**AGENDA**

**DC-20: Building Design Committee**

*Thursday, October 3, 2019*

*1:00 p.m. - 5:00 p.m.*

*Hilton Santa Fe Historic Plaza*

### Voting Members Present (x of 13)

<table>
<thead>
<tr>
<th>Carol Hayek - Chair</th>
<th>Bryan Allred</th>
<th>Don Kline</th>
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<tbody>
<tr>
<td>CCL</td>
<td>Seneca Structural Engineering Inc.</td>
<td>Kline Engineering and Consulting, LLC</td>
</tr>
<tr>
<td>Tim Christle</td>
<td>Asit Baxi</td>
<td>Frank Malits</td>
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<tr>
<td>Post-Tensioning Institute Representative</td>
<td>Baxi Engineering Inc.</td>
<td>Cagley and Associates Inc.</td>
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<td>Hamid Ahmady</td>
<td>Martin A Cuadra</td>
<td>Martin Maingot - Secretary</td>
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<tr>
<td>Suncoast Post-Tension Ltd</td>
<td>Uzun and Case, LLC</td>
<td>SCA Consulting Engineers</td>
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<tr>
<td>Rashid Ahmed</td>
<td>Jonathan Hirsch</td>
<td>Eric Ober</td>
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<td>Walker Consultants</td>
<td>Bentley Systems, Inc.</td>
<td>SGH</td>
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<td></td>
<td>Thomas Kang</td>
<td>Zuming Xia</td>
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<td></td>
<td>Seoul National University</td>
<td>Structural Technologies, Inc.</td>
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### Associate Members Present

<table>
<thead>
<tr>
<th>Joseph Ales</th>
<th>Spencer Lee</th>
<th>Srinivasan Neelamegam</th>
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<tbody>
<tr>
<td>Walter P Moore and Assoc Inc</td>
<td>ADAPT Structural Concrete Software</td>
<td>CCL USA</td>
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<tr>
<td>Ryan Bonniwell</td>
<td>Ralf Leistikow</td>
<td>Otto Schwarz</td>
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<tr>
<td>Ericksen Roed and Associates</td>
<td>Wiss, Janney, Elstner Associates, Inc.</td>
<td>Ryan Biggs Clark Davis</td>
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<tr>
<td>Kyle Boyd</td>
<td>Rafael Machado</td>
<td>Engineering and Surveying</td>
</tr>
<tr>
<td>S.A. Miro</td>
<td>Ellinwood Machado, LLC</td>
<td>Mr Roberto Suarez</td>
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<tr>
<td>Mr Anantha Chittur</td>
<td>Carine Magalhaes</td>
<td>Martinmartin</td>
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<tr>
<td>BASE</td>
<td>ADAPT Corporation</td>
<td>Todd Whisenhunt</td>
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<tr>
<td>Michael Hopper</td>
<td>Marcos Martinez</td>
<td>Thornton Tomasetti</td>
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<td>Leslie E. Robertson Associates</td>
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<td>Mark Yerges</td>
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<tr>
<td>Mr Chad Andrew Konger</td>
<td>Siva Munuswamy</td>
<td>CVM Engineers, Inc.</td>
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<td>Strand Systems Engineering</td>
<td>Thornton Tomasetti Inc</td>
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**ACTION ITEMS FROM LAST / THIS MEETING**

<table>
<thead>
<tr>
<th>Item #</th>
<th>Subject</th>
<th>Action</th>
<th>Responsible</th>
<th>Deadline / Completed</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>Design Webinar Modules</td>
<td>Final QA/QC reviews complete and all 10 modules posted on PTI website</td>
<td>PTI Staff</td>
<td>September 1, 2019</td>
</tr>
<tr>
<td>2</td>
<td>PTI DC20.2-88: Restraint Cracks and their Mitigation</td>
<td>Remaining contributions completed.</td>
<td>All</td>
<td>May 31, 2019</td>
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<tr>
<td></td>
<td></td>
<td>Compiled into final draft document ready for balloting. Each chapter a separate ballot item.</td>
<td>Hayek</td>
<td>June 15, 2019</td>
</tr>
<tr>
<td>3</td>
<td>DC20.7-01: Design, Construction and Maintenance of CIP PT Concrete</td>
<td>Will be removed from DC-20 agenda and placed on agenda of new DC-25 Parking Structures</td>
<td>PTI Staff</td>
<td>Remove from DC-20</td>
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<tr>
<td></td>
<td>Parking Structures</td>
<td>committee</td>
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<tr>
<td>4</td>
<td>ACI 318 Reference</td>
<td>Create an FAQ or Technical Paper on changes between 318-14 and 318-19.</td>
<td>Baxi</td>
<td>October 1, 2019</td>
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<td>Create summary of all PT provisions in 318-19</td>
<td>Kang</td>
<td>September 1, 2019</td>
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<tr>
<td>5</td>
<td>PT Manual 7th Edition</td>
<td>Summary of chapters being authored by and reviewed by DC-20 members</td>
<td>All</td>
<td>Ongoing</td>
</tr>
<tr>
<td>6</td>
<td>Dual-Banded PT Layout</td>
<td>Submit final Tech Note to TAB Testing: Two-Way Banded TG to continue efforts and assisting in test configuration.</td>
<td>Hirsch/Boyd</td>
<td>June 15, 2019</td>
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<td>Dual-Banded TG</td>
<td>Ongoing</td>
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<td>7</td>
<td>Temperature Tendons</td>
<td>Submit final Tech Note to TAB</td>
<td>PTI Staff</td>
<td>July 15, 2019</td>
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<td>8</td>
<td>Structural Detailing for PT Buildings</td>
<td>Tim is starting direct discussions with PT suppliers to obtain their input on problematic details. Ultimate publication goal is before the end of 2019</td>
<td>PTI Staff</td>
<td>Ongoing</td>
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<tr>
<td>9</td>
<td>Update from DC-20A BIM Subcommittee</td>
<td>Starting IFC in Exchange Model work with ACI 131</td>
<td>Malits</td>
<td>Ongoing</td>
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<td>10</td>
<td>FAQ: Two-way PT slabs restraint effects exclusion, 125 psi min.</td>
<td>Write initial draft of FAQ to provide technical guidance on this subject</td>
<td>Hirsch</td>
<td>November 1, 2019</td>
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<td>Agenda Item</td>
<td>Expected Outcome / Actions Taken</td>
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<td><strong>A. General</strong></td>
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<td>A.1 Call to Order</td>
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<td>A.2 Introductions</td>
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<td>A.3 Committee Roster / Changes</td>
<td>A.3 Welcome new Associate Members Chad Konger and Marcos Martinez</td>
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<td>A.4 PTI Antitrust Policy</td>
<td>A.4 Policy reminder included with agenda</td>
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<td>A.5 Annual Report</td>
<td>A.5 Committee Annual Report to be completed and submitted within 30 days after this meeting</td>
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<td><strong>B. Agenda &amp; Minutes</strong></td>
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<td>B.1 Approval of Agenda</td>
<td>B.2 Vote on Minutes from 5/7/19 Seattle meeting approval</td>
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<td>B.2 Approval of Minutes from 5/7/19 (Meeting ballot required)</td>
<td>Motion / Second: Name / Name Result: X-X-X (Y-N-A)</td>
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<td><strong>C. Actions Taken Between Meetings</strong></td>
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<td>C.1 Letter Ballots</td>
<td>C.1 Ballot DC-20-1901 (DC20.2-20 Restraint Cracks and their Mitigation) will open 10/4/19 and close 11/4/19</td>
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<td>C.2 Web Meetings</td>
<td>C.2 None</td>
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<tr>
<td><strong>1. Action Item 1: (Design Webinar Modules)</strong></td>
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<td>1.1 Module production</td>
<td>1.1 Modules 1-9 recorded. Alternative speaker desired for Module 10. Posting to PTI website will occur after final PTI staff QA/QC review edits.</td>
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<td><strong>2. Action Item 2: (PTI DC20.2-88, Restraint Cracks and their Mitigation)</strong></td>
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<tr>
<td>2.1 Review progress</td>
<td>2.1 Ballot DC-20-1901 (DC20.2-20 Restraint Cracks and their Mitigation) will open 10/4/19 and close 11/4/19</td>
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| 2.2 Need to publish updated document as DC20.2-20 before 2020 convention | 2.2 Updated chapters have been submitted. Each updated chapter will be setup as an individual ballot item.  
  - Chapter 1: Introduction, Scope and Limitation  
  - Chapter 2: Causes and Categorization of Cracking  
  - Chapter 3: Crack Mitigation  
  - Chapter 6: Computation of Shortening Movement  
  - Chapter 8: Conclusions and Recommendations  
  - Chapter 9: References  
  2.3 This chapter will be balloted separately once it is complete  
  - Chapter 7: Consideration of Shortening in Finite Element |
<table>
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<tr>
<th>Agenda Item</th>
<th>Expected Outcome / Actions Taken</th>
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</table>
| **2.** | These unchanged chapters will not be balloted  
- Chapter 4: Structural Evaluation of Cracks  
- Chapter 5: Crack Repair  
2.5 Web meetings will be expedited in November and December to resolve all negatives and allow final editing at the beginning of 2020 and publishing before May 2020 |
| **3. Action Item 3: (DC20.7-01, Design, Construction and Maintenance of CIP PT Concrete Parking Structures)** | 3.1 After further review, this document will likely require a Joint Committee effort (DC-20 and DC-25) in order to update it. However, other publication priorities must be cleared first.  
3.2 Most likely a Task Group consisting of members from each committee will be formed. Update of this document remains a PTI Board priority. |
| **4. Action Item 4: (ACI 318 Reference – New PTI publication)** | 4.1 Further discussion regarding ACI 318 companion document was tabled until these Committee Days in Santa Fe, NM.  
4.1.1 A reference document that guides the reader to all PT related sections of 318 and serves as a companion document to 318. This would begin with 318-14 and then be updated with each new release of 318, 318-19 being the latest. Need to start on creation of this document after DC20.2-20 goes to ballot. Task Group (maximum of 5 people) needs to be established.  
4.1.2 Locate all code requirements related to PT and create a cheat sheet based on ACI 318-19 provisions in an effort to help engineers navigate the code regarding PT design. Thomas Kang volunteered to create summary of all PT provisions included in ACI 318-19. See attached draft document.  
4.1.3 Carol previously suggested Asit create an FAQ or a paper on technical changes between ACI 318-14 and ACI 318-19 since this effort is essentially complete from Asit’s presentation. Final format and presentation to be discussed with PTI for future publication. |
| **5. Action Item 5: (PT Manual 7th Edition)** | 5.1 Tim to provide update on TAB review ballot progress to date; PTI major priority with 2 chapter per month ballot review cycle  
5.2 PT Manual 7th Edition must publish in 2020. The list below represents a summary of the “DC-20 reviewers” responsibility matrix in item 5.1 with remaining Chapters to be released for review |
<table>
<thead>
<tr>
<th>Agenda Item</th>
<th>Expected Outcome / Actions Taken</th>
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</table>
| **TAB balloting complete:** | **Chapter 5:** Analysis and Design Fundamentals; Martin C, Jonathan Hamid and Siva  
**Chapter 9:** PT Concrete Floors; Hamid, Carine  
**Chapter 10:** PT Parking Structures; Otto, Martin C, Bryan, and Eric  
**Chapter 18:** Fire resistance – Martin C; CRSI to potentially assist with review  
**Chapter 19:** Durability (buildings & bridges); Martin M, Kyle B, Ralph L  
**Chapter 8:** Seismic Design of PT Concrete Structures; Martin M, Bryan Allred, Kyle B  
**Chapter XZ:** BIM; Carine, Srini, Rob |
| **Ballot will close October 9:** | **Chapter XX:** Sustainability; Martin M, Srini |
| **Ballot needs to start in October:** | **Chapter 7:** Design examples; Siva and Eric  
**Chapter 6:** Detailing and Construction Procedures for Buildings; Zuming, Mark, Rashid; (Brian Allred volunteered to review) |
| **Ballot needs to start in November:** | **Chapter XY:** Anchorage Zone Design; Zuming, Siva;  
**Chapter 16:** Design of Prestressed Barrier Cable Systems; reviewers TBD |
| 6. Action Item 6: (Dual-Banded PT Layout) | 6.1 Review progress of testing  
6.2 Review progress of Tech Note |
<p>| 6.1 Martin C., Carol, Jonathan to provide update. | 6.2 Ballot TAB-1907-DC20-TN-Dual Banded PT closed on 9/18/19. 3 Affirmative, 3 Affirmative with Comments, 1 Negative, 1 Abstain and 4 Not Returned. Will resolve in TAB web meeting here in October and publish before year end. |
| 7. Action Item 7: (Temperature Tendons) | 7.1 Review progress of Tech Note |
| 7.1 Final draft of Tech Note is nearly complete and will be submitted for TAB ballot as soon as possible. | |
| 8. Action Item 8: (Structural Detailing for PT Buildings) | 8.1 Review progress of proposed new publication |
| 8.1 Tim is in the process of procuring details from PT suppliers in different geographical areas/regions to educate SEs on how best to represent PT on their design drawings and thus create a best practices type document. Tim has had direct discussions with multiple PT suppliers to obtain their input on problematic details. Ultimate publication goal is now more realistically mid-2020. | |
| 9. Action Item 9: (Update from DC-20A BIM Subcommittee) | 9.1 Subcommittee Chair Malits to provide update. |</p>
<table>
<thead>
<tr>
<th>Agenda Item</th>
<th>Expected Outcome / Actions Taken</th>
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<tbody>
<tr>
<td>9.1 Update on current work of the BIM subcommittee</td>
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<tr>
<td>10. Action Item 10: (FAQ or Tech Note, two-way PT slabs, exclusion of restraint effects in the min 125 psi calculation)</td>
<td>10.1 Jonathan to provide update on initial draft document</td>
</tr>
<tr>
<td>11. Action Item 11: (Restrained vs. Unrestrained FAQ)</td>
<td>11.1 Srin and Martin C. to provide update on initial draft document. See attached.</td>
</tr>
<tr>
<td>D. New Business</td>
<td>D.1 List of documents for review:</td>
</tr>
<tr>
<td>D.1 Review of documents and identification of those in need of revision</td>
<td>PTI DC20.1-87 Strength and Behavior of Closely Spaced Post-Tensioned Mono-strand Anchorages</td>
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<tr>
<td>D.2 FAQ for practical vs. ACI 318 provisions for column punching shear integrity reinforcement in PT slabs</td>
<td>PTI DC20.2-88 Restraint Cracks and their Mitigation in Unbonded Post-Tensioned Building Structures (Revision in Progress)</td>
</tr>
<tr>
<td>D.3 FAQ for ACI 318 8.6.2 provision for minimum P/A and bonded reinforcement</td>
<td>PTI DC20.3-89 Structural Integrity of Building Constructed with Unbonded Tendons</td>
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<tr>
<td>D.4 Intermediate PT Anchors</td>
<td>PTI DC20.4-90 Earthquake Performance of Unbonded Post-Tensioned Buildings</td>
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<td>PTI DC20.5-90 Evaluation of the Condition of a Post-Tensioned Concrete Parking Structure After 15 Years of Service</td>
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<td>PTI DC20.6-99 Design Fundamentals of Post-Tensioned Concrete Floors (Next priority after parking structures)</td>
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<td>PTI DC20.7-01 Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures (Priority starting in 2019)</td>
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<td>PTI DC20.8-04 Design of Post-Tensioned Slabs Using Unbonded Tendons</td>
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<td>PTI DC20.9-11 Guide for Design of Post-Tensioned Buildings</td>
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<td>Create spreadsheet or Gantt chart to show schedule slated for review/updating activity for various documents.</td>
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<tr>
<td>D.2 Discuss need for FAQ document to address this issue</td>
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<tr>
<td>D.3 Discuss need for FAQ document to address this issue</td>
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<tr>
<td>D.4 There is a TAB Task Group that is reviewing and evaluating this matter</td>
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<td>Agenda Item</td>
<td>Expected Outcome / Actions Taken</td>
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<td>E. Next Meeting</td>
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<td>2020 PTI Convention – Miami, FL—May 3-6, 2020</td>
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<tr>
<td>Web Meetings:</td>
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<tr>
<td>F. Adjourn</td>
<td>Meeting adjourned at</td>
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</tbody>
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**AGENDA / MEETING EXHIBITS**

<table>
<thead>
<tr>
<th>Exhibit #</th>
<th>Subject</th>
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<tbody>
<tr>
<td>Roster / A.4</td>
<td>Sign-In Sheet / PTI Anti-Trust Policy</td>
</tr>
<tr>
<td>A.5</td>
<td>Annual Report</td>
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<tr>
<td>B.2</td>
<td>Minutes from 5/7/19 Seattle Meeting</td>
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<tr>
<td>4.1.2</td>
<td>ACI 318-19_Prestressed Concrete</td>
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<tr>
<td>11.1</td>
<td>Draft 02 FAQ</td>
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At a meeting on October 8, 1980, the Board of Directors first discussed the Institute's status and policies regarding compliance with antitrust laws. After review of both the internal and external compliance procedures, the following resolution was approved:

"The staff, officers, directors and members of the Post-Tensioning Institute are reminded that they are required to comply with the spirit and specific requirements of the antitrust laws on all activities within the scope of, and related to, the official functions of PTI. Further, this restated position, along with appropriate explanatory material, should be placed in all meeting folders/books periodically, beginning with the 8th of October meeting of PTI."

On July 24, 2012 and again on October 7, 2015, the Executive Committee authorized Legal Counsel to review and update this Policy Statement in the perspective of the Department of Justice Business Review Letter of July 30, 1997 and current case law. As a continuing guide for your participation in PTI's meetings, please review and continue to adhere to the following "Legal Limitation on Discussions at PTI Meetings."

**LEGAL LIMITATION ON DISCUSSIONS AT PTI MEETINGS AND EVENTS**

A free exchange of ideas on matters of mutual interest to the members is necessary for the success of all meetings. Indeed, such an exchange of views is essential to the successful operation of every trade association and the law specifically allows legitimate exchange of views pertaining to, e.g., quality control, safety, building design and construction integrity, etc.

It is not the purpose of this memorandum to discourage the exploration in depth of any matters of legitimate concern to meeting participants. Nevertheless, to ignore certain antitrust ground rules, either through ignorance or otherwise, is to create a civil and criminal hazard businessmen simply cannot afford.

It is for these reasons that PTI provides you with a reminder that certain areas of formal and informal communication between competitors or between manufacturers and their suppliers and customers must be avoided, as posing potential antitrust problems.

The Sherman Antitrust Act, the Clayton Act, the Federal Trade Commission Act, and the Robinson-Patman Act comprise the basic federal antitrust laws, which set forth the broad areas of conduct considered illegal as restraints of trade. In general, agreements or understandings between competitors that operate as an impediment to free and open competition are forbidden. Federal antitrust prohibitions forbid any "agreement or understanding...to substantially lessen competition or tend to create a monopoly in any line of commerce." An important point to keep in mind is that communications and discussions between competitors or between sellers and customers, about matters which may be considered anti-competitive, often comprise the evidence from which courts infer antitrust violations. *It is the policy of the Post-Tensioning Institute that such agreements, understandings or communications shall not be tolerated at any formal or informal meetings or social events of the Institute.*

The general prohibitions contained in the federal antitrust laws, have been particularized in the form of a series of consent decrees, originally entered against a number of member companies of various trade associations and the associations themselves. It is important to note that these laws not only apply to PTI members, but also to PTI itself. Often trade associations have been and are presently co-defendants in cases brought by the Justice Department and the Federal Trade Commission ("FTC"). Recently, the FTC has stated: "Because trade associations are by their nature collaborations among competitors, the Commission and courts have long been concerned with anti-competitive restraints imposed by such organizations under the guise of codes of conduct. Competing for customers, cutting prices, and recruiting employees are hallmarks of vigorous competition. Agreements among competitors not to engage in these activities injure consumers by increasing prices and reducing quality and choice." Similar "codes" or policies and requirements that encourage directly or indirectly members’ unlawful activity are strictly forbidden by PTI in the course of its business with its members.
SPECIFIC EXAMPLES OF ACTIVITIES AND PRACTICES PROHIBITED
AT ALL PTI MEETINGS AND EVENTS:

Included in activities and practices which are forbidden, and are contrary to the policy of the Institute, both under the general antitrust laws and the consent decrees, subject to the said Business Review Letter, are the following:

- Agreeing to allocate markets, customers or suppliers among competitors, classify certain customers or suppliers being entitled to preferential treatment by manufacturers, and establish geographic trading areas.

- Participating in any plan designed to induce any manufacturer or distributor to sell or refrain from selling, or discriminate in favor of, or against any particular customer or class of customers.

- Agreeing in any manner to fix or otherwise establish bids, prices (including price increases, decreases, standardization or stabilization), profits, costs, contract terms affecting price (such as discounts and credit terms), etc. because, e.g. prices were too low, with the exception of certain resale pricing agreements between manufacturers and retailers or distributors.

- Agreeing in any manner to limit or restrict the quality of products to be produced (e.g., restrictions on selling coated strand to certain customers).

- Participating in any plan which has the effect of discriminating against, or excluding competitors, suppliers or customers.

These examples are provided to guide you in your discussions during formal and informal PTI meetings and social events. If the occasion arises, more specific advice will be provided by legal counsel, who is required by Article IV, Section 7 of the PTI By-Laws to be present at all meetings of the Board of Directors and the Executive Committee.
1. List the progress on goals of your committee during last year:

<table>
<thead>
<tr>
<th>2018-2019 Goal</th>
<th>Progress</th>
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2. List at least three goals for the upcoming year. Note – all goals are subject to TAB/CAB Approval:

<table>
<thead>
<tr>
<th>2019-2020 Goals</th>
<th>Tasks Champion / Expected Completion Date</th>
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<tbody>
<tr>
<td>(New documents, revisions of documents, convention presentations or sessions, PTI Journal case studies, research proposals, PT Treasures or Technical Papers, etc.)</td>
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3. Report detailed progress on already approved document revisions / new documents / technical sessions / PTI Journal Contributions / certification program development, etc.:

<table>
<thead>
<tr>
<th>Title</th>
<th>Progress in Last 12 Months</th>
<th>Task Champion / Expected Completion Date</th>
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4. List new and updated documents you expect to submit to TAB/CAB for review in the next 12 months:

<table>
<thead>
<tr>
<th>Document Title</th>
<th>Document Champion / Expected Completion Date</th>
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5. List old documents needing revision.

<table>
<thead>
<tr>
<th>Document Number / Title</th>
<th>Notes</th>
<th>Task Champion / Expected Completion Date</th>
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6. List subjects for FAQs and/or TNs that would reflect “PTI Position” on issues

<table>
<thead>
<tr>
<th>Subject</th>
<th>Notes</th>
<th>Task Champion / Expected Completion Date</th>
</tr>
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7. List Technical Session ideas

<table>
<thead>
<tr>
<th>Topic / Brief Synopsis</th>
<th>Presenter</th>
</tr>
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<tbody>
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</tbody>
</table>

8. List any liaisons or scope conflicts with other committees in PTI or other organizations:

9. List anything you need from PTI Staff:

Please return to: Miroslav Vejvoda  
E-mail: miroslav.vejvoda@post-tensioning.org
PTI DC-20 Building Design Committee
Tuesday, May 7, 2019
1:30 p.m. to 5:30 p.m.
Hyatt Regency Seattle

Voting Members Present (13 of 14)

Carol Hayek - Chair, TAB Contact
CCL

Martin Maingot - Secretary, V
SCA Consulting Engineers

Tim Christie, NV
Post-Tensioning Institute

Hamid Ahmady
Suncoast Post-Tension Ltd

Rashid Ahmed
Walker Consultants

Bryan Allred
Seneca Structural Engineering
Inc.

Asit Baxi
Baxi Engineering Inc.

Martin A Cuadra
Uzun and Case, LLC

Jonathan Hirsch
Bentley Systems, Inc.

Thomas Kang
Seoul National University

Don Kline
Kline Engineering and Consulting
LLC

Frank Malits
Cagley and Associates Inc.

J.R. Mujagic
Bekaert Corporation

Eric Ober
SGH

Zuming Xia
Structural Technologies

Associate Members Present

Joseph Ales
Walter P Moore and Assoc., Inc

Ryan Bonniwell
Ericksen Roed and Associates

Kyle Boyd
S.A. Miro

Anantha Chittur
BASE

Michael Hopper
Leslie E. Robertson Associates

Spencer Lee
ADAPT

Ralf Leistikow
Wiss, Janney, Elstner Associates, Inc.

