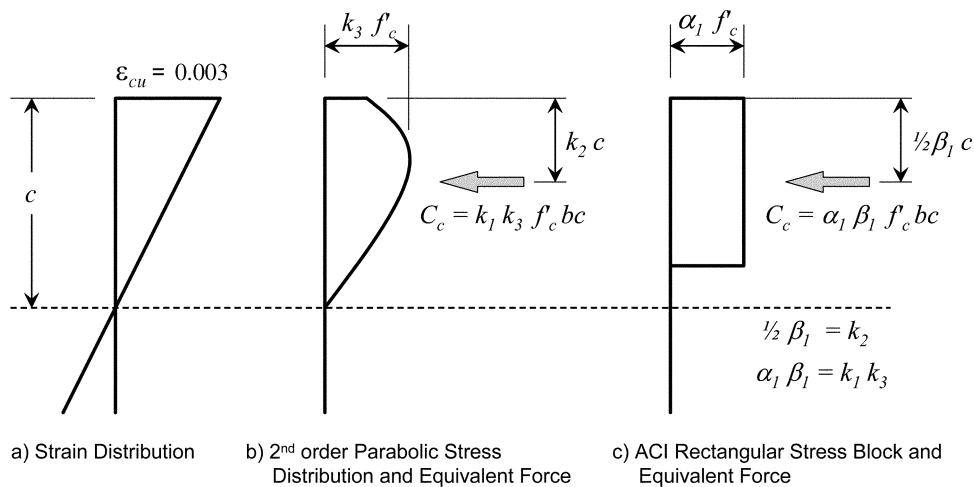


Fig. 4.1—Parameters for rectangular stress block.

assumption that plane sections remain plane after bending and by calculating the stresses corresponding to the strains in the compression zone from the stress-strain relationship. Because it is cumbersome to use a nonlinear stress-strain relationship, ACI 318-05 provides an equivalent stress block for ease in design calculations. This stress block is derived such that both the area under the actual nonlinear stress distribution (force) and the centroid of this area (point of application of force) correspond to those of the stress block as closely as possible. The stress block adopted by ACI 318-05 is of rectangular geometry. Other equivalent stress blocks with various different shapes, such as triangular and trapezoidal, have been proposed in the literature. A historical review of this topic has been presented by Hognestad (1951).

4.1—Parameters of equivalent rectangular stress block

The column design provisions of ACI 318-05 are based on an extensive column investigation conducted jointly by the University of Illinois, Lehigh University, and ACI. The initial results of the study were published in 1931 (Slater and Lyse 1931a,b), with a more comprehensive follow-up report in 1934 (Richart and Brown 1934). Subsequently, Hognestad (1951) conducted a large number of column tests and developed the parameters for a rectangular stress block.

Figure 4.1 shows the parameters that define the equivalent rectangular stress block according to ACI 318-05. A parabolic stress distribution, shown in Fig. 4.1(b), results in values of $k_2 = 0.375$ ($\beta_1 = 0.75$) and $k_1 = 0.67$ ($\alpha_1 = 0.9k_3$). A linear stress distribution yields values of $k_2 = 0.333$ ($\beta_1 = 0.667$) and $k_1 = 0.50$ ($\alpha_1 = 0.75k_3$). ACI 318-05 stipulates that the average stress factor α_1 is not sensitive to compressive strength and remains constant at 0.85, while the β_1 factor decreases from 0.85 ($k_1 k_3 = 0.723$) for a compressive strength of 4000 psi (28 MPa) to 0.65 ($k_1 k_3 = 0.553$) for a compressive strength of 8000 psi (55 MPa). According to ACI 318-05, the strain at the extreme compression fiber in the concrete at the onset of crushing is 0.003 (Fig. 4.1(a)).

Fasching and French (1998) presented a summary of several proposals for modifying the parameters of the equivalent

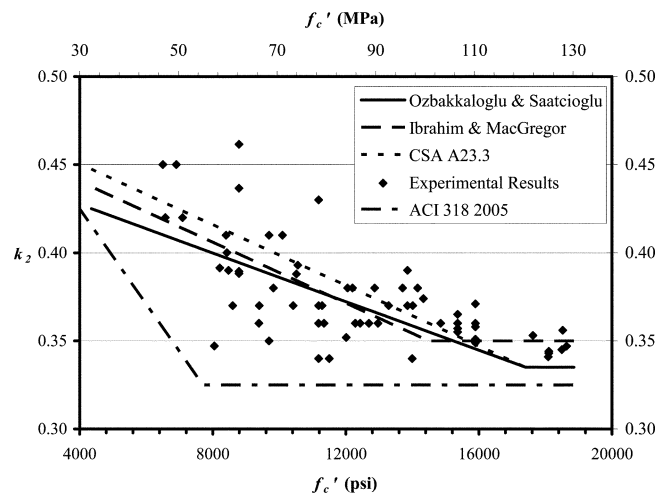


Fig. 4.2—Variation of k_2 with concrete strength (Ozbakkaloglu and Saatcioglu 2004).

rectangular stress block for high-strength concrete. They reported average values of $k_2 = 0.381$ ($\beta_1 = 0.762$) and $k_1 k_3 = 0.647$ ($\alpha_1 = 0.849$) from tests of C-shaped specimens (column specimens in which axial load and bending are induced by applying a load eccentrically at both ends) by several researchers, in which compressive strengths varied from 8400 to 14,400 psi (58 to 99 MPa). The aforementioned values are very close to those corresponding to a parabolic distribution. Specimens with higher strengths tested by Ibrahim and MacGregor (1994, 1996a), with concrete compressive strengths ranging between 17,600 and 18,600 psi (121 to 128 MPa), had values of $k_2 = 0.347$ ($\beta_1 = 0.694$) and $k_1 k_3 = 0.524$ ($\alpha_1 = 0.755$), close to those corresponding to a linear distribution. Ozbakkaloglu and Saatcioglu (2004) summarized the variation of experimentally obtained values for k_2 and the product $k_1 k_3$ with concrete compressive strength. They also presented a comparison with various design expressions, including those of ACI 318-05 and CSA A23.3-94 (Canadian Standards Association 1994). These are shown in Fig. 4.2 and 4.3 and indicate a gradual variation of k_2 and $k_1 k_3$ with increasing concrete strength.

While the product of the terms k_1 and k_3 is often used as a single parameter in the formulation of an equivalent rectangular stress block, researchers in the past identified the values for k_3 separately. The parameter k_3 represents the ratio of the in-place strength of concrete in a structural member to the compressive strength measured using standard cylinder tests. Saatcioglu et al. (1998) reported values of the k_3 factor for high-strength concrete measured by several researchers for unconfined concrete members subjected to concentric loading. Two 10 in. (250 mm) square columns with compressive strengths of 11,700 and 17,600 psi (81 and 121 MPa), tested by Saatcioglu and Razvi (1998), had k_3 factors of 0.89 and 0.92, respectively. The average value reported by Cusson and Paultre (1994) was 0.88 for columns with compressive strengths of 14,500 psi (100 MPa). Tests by Yong et al. (1988) indicated values of 0.87 and 0.97 for compressive strengths of 12,100 and 13,600 psi (83 and 94 MPa), respectively. Sun and Sakino (1994) obtained values of 0.93 and 0.91 for compressive strengths of 7500 and 19,000 psi (52 and 131 MPa), respectively. Saatcioglu and Razvi (1998) indicated that similar values of k_3 were obtained under eccentric loading.

Other tests performed to measure the value of k_3 include those by Ibrahim and MacGregor (1994, 1996b), Kaar et al. (1977), Schade (1992), and Swartz et al. (1985). The aforementioned series of tests resulted in average k_3 values of 0.91, 1.00, 0.93, and 0.98, respectively. Ibrahim and MacGregor (1994, 1996b) reported mean k_3 values of 0.932 for specimens with concrete compressive strengths between 8400 and 14,400 psi (58 and 99 MPa), and 0.919 for specimens with higher compressive strengths ranging between 17,600 and 18,600 psi (121 and 128 MPa).

4.2—Stress intensity factor α_1

According to Fasching and French (1998), experimental results show that the nominal strength of beams calculated using the stress intensity factor α_1 of ACI 318-05 is conservative for high-strength concrete. Data reported by Kaar et al. (1977) had a mean value of $\alpha_1 = 1.0$, and the data reported by Swartz et al. (1985) had a mean value of $\alpha_1 = 0.96$.