Rafael Machado
Ellinwood Machado, LLC

Carine Magalhaes
ADAPT

Siva Munuswamy
Thornton Tomasetti, Inc

Srinivasan Neelamegam
CCL USA

Otto Schwarz
Ryan Biggs Clark Davis
Engineering and Surveying

Roberto Suarez
Martin-Martin

Todd Whisenhunt
Thornton Tomasetti

Mark Yerges
CVM Engineers, Inc.
<table>
<thead>
<tr>
<th>Visitors Present</th>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Lance Osborne</td>
<td>Mohammed TiBi</td>
<td>Gillian Love</td>
</tr>
<tr>
<td>Halfen USA</td>
<td>TiBiCo Prestressed</td>
<td>SGH</td>
</tr>
<tr>
<td>Rattan Khosa</td>
<td>Daniel Schafer</td>
<td>Kyle Poklar</td>
</tr>
<tr>
<td>Amsysco, Inc.</td>
<td>PES Structural Engineers</td>
<td>Graef</td>
</tr>
<tr>
<td>Chad Konger</td>
<td>Bjorn Vors</td>
<td>Nick Bemister</td>
</tr>
<tr>
<td>Strand Systems</td>
<td>Univ. of Saskatchewan</td>
<td>LMS</td>
</tr>
<tr>
<td>Tony Johnson</td>
<td>Marta Dzheneva</td>
<td>Gaelyn Krauser</td>
</tr>
<tr>
<td>PTI</td>
<td>OAC Services</td>
<td>DCI Engineers</td>
</tr>
<tr>
<td>David Peralta</td>
<td>Artjoms Samarins</td>
<td></td>
</tr>
<tr>
<td>Unintech Cons. Engineers</td>
<td>Strandeck</td>
<td></td>
</tr>
</tbody>
</table>
## ACTION ITEMS FROM LAST / THIS MEETING

<table>
<thead>
<tr>
<th>Item #</th>
<th>Subject</th>
<th>Action</th>
<th>Responsible</th>
<th>Deadline / Completed</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Design Webinar Modules</td>
<td>Final QA/QC reviews complete and all 10 modules posted on PTI website</td>
<td>PTI Staff</td>
<td>September 1, 2019</td>
</tr>
<tr>
<td>2</td>
<td>PTI DC20.2-88: Restraint Cracks and their Mitigation</td>
<td>Remaining contributions completed. Compiled into final draft document ready for balloting. Each chapter a separate ballot item.</td>
<td>All</td>
<td>May 31, 2019</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Hayek</td>
<td>June 15, 2019</td>
</tr>
<tr>
<td>3</td>
<td>DC20.7-01: Design, Construction and Maintenance of CIP PT Concrete Parking Structures</td>
<td>Will be removed from DC-20 agenda and placed on agenda of new DC-25 Parking Structures committee</td>
<td>PTI Staff</td>
<td>Remove from DC-20</td>
</tr>
<tr>
<td>4</td>
<td>ACI 318 Reference</td>
<td>Create an FAQ or Technical Paper on changes between 318-14 and 318-19. Create summary of all PT provisions in 318-19</td>
<td>Baxi</td>
<td>October 1, 2019</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Kang</td>
<td>September 1, 2019</td>
</tr>
<tr>
<td>5</td>
<td>PT Manual 7th Edition</td>
<td>Summary of chapters being authored by and reviewed by DC-20 members</td>
<td>All</td>
<td>Ongoing</td>
</tr>
<tr>
<td>6</td>
<td>Dual-Banded PT Layout</td>
<td>Submit final Tech Note to TAB Testing: Two-Way Banded TG to continue efforts and assisting in test configuration.</td>
<td>Hirsch/Boyd</td>
<td>June 15, 2019</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Dual-Banded TG</td>
<td>Ongoing</td>
</tr>
<tr>
<td>7</td>
<td>Temperature Tendons</td>
<td>Submit final Tech Note to TAB</td>
<td>PTI Staff</td>
<td>July 15, 2019</td>
</tr>
<tr>
<td>8</td>
<td>Structural Detailing for PT Buildings</td>
<td>Tim is starting direct discussions with PT suppliers to obtain their input on problematic details. Ultimate publication goal is before the end of 2019</td>
<td>PTI Staff</td>
<td>Ongoing</td>
</tr>
<tr>
<td>9</td>
<td>Update from DC-20A BIM Subcommittee</td>
<td>Starting IFC in Exchange Model work with ACI 131</td>
<td>Malits</td>
<td>Ongoing</td>
</tr>
<tr>
<td>E.4</td>
<td>FAQ: Two-way PT slabs restraint effects exclusion, 125 psi min.</td>
<td>Write initial draft of FAQ to provide technical guidance on this subject</td>
<td>Hirsch</td>
<td>November 1, 2019</td>
</tr>
</tbody>
</table>
## E.5 FAQ: Restained vs. Unrestrained

Write FAQ based on ACI 216 provisions

<table>
<thead>
<tr>
<th>Agenda Item</th>
<th>Expected Outcome / Actions Taken</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. General</strong></td>
<td></td>
</tr>
<tr>
<td>A.1 Call to Order</td>
<td>A.1 Meeting was called to order at 1:35 pm</td>
</tr>
<tr>
<td>A.2 Introductions</td>
<td>A.2</td>
</tr>
<tr>
<td>A.3 Committee Roster / Changes</td>
<td>A.3.1 Anantha Chittur of BASE, Inc. has joined as a new Associate Member.</td>
</tr>
<tr>
<td>A.4 PTI Antitrust Policy</td>
<td>A3.2 For all members, please remember that meeting attendance, return of ballots and task group/committee assignments participation is monitored and evaluated by the Chair and PTI Staff on a regular basis.</td>
</tr>
<tr>
<td><strong>B. Agenda &amp; Minutes</strong></td>
<td></td>
</tr>
<tr>
<td>B.1 Approval of Agenda</td>
<td>B.1</td>
</tr>
<tr>
<td>B.2 Approval of Minutes from 9/27/18 (Meeting ballot required)</td>
<td>B.2 Vote on Minutes from 9/27/18 Colorado Springs meeting approval Motion / Second: Maingot / Cuadra Result: 13-0-0 (Y-N-A)</td>
</tr>
<tr>
<td><strong>C. Actions Taken Between Meetings</strong></td>
<td></td>
</tr>
<tr>
<td>C.1 Letter Ballots (none)</td>
<td>C.1.1 Ballot DC-20-1801 (Technical Note – Dual Banded PT Tendon Layout) ended 10/12/18.</td>
</tr>
<tr>
<td>C.2 Web Meetings (none)</td>
<td>C.1.2 Ballot DC-20-1802 (Technical Note – Shrinkage &amp; Temperature PT) ended 1/12/19.</td>
</tr>
<tr>
<td></td>
<td>C.2.1 Web meetings were held on 12/11/18 and 1/31/19 to resolve negatives and substantive comments from Ballot DC-20-1801.</td>
</tr>
<tr>
<td></td>
<td>C.2.2 Web meetings were held on 2/12/19 and 3/1/19 to resolve negatives and substantive comments from Ballot DC-20-1802.</td>
</tr>
<tr>
<td><strong>1. Action Item 1: (Design Webinar Modules)</strong></td>
<td></td>
</tr>
</tbody>
</table>
| 1.1. Module production | 1.1 Modules 1, 2, 3 and 6 have been completed and submitted to other PTI staff for review. Modules 4 and 7 will be submitted in May for other PTI staff review. Modules 5, 8 and 9 are still in production, and an alternative speaker might be sought for Module 10. After other PTI
<table>
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<th>Agenda Item</th>
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<td>staff review, completed modules will go to select committee members and original module authors for final QA/QC review before proceeding with post to PTI website.</td>
</tr>
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</table>

2. **Action Item 2**: (PTI DC20.2-88, Restraint Cracks and their Mitigation)

2.1. Review progress

2.2. Need to publish updated document as DC20.2-19 before the end of 2019

<table>
<thead>
<tr>
<th>Expected Outcome / Actions Taken</th>
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<tbody>
<tr>
<td>2.1 Updates from each task group: Document to be submitted for review in its entirety, yet each chapter will be balloted individually in an effort to more effectively resolve negatives. Per Carol, the RTS document will be ready for balloting by mid-June 2019 if the contributions are submitted in time. All contributions should be in by end of May 2019. Once received, Carol will go through the whole document to fit all the Chapters and any missing info to complete the full draft. The intent is to produce a more comprehensive RTS document than currently offered by ACI 224-14, state of the art.</td>
</tr>
<tr>
<td>2.1.1 Causes of Cracking (Martin C., Eric) – Complete except for obtaining figures needed from ACI 224.4R-13 document in response to request that was submitted to ACI.</td>
</tr>
<tr>
<td>2.1.2 Connection Details (Kyle, Mark Y., Asit) – Kyle and Asit have finalized the details and will try to fit the details and text within the current RTS sections. Brian Allred volunteered to provide pictures/details of slip connections used in high seismic regions. Brian also volunteered to provide pictures of conditions where no slip details were used (or not installed correctly) resulting in heavy shrinkage cracking.</td>
</tr>
<tr>
<td>2.1.3 Concrete Drying Shrinkage Mitigation Strategies (Martin M.) –</td>
</tr>
<tr>
<td>2.1.4 Equations, ¼” value and Examples (Asit, Rashid, Eric, Carol) – Asit has added a theoretical background to the section on how to estimate/calculate shrinkage with design examples using ACI 209, PCI Manual and the Simplified Method of analysis. Since all methods borrow from each other, final results are quite similar. Per Asit, the volume-to-surface calculation indicated in greater detail since it isn’t always calculated correctly. On a separate note, the calculations do not consider structural restraint from supporting vertical elements.</td>
</tr>
<tr>
<td>2.1.5 FEM Analysis (Mike H., Mark Y.) – There is no correlation between FEM model and observed cracking (per the provided crack map) attributable to a large number of unknowns related to design, construction and maintenance. General consensus is to scrap the comparison of the FEM model with the original crack map and replace with stress “heat” maps using the same/similar colors and scale to best illustrate the differences between high restraint (no pour strips or slip details), moderate restraint (pour strip only) and low restraint (pour strips and slip details). The new comparison that Mike and Mark have submitted is interesting and should be included in the RTS document.</td>
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<tr>
<td>Agenda Item</td>
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<td>2.1.6 Lance Osborne to provide description of lockable dowels and include language on ICC/IAMPO (or similar) approvals and horizontal/vertical shear transfer mechanisms.</td>
</tr>
<tr>
<td>3. Action Item 3: (DC20.7-01, Design, Construction and Maintenance of CIP PT Concrete Parking Structures) 3.1 Need to start on update of this document after Action Item 2 goes to ballot</td>
</tr>
<tr>
<td>4. Action Item 4: (ACI 318 Reference – New PTI publication) 4.1 Need to start on creation of this document after Action Item 2 goes to ballot</td>
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<td>Agenda Item</td>
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<td><strong>5. Action Item 5: (PT Manual 7th Edition)</strong></td>
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<tr>
<td>5.1 Review progress</td>
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<tr>
<td>5.2 Plan for review process</td>
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<td><strong>6. Action Item 6: (Dual-Banded PT Layout)</strong></td>
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<tr>
<td>6.1 Review progress of testing</td>
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<tr>
<td>6.2 Review progress of Tech Note</td>
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<tr>
<td>Agenda Item</td>
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7. Action Item 7: (Temperature Tendons)  
7.1 Review progress of Tech Note  
7.1 Tech Note ballot resolved in March 2019. Tim to provide update. Tim absent from meeting, but final draft of Tech Note is in progress and will be submitted to TAB for final review.  

8. Action Item 8: (Structural Detailing for PT Buildings)  
8.1 Review progress of proposed new publication  
8.1 Tim is starting direct discussions with PT suppliers to obtain their input on problematic details. Ultimate publication goal is before the end of 2019.  
8.1.1 Tim is in the process of procuring details from PT suppliers in different geographical areas/regions in an effort to educate SEs on how best to represent PT on their design drawings and thus create a best practices type document. At this time, planning stages only.  

9. Action Item 9: (Update from DC-20A BIM Subcommittee)  
9.1 Update on current work of the BIM subcommittee  
9.1 Subcommittee Chair Malits to provide update. Frank M. discussed the current status of his committee’s efforts in attempting to incorporate PT elements into BIM. Very challenging process that will take over 2 years to complete and is currently being coordinated with ACI 131.  

E. New Business  
E.1 List of documents for review:  
PTI DC20.1-87 Strength and Behavior of Closely Spaced Post-Tensioned Mono-strand Anchorages  
PTI DC20.2-88 Restraint Cracks and their Mitigation in Unbonded Post-Tensioned Building Structures (Revision in Progress)  
PTI DC20.3-89 Structural Integrity of Building Constructed with Unbonded Tendons  
PTI DC20.4-90 Earthquake Performance of Unbonded Post-Tensioned Buildings  
PTI DC20.5-90 Evaluation of the Condition of a Post-Tensioned Concrete Parking Structure After 15 Years of Service  
PTI DC20.6-99 Design Fundamentals of Post-Tensioned Concrete Floors (Next priority after parking structures)  
PTI DC20.7-01 Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking
<table>
<thead>
<tr>
<th>Agenda Item</th>
<th>Expected Outcome / Actions Taken</th>
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</thead>
<tbody>
<tr>
<td>PTI DC20.8-04</td>
<td>Design of Post-Tensioned Slabs Using Unbonded Tendons</td>
</tr>
<tr>
<td>PTI DC20.9-11</td>
<td>Guide for Design of Post-Tensioned Buildings</td>
</tr>
<tr>
<td>Create spreadsheet or Gantt chart to show schedule slated for review/updating activity for various documents.</td>
<td></td>
</tr>
<tr>
<td>E.2 Discuss need for FAQ document to address this issue</td>
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</tr>
<tr>
<td>E.3 Discuss need for FAQ document to address this issue</td>
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</tr>
<tr>
<td>E.4 Carol and Jonathan discussed minimum pre-compression requirements for two-way post-tensioned slabs and the need for an ACI 318 code interpretation FAQ or Tech Note addressing exclusion of restraint effects in the 125 psi min calculation. Jonathan was volunteered to write the initial draft yet won’t be able to start until mid to late summer 2019. No specific timeline given for completing the effort.</td>
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</tr>
<tr>
<td>E.5 Srinil and Martin C volunteered to write FAQ draft based on ACI 216 provisions yet won’t be able to start until late summer/early fall 2019.</td>
<td></td>
</tr>
<tr>
<td>E.6 TAB recently formed a task group to create a Tech Note to address the technical issues and challenges with a very recent market shift away from intermediate stressing anchors to couplers. There are several technical concerns like the effect of voids (that are created in the section at the couplers) on shear capacity and risk of bad installation and so on. Per Brian Allred, the approach has been used on the West Coast for many years in slabs, but not sure if it was used in beams. However, it is very new to other parts of the country and will likely be received with increases in material and installation costs as well as technical scrutiny.</td>
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</table>

**F. Next Meeting**
2019 PTI Committee Days,
Santa Fe, New Mexico
October 2-4, 2019
Web Meetings:

**G. Adjourn**
Meeting adjourned at 5:20 pm.
## AGENDA / MEETING EXHIBITS

<table>
<thead>
<tr>
<th>Exhibit #</th>
<th>Subject</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roster / A.4</td>
<td>Sign-In Sheet / PTI Anti-Trust Policy</td>
</tr>
</tbody>
</table>
Committee Attendance Sheet

Committee: DC-20  Date: 5/7/19

Meeting Location: HYATT REGENCY SEATTLE (CLEARWATER CONF.)

* I have read, understand, and agree to comply with PTI Anti-Trust Policy (attached).

Note to Committee members and visitors: All Committee meetings of the Post-Tensioning Institute should be conducted in a manner encouraging free and open discussion and debate of agenda items and matter properly before that Committee. Committee members and visitors are cautioned that such discussion and debate is solely for the purposes of the Charter of the Committee and PTI business. To that end, Committee discussions and debates are not considered public in nature, and, as such are to be held in confidence and do not become the official policy of PTI until properly reported, balloted, and published pursuant to procedures established by or by the adoption by the PTI Board of Directors. Committee members and visitors shall not quote, publish, use, or make use of, any oral or written drafts, drawings, calculations, or other materials, which are uttered or transcribed during the course of such meetings.

<table>
<thead>
<tr>
<th>#</th>
<th>Name</th>
<th>Company</th>
<th>Voting/ Associate/ Guest</th>
<th>E-Mail</th>
<th>Policy*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Martin Mainot</td>
<td>SCA</td>
<td>✔</td>
<td><a href="mailto:MAINAOTME@SCAENGINERERS.COM">MAINAOTME@SCAENGINERERS.COM</a></td>
<td>✔</td>
</tr>
<tr>
<td>2</td>
<td>Lance Osborne</td>
<td>HALFEN USA</td>
<td>Guest</td>
<td><a href="mailto:Osborne@halfenusa.com">Osborne@halfenusa.com</a></td>
<td>✔</td>
</tr>
<tr>
<td>3</td>
<td>Mohamed Tibi</td>
<td>Tibico Proposel</td>
<td>Guest</td>
<td><a href="mailto:Tibico@tibico.com">Tibico@tibico.com</a></td>
<td>✔</td>
</tr>
<tr>
<td>4</td>
<td>Abit Baxi</td>
<td>SCA</td>
<td>✔</td>
<td><a href="mailto:ABIT@SCA.COM">ABIT@SCA.COM</a></td>
<td>✔</td>
</tr>
<tr>
<td>5</td>
<td>Bryan Allred</td>
<td>SeneCA</td>
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</tr>
<tr>
<td>6</td>
<td>Frank Malits</td>
<td>Cagley &amp; Assoc.</td>
<td>✔</td>
<td><a href="mailto:frank@Cagleymalits.com">frank@Cagleymalits.com</a></td>
<td>✔</td>
</tr>
<tr>
<td>7</td>
<td>Martin Carper</td>
<td>Lien &amp; Case, LLC</td>
<td>✔</td>
<td>MCarper@lien&amp;case.com</td>
<td>✔</td>
</tr>
<tr>
<td>8</td>
<td>Eric Offer</td>
<td>SGH</td>
<td>✔</td>
<td><a href="mailto:eric@SGH.com">eric@SGH.com</a></td>
<td>✔</td>
</tr>
<tr>
<td>9</td>
<td>Hamid Almada</td>
<td>Suncoast PT</td>
<td>✔</td>
<td>HAI@Suncoast-PT</td>
<td>✔</td>
</tr>
<tr>
<td>10</td>
<td>Zuoming Xia</td>
<td>Structural Technique</td>
<td>✔</td>
<td><a href="mailto:zxia@structuraltec.com">zxia@structuraltec.com</a></td>
<td>✔</td>
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<tr>
<td>11</td>
<td>Swagun Munsamy</td>
<td>Thornton Tornesehi</td>
<td>✔</td>
<td>Tornesehi@ Thornton</td>
<td>✔</td>
</tr>
<tr>
<td>12</td>
<td>Gillian Love</td>
<td>Simpson Guenther</td>
<td>guest</td>
<td><a href="mailto:Love@SimpsonGuenther.com">Love@SimpsonGuenther.com</a></td>
<td>✔</td>
</tr>
<tr>
<td>13</td>
<td>Ryan Bonikiew</td>
<td>Bricekeep Road Assoc</td>
<td>✔</td>
<td><a href="mailto:Bonikiew@Bricekeep.com">Bonikiew@Bricekeep.com</a></td>
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**Committee Attendance Sheet**

**Committee:** DC-20  
**Date:** 5/7/19  
**Meeting Location:** Seattle, WA

* I have read, understand, and agree to comply with PTI Anti-Trust Policy (attached).

**Note to Committee members and visitors:** All Committee meetings of the Post-Tensioning Institute should be conducted in a manner encouraging free and open discussion and debate of agenda items and matter properly before that Committee. Committee members and visitors are cautioned that such discussion and debate is solely for the purposes of the Charter of the Committee and PTI business. To that end, Committee discussions and debates are not considered public in nature, and, as such are to be held in confidence and do not become the official policy of PTI until properly reported, balloted, and published pursuant to procedures established by or by the adoption by the PTI Board of Directors. Committee members and visitors shall not quote, publish, use, or make use of, any oral or written drafts, drawings, calculations, or other materials, which are uttered or transcribed during the course of such meetings.

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PTI POLICY STATEMENT ON COMPLIANCE WITH ANTITRUST LAWS

At a meeting on October 8, 1980, the Board of Directors first discussed the Institute's status and policies regarding compliance with antitrust laws. After review of both the internal and external compliance procedures, the following resolution was approved:

"The staff, officers, directors and members of the Post-Tensioning Institute are reminded that they are required to comply with the spirit and specific requirements of the antitrust laws on all activities within the scope of, and related to, the official functions of PTI. Further, this restated position, along with appropriate explanatory material, should be placed in all meeting folders/books periodically, beginning with the 8th of October meeting of PTI."

On July 24, 2012 and again on October 7, 2015, the Executive Committee authorized Legal Counsel to review and update this Policy Statement in the perspective of the Department of Justice Business Review Letter of July 30, 1997 and current case law. As a continuing guide for your participation in PTI's meetings, please review and continue to adhere to the following "Legal Limitation on Discussions at PTI Meetings."

**LEGAL LIMITATION ON DISCUSSIONS AT PTI MEETINGS AND EVENTS**

A free exchange of ideas on matters of mutual interest to the members is necessary for the success of all meetings. Indeed, such an exchange of views is essential to the successful operation of every trade association and the law specifically allows legitimate exchange of views pertaining to, e.g., quality control, safety, building design and construction integrity, etc.

It is not the purpose of this memorandum to discourage the exploration in depth of any matters of legitimate concern to meeting participants. Nevertheless, to ignore certain antitrust ground rules, either through ignorance or otherwise, is to create a civil and criminal hazard businessmen simply cannot afford.

It is for these reasons that PTI provides you with a reminder that certain areas of formal and informal communication between competitors or between manufacturers and their suppliers and customers must be avoided, as posing potential antitrust problems.

The Sherman Antitrust Act, the Clayton Act, the Federal Trade Commission Act, and the Robinson-Patman Act comprise the basic federal antitrust laws, which set forth the broad areas of conduct considered illegal as restraints of trade. In general, agreements or understandings between competitors that operate as an impediment to free and open competition are forbidden. Federal antitrust prohibitions forbid any "agreement or understanding...to substantially lessen competition or tend to create a monopoly in any line of commerce." An important point to keep in mind is that communications and discussions between competitors or between sellers and customers, about matters which may be considered anti-competitive, often comprise the evidence from which courts infer antitrust violations. **It is the policy of the Post-Tensioning Institute that such agreements, understandings or communications shall not be tolerated at any formal or informal meetings or social events of the Institute.**

The general prohibitions contained in the federal antitrust laws, have been particularized in the form of a series of consent decrees, originally entered against a number of member companies of various trade associations and the associations themselves. It is important to note that these laws not only apply to PTI members, but also to PTI itself. Often trade associations have been and are presently co-defendants in cases brought by the Justice Department and the Federal Trade Commission ("FTC"). Recently, the FTC has stated: “Because trade associations are by their nature collaborations among competitors, the Commission and courts have long been concerned with anti-competitive restraints imposed by such organizations under the guise of codes of conduct. Competing for customers, cutting prices, and recruiting employees are hallmarks of vigorous competition. Agreements among competitors not to engage in these activities injure consumers by increasing prices and reducing quality and choice.” Similar “codes” or policies and requirements that encourage directly or indirectly members’ unlawful activity are strictly forbidden by PTI in the course of its business with its members.
SPECIFIC EXAMPLES OF ACTIVITIES AND PRACTICES PROHIBITED
AT ALL PTI MEETINGS AND EVENTS:

Included in activities and practices which are forbidden, and are contrary to the policy of the Institute, both under the general antitrust laws and the consent decrees, subject to the said Business Review Letter, are the following:

- Agreeing to allocate markets, customers or suppliers among competitors, classify certain customers or suppliers being entitled to preferential treatment by manufacturers, and establish geographic trading areas.

- Participating in any plan designed to induce any manufacturer or distributor to sell or refrain from selling, or discriminate in favor of, or against any particular customer or class of customers.

- Agreeing in any manner to fix or otherwise establish bids, prices (including price increases, decreases, standardization or stabilization), profits, costs, contract terms affecting price (such as discounts and credit terms), etc. because, e.g. prices were too low, with the exception of certain resale pricing agreements between manufacturers and retailers or distributors.

- Agreeing in any manner to limit or restrict the quality of products to be produced (e.g., restrictions on selling coated strand to certain customers).

- Participating in any plan which has the effect of discriminating against, or excluding competitors, suppliers or customers.

These examples are provided to guide you in your discussions during formal and informal PTI meetings and social events. If the occasion arises, more specific advice will be provided by legal counsel, who is required by Article IV, Section 7 of the PTI By-Laws to be present at all meetings of the Board of Directors and the Executive Committee.
ACI 318-19 Provisions on
Design and Construction of
Prestressed Concrete Structures
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PART I. NOTATION

1.1. Notation

\( A_{cf} = \) greater gross cross-sectional area of the two orthogonal slab-beam strips intersecting at a column of a two-way prestressed slab, in.\(^2\)

\( A_{cp} = \) area enclosed by outside perimeter of concrete cross section, in.\(^2\)

\( A_{c} = \) area of that part of cross section between the flexural tension face and centroid of gross section, in.\(^2\)

\( A_{cv} = \) gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered in the case of walls, and gross area of concrete section in the case of diaphragms. Gross area is total area of the defined section minus area of any openings, in.\(^2\)

\( A_{g} = \) gross area of concrete section, in.\(^2\)

For a hollow section, \( A_{g} \) is the area of the concrete only and does not include the area of the void(s)

\( A_{oh} = \) area enclosed by centerline of the outermost closed transverse torsional reinforcement, in.\(^2\)

\( A_{ps} = \) area of prestressed longitudinal tension reinforcement, in.\(^2\)

\( A_{s} = \) area of nonprestressed longitudinal tension reinforcement, in.\(^2\)

\( A_{s,\text{min}} = \) minimum area of flexural reinforcement, in.\(^2\)

\( A_{t} = \) area of one leg of a closed stirrup, hoop, or tie resisting torsion within spacing \( s \), in.\(^2\)

\( A_{v} = \) area of shear reinforcement within spacing \( s \), in.\(^2\)

\( A_{v,\text{min}} = \) minimum area of shear reinforcement within spacing \( s \), in.\(^2\)

\( b = \) width of compression face of member, in.

\( b_{o} = \) perimeter of critical section for two-way shear in slabs and footings, in.

\( b_{\text{slab}} = \) effective slab width, in.

\( b_{1} = \) width of that part of cross section containing the closed stirrups resisting torsion, in.

\( b_{w} = \) web width or diameter of circular section, in.

\( b_{2} = \) dimension of the critical section \( b_{0} \) measured in the direction perpendicular to \( b_{1} \), in.

\( c = \) distance from extreme compression fiber to neutral axis, in.

\( c_{c} = \) clear cover of reinforcement, in.

\( c_{1} = \) dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in.

\( c_{2} = \) dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to \( c_{1} \), in.

\( d = \) distance from extreme compression fiber to

\( C = \) compressive force acting on a nodal zone, lb
centroid of longitudinal tension reinforcement, in.

$d'' = \text{distance from extreme compression fiber to centroid of longitudinal compression reinforcement, in.}$

$d_o = \text{nominal diameter of bar, wire, or prestressing strand, in.}$

$d_p = \text{distance from extreme compression fiber to centroid of prestressed reinforcement, in.}$

$f' c = \text{specified compressive strength of concrete, psi}$

$\sqrt{f_c} = \text{square root of specified compressive strength of concrete, psi}$

$f_{ci}' = \text{specified compressive strength of concrete at time of initial prestress, psi}$

$\sqrt{f_{ci}} = \text{square root of specified compressive strength of concrete at time of initial prestress, psi}$

$f_d = \text{stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, psi}$

$f_{dc} = \text{decompression stress; stress in the prestressed reinforcement if stress is zero in the concrete at the same level as the centroid of the prestressed reinforcement, psi}$

$f_{pc} = \text{compressive stress in concrete, after allowance for all prestress losses, at centroid of cross section resisting externally applied loads or at junction of web and flange where the centroid lies within the flange, psi. In a composite member, } f_{pc} \text{ is the resultant compressive stress at centroid of composite section, or at junction of web and flange where the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone}$

$f_{pe} = \text{compressive stress in concrete due only to effective prestress forces, after allowance for all prestress losses, at extreme fiber of section if tensile stress is caused by externally applied loads, psi}$

$f_p = \text{stress in prestressed reinforcement at nominal flexural strength, psi}$

$f_{pu} = \text{specified tensile strength of prestressing reinforcement, psi}$

$f_{py} = \text{specified yield strength of prestressing reinforcement, psi}$

$f_r = \text{modulus of rupture of concrete, psi}$

$f_s = \text{tensile stress in reinforcement at service loads, excluding prestressed reinforcement, psi}$

$f_e = \text{effective stress in prestressed reinforcement, after allowance for all prestress losses, psi}$

$f_t = \text{extreme fiber stress in the precompressed tension zone calculated at service loads using gross section properties after allowance of all prestress}$

$d_{burst} = \text{distance from the anchorage device to the centroid of the bursting force, } T_{burst}, \text{ in.}$

$\epsilon_{anc} = \text{eccentricity of the anchorage device or group of devices with respect to the centroid of the cross section, in.}$
losses, psi

\( f_Y \) = specified yield strength for nonprestressed reinforcement, psi

\( f_yt \) = specified yield strength of transverse reinforcement, psi

\( h \) = overall thickness, height, or depth of member, in.

\( I \) = moment of inertia of section about centroidal axis, in.\(^4\)

\( I_{cr} \) = moment of inertia of cracked section transformed to concrete, in.\(^4\)

\( I_e \) = effective moment of inertia for calculation of deflection, in.\(^4\)

\( I_g \) = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.\(^4\)

\( \ell \) = span length of beam or one-way slab; clear projection of cantilever, in.

\( \ell_n \) = length of clear span measured face-to-face of supports, in.

\( \ell_{tr} \) = transfer length of prestressed reinforcement, in.

\( L \) = effect of service live load

\( L_r \) = effect of service roof live load

\( M_a \) = maximum moment in member due to service loads at stage deflection is calculated, in.-lb

\( M_{cr} \) = cracking moment, in.-lb

\( M_{cre} \) = moment causing flexural cracking at section due to externally applied loads, in.-lb

\( M_{max} \) = maximum factored moment at section due to externally applied loads, in.-lb

\( M_n \) = nominal flexural strength at section, in.-lb

\( M_n \) = factored slab moment that is resisted by the column at a joint, in.-lb

\( M_n \) = factored moment at section, in.-lb

\( n \) = number of items, such as, bars, wires, monostrand anchorage devices, or anchors

\( N_t \) = resultant tensile force acting on the portion of the concrete cross section that is subjected to tensile stresses due to the combined effects of service loads and effective prestress, lb

\( N_a \) = factored axial force normal to cross section occurring simultaneously with \( V_a \) or \( T_a \); to be taken as positive for compression and negative for tension, lb

\( p_h \) = perimeter of centerline of outermost closed transverse torsional reinforcement, in.

\( P_a \) = nominal axial compressive strength of member, lb

\( P_{pu} \) = factored prestressing force at anchorage device, lb

\( R \) = cumulative load effect of service rain load

\( s \) = center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, tendons, or anchors, in.

\( S \) = effect of service snow load

\( S_n \) = nominal moment, shear, axial, torsion, or

\( h_{anc} \) = dimension of anchorage device or single group of closely spaced devices in the direction of bursting being considered, in.

\( M \) = moment acting on anchor or anchor group, in.-lb

\( T \) = tension force acting on a nodal zone in a strut
bearing strength

$T = \text{cumulative effects of service temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete}$

and-tie model, lb ($T$ is also used to define the cumulative effects of service temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete in the load combinations defined in 5.3.6.)

$T_{burst} = \text{tensile force in general zone acting ahead of the anchorage device caused by spreading of the anchorage force, lb}$
\[ \beta_i = \text{factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis} \]

\[ \gamma_f = \text{factor used to determine the fraction of } M_{sc} \text{ transferred by slab flexure at slab-column connections} \]

\[ \gamma_p = \text{factor used for type of prestressing reinforcement} \]

\[ \gamma_v = \text{factor used to determine the fraction of } M_{sc} \text{ transferred by eccentricity of shear at slab-column connections} \]

\[ \Delta f_{ps} = \text{stress in prestressed reinforcement at service loads less decompression stress, psi} \]

\[ \varepsilon_t = \text{net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature} \]

\[ \varepsilon_{ty} = \text{value of net tensile strain in the extreme layer of longitudinal tension reinforcement used to define a compression-controlled section} \]

\[ \lambda = \text{modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normalweight concrete of the same compressive strength} \]

\[ \lambda_s = \text{factor used to modify shear strength based on the effects of member depth, commonly referred to as the size effect factor.} \]

\[ \xi = \text{time-dependent factor for sustained load} \]

\[ \rho = \text{ratio of } A_s \text{ to } bd \]

\[ \rho' = \text{ratio of } A_s' \text{ to } bd \]

\[ \rho_{tr} = \text{ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement} \]

\[ \rho_p = \text{ratio of } A_{ps} \text{ to } bd_p \]

\[ \rho_t = \text{ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement} \]

\[ \rho_{tw} = \text{ratio of } A_t \text{ to } b_w d \]

\[ \phi = \text{strength reduction factor} \]

\[ \varepsilon_{cu} = \text{maximum usable strain at extreme concrete compression fiber} \]

\[ \lambda = \text{in most cases, the reduction in mechanical properties is caused by the reduced ratio of tensile-to-compressive strength of lightweight concrete compared to normalweight concrete. There are instances in the Code where } \lambda \text{ is used as a modifier to reduce expected performance of lightweight concrete where the reduction is not related directly to tensile strength.} \]
PART II. GENERAL

2.1. Scope (Corresponds to ACI 318-11 Section 18.1)

1.1—Scope of ACI 318
1.1.1 This chapter addresses (a) through (h):
(a) General requirements of this Code
(b) Purpose of this Code
(c) Applicability of this Code
(d) Interpretation of this Code
(e) Definition and role of the building official and the licensed design professional
(f) Construction documents
(g) Testing and inspection
(h) Approval of special systems of design, construction, or alternative construction materials

R1.1—Scope of ACI 318
R1.1.1 This Code includes provisions for the design of concrete used for structural purposes, including plain concrete; concrete containing nonprestressed reinforcement, prestressed reinforcement, or both; and anchoring to concrete. This chapter includes a number of provisions that explain where this Code applies and how it is to be interpreted.

2.2. General (Corresponds to ACI 318-11 Section 18.2)

4.12—Requirements for specific types of construction
4.12.2 Prestressed concrete systems
4.12.2.1 Design of prestressed members and systems shall be based on strength and on behavior at service conditions at all critical stages during the life of the structure from the time prestress is first applied.

4.12.2.2 Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length, and rotations due to prestressing. Effects of temperature change, restraint of attached structural members, foundation settlement, creep, and shrinkage shall also be considered.

4.12.2.3 Stress concentrations due to prestressing shall be considered in design.

4.12.2.4 Effect of loss of area due to open ducts shall be considered in computing section properties before grout in post-tensioning ducts has attained design strength.

4.12.2.5 Post-tensioning tendons shall be permitted to be external to any concrete section of a member. Strength and serviceability design requirements of this Code shall be used to evaluate the effects of external tendon forces on the concrete structure.

R4.12—Requirements for specific types of construction
R4.12.2 Prestressed concrete systems
Prestressing, as used in the Code, may apply to pretensioning, bonded post-tensioning, or unbonded posttensioning. All requirements in the Code apply to prestressed systems and members, unless specifically excluded. This section contains specific requirements for prestressed concrete systems. Other sections of this Code also provide specific requirements, such as required concrete cover for prestressed systems.

Creep and shrinkage effects may be greater in prestressed than in nonprestressed concrete structures because of the prestressing forces and because prestressed structures typically have less bonded reinforcement. Effects of movements due to creep and shrinkage may require more attention than is normally required for nonprestressed concrete. These movements may increase prestress losses.

Design of externally post-tensioned construction should consider aspects of corrosion protection and fire resistance that are applicable to this structural system.

9.2—General
9.2.3 Stability
9.2.3.2 In prestressed beams, buckling of thin webs and flanges shall be considered. If there is

R9.2—General
R9.2.3 Stability
R9.2.3.2 In post-tensioned members where the prestressed reinforcement has intermittent contact
intermittent contact between prestressed reinforcement and an oversize duct, member buckling between contact points shall be considered.

with an oversize duct, the member can buckle due to the axial prestressing force, as the member can deflect laterally while the prestressed reinforcement does not. If the prestressed reinforcement is in continuous contact with the member being prestressed or is part of an unbonded tendon with the sheathing not excessively larger than the prestressed reinforcement, the prestressing force cannot buckle the member.

2.3. Prestressing steel materials

20.3—Prestressing strands, wires, and bars
20.3.1 Material properties
20.3.1.1 Except as required in 20.3.1.3 for special moment frames and special structural walls, prestressing reinforcement shall conform to (a), (b), (c), or (d):

(a) ASTM A416 – strand
(b) ASTM A421 – wire
(c) ASTM A421 – low-relaxation wire including Supplementary Requirement S1, “Low-Relaxation Wire and Relaxation Testing”
(d) ASTM A722 – high-strength bar

20.3.1.2 Prestressing strands, wires, and bars not listed in ASTM A416, A421, or A722 are permitted provided they conform to minimum requirements of these specifications and are shown by test or analysis not to impair the performance of the member.

20.3.1.3 Prestressing reinforcement resisting earthquake-induced moment, axial force, or both, in special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers, cast using precast concrete shall comply with ASTM A416 or ASTM A722.

R20.3—Prestressing strands, wires, and bars
R20.3.1 Material properties
R20.3.1.1 Because low-relaxation prestressing reinforcement is addressed in a supplementary requirement to ASTM A421, which applies only if low-relaxation material is specified, the appropriate ASTM reference is listed as a separate entity.
### PART III. GENERAL DESIGN

#### 3.1. Design assumptions (Corresponds to ACI 318-11 Section 18.3)

<table>
<thead>
<tr>
<th>Code</th>
<th>Commentary</th>
</tr>
</thead>
</table>
| **22.2**—Design assumptions for moment and axial strength  
22.2.1 *Equilibrium and strain compatibility*  
22.2.1.1 Equilibrium shall be satisfied at each section. |  
**R22.2**—Design assumptions for moment and axial strength  
**R22.2.1** *Equilibrium and strain compatibility*  
The flexural and axial strength of a member calculated by the strength design method of the Code requires that two basic conditions be satisfied: 1) equilibrium; and 2) compatibility of strains. Equilibrium refers to the balancing of forces acting on the cross section at nominal strength. The relationship between the stress and strain for the concrete and the reinforcement at nominal strength is established within the design assumptions allowed by 22.2. |
| **24.5**—Permissible stresses in prestressed concrete flexural members  
24.5.1 *General*  
24.5.1.1 Concrete stresses in prestressed flexural members shall be limited in accordance with 24.5.2 through 24.5.4 unless it is shown by test or analysis that performance will not be impaired. |  
**R24.5**—Permissible stresses in prestressed concrete flexural members  
**R24.5.1** *General*  
**R24.5.1.1** Permissible stresses in concrete address serviceability but do not ensure adequate design strength, which should be checked in accordance with other Code requirements. A mechanism is provided such that Code limits on stress need not inhibit the development of new products, materials, and techniques in prestressed concrete construction. Approvals for the design should be in accordance with 1.10 of the Code. |

24.5.1.2 For calculation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with assumptions (a) and (b):  
(a) Strains vary linearly with distance from neutral axis in accordance with 22.2.1.  
(b) At cracked sections, concrete resists no tension.  
22.2.1.2 Strain in concrete and nonprestressed reinforcement shall be assumed proportional to the distance from neutral axis.  
**R22.2.1.2** It is reasonable to assume a linear distribution of strain across a reinforced concrete cross section (plane sections remain plane), even near nominal strength except in cases as described in Chapter 23. The strain in both nonprestressed reinforcement and in concrete is assumed to be directly proportional to the distance from the neutral axis. This assumption is of primary importance in design for determining the strain and corresponding stress in the reinforcement.  
22.2.1.3 Strain in prestressed concrete and in bonded and unbonded prestressed reinforcement shall include the strain due to effective prestress.  
22.2.1.4 Changes in strain for bonded prestressed reinforcement shall be assumed proportional to the  
**R22.2.1.4** The change in strain for bonded prestressed reinforcement is influenced by the change |
22.2.2 Design assumptions for concrete
22.2.2.2 Tensile strength of concrete shall be neglected in flexural and axial strength calculations.

R22.2.2 Design assumptions for concrete

R22.2.2.2 The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength and is approximately 10 to 15 percent of the compressive strength. Tensile strength of concrete in flexure is conservatively neglected in calculating the nominal flexural strength. The strength of concrete in tension, however, is important in evaluating cracking and deflections at service loads.

24.5—Permissible stresses in prestressed concrete flexural members
24.5.2 Classification of prestressed flexural members

24.5.2.1 Prestressed flexural members shall be classified as Class U, T, or C in accordance with Table 24.5.2.1, based on the extreme fiber stress in tension \( f_t \) in the precompressed tension zone calculated at service loads assuming an uncracked section.

Table 24.5.2.1—Classification of prestressed flexural members based on \( f_t \)

<table>
<thead>
<tr>
<th>Assumed behavior</th>
<th>Class</th>
<th>Limits of ( f_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uncracked</td>
<td>U(1)</td>
<td>( f_t \leq 7.5 \sqrt{f_c} )</td>
</tr>
<tr>
<td>Transition between uncracked and cracked</td>
<td>T</td>
<td>( 7.5 \sqrt{f_c} &lt; f_t \leq 12 \sqrt{f_c} )</td>
</tr>
<tr>
<td>Cracked</td>
<td>C</td>
<td>( f_t &gt; 12 \sqrt{f_c} )</td>
</tr>
</tbody>
</table>

(1) Prestressed two-way slabs shall be designed as Class U with \( f_t \leq 6 \sqrt{f_c} \).

R24.5—Permissible stresses in prestressed concrete flexural members
R24.5.2 Classification of prestressed flexural members

R24.5.2.1 Three classes of behavior of prestressed flexural members are defined. Class U members are assumed to behave as uncracked members. Class C members are assumed to behave as cracked members. The behavior of Class T members is assumed to be in transition between uncracked and cracked. These classes apply to both bonded and unbonded prestressed flexural members, but prestressed two-way slab systems are required to be designed as Class U with \( f_t \leq 6 \sqrt{f_c} \).

The serviceability requirements for each class are summarized in Table R24.5.2.1. For comparison, Table R24.5.2.1 also shows corresponding requirements for nonprestressed members. Due to lack of strain compatibility, it is inappropriate to include the area of unbonded prestressed reinforcement in the calculation of gross or cracked section properties, although the effective prestress force should be considered when determining the location of the neutral axis. Conversely, the calculation of section properties should account for the area of the voids created by the sheathing or duct for unbonded prestressed reinforcement. A method for evaluating stresses, deflections, and crack control in cracked prestressed members is given in Mast (1998).

The precompressed tension zone is that portion of a prestressed member where flexural tension, calculated using gross section properties, would occur under unfactored dead and live loads if the prestress force was not present. Prestressed concrete is usually designed so that the prestress force introduces compression into this zone, thus effectively reducing the magnitude of the tensile...
stress.

For corrosive environments, defined as an environment in which chemical attack (such as seawater, corrosive industrial atmosphere, or sewer gas) is encountered, cracking at service loads becomes more critical to long-term performance. For these conditions, cover should be increased in accordance with 20.5.1.4, and tensile stresses in the concrete reduced to minimize possible cracking at service loads.

Table R24.5.2.1—Serviceability design requirements

<table>
<thead>
<tr>
<th>Assumed behavior</th>
<th>Prestressed</th>
<th>Nonprestressed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Class U</td>
<td>Class T</td>
</tr>
<tr>
<td>Section properties for stress calculation at service loads</td>
<td>Gross section</td>
<td>Gross section</td>
</tr>
<tr>
<td>Allowable stress at transfer</td>
<td>24.5.3</td>
<td>24.5.3</td>
</tr>
<tr>
<td>Allowable compressive stress based on uncracked section properties</td>
<td>24.5.4</td>
<td>24.5.4</td>
</tr>
<tr>
<td>Tensile stress at service loads</td>
<td>≤ 7.5 ( \sqrt{f_{c}} )</td>
<td>7.5 ( \sqrt{f_{c}} ) ≤ ( \sqrt{f_{c}} ) ≤ 12 ( \sqrt{f_{c}} )</td>
</tr>
<tr>
<td>Deflection calculation basis</td>
<td>24.2.3.8, 24.2.4.2</td>
<td>Gross section</td>
</tr>
<tr>
<td>Crack control</td>
<td>No requirement</td>
<td>No requirement</td>
</tr>
<tr>
<td>Computation of ( \Delta u ) or ( f_{u} ) for crack control</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Side skin reinforcement</td>
<td>No requirement</td>
<td>No requirement</td>
</tr>
</tbody>
</table>

8.3—Design limits
8.3.4 Stress limits in prestressed slabs
8.3.4.1 Prestressed slabs shall be designed as Class U with \( f' \leq 6 \sqrt{f_{c}} \). Other stresses in prestressed slabs immediately after transfer and at service loads shall not exceed the permissible stresses in 24.5.3 and 24.5.4.

24.5.2.2 For Class U and T members, stresses at service loads shall be permitted to be calculated using the uncracked section.

24.5.2.3 For Class C members, stresses at service loads shall be calculated using the cracked transformed section.

24.2—Deflections due to service-level gravity loads
24.2.2 Deflections calculated in accordance with 24.2.3 through 24.2.5 shall not exceed the limits in Table 24.2.2.

R24.2.2 It should be noted that the limitations given in Table 24.2.2 relate only to supported or attached nonstructural elements. For those structures in which structural members are likely to be affected by deflection or deformation of members to which they are attached in such a manner as to affect adversely the strength of the structure, these deflections and the resulting forces should be considered explicitly in the analysis and design of the structures as required by 24.2.1 (ACI 209R). When time-dependent deflections are calculated, the
portion of the deflection before attachment of the nonstructural elements may be deducted. In making this correction, use may be made of the curve in Fig. R24.2.4.1 for members of usual sizes and shapes.

![Fig. R24.2.4.1-Multipliers for time-dependent deflections.](image)

Table 24.2.2—Maximum permissible calculated deflections

<table>
<thead>
<tr>
<th>Member</th>
<th>Condition</th>
<th>Deflection to be considered</th>
<th>Deflection limitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat roofs</td>
<td>Not supporting or attached to nonstructural elements likely to be damaged by large deflections</td>
<td>Immediate deflection due to maximum of $L_c$, $S$, and $R$</td>
<td>$\ell/180$</td>
</tr>
<tr>
<td>Floors</td>
<td>Supporting or attached to nonstructural elements likely to be damaged by large deflections</td>
<td>That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflection due to all sustained loads and the immediate deflection due to any additional live load</td>
<td>$\ell/480$</td>
</tr>
<tr>
<td>Roof or floors</td>
<td>Supporting or attached to nonstructural elements not likely to be damaged by large deflections</td>
<td>That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflection due to all sustained loads and the immediate deflection due to any additional live load</td>
<td>$\ell/240$</td>
</tr>
</tbody>
</table>

1 Limit not intended to safeguard against ponding. Ponding shall be checked by calculations of deflection, including added deflections due to ponded water, and considering time-dependent effects of sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

2 Time-dependent deflection shall be calculated in accordance with 24.2.4, but shall be permitted to be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be calculated on basis of accepted engineering data relating to time-dependent characteristics of members similar to those being considered.

3 Limit shall be permitted to be exceeded if measures are taken to prevent damage to supported or attached elements.

4 Limit shall not exceed tolerance provided for nonstructural elements.

24.2.3 Calculation of immediate deflections

24.2.3.8 For prestressed Class U slabs and beams as defined in 24.5.2, it shall be permitted to calculate deflections based on $I_e$.

24.2.3.9 For prestressed Class T and Class C slabs and beams as defined in 24.5.2, deflection calculations shall be based on a cracked transformed section analysis. It shall be permitted to base deflection calculations on a bilinear moment-deflection relationship or $I_e$ in accordance with Eq. (24.2.3.9a)

$$I_e = \left( \frac{M}{M_o} \right)^3 I_e + \left[ 1 - \left( \frac{M}{M_o} \right)^3 \right] I_e$$  \hspace{1em} (24.2.3.9a)

where $M_o$ is calculated as

24.2.3.9 The effective moment of inertia equation in 24.2.3.5 was revised in the 2019 Code. The revision is not applicable to prestressed members. Equation (24.2.3.9a) maintains the provisions of previous editions of the Code for these types of members. The *PCI Design Handbook (PCI MNL 120)* gives information on deflection calculations using a bilinear moment-deflection relationship and using an effective moment of inertia. *Mast (1998)* gives additional information on deflection of cracked prestressed concrete members.

*Shaikh and Branson (1970)* shows that the $I_e$ method can be used to calculate deflections of Class C and Class T prestressed members loaded above the cracking load. For this case, the cracking moment should take into account the effect of prestress as...
24.2.4 Calculation of time-dependent deflections

24.2.4.2 Prestressed members

24.2.4.2.1 Additional time-dependent deflection of prestressed concrete members shall be calculated considering stresses in concrete and reinforcement under sustained load, and the effects of creep and shrinkage of concrete and relaxation of prestressed reinforcement.

\[ M_{cr} = \frac{(f_c + f_{pc})L}{y_c} \tag{24.2.3.9b} \]

provided in Eq. (24.2.3.9).

A method for predicting the effect of nonprestressed tension reinforcement in reducing creep camber is also given in Shaikh and Branson (1970), with approximate forms given in ACI 209R and Branson (1970).

R24.2.4 Calculation of time-dependent deflections
R24.2.4.2 Prestressed members
R24.2.4.2.1 Calculation of time-dependent deflections of prestressed concrete flexural members is challenging. The calculations should consider not only the increased deflections due to flexural stresses, but also the additional time-dependent deflections resulting from time-dependent shortening of the flexural member.