Ibrahim and MacGregor (1994, 1996a) conducted extensive tests of concentrically and eccentrically loaded high-strength concrete columns and developed an expression for α_1 . They found lower stress intensity factors in concentrically loaded columns, which resulted in the following expression for the stress intensity factor

$$\alpha_1 = 0.85 - \frac{0.00862f'_c}{1000} \geq 0.725 \quad (f'_c \text{ in psi}) \quad (4-1)$$

$$\alpha_1 = 0.85 - 0.00125f'_c \geq 0.725 \quad (f'_c \text{ in MPa})$$

The equation by Ibrahim and MacGregor was used as the basis for the Canadian Standard CSA A23.3-94 (Canadian Standards Association 1994), where the value of the stress intensity factor is

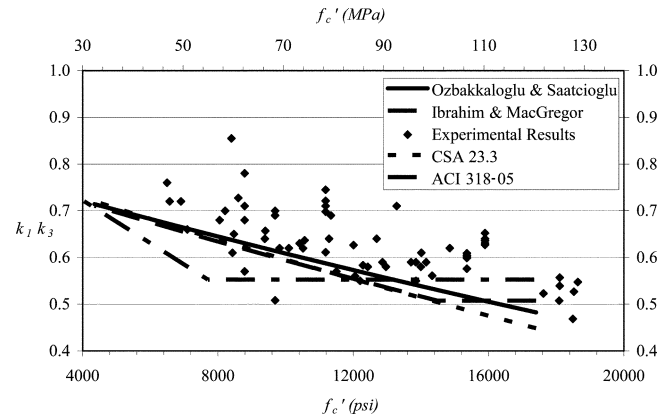


Fig. 4.3—Variation of k_1k_3 with concrete strength (Ozbakkaloglu and Saatcioglu 2004).

$$\alpha_1 = 0.85 - \frac{0.01f'_c}{1000} \geq 0.67 \quad (f'_c \text{ in psi}) \quad (4-2)$$

$$\alpha_1 = 0.85 - 0.0015f'_c \geq 0.67 \quad (f'_c \text{ in MPa})$$

Park et al. (1998) described the background considerations of the NZS 3101:1995 design provisions (Standards Association of New Zealand 1995) regarding the shape of the equivalent rectangular stress block, which is very similar to that used in ACI 318-05. As stated previously, for a linear stress distribution, the equivalent rectangular stress block has values of $k_2 = 0.333$ ($\beta_1 = 0.667$) and $k_1 = 0.5$ ($\alpha_1 = 0.75k_3$). The following expression for the stress factor is used in the New Zealand Standard, which is close to that corresponding to a linear stress distribution for high-strength concrete

$$\alpha_1 = 0.85, \text{ for } f'_c \leq 8000 \text{ psi (55 MPa)} \quad (4-3)$$

$$\alpha_1 = 0.85 - \frac{0.0275(f'_c - 8000)}{1000} \geq 0.75 \text{ for } f'_c > 8000 \text{ (} f'_c \text{ in psi)} \quad (4-4)$$

$$\alpha_1 = 0.85 - 0.004(f'_c - 55) \geq 0.75 \text{ for } f'_c > 55 \text{ (} f'_c \text{ in MPa)}$$

Azizinamini et al. (1994) investigated columns subjected to axial load and flexure, and observed that the ACI 318-05 equivalent stress block resulted in conservative estimates of strength for columns with normal-strength concrete, while it overestimated the strength of columns with high-strength concrete. Based on this observation, they recommended maintaining the value of $\alpha_1 = 0.85$ for $f'_c \leq 10,000$ psi (69 MPa) and changing it for $f'_c > 10,000$ psi (69 MPa) using the following expression

$$\alpha_1 = 0.85 - \frac{0.50(f'_c - 10,000)}{1000} \geq 0.60 \text{ for } f'_c > 10,000 \text{ (} f'_c \text{ in psi)} \quad (4-5)$$

$$\alpha_1 = 0.85 - 0.00725(f'_c - 69) \geq 0.60 \text{ for } f'_c > 69 \text{ (} f'_c \text{ in MPa)}$$

Bae and Bayrak (2003) developed a proposed stress-strain relationships for high-strength concrete. The

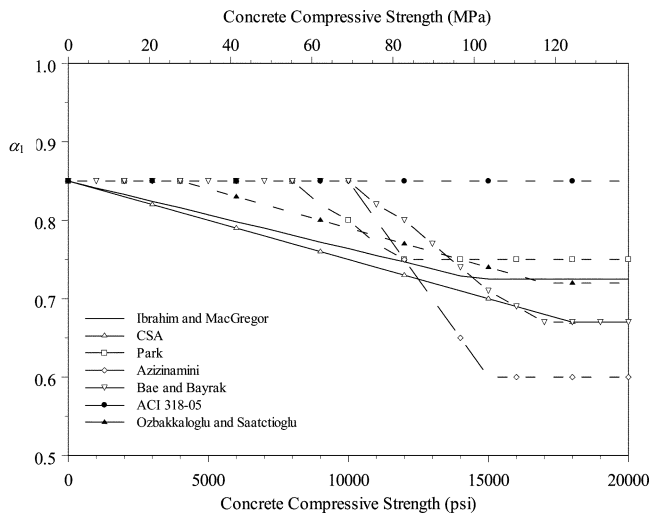


Fig. 4.4—Comparison of proposed expressions for stress intensity factor α_1 .

stress intensity factor α_1 was derived by finding the total area underneath the theoretical stress-strain curve. According to Bae and Bayrak (2003)

$$\alpha_1 = 0.85 - 2.75 \times 10^{-5}(f'_c - 10,000), 0.67 \leq \alpha_1 \leq 0.85 \quad (f'_c \text{ in psi}) \quad (4-6)$$

$$\alpha_1 = 0.85 - 0.004(f'_c - 70), 0.67 \leq \alpha_1 \leq 0.85 \quad (f'_c \text{ in MPa})$$

Ozbakkaloglu and Saatcioglu (2004) developed a rectangular stress block for high-strength and normal-strength concretes based on a large volume of experimental data and an analytical stress-strain relationship. They suggested varying α_1 with concrete compressive strength to reflect the change in the shape of the stress-strain relationship. Accordingly

$$\alpha_1 = 0.85 - (f'_c - 4000) \times 10^{-5}, 0.72 \leq \alpha_1 \leq 0.85 \quad (f'_c \text{ in psi}) \quad (4-7)$$

$$\alpha_1 = 0.85 - 0.0014(f'_c - 30), 0.72 \leq \alpha_1 \leq 0.85 \quad (f'_c \text{ in MPa})$$

A comparison of the ACI 318-05 stress intensity factor α_1 and the aforementioned recommended changes for the stress intensity factor is shown in Fig. 4.4.

4.3—Stress block depth parameter β_1

The parameter β_1 defines the ratio of the depth of the equivalent rectangular stress block to that of the neutral axis. For a constant value of the stress intensity factor α_1 , the effect of assuming a theoretical value of β_1 smaller than the actual value is that the calculated lever arm is increased, resulting in unconservative estimates of the moment strength.

Fasching and French (1998) evaluated the ACI 318-95 expression (same as in ACI 318-05) for factor β_1 using experimental results reported by Ibrahim and MacGregor (1994), Kaar et al. (1977), and Swartz et al. (1985). Fasching and French concluded that the ACI 318 expression for β_1 underestimated the experimentally observed values in the data set used for the evaluation.

A similar conclusion was derived by Ibrahim and MacGregor (1994), who proposed the following expression for β_1

$$\beta_1 = 0.95 - \frac{0.0172f'_c}{1000} \geq 0.70 \quad (f'_c \text{ in psi}) \quad (4-8)$$

$$\beta_1 = 0.95 - 0.0025f'_c \geq 0.70 \quad (f'_c \text{ in MPa})$$

The previous equation served as the basis for and is very similar to the equation adopted in CSA A23.3-94 (Canadian Standards Association 1994)

$$\beta_1 = 0.97 - \frac{0.0172f'_c}{1000} \geq 0.67 \quad (f'_c \text{ in psi}) \quad (4-9)$$

$$\beta_1 = 0.97 - 0.0025f'_c \geq 0.67 \quad (f'_c \text{ in MPa})$$

The stress block depth parameter recommended by Park (1998), and subsequently adopted in NZS 3101:1995 design provisions (Standards Association of New Zealand 1995), has the same definition as the depth parameter β_1 in ACI 318-05. Similarly, Azizinamini et al. (1994) recommended no change to the definition of β_1 used in ACI 318-05. In effect, these authors implied that changing the location of the equivalent force C_c (Fig. 4.1) relative to the extreme compression fiber has a negligible effect on the nominal moment strength because the term $(1/2)\beta_1 c$ is small in comparison to the moment arm $jd = (d - [1/2]\beta_1 c)$. In columns with small eccentricities, the precision of β_1 will have a more significant influence on the moment arm and, consequently, on the nominal moment strength. The overall effect of reducing the stress intensity factor α_1 while maintaining the parameter β_1 similar to that in ACI 318-05 is that a larger neutral axis depth is calculated for a given amount of reinforcement and axial load, reducing the lever arm and the nominal moment strength of the section.