Prestressed concrete members shorten more with time than similar nonprestressed members due to the precompression in the slab or beam, which causes creep. This creep, together with concrete shrinkage, results in significant shortening of the flexural members that continues for several years after construction and should be considered in design. The shortening tends to reduce the tension in the prestressed reinforcement, reducing the precompression in the member and thereby causing increased time-dependent deflections.

Another factor that can influence time-dependent deflections of prestressed flexural members is adjacent concrete or masonry that is nonprestressed in the direction of the prestressed member. This can be a slab nonprestressed in the beam direction adjacent to a prestressed beam or a nonprestressed slab system. As the prestressed member tends to shrink and creep more than the adjacent nonprestressed concrete, the structure will tend to reach a compatibility of the shortening effects. This results in a reduction of the precompression in the prestressed member as the adjacent concrete absorbs the compression. This reduction in precompression of the prestressed member can occur over a period of years and will result in additional time-dependent deflections and an increase in tensile stresses in the prestressed member.

Any suitable method for calculating time-dependent deflections of prestressed members may be used, provided all effects are considered. Guidance may be found in ACI 209R, ACI Committee 435 (1963), Branson et al. (1970), and Ghali and Favre (1986).

3.2 Serviceability requirements – Flexural members (Corresponds to ACI 318-11 Section 18.4)

24.5—Permissible stresses in prestressed concrete  R24.5—Permissible stresses in prestressed
**24.5.3** Permissible concrete stresses at transfer of prestress

The concrete stresses at this stage are caused by the weight of the member and the force in the prestressed reinforcement after jacking reduced by the losses due to seating of the prestressed reinforcement and elastic shortening of the concrete. Shrinkage, creep, and relaxation effects are generally not included at this stage. These stresses apply to both pretensioned and post-tensioned concrete with proper modifications of the losses at transfer.

**R24.5.3.1** The permissible concrete compressive stresses at transfer of prestress are higher at ends of simply supported members than at other locations based on research in the precast, prestressed concrete industry (Castro et al. 2004; Dolan and Krohn 2007; Hale and Russell 2006).

<table>
<thead>
<tr>
<th>Location</th>
<th>Concrete compressive stress limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of simply-supported members</td>
<td>$0.70f'c$</td>
</tr>
<tr>
<td>All other locations</td>
<td>$0.60f'c$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Concrete tensile stress limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of simply-supported members</td>
<td>$6f_y$</td>
</tr>
<tr>
<td>All other locations</td>
<td>$3f_y$</td>
</tr>
</tbody>
</table>

The limits in Table 24.5.3.2 shall be permitted to be exceeded where additional bonded reinforcement in the tension zone resists the total tensile force in the concrete calculated with the assumption of an uncracked section.

**24.5.4** Permissible concrete compressive stresses at service loads

**24.5.4.1** For Class U and T members, the calculated extreme concrete fiber stress in compression at service loads, after allowance for all prestress losses, shall not exceed the limits in Table 24.5.4.1.

<table>
<thead>
<tr>
<th>Location</th>
<th>Concrete compressive stress limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of simply-supported members</td>
<td>$0.45f'c$</td>
</tr>
<tr>
<td>All other locations</td>
<td>$0.60f'c$</td>
</tr>
</tbody>
</table>

The compressive stress limit of $0.45f'c$ was originally established to decrease the probability of failure of prestressed concrete members due to repeated loads. This limit also seemed reasonable to preclude excessive creep deformation. At higher values of stress, creep strains tend to increase more rapidly as applied stress increases.

Fatigue tests of prestressed concrete beams have shown that concrete compressive failures are not the controlling criterion. Therefore, the stress limit of $0.60f'c$ permits a onethird increase in allowable compressive stress for members subject to transient...
Code

<table>
<thead>
<tr>
<th>Load condition</th>
<th>Concrete compressive stress limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress plus sustained load</td>
<td>0.45$f_c'$</td>
</tr>
<tr>
<td>Prestress plus total load</td>
<td>0.60$f_c'$</td>
</tr>
</tbody>
</table>

**24.5.1 General**
24.5.1.1 Concrete stresses in prestressed flexural members shall be limited in accordance with 24.5.2 through 24.5.4 unless it is shown by test or analysis that performance will not be impaired.

**R24.5.1 General**
R24.5.1.1 Permissible stresses in concrete address serviceability but do not ensure adequate design strength, which should be checked in accordance with other Code requirements.

A mechanism is provided such that Code limits on stress need not inhibit the development of new products, materials, and techniques in prestressed concrete construction. Approvals for the design should be in accordance with 1.10 of the Code.

**24.3—Distribution of flexural reinforcement in one-way slabs and beams**
24.3.1 Bonded reinforcement shall be distributed to control flexural cracking in tension zones of nonprestressed and Class C prestressed slabs and beams reinforced for flexure in one direction only.

24.3.3 If there is only one bonded bar, pretensioned strand, or bonded tendon nearest to the extreme tension face, the width of the extreme tension face shall not exceed $s$ determined in accordance with Table 24.3.2.

**R24.3—Distribution of flexural reinforcement in one-way slabs and beams**
R24.3.1 Where service loads result in high stresses in the reinforcement, visible cracks should be expected, and steps should be taken in detailing of the reinforcement to control cracking. For reasons of durability and appearance, many fine cracks are preferable to a few wide cracks. Detailing practices limiting bar spacing will usually lead to adequate crack control where Grade 60 reinforcement is used.

Extensive laboratory work (Gergely and Lutz 1968; Kaar 1966; Base et al. 1966) involving deformed bars demonstrated that crack width at service loads is proportional to reinforcement stress. The significant variables reflecting reinforcement detailing were found to be thickness of concrete cover and the spacing of reinforcement.

Crack width is inherently subject to wide scatter even in careful laboratory work and is influenced by shrinkage and other time-dependent effects. Improved crack control is obtained where the reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

**24.5.1 General**
24.5.1.1 Concrete compressive stress limits at service loads

Sustained live load is any portion of the service live load that will be sustained for a sufficient period to cause significant time-dependent deflections. Thus, when the sustained live and dead loads are a large percentage of the total service load, the $0.45f_c'$ limit of Table 24.5.4.1 typically controls. On the other hand, when a large portion of the total service load consists of a transient or temporary service live load, the increased stress limit of $0.60f_c'$ typically controls.

The compression limit of $0.45f_c'$ for prestress plus sustained loads will continue to control the time-dependent behavior of prestressed members.
24.3.5 The spacing of bonded flexural reinforcement in nonprestressed and Class C prestressed one-way slabs and beams subject to fatigue, designed to be watertight, or exposed to corrosive environments, shall be selected based on investigations and precautions specific to those conditions and shall not exceed the limits of 24.3.2.

24.3.2 Spacing of bonded reinforcement closest to the tension face shall not exceed the limits in Table 24.3.2, where \( c_t \) is the least distance from surface of deformed or prestressed reinforcement to the tension face. Calculated stress in deformed reinforcement, \( f_d \), and calculated change in stress in bonded prestressed reinforcement, \( \Delta f_{ps} \), shall be in accordance with 24.3.2.1 and 24.3.2.2, respectively.

Table 24.5.4.1—Maximum spacing of bonded reinforcement in nonprestressed and Class C prestressed one-way slabs and beams

<table>
<thead>
<tr>
<th>Reinforcement type</th>
<th>Maximum spacing ( c )</th>
</tr>
</thead>
</table>
| Deformed bars or wires | Lesser of: \(
\frac{15 \left( \frac{40,000}{f_d} \right) - 2 S_c}{12 \left( \frac{40,000}{f_d} \right)}
\) |
| Bonded prestressed reinforcement | Lesser of: \(
\frac{\left( \frac{2}{3} \right) \left( \frac{15 \left( \frac{40,000}{f_d} \right) - 2 S_c}{12 \left( \frac{40,000}{f_d} \right)} \right)}{\left( \frac{2}{3} \right) \left( \frac{12 \left( \frac{40,000}{f_d} \right)}{12 \left( \frac{40,000}{f_d} \right)} \right)}
\) |
| Combined deformed bars or wires and bonded prestressed reinforcement | Lesser of: \(
\frac{\left( \frac{5}{6} \right) \left( \frac{15 \left( \frac{40,000}{f_d} \right) - 2 S_c}{12 \left( \frac{40,000}{f_d} \right)} \right)}{\left( \frac{5}{6} \right) \left( \frac{12 \left( \frac{40,000}{f_d} \right)}{12 \left( \frac{40,000}{f_d} \right)} \right)}
\) |

24.3.2.1 Stress \( f_d \) in deformed reinforcement closest to the tension face at service loads shall be calculated based on the unfactored moment, or it shall be permitted to take \( f_d \) as \((2/3)f_s\).

24.3.2.2 Change in stress, \( \Delta f_{ps} \), in bonded prestressed reinforcement at service loads shall be equal to the calculated stress based on a cracked section analysis minus the decompression stress \( f_{dc} \). It shall be permitted to take \( f_{dc} \) equal to the effective stress in

<table>
<thead>
<tr>
<th>Commentary</th>
</tr>
</thead>
</table>
| R24.3.5 Although a number of studies have been conducted, clear experimental evidence is not available regarding the crack width beyond which a corrosion danger exists (ACI 222R). Exposure tests indicate that concrete quality, adequate consolidation, and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface (Schießl and Raupach 1997).

Provisions related to increased concrete cover and durability of reinforcement is covered in 20.5, while durability of concrete is covered in 19.3.

R24.3.2 The spacing of reinforcement is limited to control cracking (Beeby 1979; Frosch 1999; ACI Committee 318 1999). For the case of beams with Grade 60 reinforcement and 2 in. clear cover to the primary reinforcement, with \( f_c = 40,000 \text{ psi} \), the maximum bar spacing is 10 in.

Crack widths in structures are highly variable. The Code provisions for spacing are intended to limit surface cracks to a width that is generally acceptable in practice but may vary widely in a given structure.

The role of cracks in the corrosion of reinforcement is controversial. Research (Darwin et al. 1985; Oesterle 1997) shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. For this reason, the Code does not differentiate between interior and exterior exposures. Only tension reinforcement nearest the tension face need be considered in selecting the value of \( c_t \) used in calculating spacing requirements. To account for prestressed reinforcement, such as strand, having bond characteristics less effective than deformed reinforcement, a two-thirds effectiveness factor is used in Table 24.3.2.

For post-tensioned members designed as cracked members, it will usually be advantageous to provide crack control by the use of deformed reinforcement, for which the provisions in Table 24.3.2 for deformed bars or wires may be used. Bonded reinforcement required by other provisions of the Code may also be used as crack control reinforcement.

R24.3.2.1 For applications in which crack control is critical, the designer should consider reducing the value of \( f_c \) to help control cracking. Research by Frosch et al. (2014) and Puranam (2018) supports the use of these design provisions for Grade 100 reinforcement.

R24.3.2.2 It is conservative to take the decompression stress \( f_{dc} \) equal to \( f_{dc} \), the effective stress in the prestressed reinforcement. The maximum limitation of 36,000 psi for \( \Delta f_{ps} \) is intended to be similar to the maximum allowable stress in Grade 60 reinforcement (\( f_c = 40,000 \text{ psi} \)).
the prestressed reinforcement $f_{w}$. The value of $\Delta f_{ps}$ shall not exceed 36,000 psi. If $\Delta f_{ps}$ does not exceed 20,000 psi, the spacing limits in Table 24.3.2 need not be satisfied.

9.7—Reinforcement detailing
9.7.2 Reinforcement spacing
9.7.2.3 For nonprestressed and Class C prestressed beams with $h$ exceeding 36 in., longitudinal skin reinforcement shall be uniformly distributed on both side faces of the beam for a distance $h/2$ from the tension face. Spacing of skin reinforcement shall not exceed $s$ given in 24.3.2, where $c_{c}$ is the clear cover from the skin reinforcement to the side face. It shall be permitted to include skin reinforcement in strength calculations if a strain compatibility analysis is made.

exemption for members with $\Delta f_{ps}$ less than 20,000 psi reflects that many structures designed by working stress methods and with low reinforcement stress served their intended functions with very limited flexural cracking.

R9.7—Reinforcement detailing
R9.7.2 Reinforcement spacing
R9.7.2.3 For relatively deep beams, some reinforcement should be placed near the vertical faces of the tension zone to control cracking in the web (Frantz and Breen 1980; Frosch 2002), as shown in Fig. R9.7.2.3. Without such auxiliary reinforcement, the width of the cracks in the web may exceed the crack widths at the level of the flexural tension reinforcement.

The size of the skin reinforcement is not specified; research has indicated that the spacing rather than bar size is of primary importance (Frosch 2002). Bar sizes No. 3 to No. 5, or welded wire reinforcement with a minimum area of 0.1 in.$^{2}$ per foot of depth, are typically provided.

Fig. R9.7.2.3-Skin reinforcement for beams and joints with $h > 36$ in.

3.3. Permissible stresses in prestressing steel (Corresponds to ACI 318-11 Section 18.5)

20.3—Prestressing strands, wires, and bars
20.3.2 Design properties
20.3.2.5 Permissible tensile stresses in prestressed reinforcement
20.3.2.5.1 The tensile stress in prestressed reinforcement shall not exceed the limits in Table 20.3.2.5.1.

R20.3—Prestressing strands, wires, and bars
R20.3.2 Design properties
R20.3.2.5 Permissible tensile stresses in prestressed reinforcement
R20.3.2.5.1 Because of the high yield strength of low-relaxation strand and wire meeting the requirements of ASTM A416 and ASTM A421 including Supplementary Requirement S1 “Low-
Table 20.3.2.5.1—Maximum permissible tensile stresses in prestressed reinforcement

<table>
<thead>
<tr>
<th>Stage</th>
<th>Location</th>
<th>Maximum tensile stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>During Stressing</td>
<td>At jacking end</td>
<td>Least of $0.94f_y$, $0.80f_u$, Maximum jacking force recommended by the supplier of anchorage device</td>
</tr>
<tr>
<td>Immediately after force transfer</td>
<td>At post-tensioning anchorage devices and couplers</td>
<td>$0.70f_u$</td>
</tr>
</tbody>
</table>

25.9—Anchorage zones for post-tensioned tendons

25.9.2 Required strength

25.9.2.1 Factored prestressing force at the anchorage device, $P_{pu}$, shall exceed the least of (a) through (c), where 1.2 is the load factor from 5.3.12:

(a) $1.2(0.94f_y)A_{ps}$
(b) $1.2(0.80f_u)A_{ps}$
(c) Maximum jacking force designated by the supplier of anchorage devices multiplied by 1.2

3.4. Loss of prestress (Corresponds to ACI 318-11 Section 18.6)

20.3—Prestressing strands, wires, and bars

20.3.2 Design properties

20.3.2.6 Prestress losses

20.3.2.6.1 Prestress losses shall be considered in the calculation of the effective tensile stress in the prestressed reinforcement, $f_{se}$, and shall include (a) through (f):

(a) Prestressed reinforcement seating at transfer
(b) Elastic shortening of concrete
(c) Creep of concrete
(d) Shrinkage of concrete
(e) Relaxation of prestressed reinforcement
(f) Friction loss due to intended or unintended curvature in post-tensioning tendons

20.3.2.6.2 Calculated friction loss in post-tensioning tendons shall be based on experimentally determined wobble and curvature friction coefficients.

R20.3—Prestressing strands, wires, and bars

R20.3.2 Design properties

R20.3.2.6 Prestress losses

R20.3.2.6.1 ACI 423.10R provides a comprehensive treatment of the estimation of prestress losses.

Actual losses, greater or smaller than the calculated values, have little effect on the design strength of the member, but affect service load behavior (deflections, camber, cracking load) and connections. At service loads, overestimation of prestress losses can be almost as detrimental as underestimation because the former can result in excessive camber and horizontal movement.

R20.3.2.6.2 Estimation of friction losses in post-tensioned tendons is addressed in the Post-Tensioning Manual (TAB.1). Values of the wobble and curvature friction coefficients to be used for the particular types of prestressing reinforcement and particular types of ducts should be obtained from the manufacturers of the tendons. An unrealistically low estimate of the friction loss can lead to improper camber, or potential deflection, of the member and inadequate prestress. Overestimation of the friction may result in extra prestressing force. This could lead to excessive camber and excessive shortening of a member. If the friction factors are determined to be
20.3.2.6.3 Where loss of prestress in a member is anticipated due to connection of the member to adjoining

26.10—Additional requirements for prestressed concrete
26.10.1 Design information:
(a) Magnitude and location of prestressing forces.

26.10.2 Compliance requirements:
(e) Prestressing force and friction losses shall be verified by (1) and (2).

(1) Measured elongation of prestressed reinforcement compared with elongation calculated using the modulus of elasticity determined from tests or as reported by the manufacturer.
(2) Jacking force measured using calibrated equipment such as a hydraulic pressure gauge, load cell, or dynamometer.

(f) The cause of any difference in force determination between (1) and (2) of 26.10.2(e) that exceeds 5 percent for pretensioned construction or 7 percent for posttensioned construction shall be ascertained and corrected, unless approved by the licensed design professional.

R26.10—Additional requirements for prestressed concrete

R26.10.2 Elongation measurements for prestressing should be in accordance with the procedures outlined in the Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (MNL 117), published by the Precast/Prestressed Concrete Institute.

R26.10.2(f) The 5 percent tolerance for pretensioned construction reflects experience with production of those members. Because prestressing reinforcement for pretensioned construction is usually stressed in air with minimal friction effects, a 5 percent tolerance is considered reasonable. For posttensioned construction, a slightly higher tolerance is permitted. Elongation measurements for posttensioned construction are affected by several factors that are less significant or that do not exist for pretensioned construction. The friction along prestressing reinforcement in post-tensioning applications may be affected to varying degrees by placing tolerances and small irregularities in tendon profile due to tendon and concrete placement. The friction coefficients between the prestressing reinforcement and the duct are also subject to variation.
3.5. Flexural strength (Corresponds to ACI 318-11 Section 18.7)

22.2—Design assumptions for moment and axial strength
22.2.1 Equilibrium and strain compatibility
22.2.1.3 Strain in prestressed concrete and in bonded and unbonded prestressed reinforcement shall include the strain due to effective prestress.

20.3—Prestressing strands, wires, and bars
20.3.2 Design properties
20.3.2.3 Stress in bonded prestressed reinforcement at nominal flexural strength, \( f_{pu} \)
20.3.2.3.1 As an alternative to a more accurate calculation of \( f_{pu} \) based on strain compatibility, values of \( f_{pu} \) calculated in accordance with Eq. (20.3.2.3.1) shall be permitted for members with bonded prestressed reinforcement if all prestressed reinforcement is in the tension zone and \( f_{pc} \geq 0.5f_{pu} \).

\[
f_{pu} = f_{pm} \left[ 1 - \gamma_p \left( \frac{f_{pm}}{f_p} + \frac{d}{d_p} \left( \rho - \rho' \right) \right) \right]^{(20.3.2.3.1)}
\]

where \( \gamma_p \) is in accordance with Table 20.3.2.3.1. If compression reinforcement is considered for the calculation of \( f_{pu} \) by Eq. (20.3.2.3.1), (a) and (b) shall be satisfied.

(a) If \( d' \) exceeds 0.15\( d_p \), the compression reinforcement shall be neglected in Eq. (20.3.2.3.1).

(b) If compression reinforcement is included in Eq. (20.3.2.3.1), the term

\[
\left( \rho_p \frac{f_{pm}}{f_p} + \frac{d}{d_p} \frac{f_p}{f_p} \left( \rho - \rho' \right) \right)
\]

shall not be taken less than 0.17.

Table 20.3.2.3.1—Values of \( \gamma_p \) for use in Eq. (20.3.2.3.1)

<table>
<thead>
<tr>
<th>( f_{pm}/f_p )</th>
<th>( \gamma_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \geq 0.80 )</td>
<td>0.55</td>
</tr>
<tr>
<td>( \geq 0.85 )</td>
<td>0.40</td>
</tr>
<tr>
<td>( \geq 0.90 )</td>
<td>0.28</td>
</tr>
</tbody>
</table>

R20.3—Prestressing strands, wires, and bars
R20.3.2 Design properties
R20.3.2.3 Stress in bonded prestressed reinforcement at nominal flexural strength, \( f_{pu} \)
R20.3.2.3.1 Use of Eq. (20.3.2.3.1) may underestimate the strength of beams with high percentages of reinforcement and, for more accurate evaluations of their strength, the strain compatibility and equilibrium method should be used. If part of the prestressed reinforcement is in the compression zone, a strain compatibility and equilibrium method should be used.

The \( \gamma_p \) term in Eq. (20.3.2.3.1) and Table 20.3.2.3.1 reflects the influence of different types of prestressing reinforcement on the value of \( f_{pu} \). Table R20.3.2.3.1 shows prestressing reinforcement type and the associated ratio \( f_{pu}/f_{pu} \).

R20.3.2.3.1(a) If \( d' \) is large, the strain in compression reinforcement can be considerably less than its yield strain. In such a case, the compression reinforcement does not influence \( f_{pu} \) as favorably as implied by Eq. (20.3.2.3.1). For this reason, if \( d' \) exceeds 0.15\( d_p \), Eq. (20.3.2.3.1) is applicable only if the compression reinforcement is neglected.

R20.3.2.3.1(b) The \( \rho' \) term in Eq. (20.3.2.3.1) reflects the increased value of \( f_{pu} \) obtained when compression reinforcement is provided in a beam with a large reinforcement index. If the term \( \rho_p (f_{pu}/f_p) + (d/d_p)(f_p/f_p)(\rho - \rho') \) is small, the neutral axis depth is small, the compressive reinforcement does not develop its yield strength, and Eq. (20.3.2.3.1) becomes unconservative. For this reason, the term \( \rho_p (f_{pu}/f_p) + (d/d_p)(f_p/f_p)(\rho - \rho') \) may not be taken less than 0.17 if compression reinforcement is taken into account when calculating \( f_{pu} \). The compression reinforcement may be conservatively neglected when using Eq. (20.3.2.3.1) by taking \( \rho' \) as zero, in which case the term \( \rho_p (f_{pu}/f_p) + (d/d_p)(f_p/f_p)(\rho) \) may be less than 0.17 and an acceptable value of \( f_{pu} \) is obtained.
20.3.2.4 Stress in unbonded prestressed reinforcement at nominal flexural strength, $f_{ps}$

20.3.2.4.1 As an alternative to a more accurate calculation of $f_{ps}$, values of $f_{ps}$ calculated in accordance with Table 20.3.2.4.1 shall be permitted for members prestressed with unbonded tendons if $f_{se} \geq 0.5f_{pu}$.

Table 20.3.2.4.1—Approximate values of $f_{ps}$ at nominal flexural strength for unbonded tendons

<table>
<thead>
<tr>
<th>$f_{se}/n_h$</th>
<th>$f_{ps}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 35$</td>
<td>The least of:</td>
</tr>
<tr>
<td></td>
<td>$f_{se} + 10,000 + f_{c}'/(300\rho)$</td>
</tr>
<tr>
<td></td>
<td>$f_{se} + 60,000$</td>
</tr>
<tr>
<td></td>
<td>$f_{ps}$</td>
</tr>
<tr>
<td>$&gt; 35$</td>
<td>The least of:</td>
</tr>
<tr>
<td></td>
<td>$f_{se} + 10,000 + f_{c}'/(300\rho)$</td>
</tr>
<tr>
<td></td>
<td>$f_{se} + 30,000$</td>
</tr>
<tr>
<td></td>
<td>$f_{ps}$</td>
</tr>
</tbody>
</table>

22.3—Flexural strength

22.3.2 Prestressed concrete members

22.3.2.1 Deformed reinforcement conforming to 20.2.1, provided in conjunction with prestressed reinforcement, shall be permitted to be considered to contribute to the tensile force and be included in flexural strength calculations at a stress equal to $f_y$.

22.3.2.2 Other nonprestressed reinforcement shall be permitted to be considered to contribute to the flexural strength if a strain compatibility analysis is performed to calculate stresses in such reinforcement.

Table R20.3.2.3.1—Ratio of $f_{py}/f_{pu}$ associated with reinforcement type

<table>
<thead>
<tr>
<th>Prestressing reinforcement type</th>
<th>$f_{py}/f_{pu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-strength prestressing bars</td>
<td>$f_{py}/f_{pu}$</td>
</tr>
<tr>
<td>ASTM A722 Type I (Plain)</td>
<td>$\geq 0.85$</td>
</tr>
<tr>
<td>ASTM A722 Type II (Deformed)</td>
<td>$\geq 0.80$</td>
</tr>
<tr>
<td>Stress-relieved strand and wire</td>
<td>$\geq 0.85$</td>
</tr>
<tr>
<td>ASTM A416</td>
<td></td>
</tr>
<tr>
<td>ASTM A421</td>
<td>$\geq 0.90$</td>
</tr>
</tbody>
</table>

R20.3.2.4 Stress in unbonded prestressed reinforcement at nominal flexural strength, $f_{ps}$

R20.3.2.4.1 The term $[f_{se} + 10,000 + f_{c}'/(300\rho)]$ reflects results of tests on members with unbonded tendons and span-to-depth ratios greater than 35 (one-way slabs, flat plates, and flat slabs) (Mojtahedi and Gamble 1978). These tests also indicate that the term $[f_{se} + 10,000 + f_{c}'/(300\rho)]$, formerly used for all span-depth ratios, overestimates the amount of stress increase in such members. Although these same tests indicate that the moment strength of those shallow members designed using $[f_{se} + 10,000 + f_{c}'/(100\rho)]$ meets the factored load strength requirements, this reflects the effect of the Code requirements for minimum bonded reinforcement as well as the limitation on concrete tensile stress that often control the amount of prestressing force provided.