Bae and Bayrak (2003) suggested the following expression for the parameter β_1 by finding the location of the compression resultant for the theoretical stress-strain curve

$$\beta_1 = 0.85 - 2.75 \times 10^{-5}(f'_c - 4000), 0.67 \leq \beta_1 \leq 0.85 \quad (f'_c \text{ in psi}) \quad (4-10)$$

$$\beta_1 = 0.85 - 0.004(f'_c - 30), 0.67 \leq \beta_1 \leq 0.85 \quad (f'_c \text{ in MPa})$$

Ozbakkaloglu and Saatcioglu (2004) recommended a gradual change in β_1 starting at 4000 psi (28 MPa) to reflect the variation in internal lever arm with the changing shape of the stress-strain relationship of concrete. Their recommended relationship for β_1 is

$$\beta_1 = 0.85 - 1.3 \times 10^{-5}(f'_c - 4000), 0.67 \leq \beta_1 \leq 0.85 \quad (f'_c \text{ in psi}) \quad (4-11)$$

$$\beta_1 = 0.85 - 0.020(f'_c - 30), 0.67 \leq \beta_1 \leq 0.85 \quad (f'_c \text{ in MPa})$$

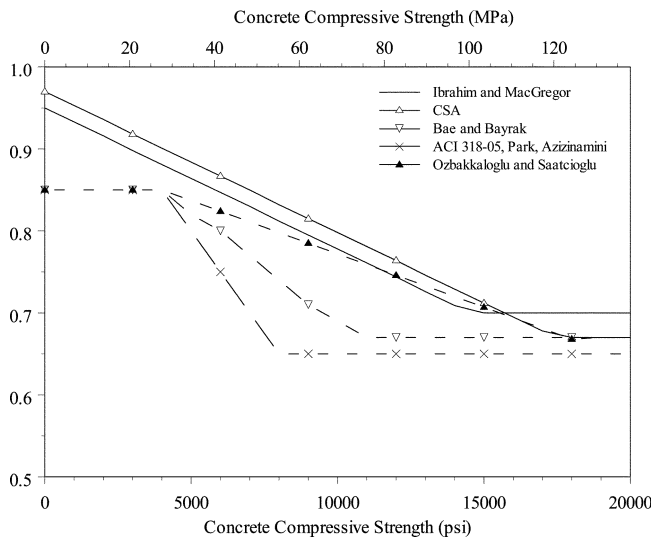


Fig. 4.5—Comparison of proposed expressions for stress block depth factor β_1 .

A comparison of the ACI 318-05 stress block depth parameter β_1 and the aforementioned recommended changes to the depth parameter are shown in Fig. 4.5.

4.4—Stress block area $\alpha_1\beta_1$

The product $\alpha_1\beta_1$ is an indication of the area of the stress block. Fasching and French (1998), using the data from Ibrahim and MacGregor (1994), Kaar et al. (1977), and Swartz et al. (1985), showed that the product $\alpha_1\beta_1$ decreased with increasing compressive strength. The decrease was approximately linear from a value of 0.75 for 6000 psi (41 MPa) to 0.5 for 18,000 psi (124 MPa). The provisions in ACI 318-05 include a steeper descent in the product $\alpha_1\beta_1$ from 4000 to 8000 psi (28 to 55 MPa) than results from stress block parameters proposed by several authors for high-strength concrete (Bae and Bayrak 2003; Ibrahim and MacGregor 1997; Ozbakkaloglu and Saatcioglu 2004). Fasching and French (1998) indicated that the steeper descent in the product $\alpha_1\beta_1$ resulted in underestimating the area of the compression block for specimens with concrete compressive strengths up to 14,000 psi (97 MPa), and overestimating the area of the compression block for specimens with concrete compressive strengths of 18,000 psi (124 MPa). For concrete compressive strengths on the order of 18,000 psi (124 MPa), the inferred values of the coefficients α_1 and β_1 were similar to those corresponding to a linear stress distribution.

4.5—Limiting strain ϵ_{cu}

The limiting strain at the extreme compression fiber at the onset of concrete crushing, ϵ_{cu} , is a significant parameter for calculating the nominal moment strength of columns because it defines the strains throughout the cross section, particularly the strains in the longitudinal reinforcement. Calculated strains have a direct effect on the calculated stresses in the longitudinal reinforcement and also on the magnitude of the strength reduction factor ϕ . ACI 318-05 indicates that the magnitude of the strain at the extreme compression fiber ϵ_{cu} is independent of compressive

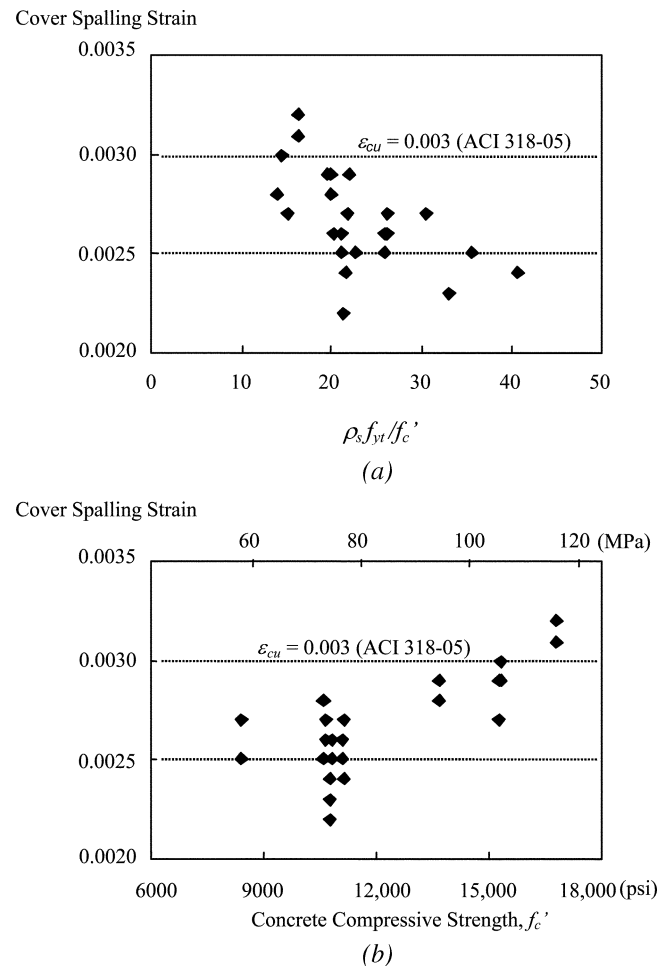


Fig. 4.6—Cover spalling strains for high-strength concrete columns (Bae and Bayrak 2003).

strength, and should be taken as 0.003. The majority of design provisions and proposals presented (Ibrahim and MacGregor 1994; Standards Association of New Zealand 1995; Aziznamini et al. 1994; Ozbakkaloglu and Saatcioglu 2004) adopt the same limiting strain of 0.003 as ACI 318-05, whereas CSA A23.3-94 adopts a limiting strain of 0.0035. Fasching and French (1998) indicated that past research on the magnitude of ϵ_{cu} for high-strength concrete resulted in mixed conclusions, with some researchers indicating that the limiting strain increases with compressive strength, and others indicating that it decreases. A review of test data by Fasching and French showed that the limiting strain was more sensitive to the type of aggregate than the concrete compressive strength, with limiting strains ranging between 0.002 and 0.005 for compressive strengths greater than 8000 psi (55 MPa). Average values for each type of aggregate were all above 0.003, and the average for all types of aggregate was 0.0033.

Bae and Bayrak (2003) suggested adopting a lower value of ϵ_{cu} due to observed spalling at lower strains in highly confined high-strength concrete columns (Fig. 4.6). They proposed using a limiting strain of 0.0025 for concrete compressive strengths greater than 8000 psi (55 MPa), and 0.003 for lower compressive strengths.