R22.3—Flexural strength

R22.3.2 Prestressed concrete members

R22.3.2.2 Bond length for nontensioned prestressing strand (Salmons and McCrate 1977; PCA 1980) should be sufficient to develop the stress consistent with strain compatibility analysis at the critical section.
3.6. Limits for reinforcement of flexural members (Corresponds to ACI 318-11 Section 18.8)

21.2—Strength reduction factors for structural concrete members and connections

21.2.2 Strength reduction factor for moment, axial force, or combined moment and axial force shall be in accordance with Table 21.2.2.

R21.2—Strength reduction factors for structural concrete members and connections

R21.2.2 The nominal strength of a member that is subjected to moment or combined moment and axial force is determined for the condition where the strain in the extreme compression fiber is equal to the assumed strain limit of 0.003. The net tensile strain $\varepsilon_t$ is the tensile strain calculated in the extreme tension reinforcement at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension reinforcement is determined from a linear strain distribution at nominal strength, shown in Fig. R21.2.2a for a nonprestressed member.

Members subjected to only axial compression are considered to be compression-controlled and members subjected to only axial tension are considered to be tension-controlled.

If the net tensile strain in the extreme tension reinforcement is sufficiently large ($\geq \varepsilon_t + 0.003$), the section is defined as tension-controlled, for which warning of failure by excessive deflection and cracking may be expected. The limit $\geq \varepsilon_t + 0.003$ provides sufficient ductility for most applications. Before the 2019 Code, the tension-controlled limit on $\varepsilon_t$ was defined as 0.005 established primarily on the basis of Grade 60 nonprestressed reinforcement and prestressed reinforcement, with some consideration given to higher grades of nonprestressed reinforcement (Mast 1992). Beginning with the 2019 Code, to accommodate nonprestressed reinforcement of higher grades, the tension-controlled limit on $\varepsilon_t$ in Table 21.2.2 is defined as $\varepsilon_t + 0.003$. This expression is consistent with the recommendations of Mast (1992) for the general case of reinforcement other than Grade 60, and test data show that the expression leads to elements with adequate ductility.

One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames, which is addressed in 6.6.5. Because redistribution of moment depends on the ductility available in the hinge regions, redistribution of moment is limited to sections that have a net tensile strain of at least 0.0075.

If the net tensile strain in the extreme tension reinforcement is small ($\leq \varepsilon_t$), a brittle compression failure condition is expected, with little warning of impending failure. Before ACI 318-14, the compression-controlled strain limit was defined as 0.002 for Grade 60 reinforcement and all prestressed reinforcement, but it was not explicitly defined for other types of reinforcement. The compression-controlled strain limit $\varepsilon_t$ is defined in 21.2.2.1 and 21.2.2.2 for deformed and prestressed reinforcement,
respectively. Beams and slabs are usually tension-controlled, whereas columns may be compression-controlled. Some members, such as those with small axial forces and large bending moments, experience net tensile strain in the extreme tension reinforcement between the limits of $\varepsilon_{tu}$ and $(\varepsilon_{tu} + 0.003)$. These sections are in a transition region between compression-controlled and tension-controlled.

This section specifies the appropriate strength reduction factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region. Beginning with the 2019 Code, the expression $(\varepsilon_{tu} + 0.003)$ defines the limit on $\varepsilon_t$ for tension-controlled behavior in Table 21.2.2. For sections subjected to combined axial force and moment, design strengths are determined by multiplying both $P_n$ and $M_n$ by the appropriate single value of $\phi$.

A lower $\phi$-factor is used for compression-controlled sections than for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections. Columns with spiral reinforcement are assigned a higher $\phi$-factor than columns with other types of transverse reinforcement because spiral columns have greater ductility and toughness. For sections within the transition region, the value of $\phi$ may be determined by linear interpolation, as shown in Fig. R21.2.2b.

### Table 21.2.2—Strength reduction factor $\phi$ for moment, axial force, or combined moment and axial force

<table>
<thead>
<tr>
<th>Net tensile stain $\varepsilon_t$</th>
<th>Classification</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_t \leq \varepsilon_{tu}$</td>
<td>Compression-controlled</td>
<td>$0.75$</td>
</tr>
<tr>
<td>$\varepsilon_{tu} &lt; \varepsilon_t &lt; \varepsilon_{tu} + 0.003$</td>
<td>Transition$^1$</td>
<td>$0.75 + 0.15 \frac{\varepsilon_t - \varepsilon_{tu}}{0.003}$ (a)</td>
</tr>
<tr>
<td>$\varepsilon_t \geq \varepsilon_{tu} + 0.003$</td>
<td>Tension-controlled</td>
<td>$0.90$</td>
</tr>
</tbody>
</table>

$^1$For sections classified as transition, it shall be permitted to use $\phi$ corresponding to compression-controlled sections.

---

**Fig. R21.2.2a**—Strain distribution and net tensile strain.
7.6—Reinforcement limits
7.6.2 Minimum flexural reinforcement in prestressed slabs
7.6.2.1 For slabs with bonded prestressed reinforcement, total quantity of \( A_s \) and \( A_{ps} \) shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of \( f_r \) as given in 19.2.3.

7.6.2.2 For slabs with both flexural and shear design strength at least twice the required strength, 7.6.2.1 need not be satisfied.

8.6—Reinforcement limits
8.6.2 Minimum flexural reinforcement in prestressed slabs
8.6.2.2 For slabs with bonded prestressed reinforcement, total quantity of \( A_s \) and \( A_{ps} \) shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of \( f_r \) defined in 19.2.3.

8.6.2.1 For slabs with both flexural and shear design strength at least twice the required strength, 8.6.2.2 need not be satisfied.

9.6—Reinforcement limits
9.6.2 Minimum flexural reinforcement in prestressed beams
9.6.2.1 For beams with bonded prestressed reinforcement, total quantity of \( A_s \) and \( A_{ps} \) shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of \( f_r \) defined in 19.2.3.

R7.6—Reinforcement limits
R7.6.2 Minimum flexural reinforcement in prestressed slabs
The requirements for minimum flexural reinforcement for prestressed one-way slabs are the same as those for prestressed beams. Refer to R9.6.2 for additional information.

R8.6—Reinforcement limits
R8.6.2 Minimum flexural reinforcement in prestressed slabs
R8.6.2.2 This provision is a precaution against abrupt flexural failure developing immediately after cracking. A flexural member designed according to Code provisions requires considerable additional load beyond cracking to reach its flexural strength. Thus, considerable deflection would warn that the member strength is approaching. If the flexural strength were reached shortly after cracking, the warning deflection would not occur. Transfer of force between the concrete and the prestressed reinforcement, and abrupt flexural failure immediately after cracking, does not occur when the prestressed reinforcement is unbonded (ACI 423.3R); therefore, this requirement does not apply to members with unbonded tendons.

R9.6—Reinforcement limits
R9.6.2 Minimum flexural reinforcement in prestressed beams
R9.6.2.1 Minimum flexural reinforcement is required for reasons similar to nonprestressed beams as discussed in R9.6.1.1. Abrupt flexural failure immediately after cracking does not occur when the prestressed reinforcement is unbonded (ACI 423.3R); therefore, this requirement does not apply to members with unbonded tendons.

![Fig. R21.2.2b-Variation of \( \phi \) with net tensile strain in extreme tension reinforcement, \( \varepsilon_t \).](image-url)
9.6.2.2 For beams with both flexural and shear design strength at least twice the required strength, 9.6.2.1 need not be satisfied.

7.7—Reinforcement detailing
7.7.2 Reinforcement spacing
7.7.2.2 For nonprestressed and Class C prestressed slabs, spacing of bonded longitudinal reinforcement closest to the tension face shall not exceed $s$ given in 24.3.

7.7.2.3 For nonprestressed and Class T and C prestressed slabs with unbonded tendons, maximum spacing $s$ of deformed longitudinal reinforcement shall be the lesser of $3h$ and 18 in.

11.7—Reinforcement detailing
11.7.2 Spacing of longitudinal reinforcement
11.7.2.4 Flexural tension reinforcement shall be well distributed and placed as close as practicable to the tension face.

24.3—Distribution of flexural reinforcement in one-way slabs and beams
24.3.1 Bonded reinforcement shall be distributed to control flexural cracking in tension zones of nonprestressed and Class C prestressed slabs and beams reinforced for flexure in one direction only.

R7.7—Reinforcement detailing
R7.7.2 Reinforcement spacing

R7.7.2.3 Editions of ACI 318 prior to 2019 excluded the provisions of 7.7.2.3 for prestressed concrete. However, Class T and C slabs prestressed with unbonded tendons rely solely on deformed reinforcement for crack control. Consequently, the requirements of 7.7.2.3 have been extended to apply to Class T and C slabs prestressed with unbonded tendons.

R24.3—Distribution of flexural reinforcement in one-way slabs and beams
R24.3.1 Where service loads result in high stresses in the reinforcement, visible cracks should be expected, and steps should be taken in detailing of the reinforcement to control cracking. For reasons of durability and appearance, many fine cracks are preferable to a few wide cracks. Detailing practices limiting bar spacing will usually lead to adequate crack control where Grade 60 reinforcement is used. Extensive laboratory work (Gergely and Lutz 1968; Kaar 1966; Base et al. 1966) involving deformed bars demonstrated that crack width at service loads is proportional to reinforcement stress. The significant variables reflecting reinforcement detailing were found to be thickness of concrete cover and the spacing of reinforcement. Crack width is inherently subject to wide scatter even in careful laboratory work and is influenced by shrinkage and other time-dependent effects. Improved crack control is obtained where the reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

3.7. Minimum bonded reinforcement (Corresponds to ACI 318-11 Section 24)
18.9)  

7.6—Reinforcement limits  

7.6.2 Minimum flexural reinforcement in prestressed slabs  

7.6.2.3 For slabs with unbonded tendons, the minimum area of bonded deformed longitudinal reinforcement, \( A_{s,min} \), shall be:  

\[
A_{s,min} \geq 0.004 A_y \quad (7.6.2.3)
\]

where \( A_y \) is the area of that part of the cross section between the flexural tension face and the centroid of the gross section.

8.6—Reinforcement limits  

8.6.2 Minimum flexural reinforcement in prestressed slabs  

8.6.2.3 For prestressed slabs, a minimum area of bonded deformed longitudinal reinforcement, \( A_{s,min} \), shall be provided in the precompressed tension zone in the direction of the span under consideration in accordance with Table 8.6.2.3.

Table 8.6.2.3—Minimum bonded deformed longitudinal reinforcement \( A_{s,min} \) in two-way slabs with bonded or unbonded tendons

<table>
<thead>
<tr>
<th>Region</th>
<th>Calculated ( f_y ) after all losses, psi</th>
<th>( A_{s,min} ) in.²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Positive moment</td>
<td>( f_y \leq 2\sqrt{f_c} )</td>
<td>Not required</td>
</tr>
<tr>
<td></td>
<td>( 2\sqrt{f_c} &lt; f_y \leq 6\sqrt{f_c} )</td>
<td>( \frac{N}{0.075f_y} )</td>
</tr>
<tr>
<td>Negative moment</td>
<td>( f_y \leq 6\sqrt{f_c} )</td>
<td>0.00075A_y</td>
</tr>
</tbody>
</table>

¹The value of \( f_y \) shall not exceed 60,000 psi  
²For slabs with bonded tendons, it shall be permitted to reduce \( A_{s,min} \) by the area of the bonded prestressed reinforcement located within the area used to determine \( N_y \), for positive moment, or within the width of slab defined in 8.7.5.3(a) for negative moment.

R7.6—Reinforcement limits  

R7.6.2 Minimum flexural reinforcement in prestressed slabs  

The requirements for minimum flexural reinforcement for prestressed one-way slabs are the same as those for prestressed beams. Refer to R9.6.2 for additional information.

R8.6—Reinforcement limits  

R8.6.2 Minimum flexural reinforcement in prestressed slabs  

R8.6.2.3 Some bonded reinforcement is required by the Code in prestressed slabs to limit crack width and spacing at service load when concrete tensile stresses exceed the modulus of rupture and, for slabs with unbonded tendons, to ensure flexural performance at nominal strength, rather than performance as a tied arch. Providing the minimum bonded reinforcement as stipulated in this provision helps to ensure adequate performance.

The minimum amount of bonded reinforcement in two-way flat slab systems is based on reports by Joint ACI-ASCE Committee 423 (1958) and ACI 423.3R. Limited research available for two-way flat slabs with drop panels (Odello and Mehta 1967) indicates that behavior of these particular systems is similar to the behavior of flat plates. For usual loads and span lengths, flat plate tests summarized in Joint ACI-ASCE Committee 423 (1958) and experience since the 1963 Code was adopted indicate satisfactory performance without bonded reinforcement in positive moment regions where \( f_y \leq 2\sqrt{f_c} \). In positive moment regions where \( 2\sqrt{f_c} \leq f_y \leq 6\sqrt{f_c} \), a minimum bonded reinforcement area proportioned to resist \( N_y \) according to Eq. (8.6.2.3(b)) is required. The tensile force \( N_y \) is calculated at service load on the basis of an uncracked, homogeneous section.

Research on unbonded post-tensioned two-way flat slab systems (Joint ACI-ASCE Committee 423 1958, 1974; ACI 423.3R; Odello and Mehta 1967) shows that bonded reinforcement in negative moment regions, proportioned on the basis of 0.075 percent of the cross-sectional area of the slab-beam strip, provides sufficient ductility and reduces crack width and spacing. The same area of bonded reinforcement is required in slabs with either bonded or unbonded tendons. The minimum bonded reinforcement area required by Eq. (8.6.2.3(c)) is a minimum area...
9.6—Reinforcement limits
9.6.2 Minimum flexural reinforcement in prestressed beams
9.6.2.3 For beams with unbonded tendons, the minimum area of bonded deformed longitudinal reinforcement $A_{s,min}$ shall be:

$$A_{s,min} \geq 0.004 A_{t}$$  \hspace{1cm} (9.6.2.3)

where $A_{t}$ is the area of that part of the cross section between the flexural tension face and the centroid of the gross section.

11.7—Reinforcement detailing
11.7.2 Spacing of longitudinal reinforcement
11.7.2.4 Flexural tension reinforcement shall be well distributed and placed as close as practicable to the tension face.
24.3—Distribution of flexural reinforcement in one-way slabs and beams
24.3.1 Bonded reinforcement shall be distributed to control flexural cracking in tension zones of nonprestressed and Class C prestressed slabs and beams reinforced for flexure in one direction only.

R24.3—Distribution of flexural reinforcement in one-way slabs and beams
R24.3.1 Where service loads result in high stresses in the reinforcement, visible cracks should be expected, and steps should be taken in detailing of the reinforcement to control cracking. For reasons of durability and appearance, many fine cracks are preferable to a few wide cracks. Detailing practices limiting bar spacing will usually lead to adequate crack control where Grade 60 reinforcement is used.

Extensive laboratory work (Gergely and Lutz 1968; Kaar 1966; Base et al. 1966) involving deformed bars demonstrated that crack width at service loads is proportional to reinforcement stress. The significant variables reflecting reinforcement detailing were found to be thickness of concrete cover and the spacing of reinforcement.

Crack width is inherently subject to wide scatter even in careful laboratory work and is influenced by shrinkage and other time-dependent effects. Improved crack control is obtained where the reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

8.7—Reinforcement detailing
8.7.5 Flexural reinforcement in prestressed slabs
8.7.5.3 Bonded longitudinal reinforcement required by Eq. (8.6.2.3(c)) shall be placed in the top of the slab, and shall be in accordance with (a) through (c):

(a) Reinforcement shall be distributed between lines that are 1.5h outside opposite faces of the column support.
(b) At least four deformed bars, deformed wires, or bonded strands shall be provided in each direction.
(c) Maximum spacing s between bonded longitudinal reinforcement shall not exceed 12 in.

7.7—Reinforcement detailing
7.7.4 Flexural reinforcement in prestressed slabs
7.7.4.4 Termination of deformed reinforcement in slabs with unbonded tendons
7.7.4.4.1 Length of deformed reinforcement required by 7.6.2.3 shall be in accordance with (a) and (b):

(a) At least ℓd/3 in positive moment areas and be centered in those areas
(b) At least ℓd/6 on each side of the face of support

R7.7—Reinforcement detailing
R7.7.4 Flexural reinforcement in prestressed slabs
R7.7.4.4 Termination of deformed reinforcement in slabs with unbonded tendons
Requirements for termination of deformed reinforcement in one-way slabs with unbonded tendons are the same as those for beams. Refer to R9.7.4.4 for additional information.

8.7—Reinforcement detailing
8.7.5 Flexural reinforcement in prestressed slabs
8.7.5.5 Termination of deformed reinforcement in slabs with unbonded tendons
8.7.5.5.1 Length of deformed reinforcement required
by 8.6.2.3 shall be in accordance with (a) and (b):
(a) In positive moment areas, length of reinforcement shall be at least \( \ell_n/3 \) and be centered in those areas
(b) In negative moment areas, reinforcement shall extend at least \( \ell_n/6 \) on each side of the face of support

9.7—Reinforcement detailing
9.7.4 Flexural reinforcement in prestressed beams
9.7.4.4 Termination of deformed reinforcement in beams with unbonded tendons
9.7.4.4.1 Length of deformed reinforcement required by 9.6.2.3 shall be in accordance with (a) and (b):
(a) At least \( \ell_n/3 \) in positive moment areas and be centered in those areas
(b) At least \( \ell_n/6 \) on each side of the face of support in negative moment areas

7.7—Reinforcement detailing
7.7.4 Flexural reinforcement in prestressed slabs
7.7.4.2 If nonprestressed reinforcement is required to satisfy flexural strength, the detailing requirements of 7.7.3 shall be satisfied.

8.7—Reinforcement detailing
8.7.5 Flexural reinforcement in prestressed slabs
8.7.5.2 If bonded deformed longitudinal reinforcement is required to satisfy flexural strength or for tensile stress conditions in accordance with Eq. (8.6.2.3(b)), the detailing requirements of 7.7.3 shall be satisfied.

9.7—Reinforcement detailing
9.7.4 Flexural reinforcement in prestressed beams
9.7.4.2 If nonprestressed reinforcement is required to satisfy flexural strength, the detailing requirements of 9.7.3 shall be satisfied.

R8.7.5.5.1 The minimum lengths apply for bonded reinforcement required by 8.6.2.3, but not required for flexural strength in accordance with 22.3.2. Research (Odello and Mehta 1967) on continuous spans shows that these minimum lengths provide adequate behavior under service load and factored load conditions.

R9.7—Reinforcement detailing
R9.7.4 Flexural reinforcement in prestressed beams
R9.7.4.4 Termination of deformed reinforcement in beams with unbonded tendons
R9.7.4.4.1 The minimum lengths apply for bonded reinforcement required by 9.6.2.3. Research (Odello and Mehta 1967) on continuous spans shows that these minimum lengths provide satisfactory behavior under service load and factored load conditions.

R8.7—Reinforcement detailing
R8.7.5 Flexural reinforcement in prestressed slabs
R8.7.5.2 Bonded reinforcement should be adequately anchored to develop the required strength to resist factored loads. The requirements of 7.7.3 are intended to provide adequate anchorage for tensile or compressive forces developed in bonded reinforcement by flexure under factored loads in accordance with 22.3.2, or by tensile stresses at service load in accordance with Eq. (8.6.2.3(b)).

R9.7—Reinforcement detailing
R9.7.4 Flexural reinforcement in prestressed beams
R9.7.4.2 Nonprestressed reinforcement should be developed to achieve factored load forces. The requirements of 9.7.3 provide that bonded reinforcement required for flexural strength under factored loads is developed to achieve tensile or compressive forces.

3.8. Statically indeterminate structures (Corresponds to ACI 318-11 Section 18.10)

4.12—Requirements for specific types of construction
4.12.2 Prestressed concrete systems
4.12.2.1 Design of prestressed members and systems shall be based on strength and on behavior at service conditions at all critical stages during the life of the structure from the time prestress is first applied.

R4.12—Requirements for specific types of construction
R4.12.2 Prestressed concrete systems
Prestressing, as used in the Code, may apply to pretensioning, bonded post-tensioning, or unbonded posttensioning. All requirements in the Code apply to prestressed systems and members, unless specifically
4.7—Serviceability
4.7.1 Evaluation of performance at service load conditions shall consider reactions, moments, shears, torsions, and axial forces induced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural members, and foundation settlement.

R4.7—Serviceability
Serviceability refers to the ability of the structural system or structural member to provide appropriate behavior and functionality under the actions affecting the system. Serviceability requirements address issues such as deflections and cracking, among others. Serviceability considerations for vibrations are discussed in R6.6.3.2.2 and R24.1.

Except as stated in Chapter 24, service-level load combinations are not defined in this Code, but are discussed in Appendix C of ASCE/SEI 7-16. Appendixes to ASCE/SEI 7 are not considered mandatory parts of the standard.

5.3—Load factors and combinations
5.3.11 Required strength $U$ shall include internal load effects due to reactions induced by prestressing with a load factor of 1.0.

R5.3—Load factors and combinations
R5.3.11 For statically indeterminate structures, the internal load effects due to reactions induced by prestressing forces, sometimes referred to as secondary moments, can be significant (Bondy 2003; Lin and Thornton 1972; Collins and Mitchell 1997).

6.6—Linear elastic first-order analysis
6.6.5 Redistribution of moments in continuous flexural members
6.6.5.1 Except where approximate values for moments are used in accordance with 6.5, where moments have been calculated in accordance with 6.8, or where moments in two-way slabs are determined using pattern loading specified in 6.4.3.3, reduction of moments at sections of maximum negative or maximum positive moment calculated by elastic theory shall be permitted for any assumed loading arrangement if (a) and (b) are satisfied:

(a) Flexural members are continuous
(b) $\varepsilon \geq 0.0075$ at the section at which moment is reduced

6.6.5.2 For prestressed members, moments include those due to factored loads and those due to reactions induced by prestressing.

R6.6—Linear elastic first-order analysis
R6.6.5 Redistribution of moments in continuous flexural members
Redistribution of moments is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at sections of maximum positive or negative moment and cause a shift in the elastic moment diagram. The usual result is a reduction in the values of maximum negative moments in the support regions and an increase in the values of positive moments between supports from those calculated by elastic analysis. However, because negative moments are typically determined for one loading arrangement and positive moments for another (6.4.3 provides an exception for certain loading conditions), economies in reinforcement can sometimes be realized by reducing maximum elastic positive moments and increasing negative moments, thus narrowing the envelope of maximum negative and positive moments at any section in the span (Bondy 2003). Plastic hinges permit utilization of the

excluded. This section contains specific requirements for prestressed concrete systems. Other sections of this Code also provide specific requirements, such as required concrete cover for prestressed systems. Creep and shrinkage effects may be greater in prestressed than in non prestressed concrete structures because of the prestressing forces and because prestressed structures typically have less bonded reinforcement. Effects of movements due to creep and shrinkage may require more attention than is normally required for non prestressed concrete. These movements may increase prestress losses.

Design of externally post-tensioned construction should consider aspects of corrosion protection and fire resistance that are applicable to this structural system.
6.6.5.4 The reduced moment shall be used to calculate redistributed moments at all other sections within the spans such that static equilibrium is maintained after redistribution of moments for each loading arrangement.

The Code permissible redistribution is shown in Fig. R6.6.5. Using conservative values of limiting concrete strains and lengths of plastic hinges derived from extensive tests, flexural members with small rotation capacities were analyzed for redistribution of moments up to 20 percent, depending on the reinforcement ratio. As shown, the permissible redistribution percentages are conservative relative to the calculated percentages available for both $f_y = 60$ ksi and 80 ksi. Studies by Cohn (1965) and Mattock (1959) support this conclusion and indicate that cracking and deflection of beams designed for redistribution of moments are not significantly greater at service loads than for beams designed by the distribution of moments according to elastic theory. Also, these studies indicate that adequate rotational capacity for the redistribution of moments allowed by the Code is available if the members satisfy 6.6.5.1.

The provisions for redistribution of moments apply equally to prestressed members (Mast 1992).

The elastic deformations caused by a nonconcordant tendon change the amount of inelastic rotation required to obtain a given amount of redistribution of moments. Conversely, for a beam with a given inelastic rotational capacity, the amount by which the moment at the support may be varied is changed by an amount equal to the secondary moment at the support due to prestressing. Thus, the Code requires that secondary moments caused by reactions generated by prestressing forces be included in determining design moments.

Redistribution of moments as permitted by 6.6.5 is not appropriate where approximate values of bending moments are used, such as provided by the simplified method of 6.5.