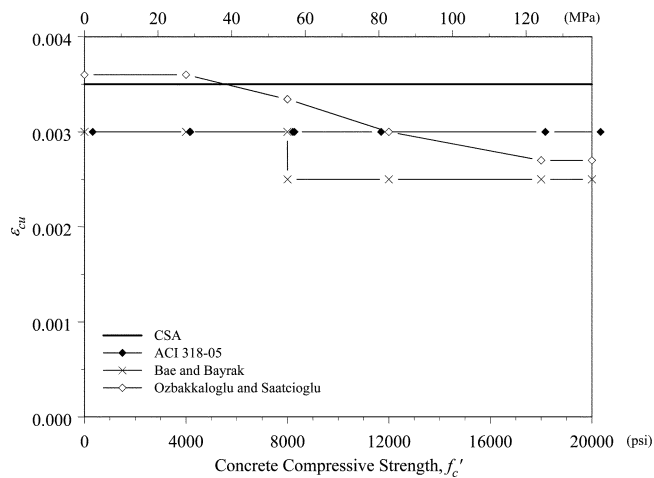


Fig. 4.7—Variation of ultimate strain with concrete strength according to various design codes and authors.

Ozbakkaloglu and Saatcioglu (2004) reported that, while the crushing strain under uniform compression, ϵ_o , increases with increasing concrete strength, the crushing strain under strain gradient, ϵ_{cu} , decreases with increasing concrete strength because of the brittleness of high-strength concretes.

Based on moment-curvature analyses of columns under different levels of axial compression, the researchers concluded that ϵ_{cu} varied between 0.0036 and 0.0027 for 4000 to 18,000 psi (28 and 124 MPa) concretes, respectively. This is shown in Fig. 4.7. The same researchers, however, also concluded that the variation in ϵ_{cu} did not appreciably affect sectional strength calculations, and hence recommended the use of a constant average value of $\epsilon_{cu} = 0.003$ for members under strain gradient.

4.6—Axial strength of high-strength concrete columns

The design expression used in ACI 318-05 to calculate the strength of concentrically loaded columns, similar in form to Eq. (4-12), is based on an extensive column investigation that was conducted jointly by the University of Illinois (Richart and Brown 1934), Lehigh University (Slater and Lyse 1931a,b), and ACI. One of the main conclusions of this research was that it was possible to express the strength of columns subjected to concentric loading in a simple form, consisting of contributions from: 1) concrete at peak stress; and 2) longitudinal steel at yield

$$P_o = 0.85f'_c(A_g - A_{st}) + A_{st}f_y \quad (4-12)$$

The concrete contribution is based on the in-place strength and the net area of concrete, including the cover. The in-place strength of concrete is assumed to be 85% of the cylinder strength. The reduction in strength is attributed to the differences in size, shape, and concrete casting practice between a standard cylinder and an actual column. This ratio of in-place strength to cylinder strength, defined as the coefficient k_3 in Section 4.1, is one of the parameters necessary to define the rectangular stress block. Experimental data are available for in-place strength of high-strength concrete, as

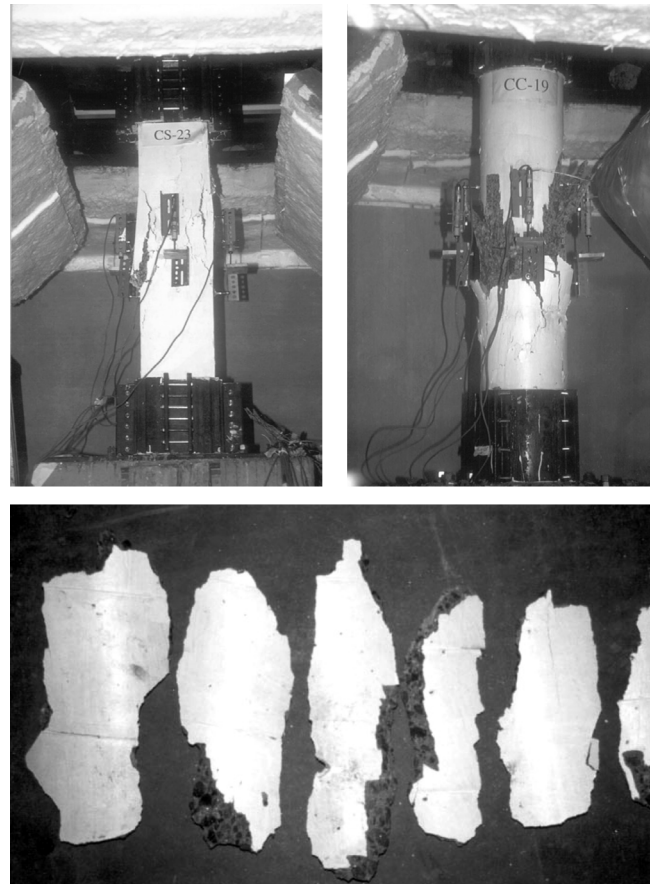


Fig. 4.8—Instability of cover concrete under concentric compression (Saatcioglu and Razvi 1998). The bottom photograph shows section of the cover that spalled off during the tests.

indicated in Section 4.1. Researchers found that the coefficient k_3 for high-strength concrete varied between 0.87 and 0.97 based on concentrically tested columns (Yong et al. 1988; Sun and Sakino 1993; Cusson and Paultre 1994; Saatcioglu and Razvi 1998). A similar variation was obtained from column tests under eccentric loading (Kaar et al. 1977; Swartz et al. 1985; Schade 1992; Ibrahim and MacGregor 1994, 1996b). Having reviewed the previous experimental data, Ozbakkaloglu and Saatcioglu (2004) concluded that $k_3 = 0.9$ provides a reasonable estimate for the ratio of concrete strength in a structural member to that determined by standard cylinder tests.

In spite of the favorable in-place strength of high-strength concrete, experimentally recorded column strengths have been shown to be below the computed values based on Eq. (4-12) unless the columns are confined by properly designed transverse reinforcement. The strain data recorded by Saatcioglu and Razvi (1998) during their tests of high-strength concrete columns indicated that premature spalling of cover concrete occurred in most columns before the development of strains associated with concrete crushing. This observation, combined with visual observations of cover spalling during tests, as shown in Fig. 4.8, suggests that the cover concrete in high-strength concrete columns suffers stability failure rather than crushing.

Saatcioglu and Razvi (1998) hypothesized that the presence of closely spaced longitudinal and transverse steel, forming a mesh of reinforcement, produced a natural plane of separation between the cover and the core. The separation along this plane was triggered by high compressive stresses associated with high-strength concrete as well as the differences in mechanical properties of core and cover concretes (Richart et al. 1929; Roy and Sozen 1963). Columns tested by Rangan et al. (1991) and some of the columns tested by Yong et al. (1988) contained widely spaced transverse reinforcement of low volumetric ratio, without a sufficient mesh of reinforcement to separate the cover from the core. These columns were able to develop unconfined column strengths P_o calculated using Eq. (4-12). Columns tested by Itakura and Yagenji (1992) without any cover consistently showed higher strengths than those computed on the basis of gross cross-sectional area and unconfined concrete because they did not suffer strength loss due to cover spalling. Columns that were sufficiently confined to offset the effects of cover spalling consistently developed higher strengths than P_o . The group that contained an insufficient volumetric ratio of closely spaced transverse reinforcement, however, could not sustain strengths computed on the basis of total cross-sectional area and unconfined concrete strength.

According to Saatcioglu and Razvi (1998), given the unfavorable circumstances described previously, the premature spalling of cover concrete could lead to reduced strength of concentrically loaded high-strength concrete columns relative to those predicted by Eq. (4-12). The effect of premature cover spalling was introduced into Eq. (4-12) by Ozbakkaloglu and Saatcioglu (2004) through a coefficient k_4 by defining the in-place strength of concrete as $k_3k_4f'_c$ instead of $k_3f'_c$, where $k_3 = 0.85$. Figure 4.9 shows the variation of the product k_3k_4 with concrete strength obtained from a large volume of test data. The test data also included moderately confined columns for which high values of the product were obtained. The strength loss associated with cover spalling is a function of the area of unconfined cover concrete. For this reason, this effect can be quantified in terms of the ratio of core area to gross area (A_c/A_g) of the column. As this ratio decreases (cover thickness increases), the strength loss increases. Figure 4.10 illustrates the variation of the product k_3k_4 with respect to the A_c/A_g ratio. The product k_3k_4 in Figure 4.10 indicates the degree of premature loss of strength in high-strength concrete columns as a function of concrete compressive strength and the A_c/A_g ratio. This premature spalling effect can be quite significant in small-scale test columns with thin covers (Ozbakkaloglu and Saatcioglu 2004).