Redistribution of moments is also not appropriate for two-way slab systems that are analyzed using the pattern loadings given in 6.4.3.3. These loadings use only 75 percent of the full factored live load, which is based on considerations of moment redistribution.
8.4—Required strength
8.4.1 General
8.4.1.3 For prestressed slabs, effects of reactions induced by prestressing shall be considered in accordance with 5.3.11.

3.9. Compression members – Combined flexure and axial loads
(Corresponds to ACI 318-11 Section 18.11)

22.4—Axial strength or combined flexural and axial strength
22.4.1 General
22.4.1.1 Nominal flexural and axial strength shall be calculated in accordance with the assumptions of 22.2.

10.6—Reinforcement limits
10.6.1 Minimum and maximum longitudinal reinforcement
10.6.1.1 For nonprestressed columns and for prestressed columns with average $f_{pe} < 225$ psi, area of longitudinal reinforcement shall be at least $0.01A_g$ but shall not exceed $0.08A_g$.

R10.6—Reinforcement limits
R10.6.1 Minimum and maximum longitudinal reinforcement
R10.6.1.1 Limits are provided for both the minimum and maximum longitudinal reinforcement ratios. Minimum reinforcement—Reinforcement is necessary to provide resistance to bending, which may exist regardless of analytical results, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Creep and shrinkage tend to transfer load from the concrete to the reinforcement, and the resultant increase in reinforcement stress becomes greater as the reinforcement ratio decreases. Therefore, a minimum limit is placed on the reinforcement ratio to prevent reinforcement from yielding under sustained service loads (Richart 1933).

Maximum reinforcement—The amount of longitudinal reinforcement is limited to ensure that concrete can be effectively consolidated around the bars and to ensure that columns designed according
10.7—Reinforcement detailing
10.7.3 Longitudinal reinforcement
10.7.3.1 For nonprestressed columns and for prestressed columns with average $f_{pe} < 225$ psi, the minimum number of longitudinal bars shall be (a), (b), or (c):

(a) Three within triangular ties
(b) Four within rectangular or circular ties
(c) Six enclosed by spirals or for columns of special moment frames enclosed by circular hoops

11.6—Reinforcement limits
11.6.1 If in-plane $V_0 \leq 0.5 \phi c \lambda \sqrt{f} A_r$, minimum $\rho_t$ and minimum $\rho_t$ shall be in accordance with Table 11.6.1. These limits need not be satisfied if adequate strength and stability can be demonstrated by structural analysis.

R10.7—Reinforcement detailing
R10.7.3 Longitudinal reinforcement
R10.7.3.1 At least four longitudinal bars are required when bars are enclosed by rectangular or circular ties. For other tie shapes, one bar should be provided at each apex or corner and proper transverse reinforcement provided. For example, tied triangular columns require at least three longitudinal bars, with one at each apex of the triangular ties. For bars enclosed by spirals, at least six bars are required. If the number of bars in a circular arrangement is less than eight, the orientation of the bars may significantly affect the moment strength of eccentrically loaded columns and should be considered in design.

R11.6—Reinforcement limits
R11.6.1 Both horizontal and vertical shear reinforcement are required for all walls. The distributed reinforcement is identified as being oriented parallel to either the longitudinal or transverse axis of the wall. Therefore, for vertical wall segments, the notation used to describe the horizontal distributed reinforcement ratio is $\rho_h$, and the notation used to describe the vertical distributed reinforcement ratio is $\rho_v$.

Transverse reinforcement is not required in precast, prestressed walls equal to or less than 12 ft in width because this width is less than that in which shrinkage and temperature stresses can build up to a magnitude requiring transverse reinforcement. In addition, much of the shrinkage occurs before the members are connected into the structure. Once in the final structure, the members are usually not as rigidly connected transversely as monolithic concrete; thus, the transverse restraint stresses due to both shrinkage and temperature change are significantly reduced.

The minimum area of wall reinforcement for precast walls has been used for many years and is recommended by the Precast/Prestressed Concrete Institute (PCI MNL-120) and the Canadian Precast Concrete Design Standard (2016). Reduced minimum reinforcement and greater spacings in 11.7.2.2 are allowed recognizing that precast wall panels have very little restraint at their edges during early stages of curing and develop less shrinkage to the Code are similar to the test specimens by which the Code was calibrated. The 0.08 limit applies at all sections, including splice regions, and can also be considered a practical maximum for longitudinal reinforcement in terms of economy and requirements for placing. Longitudinal reinforcement in columns should usually not exceed 4 percent if the column bars are required to be lap spliced, as the lap splice zone will have twice as much reinforcement if all lap splices occur at the same location.
stress than comparable cast-in-place walls.

Table 11.6.1—Minimum reinforcement for walls with in-plane $V_c \leq 0.5 \lambda_0 \sqrt{f_c} A_v$

<table>
<thead>
<tr>
<th>Wall type</th>
<th>Type of nonprestressed reinforcement</th>
<th>Bar/wire size</th>
<th>$f_y$, psi</th>
<th>Minimum longitudinal $\rho_l$</th>
<th>Minimum transverse, $\rho_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast-in-place</td>
<td>Deformed bars</td>
<td>$\leq$ No.5</td>
<td>$\geq 60,000$</td>
<td>0.0012</td>
<td>0.00020</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$&lt; 60,000$</td>
<td></td>
<td>0.0015</td>
<td>0.0025</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$&gt; No.5$</td>
<td>Any</td>
<td>0.0015</td>
<td>0.0025</td>
</tr>
<tr>
<td>Welded-wire reinforcement</td>
<td>$\leq$ W31 or D31</td>
<td>Any</td>
<td>Any</td>
<td>0.0012</td>
<td>0.0020</td>
</tr>
<tr>
<td>Precast$^2$</td>
<td>Deformed bars or welded-wire reinforcement</td>
<td>Any</td>
<td>Any</td>
<td>0.0010</td>
<td>0.0010</td>
</tr>
</tbody>
</table>

$^1$Prestressed walls with an average effective compressive stress of at least 225 psi need not meet the requirement for minimum longitudinal reinforcement $\rho_l$.

$^2$In one-way precast, prestressed walls not wider than 12 ft and not mechanically connected to cause restraint in the transverse direction, the minimum reinforcement requirement in the direction normal in the flexural reinforcement need not be satisfied.

10.7—Reinforcement detailing
10.7.6 Transverse reinforcement
10.7.6.1 General
10.7.6.1.3 For prestressed columns with average $f_{pe} \geq 225$ psi, transverse ties or hoops need not satisfy the $16d_b$ spacing requirement of 25.7.2.1.

25.7—Transverse reinforcement
25.7.2 Ties
25.7.2.1 Ties shall consist of a closed loop of deformed bar with spacing in accordance with (a) and (b):

(a) Clear spacing of at least $\left(\frac{4}{3}\right)d_{agg}$

(b) Center-to-center spacing shall not exceed the least of $16d_b$ of longitudinal bar, $48d_b$ of tie bar, and smallest dimension of member

25.7.2.2 Diameter of tie bar shall be at least (a) or (b):

(a) No. 3 enclosing No. 10 or smaller longitudinal bars

(b) No. 4 enclosing No. 11 or larger longitudinal bars or bundled longitudinal bars

25.7.2.2.1 As an alternative to deformed bars, deformed wire or welded wire reinforcement of equivalent area to that required in 25.7.2.1 shall be permitted subject to the requirements of Table 20.2.2.4(a).

10.7—Reinforcement detailing
10.7.6 Transverse reinforcement
10.7.6.2 Lateral support of longitudinal bars using ties or hoops
10.7.6.2.1 In any story, the bottom tie or hoop shall be located not more than one-half the tie or hoop spacing above the top of footing or slab.

10.7.6.2.2 In any story, the top tie or hoop shall be located not more than one-half the tie or hoop spacing below the lowest horizontal reinforcement in the slab, drop panel, or shear cap. If beams or

10.7.6.2.2 For rectangular columns, beams or brackets framing into all four sides at the same elevation are considered to provide restraint over a joint depth equal to that of the shallowest beam or
brackets frame into all sides of the column, the top tie or hoop shall be located not more than 3 in. below the lowest horizontal reinforcement in the shallowest beam or bracket.

bracket. For columns with other shapes, four beams framing into the column from two orthogonal directions are considered to provide equivalent restraint.

3.10. Slab systems (Corresponds to ACI 318-11 Section 18.12)

8.4—Required strength
8.4.1 General
8.4.1.2 Required strength shall be calculated in accordance with the analysis procedures given in Chapter 6.

7.5—Design strength
7.5.1 General
7.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$ including (a) and (b). Interaction between load effects shall be considered.

(a) $\phi M_n \geq M_u$
(b) $\phi V_n \geq V_u$

8.5—Design strength
8.5.1 General
8.5.1.1 For each applicable factored load combination, design strength shall satisfy $\phi S_n \geq U$, including (a) through (d). Interaction between load effects shall be considered.

(a) $\phi M_n \geq M_u$ at all sections along the span in each direction
(b) $\phi M_n \geq \gamma M_{oc}$ within $b_{slab}$ as defined in 8.4.2.2.3
(c) $\phi V_n \geq V_u$ at all sections along the span in each direction for one-way shear
(d) $\phi v_n \geq v_u$ at the critical sections defined in 8.4.4.1 for two-way shear

R8.4—Required strength
R8.4.1 General
R8.4.1.2 To determine service and factored moments as well as shears in prestressed slab systems, numerical analysis is required rather than simplified approaches such as the direct design method. The equivalent frame method of analysis as contained in the 2014 edition of the Code is a numerical method that has been shown by tests of large structural models to satisfactorily predict factored moments and shears in prestressed slab systems (Smith and Burns 1974; Burns and Hemakom 1977; Hawkins 1981; PTI DC20.8; Gerber and Burns 1971; Scordelis et al. 1959). The referenced research also shows that analysis using prismatic sections or other approximations of stiffness may provide erroneous and unsafe results. Moment redistribution for prestressed slabs is permitted in accordance with 6.6.5. PTI DC20.8 provides guidance for prestressed concrete slab systems.

R7.5—Design strength
R7.5.1 General
R7.5.1.1 Refer to R9.5.1.1.

R8.5—Design strength
R8.5.1 General
R8.5.1.1 Refer to R9.5.1.1.

R9.5—Design strength
R9.5.1 General
8.3—Design limits
8.3.2 Calculated deflection limits
8.3.2.1 Immediate and time-dependent deflections shall be calculated in accordance with 24.2 and shall not exceed the limits in 24.2.2 for two-way slabs given in (a) through (c):

(a) Nonprestressed slabs not satisfying 8.3.1
(b) Nonprestressed slabs without interior beams spanning between the supports on all sides and having a ratio of long-to-short span exceeding 2.0
(c) Prestressed slabs

7.2—General
7.2.1 The effects of concentrated loads, slab openings, and voids within the slab shall be considered in design.

8.2—General
8.2.2 The effects of concentrated loads, slab openings, and slab voids shall be considered in design.

8.2.3 Slabs prestressed with an average effective compressive stress less than 125 psi shall be designed as nonprestressed slabs.

8.6—Reinforcement limits
8.6.2 Minimum flexural reinforcement in prestressed slabs
8.6.2.1 For prestressed slabs, the effective prestress force $A_p f_{pc}$ shall provide a minimum average compressive stress of 125 psi on the slab section tributary to the tendon or tendon group. For slabs with varying cross section along the slab span, either parallel or perpendicular to the tendon or tendon group, the minimum average effective prestress of 125 psi is required at every cross section tributary to the tendon or tendon group along the span.

R9.5.1.1 The design conditions 9.5.1.1(a) through (d) list the typical forces and moments that need to be considered. However, the general condition $\phi S_\alpha \geq U$ indicates that all forces and moments that are relevant for a given structure need to be considered.

R8.3—Design limits
R8.3.2 Calculated deflection limits
R8.3.2.1 For prestressed flat slabs continuous over two or more spans in each direction, the span-thickness ratio generally should not exceed 42 for floors and 48 for roofs; these limits may be increased to 48 and 52, respectively, if calculations verify that both short- and long-term deflection, camber, and vibration frequency and amplitude are not objectionable. Short- and long-term deflection and camber should be calculated and checked against serviceability requirements of the structure.

R7.2—General
R7.2.1 Concentrated loads and slab openings create local moments and shears and may cause regions of one-way slabs to have two-way behavior. The influence of openings through the slab and voids within the slab (for example ducts) on flexural and shear strength as well as deflections is to be considered, including evaluating the potential for critical sections created by the openings and voids.

R8.2—General
R8.2.2 Refer to R7.2.1.

R8.6—Reinforcement limits
R8.6.2 Minimum flexural reinforcement in prestressed slabs
R8.6.2.1 The minimum average effective prestress of 125 psi was used in two-way test panels in the early 1970s to address punching shear concerns of lightly reinforced slabs. For this reason, the minimum effective prestress is required to be provided at every cross section. If the slab thickness varies along the span of a slab or perpendicular to the span of a slab, resulting in a varying slab cross section, the 125 psi minimum effective prestress and the maximum tendon spacing is required at every cross section tributary to the tendon or group of tendons along the span, considering both the thinner and the thicker slab sections. This may result in higher than the minimum $f_{pc}$ in thinner cross sections, and tendons spaced at less than the maximum in thicker cross sections along a span with varying thickness, due to the practical aspects of tendon placement in the field.
8.7—Reinforcement detailing
8.7.2 Flexural reinforcement spacing
8.7.2.3 For prestressed slabs with uniformly distributed loads, maximum spacing $s$ of tendons or groups of tendons in at least one direction shall be the lesser of $8h$ and 5 ft.

8.7.2.4 Concentrated loads and openings shall be considered in determining tendon spacing.

9.2—General
9.2.4 T-beam construction
9.2.4.3 For T-beam flanges where the primary flexural slab reinforcement is parallel to the longitudinal axis of the beam, reinforcement in the flange perpendicular to the longitudinal axis of the beam shall be in accordance with 7.5.2.3.

R8.7—Reinforcement detailing
R8.7.2 Flexural reinforcement spacing
R8.7.2.3 This section provides specific guidance concerning tendon distribution that will permit the use of banded tendon distributions in one direction. This method of tendon distribution has been shown to provide satisfactory performance by structural research (Burns and Hemakom 1977).

R9.2—General
R9.2.4 T-beam construction
R9.2.4.3 Refer to R7.5.2.3.

R7.5—Design strength
R7.5.2 Moment
R7.5.2.3 This provision applies only where a T-beam is parallel to the span of a one-way slab. For example, this beam might be used to support a wall or concentrated load that the slab alone cannot support. In that case, the primary slab reinforcement is parallel to the beam and the perpendicular reinforcement is usually sized for temperature and shrinkage. The reinforcement required by this provision is intended to consider “unintended” negative moments that may develop over the beam that exceed the requirements for temperature and shrinkage reinforcement alone.

R8.7—Reinforcement detailing
R8.7.5 Flexural reinforcement in prestressed slabs
R8.7.5.6 Structural integrity
R8.7.5.6.1 Prestressing tendons that pass through the slab-column joint at any location over the depth of the slab suspend the slab following a punching shear failure, provided the tendons are continuous through or anchored within the region bounded by the longitudinal reinforcement of the column and are prevented from bursting through the top surface of the slab (ACI 352.1R).

R8.7.5.6.2 Between column or shear cap faces, structural integrity tendons should pass below the orthogonal tendons from adjacent spans so that vertical movements of the integrity tendons are restrained by the orthogonal tendons. Where tendons...

8.7.5.6.2 Outside of the column and shear cap faces, the two structural integrity tendons required by 8.7.5.6.1 shall pass under any orthogonal tendons in adjacent spans.
8.7.5.6.3 Slabs with tendons not satisfying 8.7.5.6.1 shall be permitted if bonded bottom deformed reinforcement is provided in each direction in accordance with 8.7.5.6.3.1 through 8.7.5.6.3.3.

8.7.5.6.3.1 Minimum bottom deformed reinforcement $A_s$ in each direction shall be the larger of (a) and (b). The value of $f_y$ shall be limited to a maximum of 80,000 psi:

$$A_s = \frac{4.5f_{c}\sqrt{d}}{f_y} \quad (8.7.5.6.3.1a)$$

$$A_s = \frac{300c_s d}{f_y} \quad (8.7.5.6.3.1b)$$

where $c_s$ is measured at the column faces through which the reinforcement passes.

8.7.5.6.3.2 Bottom deformed reinforcement calculated in 8.7.5.6.3.1 shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

8.7.5.6.3.3 Bottom deformed reinforcement shall be anchored to develop $f_y$ beyond the column or shear cap face.

8.9—Lift-slab construction

8.9.1 In slabs constructed with lift-slab methods where it is impractical to pass the tendons required by 8.7.5.6.1 or the bottom bars required by 8.7.4.2 or 8.7.5.6.3 through the column, at least two post-tensioned tendons or two bonded bottom bars or wires in each direction shall pass through the lifting collar as close to the column as practicable, and be continuous or spliced using mechanical or welded splices in accordance with 25.5.7 or Class B tension lap splices in accordance with 25.5.2. At exterior columns, the reinforcement shall be anchored at the lifting collar.

R8.7.5.6.3 In some prestressed slabs, tendon layout constraints make it difficult to provide the structural integrity tendons required by 8.7.5.6.1. In such situations, the structural integrity tendons can be replaced by deformed bar bottom reinforcement (ACI 352.1R).
3.11. Temperature and shrinkage reinforcement

7.6—Reinforcement limits

7.6.4 Minimum shrinkage and temperature reinforcement

7.6.4.1 Reinforcement shall be provided to resist shrinkage and temperature stresses in accordance with 24.4.

7.6.4.2 If prestressed shrinkage and temperature reinforcement in accordance with 24.4.4 is used, 7.6.4.2.1 through 7.6.4.2.3 shall apply.

7.6.4.2.1 For monolithic, cast-in-place, post-tensioned beam-and-slab construction, gross concrete area shall consist of the total beam area including the slab thickness and the slab area within half the clear distance to adjacent beam webs. It shall be permitted to include the effective force in beam tendons in the calculation of total prestress force acting on gross concrete area.

7.6.4.2.2 If slabs are supported on walls or not cast monolithically with beams, gross concrete area is the slab section tributary to the tendon or tendon group.

7.6.4.2.3 At least one tendon is required in the slab between faces of adjacent beams or walls.

R7.6—Reinforcement limits

R7.6.4 Minimum shrinkage and temperature reinforcement

R7.6.4.2 In prestressed monolithic beam-and-slab construction, at least one shrinkage and temperature tendon is required between beams, even if the beam tendons alone provide at least 100 psi average compressive stress as required by 24.4.4.1 on the gross concrete area as defined in 7.6.4.2.1. A tendon of any size is permissible as long as all other requirements of 7.6.4.2 and 7.7.6.3 are satisfied. Application of the provisions of 7.6.4.2 and 7.7.6.3 to monolithic, cast-in-place, post-tensioned, beam-and-slab construction is illustrated in Fig. R7.6.4.2. Tendons used for shrinkage and temperature reinforcement should be positioned as close as practicable to the mid-depth of the slab. In cases where the shrinkage and temperature tendons are used for supporting the principal tendons, variations from the slab centroid are permissible; however, the resultant of the shrinkage and temperature tendons should not fall outside the middle third of the slab thickness. The effects of slab shortening should be evaluated to ensure the effectiveness of the prestressing. In most cases, the low level of prestressing recommended should not cause difficulties in a properly detailed structure. Additional attention may be required where thermal effects become significant.
7.7—Reinforcement detailing
7.7.6 Shrinkage and temperature reinforcement
7.7.6.1 Shrinkage and temperature reinforcement in accordance with 7.6.4 shall be placed perpendicular to flexural reinforcement.

7.7.6.2 Nonprestressed reinforcement
7.7.6.2.1 Spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of $5h$ and 18 in.

7.7.6.3 Prestressed reinforcement
7.7.6.3.1 Spacing of slab tendons required by 7.6.4.2 and the distance between face of beam or wall to the nearest slab tendon shall not exceed 6 ft.

7.7.6.3.2 If spacing of slab tendons exceeds 4.5 ft, additional deformed shrinkage and temperature reinforcement conforming to 24.4.3 shall be provided parallel to the tendons, except 24.4.3.4 need not be satisfied. In calculating the area of additional reinforcement, it shall be permitted to take the gross concrete area in 24.4.3.2 as the slab area between faces of beams. This shrinkage and temperature reinforcement shall extend from the slab edge for a distance not less than the slab tendon spacing.

R7.7—Reinforcement detailing
R7.7.6 Shrinkage and temperature reinforcement

R7.7.6.3 Prestressed reinforcement
R7.7.6.3.2 Widely spaced tendons result in non-uniform compressive stresses near the slab edges. The additional reinforcement is to reinforce regions near the slab edge that may be inadequately compressed. Placement of this reinforcement is illustrated in Fig. R7.7.6.3.2.
**24.4—Shrinkage and temperature reinforcement**

24.4.1 Reinforcement to resist shrinkage and temperature stresses shall be provided in one-way slabs in the direction perpendicular to the flexural reinforcement in accordance with 24.4.3 or 24.4.4.

24.4.2 If shrinkage and temperature movements are restrained, the effects of $T$ shall be considered in accordance with 5.3.6.

**R24.4—Shrinkage and temperature reinforcement**

R24.4.1 Shrinkage and temperature reinforcement is required at right angles to the principal reinforcement to minimize cracking and to tie the structure together to ensure it is acting as assumed in the design. The provisions of this section are intended for structural slabs only; they are not intended for slabs-on-ground.

R24.4.2 The area of shrinkage and temperature reinforcement required by 24.4.3.2 has been satisfactory where shrinkage and temperature movements are permitted to occur. Where structural walls or columns provide significant restraint to shrinkage and temperature movements, the restraint of volume changes causes tension in slabs, as well as displacements, shear forces, and flexural moments in columns or walls. In these cases, it may be necessary to increase the amount of slab reinforcement required by 24.4.3.2 due to the shrinkage and thermal effects in both principal directions (PCI MNL 120; Gilbert 1992). Top and bottom reinforcement are both effective in controlling cracks. Control strips during the construction period, which permit initial shrinkage to occur without causing an increase in stress, are also effective in reducing cracks caused by restraint.

Topping slabs also experience tension due to restraint of differential shrinkage between the topping and the precast elements or metal deck (which has zero shrinkage) that should be considered in reinforcing the slab. Consideration should be given to strain demands on reinforcement crossing joints of precast elements where most of the restraint...
24.4.3 Nonprestressed reinforcement
24.4.3.1 Deformed reinforcement to resist shrinkage and temperature stresses shall conform to Table 20.2.2.4(a) and shall be in accordance with 24.4.3.2 through 24.4.3.5.

24.4.3.2 The ratio of deformed shrinkage and temperature reinforcement area to gross concrete area shall be greater than or equal to 0.0018.

24.4.3.3 The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of \(5h\) and 18 in.

24.4.3.4 At all sections where required, deformed reinforcement used to resist shrinkage and temperature stresses shall develop \(f_y\) in tension.

24.4.3.5 For one-way precast slabs and one-way precast, prestressed wall panels, shrinkage and temperature reinforcement is not required in the direction perpendicular to the flexural reinforcement if (a) through (c) are satisfied.

(a) Precast members are not wider than 12 ft
(b) Precast members are not mechanically connected to cause restraint in the transverse direction
(c) Reinforcement is not required to resist transverse flexural Stresses

24.4.4 Prestressed reinforcement
24.4.4.1 Prestressed reinforcement to resist shrinkage and temperature stresses shall conform to Table 20.3.2.2, and the effective prestress after losses shall provide an average compressive stress of at least 100 psi.

R24.4.3 Nonprestressed reinforcement

R24.4.3.2 The minimum ratios of deformed bar or welded wire reinforcement area to gross concrete area of 0.0018 is empirical but has been used satisfactorily for many years. The resulting area of reinforcement may be distributed near the top or bottom of the slab, or may be distributed between the two faces of the slab as deemed appropriate for specific conditions. Previous editions of the Code permitted a reduction in shrinkage and temperature reinforcement for reinforcement with yield strength greater than 60,000 psi. However, the mechanics of cracking suggest that increased yield strength provides no benefit for the control of cracking. If crack width or leakage prevention is a design limit state, refer to ACI 224R or ACI 350 for recommended reinforcement ratios.

R24.4.3.4 Splices and end anchorages of shrinkage and temperature reinforcement are to be designed to develop the specified yield strength of the reinforcement in accordance with Chapter 25.

R24.4.3.5 For precast, prestressed concrete members not wider than 12 ft, such as hollow-core slabs, solid slabs, or slabs with closely spaced ribs, there is usually no need to provide reinforcement to withstand shrinkage and temperature stresses in the short direction. This is generally also true for precast, nonprestressed floor and roof slabs. The 12 ft width is less than that in which shrinkage and temperature stresses can build up to a magnitude requiring reinforcement. In addition, much of the shrinkage occurs before the members are tied into the structure. Once in the final structure, the members are usually not as rigidly connected transversely as monolithic concrete, thus, the transverse restraint stresses due to both shrinkage and temperature change are significantly reduced.

The waiver does not apply where reinforcement is required to resist flexural stresses, such as in thin flanges of precast single and double tees.

R24.4.4 Prestressed reinforcement
R24.4.4.1 Prestressed reinforcement requirements have been selected to provide an effective force on the slab approximately equal to the force required to yield nonprestressed shrinkage and temperature
psi on gross concrete area. This amount of prestressing—100 psi on the gross concrete area—has been used successfully on a large number of projects. The effects of slab shortening should be evaluated to ensure serviceable behavior of the structure. In most cases, the low level of prestressing recommended should not cause difficulties in a properly detailed structure. Additional attention may be required where thermal effects or restraint become significant.

### 3.12. Structural integrity reinforcement

#### 8.7—Reinforcement detailing

8.7.5 Flexural reinforcement in prestressed slabs

8.7.5.6 Structural integrity

8.7.5.6.1 Except as permitted in 8.7.5.6.3, at least two tendons with 1/2 in. diameter or larger strand shall be placed in each direction at columns in accordance with (a) or (b):

(a) Tendons shall pass through the region bounded by the longitudinal reinforcement of the column.  
(b) Tendons shall be anchored within the region bounded by the longitudinal reinforcement of the column, and the anchorage shall be located beyond the column centroid and away from the anchored span.

8.7.5.6.2 Outside of the column and shear cap faces, the two structural integrity tendons required by 8.7.5.6.1 shall pass under any orthogonal tendons in adjacent spans.

8.7.5.6.3 Slabs with tendons not satisfying 8.7.5.6.1 shall be permitted if bonded bottom deformed reinforcement is provided in each direction in accordance with 8.7.5.6.3.1 through 8.7.5.6.3.3.

\[
A_y = \frac{4.5 \sqrt{f_y c_y d}}{f_y} \quad (8.7.5.6.3.1a)
\]

\[
A_y = \frac{300c_y d}{f_y} \quad (8.7.5.6.3.1b)
\]

where \(c_2\) is measured at the column faces through

#### R8.7—Reinforcement detailing

R8.7.5 Flexural reinforcement in prestressed slabs

R8.7.5.6 Structural integrity

R8.7.5.6.1 Prestressing tendons that pass through the slab-column joint at any location over the depth of the slab suspend the slab following a punching shear failure, provided the tendons are continuous through or anchored within the region bounded by the longitudinal reinforcement of the column and are prevented from bursting through the top surface of the slab (ACI 352.1R).

R8.7.5.6.2 Between column or shear cap faces, structural integrity tendons should pass below the orthogonal tendons from adjacent spans so that vertical movements of the integrity tendons are restrained by the orthogonal tendons. Where tendons are distributed in one direction and banded in the orthogonal direction, this requirement can be satisfied by first placing the integrity tendons for the distributed tendon direction and then placing the banded tendons. Where tendons are distributed in both directions, weaving of tendons is necessary and use of 8.7.5.6.3 may be an easier approach.

R8.7.5.6.3 In some prestressed slabs, tendon layout constraints make it difficult to provide the structural integrity tendons required by 8.7.5.6.1. In such situations, the structural integrity tendons can be replaced by deformed bar bottom reinforcement (ACI 352.1R).
which the reinforcement passes.

8.7.5.6.3.2 Bottom deformed reinforcement calculated in 8.7.5.6.3.1 shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

8.7.5.6.3.3 Bottom deformed reinforcement shall be anchored to develop $f_y$ beyond the column or shear cap face.

### 3.13. Anchorage zones for post-tensioned tendons (Corresponds to ACI 318-11 Sections 18.13, 18.14, 18.15, 2.2, 9.2.7, and ACI 318-14 Sections 25.9.4.3.1 and 25.9.4.3.1)

**25.9—Anchorage zones for post-tensioned tendons**

**25.9.1 General**

**25.9.1.1** Anchorages regions of post-tensioned tendons shall consist of two zones, (a) and (b):

(a) The local zone shall be assumed to be a rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement

(b) The general zone includes the local zone and shall be assumed to be the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section

**R25.9—Anchorage zones for post-tensioned tendons**

**R25.9.1 General**

The detailed provisions in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFDUS) for analysis and reinforcement detailing of post-tensioned anchorage zones are considered to satisfy the more general requirements of this Code. In the specific areas of anchorage device evaluation and acceptance testing, this Code references the detailed AASHTO provisions.

**R25.9.1.1** Based on St. Venant’s principle, the extent of the anchorage zone may be estimated as approximately equal to the largest dimension of the cross section. Local zones and general zones are shown in Fig. R25.9.1.1a.

When anchorage devices located away from the end of the member are tensioned, large local tensile stresses are generated ahead of and behind the device. These tensile stresses are induced by incompatibility of deformations. The entire shaded region shown in Fig. R25.9.1.1b should be considered in the design of the general zone.
25.9.1.2 The local zone shall be designed in accordance with 25.9.3.

25.9.1.3 The general zone shall be designed in accordance with 25.9.4.

25.9.1.4 Compressive strength of concrete required at time of post-tensioning shall be specified as required by 26.10.

25.9.1.5 Stressing sequence shall be considered in the design process and specified as required by 26.10.

25.9.2 Required strength
25.9.2.1 Factored prestressing force at the anchorage device, $P_{pu}$, shall exceed the least of (a) through (c), where 1.2 is the load factor from 5.3.12:

(a) $1.2(0.94f_{py})A_{ps}$
(b) $1.2(0.80f_{pu})A_{ps}$
(c) Maximum jacking force designated by the supplier of anchorage devices multiplied by 1.2

R25.9.1.5 The sequence of anchorage device stressing can have a significant effect on the general zone stresses. Therefore, it is important to consider not only the final stage of a stressing sequence with all tendons stressed, but also intermediate stages during construction. The most critical bursting forces caused by each of the sequentially posttensioned tendon combinations, as well as that of the entire group of tendons, should be taken into account.
25.9.3 Local zone
25.9.3.1 The design of local zone in post-tensioned anchorages shall meet the requirements of (a), (b), or (c):

(a) Monostrand or single 5/8 in. or smaller diameter bar anchorage devices shall meet the bearing resistance and local zone requirements of ACI 423.7
(b) Basic multistrand anchorage devices shall meet the bearing resistance requirements of AASHTO LRFD Bridge Design Specifications, Article 5.8.4.4.2, except that the load factors shall be in accordance with 5.3.12 and \( \phi \) shall be in accordance with 21.2.1
(c) Special anchorage devices shall satisfy the tests required in AASHTO LRFD Bridge Design Specifications, Article 5.8.4.4.3, and described in AASHTO LRFD Bridge Construction Specifications, Article 10.3.2.3

25.9.3.2 Where special anchorage devices are used, supplementary skin reinforcement shall be provided in addition to the confining reinforcement specified for the anchorage device.

25.9.3.2.1 Supplementary skin reinforcement shall be similar in configuration and at least equivalent in volumetric ratio to any supplementary skin reinforcement used in the qualifying acceptance tests of the anchorage device.

R25.9.2 Required strength
R25.9.2.1 The factored prestressing force is the product of the load factor and the maximum prestressing force permitted. The maximum permissible tensile stresses during jacking are defined in 20.3.2.5.1.

R25.9.3 Local zone
The local zone resists very high local stresses introduced by the anchorage device and transfers them to the remainder of the anchorage zone. The behavior of the local zone is strongly influenced by the specific characteristics of the anchorage device and its confining reinforcement, and is less influenced by the geometry and loading of the overall structure. Local-zone design sometimes cannot be completed until specific anchorage devices are selected. If special anchorage devices are used, the anchorage device supplier should furnish test information to demonstrate that the device is satisfactory under Article 10.3.2.3 of the AASHTO LRFD Bridge Construction Specifications (LRFDCONS) and provide information regarding necessary conditions for use of the device. The main considerations in local-zone design are the effects of high bearing pressure and the adequacy of any confining reinforcement provided to increase concrete bearing resistance.

25.9.4 General zone
R25.9.3.2.1 Skin reinforcement is placed near the outer faces in the anchorage zone to limit local crack width and spacing. Reinforcement in the general zone for other actions (such as shrinkage and temperature) may be used in satisfying the supplementary skin reinforcement requirement. Determination of the supplementary skin reinforcement depends on the anchorage device hardware used and frequently cannot be determined until the specific anchorage devices are selected.

R25.9.4 General zone
Within the general zone, the assumption that plane sections remain plane is not valid. Tensile stresses that can be caused by the tendon anchorage device, including bursting, spalling, and edge tension, as shown in Fig. R25.9.4, should be considered in design. In addition, the compressive stresses immediately ahead of the local zone should be checked (Fig. R25.9.1.1b).
25.9.4.1 The extent of the general zone is equal to the largest dimension of the cross section. In the case of slabs with anchorages or groups of anchorages spaced along the slab edge, the depth of the general zone shall be taken as the spacing of the tendons.

25.9.4.2 For anchorage devices located away from the end of a member, the general zone shall include the disturbed regions ahead of and behind the anchorage devices.

25.9.4.3 Analysis of general zones
25.9.4.3.1 Methods (a) through (c) shall be permitted for design of general zones:

(a) The strut-and-tie method in accordance with Chapter 23
(b) Linear stress analysis, including finite element analysis or equivalent
(c) Simplified equations in AASHTO LRFD Bridge Design Specifications, Article 5.8.4.5, except where restricted by 25.9.4.3.2

The design of general zones by other methods shall be permitted, provided that the specific procedures used for design result in prediction of strength in substantial agreement with results of comprehensive tests.

R25.9.4.1 The depth of the general zone in slabs is defined in AASHTO LRFD Bridge Design Specifications (LRFDUS), Article 5.9.5.6 as the spacing of the tendons (Fig. R25.9.4.1). Refer to 25.9.4.6 for monostrand anchorages.

R25.9.4.2 The dimensions of the general zone for anchorage devices located away from the end of the member are defined in Fig. R25.9.1.1b.

R25.9.4.3 Analysis of general zones
R25.9.4.3.1 The design methods include those procedures for which guidelines have been given in AASHTO LRFDUS and Breen et al. (1994). These procedures have been shown to be conservative predictors of strength compared to test results (Breen et al. 1994). The use of the strut-and-tie method is especially helpful for general zone design (Breen et al. 1994). In many anchorage applications, where
25.9.4.3.2 Simplified equations as permitted by 25.9.4.3.1(c) shall not be used for the design of a general zone if any of the situations listed in (a) through (g) occur:

(a) Member cross sections are nonrectangular
(b) Discontinuities in or near the general zone cause deviations in the force flow path
(c) Minimum edge distance is less than 1.5 times the anchorage device lateral dimension in that direction
(d) Multiple anchorage devices are used in other than one closely spaced group
(e) Centroid of the tendons is located outside the kern
(f) Angle of inclination of the tendon in the general zone is less than –5 degrees from the centerline of axis of the member, where the angle is negative if the anchor force points away from the centroid of the section
(g) Angle of inclination of the tendon in the general zone is greater than +20 degrees from the centerline of axis of the member, where the angle is positive if the anchor force points towards the centroid of the section

Values for the magnitude of the bursting force, \( T_{\text{burst}} \), and for its centroidal distance from the major bearing surface of the anchorage, \( d_{\text{burst}} \), may be estimated from Eq. (R25.9.4.3.1a) and (R25.9.4.3.1b), respectively. The terms used in these equations are shown in Fig. R25.9.4.3.1 for a prestressing force with a small eccentricity. In the application of these equations, the specified stressing sequence should be considered if more than one tendon is present.

\[
T_{\text{burst}} = 0.25 \sum P_{\text{pu}} \left( 1 - \frac{h_{\text{anc}}}{h} \right) \quad \text{(R25.9.4.3.1a)}
\]

\[
d_{\text{burst}} = 0.5 \left( h - 2e_{\text{anc}} \right) \quad \text{(R25.9.4.3.1b)}
\]

where \( \Sigma P_{\text{pu}} \) is the sum of the \( P_{\text{pu}} \) forces from the individual tendons; \( h_{\text{anc}} \) is the depth of the anchorage device or single group of closely spaced devices in the direction considered; and \( e_{\text{anc}} \) is the eccentricity (always taken as positive) of the anchorage device or group of closely spaced devices with respect to the centroid of the cross section (Fig. R25.9.4.3.1).

Anchorage devices should be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered.

25.9.4.3.2 The simplified equations in the AASHTO LRFDUS are not applicable in several common situations listed in 25.9.4.3.2. In these cases, a detailed analysis is required. In addition, in the post-tensioning of thin sections, flanged sections, or irregular sections, or where the tendons have appreciable curvature within the general zone, more general procedures such as those of AASHTO LRFDUS Articles 5.8.2.7 and 5.8.3 are required. Detailed recommendations for design principles that apply to all design methods are given in Article 5.9.5.6.5b of the AASHTO LRFDUS.

Groups of monostrand tendons with individual

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Fig. R25.9.4.3.1-Definition of terms used to define the general zone.
25.9.4.3 Three-dimensional effects shall be considered in design and analyzed by (a) or (b):

(a) Three-dimensional analysis procedures
(b) Approximated by considering the summation of effects for two orthogonal planes

25.9.4.4 Reinforcement limits
25.9.4.4.1 Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

25.9.4.4.2 Reinforcement shall be provided in the general zone to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices, as applicable. Effects of abrupt changes in section and stressing sequence shall be considered.

25.9.4.3 For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $0.35P_p$ into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchor.

25.9.4.3.3 The provision for three-dimensional effects is to ensure that the effects perpendicular to the main plane of the member, such as bursting forces in the thin direction of webs or slabs are considered. In many cases, these effects can be determined independently for each direction, but some applications require a full three-dimensional analysis (for example, diaphragms for the anchorage of external tendons).

R25.9.4.4 Reinforcement limits

R25.9.4.4.2 In some cases, reinforcement requirements cannot be determined until specific tendon and anchorage device layouts are selected. Design and approval responsibilities should be clearly assigned in the construction documents. Abrupt changes in section can cause substantial deviation in force paths. These deviations can greatly increase tensile forces, as shown in Fig. R25.9.4.4.2.
the anchorage device and shall be fully developed both behind and ahead of the anchorage device.

**25.9.4.4** If tendons are curved in the general zone, bonded reinforcement shall be provided to resist radial and splitting forces, except for monostrand tendons in slabs or where analysis shows reinforcement is not required.

**25.9.4.5** Reinforcement with a nominal tensile strength equal to 2 percent of the factored prestressing force shall be provided in orthogonal directions parallel to the loaded face of the anchorage zone to limit spalling, except for monostrand tendons in slabs or where analysis shows reinforcement is not required.

**25.9.4.6** For monostrand anchorage devices for 1/2 in. or smaller diameter strands in normalweight concrete slabs, reinforcement satisfying (a) and (b) shall be provided in the anchorage zone, unless a detailed analysis in accordance with 25.9.4.3 shows that this reinforcement is not required:

(a) Two horizontal bars at least No. 4 in size shall be provided within the local zone parallel to the slab edge ahead of the bearing face of the anchorage device. They shall be permitted to be in contact with the bearing face of the anchorage device, the center of the bars shall be no farther than 4 in. ahead of the bearing face of the device, and the bars shall extend at least 6 in. either side of the outer edges of the device.

(b) If the center-to-center spacing of anchorage devices is 12 in. or less, the anchorage devices shall be considered as a group. For each group of six or more anchorage devices, at least \( n + 1 \) hairpin bars or closed stirrups at least No. 3 in size shall be provided, where \( n \) is the number of anchorage devices. One hairpin bar or stirrup shall be placed between adjacent anchorage devices and one on each side of the group. The hairpin bars or stirrups shall be placed with the horizontal legs extending into the slab perpendicular to the edge. The center line of the vertical leg of the hairpin bars, or the vertical leg of stirrups closest to the anchorage device, shall be placed \( 3h/8 \) to \( h/2 \) ahead of the bearing face of the anchorage device. Hairpin bars or stirrups shall be detailed in accordance with 25.7.1.1 and 25.7.1.2.

**R25.9.4.4.3** Where anchorages are located away from the end of a member, local tensile stresses are generated behind these anchorages (Fig. R25.9.1.1b) due to compatibility of deformations ahead of and behind the anchorages. Bonded tie-back reinforcement parallel to the tendon is required in the immediate vicinity of the anchorage to limit the extent of cracking behind the anchorage. The requirement of \( 0.35P_{pu} \) was derived using 25 percent of the unfactored prestressing force being resisted by reinforcement at \( 0.6f_y \) considering a load factor of 1.2. Therefore, the full yield strength of the reinforcement, \( f_y \), should be used in calculating the provided capacity.

**R25.9.4.4.5** The spalling force for tendons for which the centroid lies within the kern of the section may be estimated as 2 percent of the total factored prestressing force, except for multiple anchorage devices with center-to-center spacing greater than 0.4 times the depth of the section.

**R25.9.4.4.6** For monostrand slab tendons, the anchoragezone minimum reinforcement
requirements are based on the recommendations of Breen et al. (1994) and confirmed based on analysis of other test results by Roberts-Wollmann and Wollmann (2008). Typical details are shown in Fig. R25.9.4.4.6. For slabs not thicker than 8 in., with groups of anchors requiring hairpins, the bars parallel to the loaded face can satisfy 25.9.4.4.6(a) and also provide anchorage for the hairpin bars. Thicker slabs require two bars for 25.9.4.4.6(a) and two additional bars to provide anchorage for the hairpins in accordance with 25.7.1.2. The horizontal bars parallel to the edge required by 25.9.4.4.6(a) should be continuous where possible.

The tests on which the recommendations of Breen et al. (1994) were based were limited to anchorage devices for 1/2 in. diameter, Grade 270 strand, and unbonded tendons in normalweight concrete. For larger strand anchorage devices or for use in lightweight concrete slabs, ACI Committee 423 recommends that the amount and spacing of reinforcement should be conservatively adjusted to provide for the larger anchorage force and smaller splitting tensile strength of lightweight concrete (ACI 423.3R).

ACI 423.3R and Breen et al. (1994) both recommend that hairpin bars also be furnished for anchorages located within 12 in. of slab corners to resist edge tension forces. The meaning of “ahead of” in 25.9.4.4.6 is illustrated in Fig. R25.9.1.1b.

In those cases where multistrand anchorage devices are used for slab tendons, all provisions of 25.9.4 are to be satisfied.
Code | Commentary
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For slabs with \( h > 8 \text{ in.} \), provide #4 or larger straight bars parallel to slab edge, in contact with or not farther than 4 in. ahead of bearing face of anchorage device.

③ or larger hairpins required if \( s \leq 12 \text{ in.} \)

≥ 6 in. extension

Anchorage spacing, \( s \)

Edge of slab

⑩ Tendon (typ.)

(a) Plan view

Bars to anchor hairpins in accordance with 25.7.1.2

Bars to anchor hairpins

3h\(^n\) to h/2

h > 8 in.

⑩ (top + bottom cover)

④ or larger straight bars parallel to slab edge, in contact with or not farther than 4 in. ahead of bearing face of anchorage device

(b) Section A-A for slabs with \( h > 8 \text{ in.} \)

3h\(^n\) to h/2

h - (top + bottom cover)

④ or larger straight bars parallel to slab edge, in contact with or not farther than 4 in. ahead of bearing face of anchorage device

(c) Section A-A for slabs with \( h \leq 8 \text{ in.} \)

### 25.9.4.5 Limiting stresses in general zones

##### 25.9.4.5.1 Maximum design tensile stress

Maximum design tensile stress in reinforcement at nominal strength shall not exceed the limits in Table 25.9.4.5.1.

<table>
<thead>
<tr>
<th>Type of reinforcement</th>
<th>Maximum design tensile stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonprestressed reinforcement</td>
<td>( f_y )</td>
</tr>
<tr>
<td>Bonded, prestressed reinforcement</td>
<td>( f_{yf} )</td>
</tr>
<tr>
<td>Unbonded, prestressed reinforcement</td>
<td>( f_{yc} + 10000 )</td>
</tr>
</tbody>
</table>

### 25.9.4.5.2 Compressive stress

Compressive stress in concrete at nominal strength shall not exceed \( 0.7 \lambda f_{ci}' \), where \( \lambda \) is defined in 19.2.4.

### 25.9.4.5.3 If concrete is confined by spirals or hoops and the effect of confining reinforcement is documented by tests and analysis, it shall be permitted to use an increased value of compressive stress in concrete when calculating the nominal compressive stress introduced by auxiliary prestressing applied perpendicular to the.
strength of the general zone.

25.9.4.5.4 Prestressing reinforcement shall not be stressed until compressive strength of concrete, as indicated by tests of cylinders cured in a manner consistent with curing of the member, is at least 2500 psi for single-strand or bar tendons or at least 4000 psi for multistrand tendons unless 25.9.4.5.5 is satisfied.

25.9.4.5.5 Provisions of 25.9.4.5.4 need not be satisfied if (a) or (b) is satisfied:

(a) Oversized anchorage devices are used to compensate for a lower concrete compressive strength
(b) Prestressing reinforcement is stressed to no more than 50 percent of the final prestressing force

25.9.5 Reinforcement detailing
25.9.5.1 Selection of reinforcement size, spacing, cover, and other details for anchorage zones shall make allowances for tolerances on fabrication and placement of reinforcement; for the size of aggregate; and for adequate placement and consolidation of the concrete.

26.4—Concrete materials and mixture requirements
26.4.2 Concrete mixture requirements
26.4.2.2 Compliance requirements:
(a) The required compressive strength at designated stages of construction for each part of the structure not designed by the licensed design professional shall be submitted for review.

26.10—Additional requirements for prestressed concrete
26.10.1 Design information:
(b) Stressing sequence of tendons.

26.10.2 Compliance requirements:

(j) Prestressing reinforcement in post-tensioned construction shall not be stressed until the concrete compressive strength is at least 2500 psi for single-strand or bar tendons, 4000 psi for multistrand tendons, or a higher strength, if required. An exception to these strength requirements is provided in 26.10.2(k).
(k) Lower concrete compressive strength than

axis of the main tendons can be effective in increasing anchorage zone strength.

R25.9.4.5.4 To limit early shrinkage cracking, monostrand tendons are sometimes stressed at concrete strengths less than 2500 psi. In such cases, either oversized monostrand anchorages are used, or the strands are stressed in stages, often to levels one-third to one-half of the final prestressing force as permitted by 25.9.4.5.5.

R26.10—Additional requirements for prestressed concrete
R26.10.1(b) The sequence of anchorage device stressing can have a significant effect on general zone stresses. Therefore, it is important to consider not only the final stage of a stressing sequence with all tendons stressed, but also intermediate stages during construction. The most critical bursting forces caused by each of the sequentially posttensioned tendon combinations, as well as that of the entire group of tendons, should be taken into account.
required by 26.10.2(j) shall be permitted if (1) or (2) is satisfied:
(1) Oversized anchorage devices are used to compensate for a lower concrete compressive strength.
(2) Prestressing reinforcement is stressed to no more than 50 percent of the final prestressing force.

R26.10.2(k) To limit early shrinkage cracking, monostrand tendons are sometimes stressed at concrete strengths less than 2500 psi. In such cases, either oversized monostrand anchorages are used, or the strands are stressed in stages, often to levels one-third to one-half the final prestressing force.

26.10—Development of reinforcement
26.10.2 R26.10.2 To limit early shrinkage cracking, monostrand tendons are sometimes stressed at concrete strengths less than 2500 psi. In such cases, either oversized monostrand anchorages are used, or the strands are stressed in stages, often to levels one-third to one-half the final prestressing force.

Development of reinforcement
25.4 Reduction of development length for excess reinforcement
25.4.10 R25.4.10 Reduction of development length for excess reinforcement
25.4.10.2 R25.4.10.2 The excess reinforcement factor \( \frac{A_{\text{required}}}{A_{\text{provided}}} \), applicable to straight reinforcement, is not applicable for hooked or headed deformed reinforcement.

Prestressing, as used in the Code, may apply to pretensioning, bonded post-tensioning, or unbonded posttensioning. All requirements in the Code apply to prestressed systems and members, unless specifically excluded. This section contains specific requirements.
for prestressed concrete systems. Other sections of this Code also provide specific requirements, such as required concrete cover for prestressed systems. Creep and shrinkage effects may be greater in prestressed than in nonprestressed concrete structures because of the prestressing forces and because prestressed structures typically have less bonded reinforcement. Effects of movements due to creep and shrinkage may require more attention than is normally required for nonprestressed concrete. These movements may increase prestress losses.

Design of externally post-tensioned construction should consider aspects of corrosion protection and fire resistance that are applicable to this structural system.

7.5—Design strength
7.5.2 Moment
7.5.2.2 For prestressed slabs, external tendons shall be considered as unbonded tendons in calculating flexural strength, unless the external tendons are effectively bonded to the concrete section along the entire length.

8.5—Design strength
8.5.2 Moment
8.5.2.3 In calculating $M_n$ for prestressed slabs, external tendons shall be considered as unbonded unless the external tendons are effectively bonded to the slab along its entire length.

9.5—Design strength
9.5.2 Moment
9.5.2.3 For prestressed beams, external tendons shall be considered as unbonded tendons in calculating flexural strength, unless the external tendons are effectively bonded to the concrete along the entire length.