Because the stability of the cover improves as the cover thickness increases, columns with thick covers are less likely to be susceptible to premature spalling than those with thin covers. Given the difficulties associated with testing large-scale columns with very high concrete compressive strengths under concentric compression, there is a paucity of experimental results for large-scale high-strength concrete columns with thick concrete covers. For this reason, it was suggested by Ozbakkaloglu and Saatcioglu (2004) that, until more data become available, the ratio A_c/A_g should not be

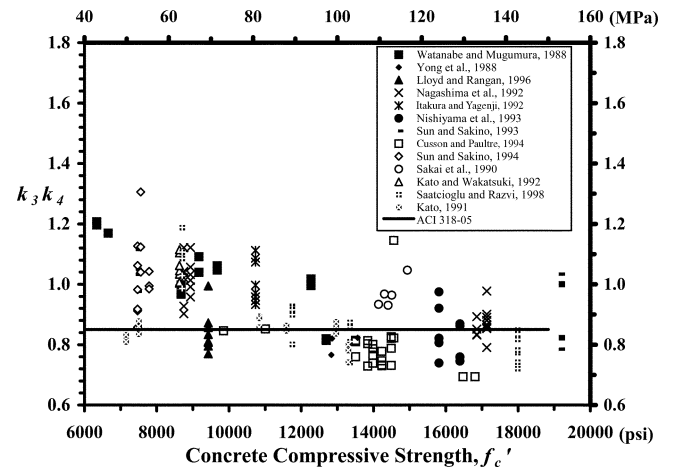


Fig. 4.9—Variation of k_3k_4 with concrete compressive strength (Ozbakkaloglu and Saatcioglu 2004).

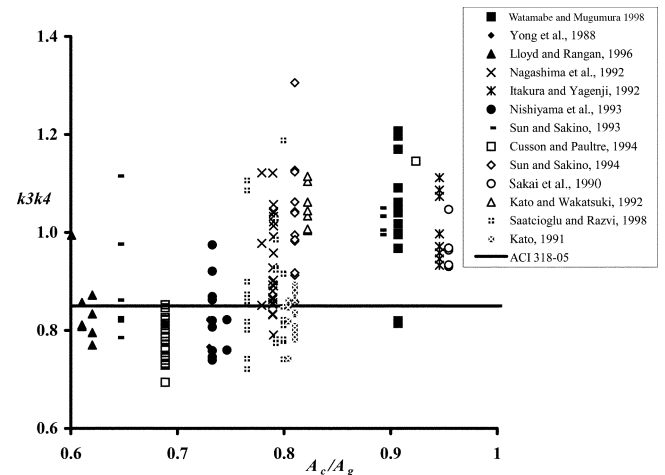


Fig. 4.10—Variation of k_3k_4 with core-to-gross area ratio (Ozbakkaloglu and Saatcioglu 2004).

taken less than 0.6, irrespective of its actual value, in assessing the premature cover spalling effect.

The test data in Fig. 4.9 and 4.10 were further examined after removing confined column data and grouping them on the basis of concrete strength (Ozbakkaloglu and Saatcioglu 2004). A regression analysis was conducted to find an expression for the coefficient k_4 . The researchers suggested the following expressions for computing concentric axial strength of high-strength concrete columns

$$P_o = k_3k_4f'_c(A_g - A_{st}) + A_{st}f_y \quad (4-13)$$

$$k_3 = 0.90 \quad (4-14)$$

$$k_4 = \gamma_c + (1 - \gamma_c) \frac{A_c}{A_g} \leq 0.95 \quad (4-15)$$

$$\gamma_c = 1.1 - \frac{f'_c}{20,000} \leq 0.8 \quad (f'_c \text{ in psi}) \quad (4-17)$$

$$\gamma_c = 1.1 - \frac{f'_c}{138} \leq 0.8 \quad (f'_c \text{ in MPa})$$

The product k_3k_4 can be as low as 0.61 for 18,000 psi (124 MPa) concrete and $A_c/A_g = 0.6$, which is 28% below the 0.85 value suggested by ACI 318-05 for normal-strength concrete columns, as reproduced in Eq. (4-12). Instead of detailed computation of the coefficient k_4 , as outlined previously, a conservative, but simple, approach was recommended for convenience in design by Ozbakkaloglu and Saatcioglu (2004). They suggested that the product k_3k_4 be taken as 0.85 for f'_c of up to 6000 psi (41 MPa), and be reduced by 0.017 for every 1000 psi (6.9 MPa) increase over 6000 psi (41 MPa), up to 18,000 psi (124 MPa). The researchers identified the premature cover spalling as a phenomenon that is prevalent in concentrically loaded high-strength concrete columns. For columns subjected to bending and axial load, Ozbakkaloglu and Saatcioglu (2004) indicated that the critical compression side of the cover would deform toward the core concrete, which would restrain the cover against buckling.

Park et al. (1998) indicated that the axial strength of columns subjected to compression is

$$P_o = \chi_1 f'_c (A_g - A_{st}) + f_y A_{st} \quad (4-18)$$

They pointed out that the k_3 values that have been measured under concentric compression are greater than the value of χ_1 in the NZS 3101:1995 provisions (Standards Association of New Zealand 1995) and, as a result, the nominal axial strength calculated using that standard is conservative. Aziznamini et al. (1994) proposed calculating the axial strength of columns in the same manner as NZS 3101:1995 by using Eq. (4-18). The premature spalling of cover concrete was recognized by CSA A23.3-94 (Canadian Standards Association 1994), and Eq. (4-18) was adopted with the stress intensity factor χ_1 decreasing as a function of concrete strength, reducing to 0.67 for 18,000 psi (124 MPa) concrete.

4.7—Comparison of different proposals for rectangular stress block

Fasching and French (1998) carried out a comparison between the measured flexural strengths of beam members and those calculated according to different stress block proposals for high-strength concrete. They found a slightly higher level of conservatism for the stress block proposals for high-strength concrete that they evaluated compared with the stress block defined in ACI 318-05. The New Zealand and Canadian proposals resulted in nearly identical average ratios of experimental-to-calculated strengths of 1.25, while the stress block of ACI 318-05 resulted in an average ratio of 1.21. Because the depth of the compression zone in beams is small compared with the depth of the member, it was

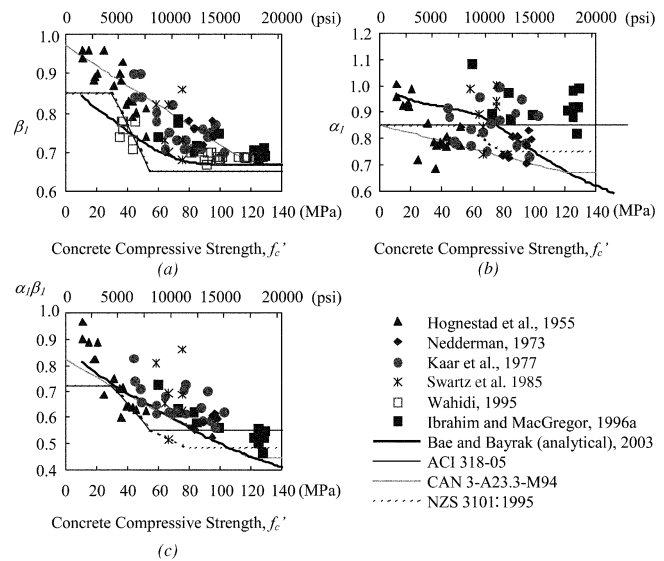


Fig. 4.11—Comparison of stress block parameters α_1 and β_1 inferred from experimental results and various expressions proposed for high-strength concrete (Bae and Bayrak 2003).

anticipated that the proposed modifications to the stress block would have a small effect on the nominal moment strength of beams. Fasching and French (1998) recommended that the stress block should be modified to avoid unconservative estimates of column strength.

Bae and Bayrak (2003) compared the measured strengths of 224 columns with the strengths calculated using the ACI 318-05 rectangular stress block and other stress blocks outlined in this review (Fig. 4.11 and 4.12). Figure 4.11 shows the variation of the factors α_1 and β_1 , and the product $\alpha_1\beta_1$ proposed by several investigators with respect to concrete compressive strength.

To estimate the accuracy of moment and axial strengths, Bae and Bayrak (2003) developed two different error indicators. They defined the error based on the experimental axial force EE_p as the ratio of the difference between the nominal and experimental moment strengths to experimental moment strength (Fig. 4.12). EE_p is calculated as

$$EE_p = \frac{M_{ncol} - M_{exp}}{M_{exp}} \times 100 \quad (4-19)$$

A negative EE_p value implies that the calculated strength was below the measured value, and consequently, the estimate was conservative.

The second error indicator was based on the experimental eccentricity (Bae and Bayrak 2003). Based on both error indicators, Bae and Bayrak concluded that estimates using the equivalent rectangular stress block of ACI 318-05 became increasingly unconservative with increasing compressive strength, particularly with concrete strengths exceeding 10,000 psi (69 MPa).

The stress blocks proposed by Ibrahim and MacGregor (1997), Park et al. (1998), Standards Association of New Zealand (1995), and Bae and Bayrak (2003) all produced

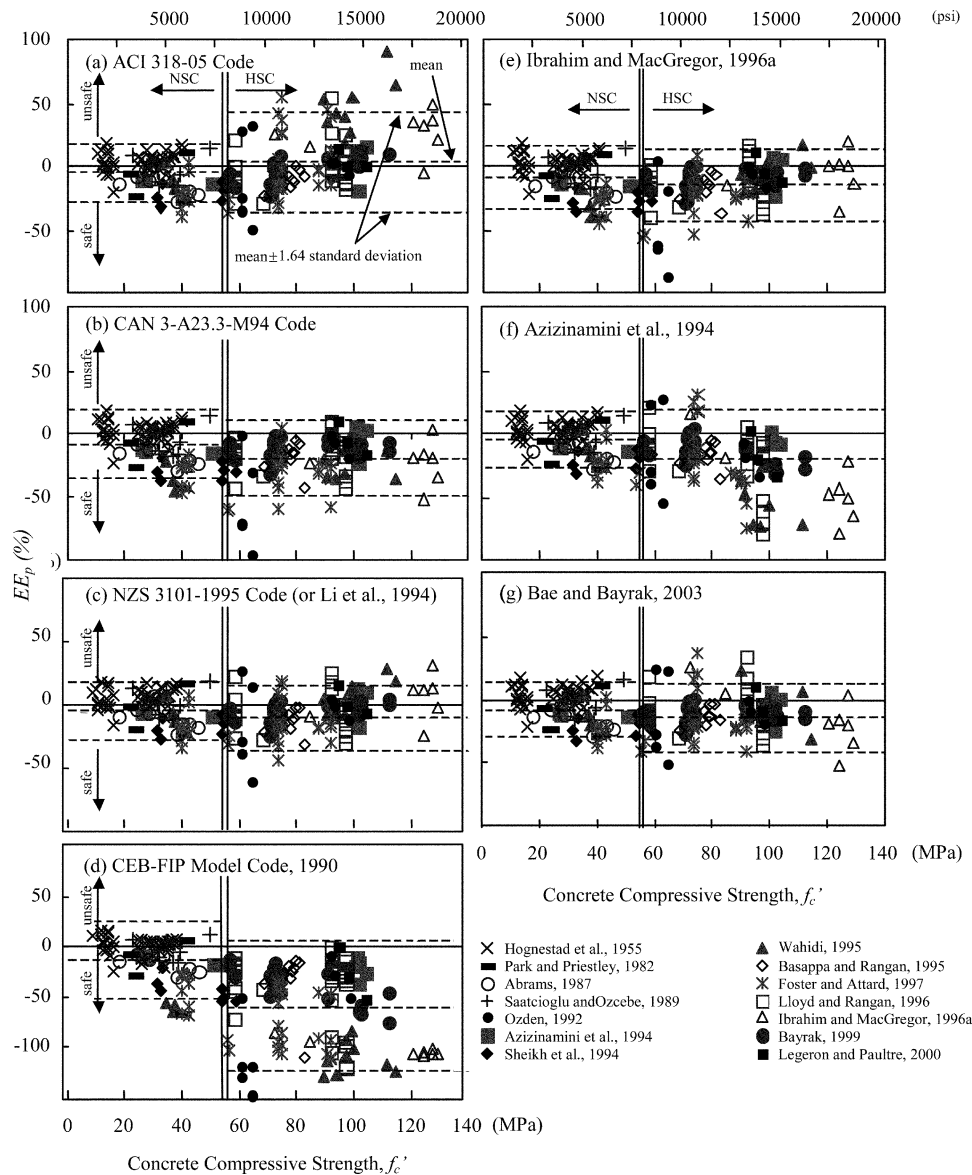


Fig. 4.12—Error parameter EE_p in estimates of column strength (Bae and Bayrak 2003).

similar levels of conservatism for all levels of concrete strength. The model proposed by Azizinamini et al. (1994) increasingly underestimated the column strengths for concrete compressive strengths beyond 13,000 psi (90 MPa). Bae and Bayrak noted that the data they used lacked a significant number of test results with high axial loads (small eccentricities). When axial loads are high, the different models provide significantly different predictions. They also noted that in seismic applications, the concern is not with high axial loads, but with relatively low axial loads (high eccentricities).

Ozbakkaloglu and Saatcioglu (2004) compared column interaction diagrams based on the rectangular stress blocks of ACI 318-05, CSA A23.3-94, and those proposed by Ibrahim and MacGregor (1997) and Ozbakkaloglu and Saatcioglu (2004).

The comparisons, shown in Fig. 4.13, indicate that the interaction diagrams generated by the equivalent rectangular stress block of ACI 318-05 and that proposed by

Ozbakkaloglu and Saatcioglu are identical for columns with a concrete compressive strength of 4000 psi (28 MPa), whereas the equivalent rectangular stress blocks recommended by CSA A23.3 and Ibrahim and MacGregor produce slightly lower estimates of strength than ACI 318-05. As concrete strength increased, Ozbakkaloglu and Saatcioglu concluded that the ACI 318-05 stress block lead to overestimating column strengths obtained from test results. Ozbakkaloglu and Saatcioglu indicated that the magnitude of the overestimation was very significant for a column with a concrete compressive strength of 17,400 psi (120 MPa). For this same column, the rectangular stress blocks proposed by Ibrahim and MacGregor and Ozbakkaloglu and Saatcioglu produced similar interaction diagrams, and the CSA A23.3 stress block resulted in a more conservative estimate of strength. The fact that the results obtained using the rectangular stress block in CSA A23.3 were consistently more conservative was attributed to the use of a lower stress intensity factor, α .

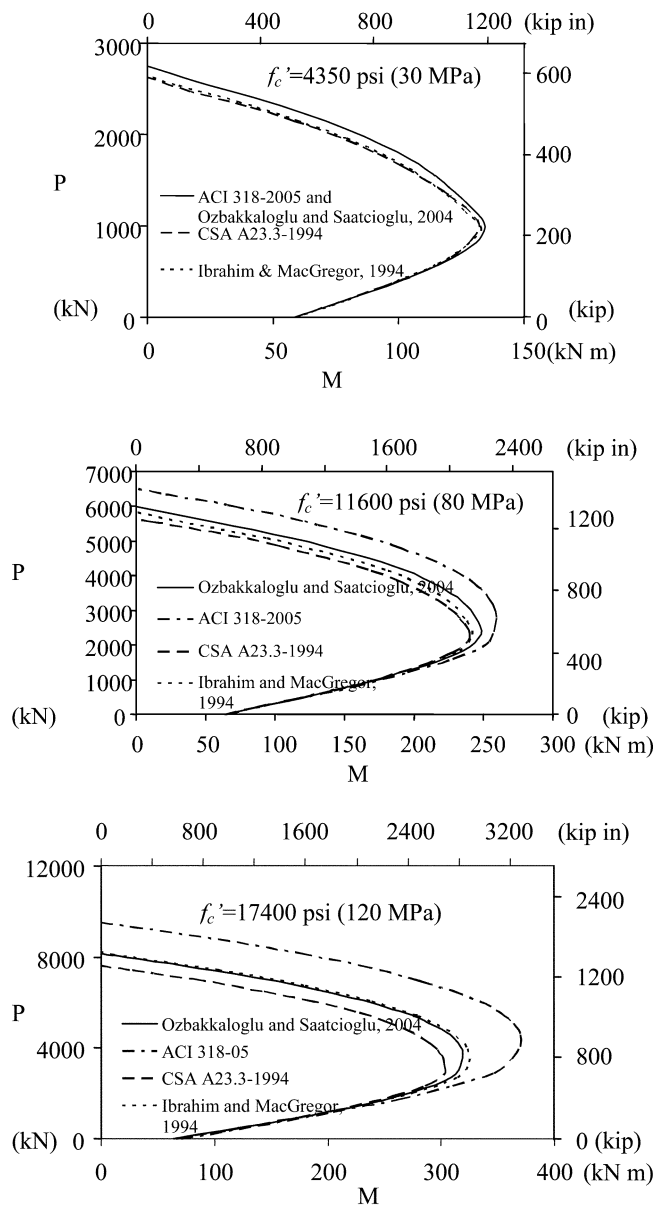
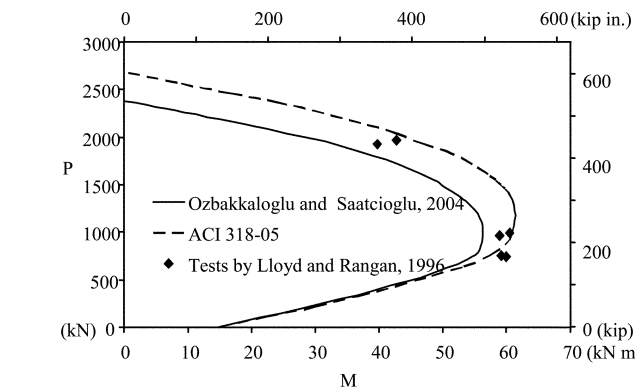


Fig. 4.13—Comparison of interaction diagrams for columns with different concrete strengths (Ozbakkaloglu and Saatcioglu 2004) ($A_c/A_g = 0.7$; $\rho = 1.33\%$; $b = h = 11.81$ in. [300 mm]).

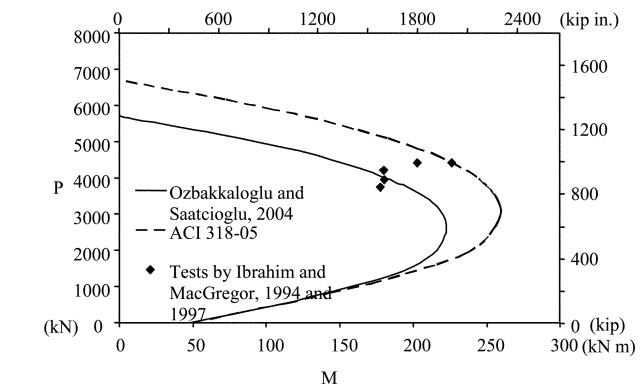
Ozbakkaloglu and Saatcioglu (2004) also provided comparisons of interaction diagrams drawn on the basis of their proposed stress block and that of ACI 318-02 (ACI Committee 318 2002) (which is the same used in ACI 318-05) for columns tested by Lloyd and Rangan (1996), Ibrahim and MacGregor (1994, 1997), and Foster and Attard (1997), under different levels of end eccentricity (Fig. 4.14).

They concluded that the stress block of ACI 318-05 overestimated column axial and moment strengths, resulting in unsafe strength values for columns with concrete strengths in excess of 10,000 psi (69 MPa), whereas their proposed stress block (Ozbakkaloglu and Saatcioglu 2004) provided very good agreement with experimental strength values.

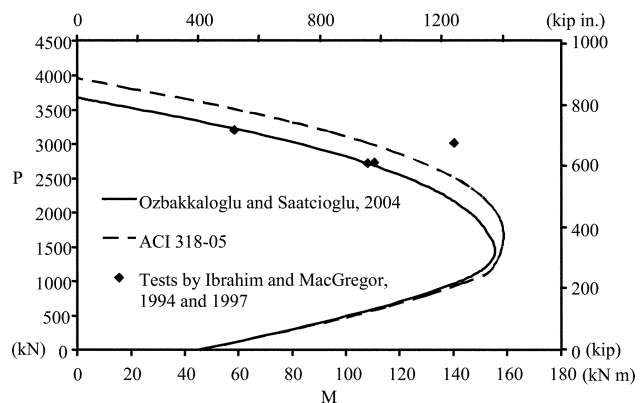
A parametric study was carried out as part of this report to provide further insight into the differences among various



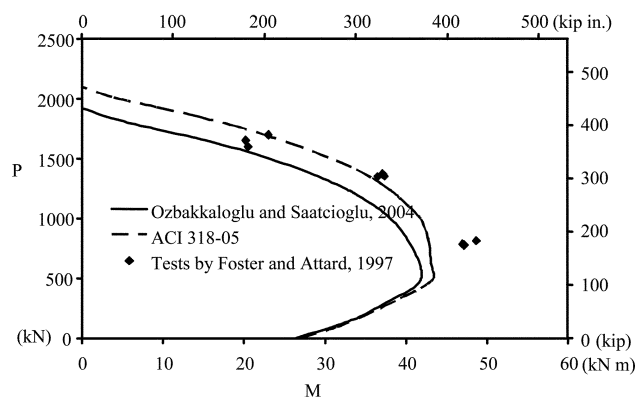
(a) $f'_c = 14,070$ psi (97 MPa), 6.9×6.9 in (175 mm), $\rho = 1.3\%$, $A_c/A_g = 0.84$



(b) $f'_c = 18,275$ psi (126 MPa), 7.9×11.8 in (200 x 300 mm), $\rho = 1.3\%$, $A_c/A_g = 0.6$



(c) $f'_c = 10,440$ psi (72 MPa), 7.9×11.8 in (200 x 300 mm), $\rho = 1.3\%$, $A_c/A_g = 0.6$



(d) $f'_c = 13,050$ psi (90 MPa), 6.9×6.9 in (175 mm), $\rho = 3.6\%$, $A_c/A_g = 0.89$

Fig. 4.14—Comparison of computed interaction diagrams with test data (Ozbakkaloglu and Saatcioglu 2004).

Table 4.1—Summary of parameters α_1 and β_1 defining different rectangular stress blocks investigated in parametric study

Concrete compressive strength, psi (MPa)	4000 (28)		6000 (41)		8000 (55)		10,000 (69)		12,000 (83)		15,000 (103)	
Equivalent rectangular stress block parameter	α_1	β_1	α_1	β_1	α_1	β_1	α_1	β_1	α_1	β_1	α_1	β_1
ACI 318-05	0.85	0.85	0.85	0.75	0.85	0.65	0.85	0.65	0.85	0.65	0.85	0.65
Ibrahim and MacGregor (1997)	0.82	0.88	0.80	0.85	0.78	0.81	0.76	0.78	0.75	0.74	0.73	0.70
Park et al. (1998)	0.85	0.85	0.85	0.75	0.85	0.65	0.80	0.65	0.75	0.65	0.75	0.65
Azizinamini et al. (1994)	0.85	0.85	0.85	0.75	0.85	0.65	0.85	0.65	0.75	0.65	0.60	0.65

proposals. Column interaction diagrams were calculated, with and without strength reduction factors ϕ , to compare the ACI 318-05 stress block with the proposals by Ibrahim and MacGregor (1997), Park et al. (1998), and Azizinamini et al. (1994). The column cross section that was analyzed is shown in Fig. 4.15, with the bending moment about the Y-Y axis. The column was analyzed for steel ratios of 1 and 2.5% and for concrete compressive strengths of 4000, 6000, 8000, 10,000, 12,000, and 15,000 psi (28, 41, 55, 69, 83, and 103 MPa). The stress block parameters for the compared models are given in Table 4.1, and the results of the parametric study are given in Fig. 4.16.

From Fig. 4.16 and Table 4.1, it can be seen that for concrete compressive strengths of 4000, 6000, and 8000 psi (28, 41, and 55 MPa), the only model that resulted in estimates of strength that were noticeably different from those obtained with the ACI 318-05 stress block was that proposed by Ibrahim and MacGregor (1997).

The Ibrahim and MacGregor (1997) model resulted in progressively smaller estimates of nominal strength as concrete compressive strength increased, which indicates that their model was the most conservative in this range. For a concrete compressive strength of 10,000 psi (69 MPa), the ACI 318-05 stress block and that proposed by Azizinamini et al. (1994) produced similar results, whereas the proposals by Ibrahim and MacGregor (1997) and Park et al. (1998) produced more conservative estimates of strength. For a concrete compressive strength of 12,000 psi (83 MPa), the models by Park et al. and Azizinamini et al. have identical stress block parameters. Consequently, strength estimates obtained with these two models were identical, and approximately the same as the nominal strength calculated using the model by Ibrahim and MacGregor. Finally, for a concrete compressive strength of 15,000 psi (103 MPa), the models by Ibrahim and MacGregor and Park et al. yielded similar results, and were slightly more conservative than the equivalent rectangular stress block of ACI 318-05. The model by Azizinamini et al. (1994) resulted in significantly lower estimates of strength than the other models.

4.8—Recommendations

It is apparent from a review of the available literature that when the equivalent rectangular stress block of ACI 318-05 is used for members with axial loads above that corresponding to balanced failure and high-strength concrete, the ratio of nominal-to-experimental column strength decreases as the axial load increases. Experimental results (Fig. 4.12(a)) indicate that the nominal moment and axial strengths of

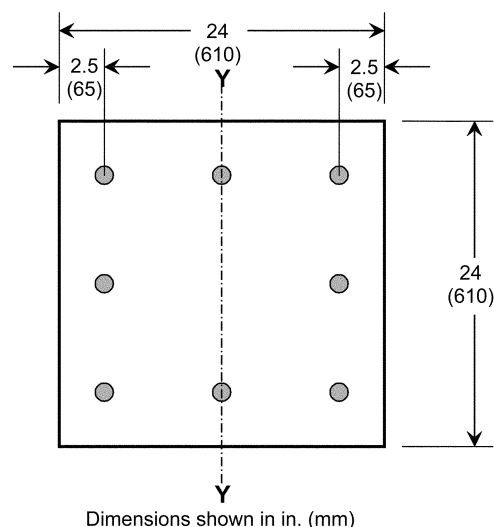


Fig. 4.15—Column cross section used in parametric study.

columns calculated with the ACI 318-05 stress block may be unconservative for compressive strengths greater than approximately 12,000 psi (83 MPa).

Two consequences of overestimating the flexural strengths of columns are that the shear demand on the column calculated on the basis of the probable flexural strength is overestimated and that the ratio of column-to-beam moment strengths is overestimated. Overestimating the shear demand is conservative because it leads to a higher amount of transverse reinforcement. Conversely, overestimating the ratio of column-to-beam moment strengths has a negative effect because it increases the probability of hinging in the columns. ACI 318-05 requires a minimum ratio of column-to-beam moment strengths of 1.2. Overestimating column flexural strength decreases that ratio, and may even result in a strong beam-weak column mechanism.

Because experimental results showed that the equivalent rectangular stress block of ACI 318-05 is appropriate for normal-strength concrete, a recommendation was developed focusing on columns with compressive strengths greater than 8000 psi (55 MPa). This was done by suggesting a stress block with a variable stress intensity factor α_1 for concrete compressive strengths greater than 8000 psi (55 MPa). Accordingly, in inch-pound units, it is recommended that: “factor α_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 8000 psi. For strengths above 8000 psi, α_1 shall be reduced continuously at a rate of 0.015 for each 1000 psi of strength in excess of 8000 psi, but α_1 shall not be less than 0.70.” In SI units, the recommendation is that:

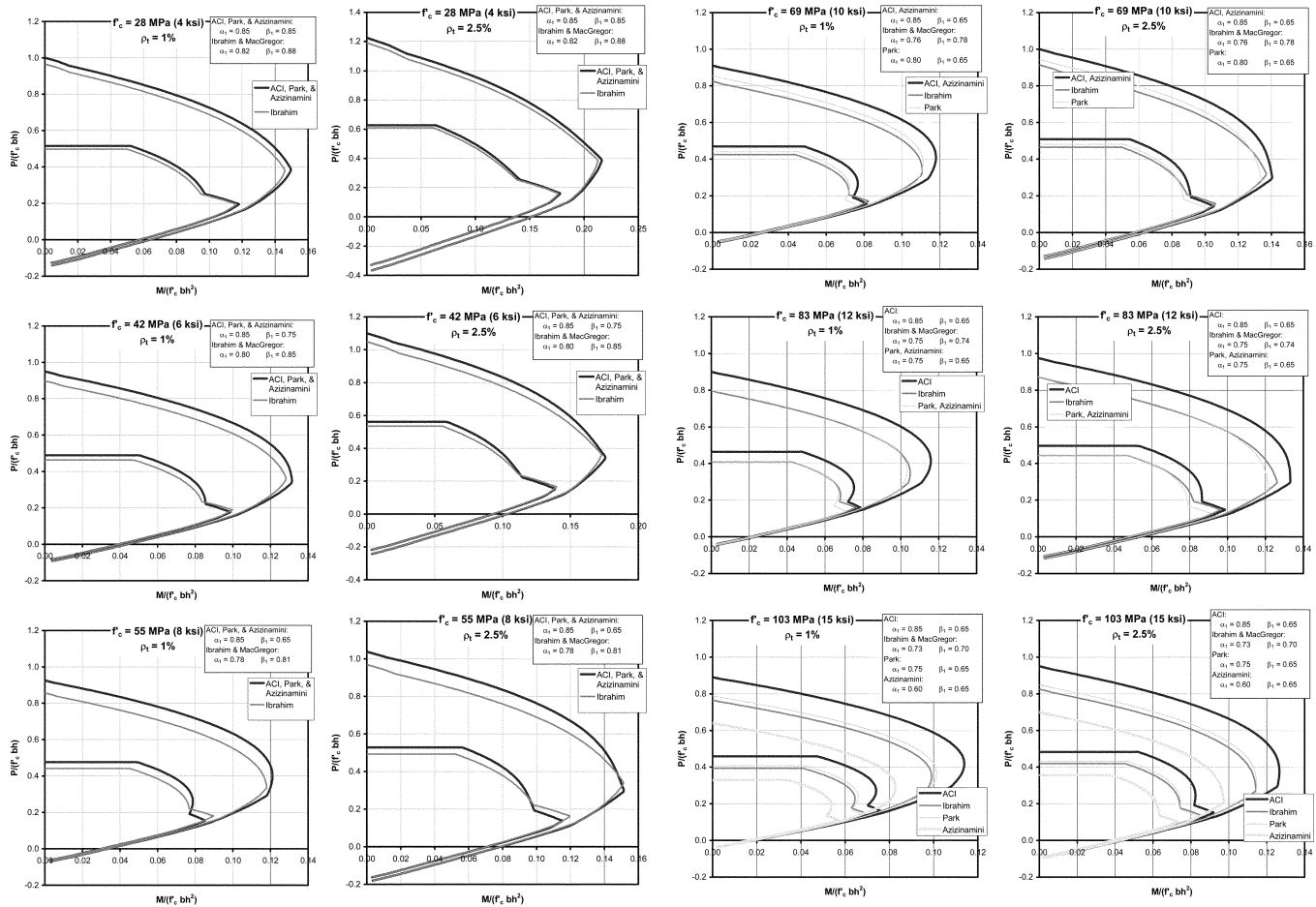


Fig. 4.16—Column strength interaction diagrams comparing different stress blocks.

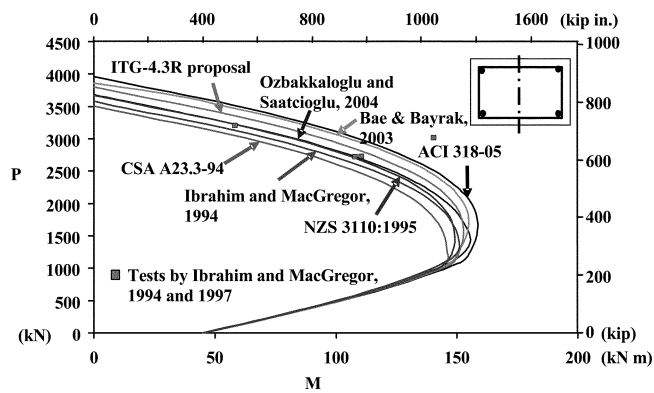


Fig. 4.17—Comparisons of column interaction diagrams and test data ($f'_c = 10,440$ psi [72 MPa], 7.8×11.8 in. (200 x 300 mm), $\rho = 1.3\%$, $A_c/A_g = 0.6$).

“factor α_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 55 MPa. For strengths above 55 MPa, α_1 shall be reduced continuously at a rate of 0.0022 for each 6.9 MPa of strength in excess of 55 MPa, but α_1 shall not be taken less than 0.70.”

A number of revisions to ACI 318-05 are proposed in Chapter 10 of this document.

The parameter β_1 , which defines the depth of the stress block, was not changed. Figures 4.17 to 4.20 show the correlation of

the proposed stress block with those proposed by others, as well as with the results of sample tests on columns using concrete strengths of up to 18,000 psi (124 MPa).

The strength intensity factor α_1 is also recommended to calculate the strength of columns subjected to concentric loading. The similarities in the values of α_1 and the coefficient that defines the in-place strength of concrete in columns under concentric compression χ_1 makes it possible to use the same value in computing column concentric strength P_o for convenience in design. The recommendations translate into Eq. (4-20) and (4-21) for spirally reinforced and tied columns, respectively

$$\phi P_{n,max} = 0.85\phi[\chi_1 f'_c (A_g - A_{st}) + f_y A_{st}] \quad (4-20)$$

$$\phi P_{n,max} = 0.80\phi[\chi_1 f'_c (A_g - A_{st}) + f_y A_{st}] \quad (4-21)$$

Accordingly, in inch-pound units, it is recommended that: “factor χ_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 8000 psi. For strengths above 8000 psi, χ_1 shall be reduced continuously at a rate of 0.015 for each 1000 psi of strength in excess of 8000 psi, but χ_1 shall not be taken less than 0.70.” In SI units, the recommendation is that: “factor χ_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 55 MPa. For strengths above 55 MPa, χ_1 shall be reduced continuously at a rate of 0.0022 for each 6.9 MPa of strength in excess of 55 MPa, but χ_1 shall not be taken less than 0.70.”