7.7—Reinforcement detailing
7.7.4 Flexural reinforcement in prestressed slabs
7.7.4.1 External tendons shall be attached to the member in a manner that maintains the specified eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.

8.7—Reinforcement detailing
8.7.5 Flexural reinforcement in prestressed slabs
8.7.5.1 External tendons shall be attached to the slab in a manner that maintains the specified eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.

9.7—Reinforcement detailing
9.7.4 Flexural reinforcement in prestressed beams
9.7.4.1 External tendons shall be attached to the member in a manner that maintains the specified eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.
eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.

20.5—Provisions for durability of steel reinforcement
20.5.6 Corrosion protection for external post-tensioning
20.5.6.1 External tendons and tendon anchorage regions shall be protected to provide resistance to corrosion.

R20.5—Provisions for durability of steel Reinforcement
R20.5.6 Corrosion protection for external post-tensioning
R20.5.6.1 Corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable to the environment in which the tendons are located. Some conditions will require that the prestressing reinforcement be protected by concrete cover or by cement grout in polyethylene or metal tubing; other conditions will permit the protection provided by coatings such as paint or grease. Corrosion protection methods should meet the fire protection requirements of the general building code, unless the installation of external post-tensioning is to only improve serviceability.

26.10—Additional requirements for prestressed concrete
26.10.1 Design information:
(e) Materials and details of corrosion protection for tendons, couplers, end fittings, post-tensioning anchorages, and anchorage regions.

R26.10—Additional requirements for prestressed concrete
R26.10.1(e) For recommendations regarding protection, refer to Sections 4.2 and 4.3 of ACI 423.3R, and Sections 3.4, 3.6, 5, 6, and 8.3 of ACI 423.7. Also refer to 20.5.1.4.2 for corrosion protection requirements. Corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable for the environment in which the tendons are located. Some conditions will require that the prestressed reinforcement be protected by concrete cover or by cement grout in metal or plastic duct; other conditions will permit the protection provided by coatings such as paint or grease. Corrosion protection methods should meet the fire protection requirements of the general building code unless the installation of external post-tensioning is to only improve serviceability.
PART IV. CONSTRUCTION AND DURABILITY

4.1. Corrosion protection for unbonded tendons (Corresponds to ACI 318-11 Section 18.16)

20.5—Provisions for durability of steel reinforcement
20.5.3 Corrosion protection for unbonded prestressing reinforcement
20.5.3.1 Unbonded prestressing reinforcement shall be encased in sheathing, and the space between the prestressing reinforcement and the sheathing shall be completely filled with a material formulated to inhibit corrosion. Sheathing shall be watertight and continuous over the unbonded length.

20.5.3.2 The sheathing shall be connected to all stressing, intermediate, and fixed anchorages in a watertight fashion.

20.5.3.3 Unbonded single-strand tendons shall be protected to provide resistance to corrosion in accordance with ACI 423.7.

R20.5—Provisions for durability of steel reinforcement
R20.5.3 Corrosion protection for unbonded prestressing reinforcement
R20.5.3.1 Material for corrosion protection of unbonded prestressing reinforcement should have the properties identified in 19.1 of Breen et al. (1994). Typically, sheathing is a continuous, seamless, high-density polyethylene material that is extruded directly onto the coated prestressing reinforcement.

4.2. Post-tensioning ducts (Corresponds to ACI 318-11 Section 18.17)

26.10—Additional requirements for prestressed concrete
26.10.1 Design information:
(f) Requirements for ducts for bonded tendons.

R26.10—Additional requirements for prestressed concrete
R26.10.1(f) Guidance for specifying duct requirements for bonded tendons is provided in PTI M50.3 and PTI M55.1.

20.5—Provisions for durability of steel reinforcement
20.5.4 Corrosion protection for grouted tendons
20.5.4.1 Ducts for grouted tendons shall be grout-tight and nonreactive with concrete, prestressing reinforcement, grout, and corrosion inhibitor admixtures.

20.5.4.2 Ducts shall be maintained free of water.

R20.5.4.2 Water in ducts may cause corrosion of the prestressing reinforcement, may lead to bleeding and segregation of grout, and may cause distress to the surrounding concrete if subjected to freezing conditions. A corrosion inhibitor should be used to provide temporary corrosion protection if prestressing reinforcement is exposed in the ducts for prolonged periods of time before grouting (ACI 423.7).

20.5.4.3 Ducts for grouted single-wire, single-strand, or single-bar tendons shall have an inside diameter at
least 1/4 in. larger than the diameter of the prestressing reinforcement.

20.5.4.4 Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area at least two times the cross-sectional area of the prestressing reinforcement.

4.3. Grout for bonded tendons (Corresponds to ACI 318-11 Section 18.18)

26.10—Additional requirements for prestressed concrete
26.10.1 Design information:
(g) Requirements for grouting of bonded tendons, including maximum water-soluble chloride ion (Cl–) content requirements in 19.4.1.

R26.10—Additional requirements for prestressed concrete
R26.10.1(g) Guidance for specifying grouting requirements for bonded tendons is provided in PTI M55.1.

4.4. Protection for prestressing steel (Corresponds to ACI 318-11 Section 18.19)

26.10—Additional requirements for prestressed concrete
26.10.2 Compliance requirements:
(d) Burning or welding operations in the vicinity of prestressing reinforcement shall be performed in such a manner that prestressing reinforcement is not subject to welding sparks, ground currents, or temperatures that degrade the properties of the reinforcement.

4.5. Application and measurement of prestressing steel (Corresponds to ACI 318-11 Section 18.20)

26.10—Additional requirements for prestressed concrete
26.10.2 Compliance requirements:
(e) Prestressing force and friction losses shall be verified by (1) and (2).

(1) Measured elongation of prestressed reinforcement compared with elongation calculated using the modulus of elasticity

R26.10—Additional requirements for prestressed concrete
R26.10.2(e) Elongation measurements for prestressing should be in accordance with the procedures outlined in the Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (MNL 117), published by the Precast/Prestressed Concrete Institute.
determined from tests or as reported by the manufacturer.

(2) Jacking force measured using calibrated equipment such as a hydraulic pressure gauge, load cell, or dynamometer.

(f) The cause of any difference in force determination between (1) and (2) of 26.10.2(e) that exceeds 5 percent for pretensioned construction or 7 percent for posttensioned construction shall be ascertained and corrected, unless approved by the licensed design professional.

(g) Loss of prestress force due to unreplaced broken prestressed reinforcement shall not exceed 2 percent of the total prestress force in prestressed concrete members, unless approved by the licensed design professional.

(h) If the transfer of force from the anchorages of the pretensioning bed to the concrete is accomplished by flame cutting prestressed reinforcement, the cutting locations and cutting sequence shall be selected to avoid undesired temporary stresses in pretensioned members.

(i) Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to the concrete.

4.6. Post-tensioning anchorages and couplers (Corresponds to ACI 318-11 Section 18.21)

25.8—Post-tensioning anchorages and couplers
25.8.1 Anchors and couplers for tendons shall develop at least 95 percent of $f_{pu}$ when tested in an unbonded condition, without exceeding anticipated set.

R25.8—Post-tensioning anchorages and couplers
R25.8.1 The required strength of the tendon-anchorage or tendon-coupler assemblies for both unbonded and bonded tendons, when tested in an unbonded state, is based on 95 percent of the
25.8.2 Anchorages and couplers for bonded tendons shall be located so that 100 percent of $f_{pu}$ shall be developed at critical sections after the post-tensioned reinforcement is bonded in the member.

R25.8.2 Anchorages and couplers for bonded tendons that develop less than 100 percent of the specified tensile strength of the prestressing reinforcement should be used only where the bond transfer length between the anchorage or coupler and critical sections equals or exceeds that required to develop the prestressed reinforcement strength. This bond length may be calculated based on the results of tests of bond characteristics of non-tensioned prestressing strand (Salmons and McCrate 1977; PCA 1980), or bond tests on other prestressing reinforcement, as appropriate.

25.8.3 In unbonded construction subject to repetitive loads, the possibility of fatigue of prestressed reinforcement in anchorages and couplers shall be considered.

R25.8.3 A discussion on fatigue loading is provided in ACI 215R. Detailed recommendations on tests for static and cyclic loading conditions for tendons and anchorage fittings of unbonded tendons are provided in ACI 423.3R (Section 4.1.3) and ACI 301 (Section 15.2.2).

25.8.4 Couplers shall be placed at locations approved by the licensed design professional and enclosed in housings long enough to permit necessary movements.

20.5—Provisions for durability of steel Reinforcement

R20.5—Provisions for durability of steel Reinforcement

20.5.5 Corrosion protection for post-tensioning anchorages, couplers, and end fittings

R20.5.5 Corrosion protection for post-tensioning anchorages, couplers, and end fittings

20.5.5.1 Anchorages, couplers, and end fittings shall be protected to provide long-term resistance to corrosion.

R20.5.5.1 For recommendations regarding protection, refer to 4.2 and 4.3 of Mojtabedi and Gamble (1978) and 3.4, 3.6, 5, 6, and 6.3 of Breen et al. (1994).

26.10—Additional requirements for prestressed concrete

R26.10—Additional requirements for prestressed concrete

26.10.1 Design information:

R26.10.1(e) For recommendations regarding protection, refer to Sections 4.2 and 4.3 of ACI
tendons, couplers, end fittings, post-tensioning anchorages, and anchorage regions.

26.10.2 Compliance requirements:
(c) Couplers shall be placed in areas approved by the licensed design professional and enclosed in housings long enough to permit necessary movements.

423.3R, and Sections 3.4, 3.6, 5, 6, and 8.3 of ACI 423.7. Also refer to 20.5.1.4.2 for corrosion protection requirements.

Corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable for the environment in which the tendons are located. Some conditions will require that the prestressed reinforcement be protected by concrete cover or by cement grout in metal or plastic duct; other conditions will permit the protection provided by coatings such as paint or grease. Corrosion protection methods should meet the fire protection requirements of the general building code unless the installation of external post-tensioning is to only improve serviceability.
5.1. One-way shear strength

22.5—One-way shear strength

22.5.1 General

22.5.1.1 Nominal one-way shear strength at a section, \( V_n \), shall be calculated by:

\[
V_n = V_c + V_s
\]  

(22.5.1.1)

22.5.1.2 Cross-sectional dimensions shall be selected to satisfy Eq. (22.5.1.2).

\[
V_s \leq \phi \left( V_c + 8\sqrt{f'_c} b_w d \right)
\]  

(22.5.1.2)

22.5.1.3 For nonprestressed members, \( V_c \) shall be calculated in accordance with 22.5.5.

22.5.1.4 For prestressed members, \( V_c, V_{ci}, \) and \( V_{cw} \) shall be calculated in accordance with 22.5.6 or 22.5.7.

22.5.1.5 For calculation of \( V_c, V_{ci}, \) and \( V_{cw}, \lambda \) shall be in accordance with 19.2.4.

22.5.1.6 \( V_s \) shall be calculated in accordance with 22.5.8.

22.5.1.7 Effect of any openings in members shall be considered in calculating \( V_n \).

42.5—One-way shear strength

R22.5.1 General

R22.5.1.1 In a member without shear reinforcement, shear is assumed to be resisted by the concrete. In a member with shear reinforcement, a portion of the shear strength is assumed to be provided by the concrete and the remainder by the shear reinforcement.

The one-way shear equations for nonprestressed concrete were changed in the 2019 Code with the primary objectives of including effect of member depth, commonly referred to as the “size effect,” and the effects of the longitudinal reinforcement ratio on shear strength.

The shear strength provided by concrete, \( V_c \), is taken as the shear causing inclined cracking (Joint ACI-ASCE Committee 426 1973; MacGregor and Hanson 1969; Joint ACI-ASCE Committee 326 1962). After cracking, \( V_c \) is attributed to aggregate interlock, dowel action, and the shear transmitted across the concrete compression zone.

The shear strength is based on an average shear stress over the effective cross section, \( b_w d \).

Chapter 23 allows the use of the strut-and-tie method in the shear design of any structural concrete member, or discontinuity region in a member.

R22.5.2 The limit on cross-sectional dimensions in 22.5.1.2 is intended to minimize the likelihood of diagonal compression failure in the concrete and limit the extent of cracking.

R22.5.1.7 Openings in the web of a member can reduce its shear strength. The effects of openings are discussed in Section 4.7 of Joint ACI-ASCE Committee 426 (1973), Barney et al. (1977), and Schlaich et al. (1987). The strut-and-tie method as addressed in Chapter 23 can be used to design members with openings.
22.5.1.8 Effect of axial tension due to creep and shrinkage in members shall be considered in calculating $V_c$.

22.5.1.9 Effect of inclined flexural compression in variable depth members shall be permitted to be considered in calculating $V_c$.

22.5.1.10 The interaction of shear forces acting along orthogonal axes shall be permitted to be neglected if (a) or (b) is satisfied.

(a) \[ \frac{v_{n,x}}{\phi v_{n,x}} \leq 0.5 \] (22.5.1.10a)

(b) \[ \frac{v_{n,y}}{\phi v_{n,y}} \leq 0.5 \] (22.5.1.10b)

22.5.1.11 If \[ \frac{v_{n,x}}{\phi v_{n,x}} > 0.5 \] and \[ \frac{v_{n,y}}{\phi v_{n,y}} > 0.5 \] then Eq. (22.5.1.11) shall be satisfied.

\[ \frac{v_{n,x}}{\phi v_{n,x}} + \frac{v_{n,y}}{\phi v_{n,y}} \leq 1.5 \] (22.5.1.11)

22.5.2 Geometric assumptions

22.5.2.1 For calculation of $V_t$ and $V_s$ in prestressed members, $d$ shall be taken as the distance from the extreme compression fiber to the centroid of prestressed and any nonprestressed longitudinal reinforcement but need not be taken less than 0.8$h$.

22.5.2.2 For calculation of $V_t$ and $V_s$, it shall be permitted to assume (a) through (c):

(a) $d$ equal to 0.8 times the diameter for circular sections
(b) $b_e$ equal to the diameter for solid circular sections
(c) $b_e$ equal to twice the wall thickness for hollow circular sections

22.5.3 Limiting material strengths

22.5.3.1 The value of $\sqrt{f}$ used to calculate $V_c$.

R22.5.1.8 Consideration of axial tension requires engineering judgment. Axial tension often occurs due to volume changes, but it may be low enough not to be detrimental to the performance of a structure with adequate expansion joints and satisfying minimum longitudinal reinforcement requirements. It may be desirable to design shear reinforcement to resist the total shear if there is uncertainty about the magnitude of axial tension.

R22.5.1.9 In a member of variable depth, the internal shear at any section is increased or decreased by the vertical component of the inclined flexural stresses.

R22.5.1.10 and R22.5.1.11 Reinforced concrete members, such as columns and beams, may be subjected to biaxial shear. For symmetrically reinforced circular sections, nominal one-way shear strength about any axis is the same. Therefore, when a circular section is subjected to shear along two centroidal axes, shear strength can be evaluated using the resultant shear. However, for rectangular and other cross sections, calculating nominal one-way shear strength along the axis of the resultant shear is not practical. Tests and analytical results for columns have indicated that for biaxial shear loading, the shear strength follows an elliptical interaction diagram that requires calculating nominal one-way shear strength along both orthogonal directions (Umehara and Jirsa 1984). Considering shear along each centroidal axis independently can be unconservative. Thus, linear interaction accounts for biaxial shear.

R22.5.2 Geometric assumptions

R22.5.2.1 Although the value of $d$ may vary along the span of a prestressed beam, studies (MacGregor and Hanson 1969) have shown that, for prestressed concrete members, $d$ need not be taken less than 0.8$h$. The beams considered had some straight prestressed reinforcement or reinforcing bars at the bottom of the section and had stirrups that enclosed the longitudinal reinforcement.

R22.5.2.2 Shear tests of members with circular sections indicate that the effective area can be taken as the gross area of the section or as an equivalent rectangular area (Joint ACI-ASCE Committee 426 1973; Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

Although the transverse reinforcement in a circular section may not consist of straight legs, tests indicate that Eq. (22.5.8.5.3) is conservative if $d$ is taken as defined in 22.5.2.2 (Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

R22.5.3 Limiting material strengths

R22.5.3.1 Because of a lack of test data and practical
$V_{ci}$ and $V_{cw}$ for one-way shear shall not exceed 100 psi, unless allowed in 22.5.3.2.

22.5.3.2 Values of $\sqrt{f_c'}$ greater than 100 psi shall be permitted in calculating $V_c$, $V_{ci}$, and $V_{cw}$ for reinforced or prestressed concrete beams and concrete joist construction having minimum web reinforcement in accordance with 9.6.3.4 or 9.6.4.2.

22.5.3.3 The values of $f_y$ and $f_{yt}$ used to calculate $V_s$ shall not exceed the limits in 20.2.2.4.

R22.5.3.2 Based on the beam test results in Mponge and Frantz (1984), Elzanaty et al. (1986), Roller and Russell (1990), Johnson and Ramirez (1989), and Ozcebe et al. (1999), an increase in the minimum amount of transverse reinforcement is required for high-strength concrete. These tests indicate a reduction in reserve shear strength occurs as $f_{c'}$ increases in beams reinforced with transverse reinforcement providing an effective shear stress of 50 psi. By providing minimum transverse reinforcement, which increases as $f_{c'}$ increases, the reduction in shear strength is offset.

R22.5.3.3 The upper limit of 60,000 psi on the value of $f_y$ and $f_{yt}$ used in design is intended to control diagonal crack widths.

R22.5.4 Composite concrete members

R22.5.4.1 The scope of Chapter 22 includes composite concrete members. In some cases with cast-in-place concrete, separate placements of concrete may be designed to act as a unit. In these cases, the interface is designed for the loads that will be transferred across the interface. Composite structural steel-concrete beams are not covered in this Code. Design provisions for such composite members are covered in AISC 360.

22.5.4.2 For calculation of $V_n$ for composite members, no distinction shall be made between shored and unshored members.

22.5.4.3 For calculation of $V_n$ for composite members where the specified concrete compressive strength, unit weight, or other properties of different elements vary, properties of the individual elements shall be used in design. Alternatively, it shall be permitted to use the properties of the element that results in the most critical value of $V_n$.

22.5.4.4 If an entire composite member is assumed to resist vertical shear, it shall be permitted to calculate $V_s$ assuming a monolithically cast member of the same cross-sectional shape.

22.5.4.5 If an entire composite member is assumed to resist vertical shear, it shall be permitted to calculate $V_s$ assuming a monolithically cast member of the same cross-sectional shape if shear reinforcement is experience with concretes having compressive strengths greater than 10,000 psi, the Code imposes a maximum value of 100 psi on $\sqrt{f_c'}$ for use in the calculation of shear strength of concrete members. Exceptions to this limit are permitted in beams and joists if the transverse reinforcement satisfies the requirements in 22.5.3.2.
fully anchored into the interconnected elements in accordance with 25.7.

22.5.5 \( V_c \) for nonprestressed members

22.5.5.1 For nonprestressed members, \( V_c \) shall be calculated in accordance with Table 22.5.5.1 and 22.5.5.1.1 through 22.5.5.1.3.

**Table 22.5.5.1—\( V_c \) for nonprestressed members**

<table>
<thead>
<tr>
<th>Criteria</th>
<th>( V_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_s \geq A_{v,min} )</td>
<td>Either of: [ \frac{2 \lambda_f \sqrt{f'_c}}{d} \sqrt{\frac{N_u}{6A_g}} b_d ] (a) [ 2 \lambda_f \sqrt{f'_c} \left( \frac{N_u}{6A_g} \right) b_d ] (b)</td>
</tr>
<tr>
<td>( A_s &lt; A_{v,min} )</td>
<td>[ \frac{2 \lambda_f \sqrt{f'_c}}{d} \sqrt{\frac{N_u}{6A_g}} b_d ] (c)</td>
</tr>
</tbody>
</table>

Notes:
1. Axial load, \( N_u \), is positive for compression and negative for tension.
2. \( V_c \) shall not be taken less than zero.

**22.5.5.1.1** \( V_c \) shall not be taken greater than \( 5 \lambda_f \sqrt{f'_c} b_d d \).

**22.5.5.1.2** In Table 22.5.5.1, the value of \( N_u/6A_g \) shall not be taken greater than 0.05\( f'_c \).

**22.5.5.1.3** The size effect modification factor, \( \lambda_s \), shall be determined by

\[ \lambda_s = \sqrt{\frac{2}{1 + \frac{d}{10}}} \leq 1 \] (22.5.5.1.3)

**22.5.6 \( V_c \) for prestressed members**

22.5.6.1 This section shall apply to the calculation of \( V_c \) for post-tensioned and pretensioned members in regions where the effective force in the prestressed reinforcement is fully transferred to the concrete. For regions of pretensioned members where the effective

**R22.5.5 \( V_c \) for nonprestressed members**

R22.5.5.1 Test results (Kuchma et al. 2019) for nonprestressed members without shear reinforcement indicate that measured shear strength, attributed to concrete, does not increase in direct proportion with member depth. This phenomenon is often referred to as the “size effect.” For example, if the member depth doubles, the shear at failure for the deeper beam may be less than twice the shear at failure of the shallower beam (Sneed and Ramirez 2010). \( A_{v,min} \) for beams and one-way slabs is defined in 9.6.3.4.

Research (Angelakos et al. 2001; Lubell et al. 2004; Brown et al. 2006; Becker and Buettner 1985; Anderson 1978; Bažant et al. 2007) has shown that shear stress at failure is lower for beams with increased depth and a reduced area of longitudinal reinforcement.

In Table 22.5.5.1, for \( A_v > A_{v,min} \), either equation for \( V_c \) may be used. Equation (a) is provided as a simpler option.

When calculating \( V_c \) by Table 22.5.5.1, an axial tension force can cause \( V_c \) to have a negative value. In those cases, the Code specifies that \( V_c \) should be taken equal to zero.

The criteria column in Table 22.5.5.1 references \( A_{v,min} \), which is defined in Table 9.6.3.4 and 10.6.2.2 and referenced throughout the Code.

When applying equations in Table 22.5.5.1, the value of \( A_v \) to be used in the calculation of \( \rho_w \) may be taken as the sum of the areas of longitudinal bars located more than two-thirds of the overall member depth away from the extreme compression fiber. Definitions for \( b_w \) and \( d \) to be used with circular sections are given in 22.5.2.2.

**R22.5.5.1.3** The parameters within the size effect modification factor, \( \lambda_s \), are consistent with fracture mechanics theory for reinforced concrete (Bažant et al. 2007; Frosch et al. 2017).

R22.5.6 \( V_c \) for prestressed members
force in the prestressed reinforcement is not fully transferred to the concrete, 22.5.7 shall govern the calculation of $V_c$.

22.5.6.2 For prestressed flexural members with $A_{psf} \geq 0.4(A_{psf} + A_f)$, $V_c$ shall be calculated in accordance with Table 22.5.6.2, but need not be less than $2\lambda\sqrt{f'_c b_w d}$. Alternatively, it shall be permitted to calculate $V_c$ in accordance with 22.5.6.3.

Table 22.5.6.2—Approximate method for calculating $V_c$

<table>
<thead>
<tr>
<th>Least of (a), (b), and (c):</th>
<th>$V_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.6\lambda\sqrt{f'_c + 700 f_p b_v d} \frac{d_p}{M_u}$</td>
<td>(a)</td>
</tr>
<tr>
<td>$(0.6\lambda\sqrt{f'_c + 700}) b_v d$</td>
<td>(b)</td>
</tr>
<tr>
<td>$5\lambda\sqrt{f'_c b_v d}$</td>
<td>(c)</td>
</tr>
</tbody>
</table>

$M_u$ occurs simultaneously with $V_c$ at the section considered.

When calculating the $V_c$ term in Eq. 22.5.6.2(a), $d_p$ is the distance from the extreme compression fiber to the centroid of prestressed reinforcement. It shall not be permitted to take $d_p$ as 0.80 as in 22.5.2.1.

R22.5.6.2 This provision offers a simple means of calculating $V_c$ for prestressed concrete beams (MacGregor and Hanson 1969). This provision may be applied to beams having prestressed reinforcement only, or to members reinforced with a combination of prestressed and nonprestressed reinforcement. Expression (a) in Table 22.5.6.2 is most applicable to members subject to uniform loading.

In applying the expression in row (a) to simply-supported members subject to uniform loads, Eq. (R22.5.6.2) can be used:

$$V_c = \frac{d_p}{M_u} \left( \frac{\ell - 2x}{x(\ell - x)} \right)$$

where $\ell$ is the span length, and $x$ is the distance from the section being investigated to the support. For concrete with $f'_c$ equal to 5000 psi, $V_c$ from 22.5.6.2 varies, as shown in Fig. R22.5.6.2. Design aids based on this equation are given in ASCE Joint Committee (1940).

Fig. R22.5.6.2—Application of Table 22.5.6.2 to uniformly loaded prestressed members with $f'_c = 5000$ psi.

22.5.6.3 For prestressed members, $V_c$ shall be permitted to be the lesser of $V_c$ calculated in accordance with 22.5.6.3.1 and $V_{cw}$ calculated in accordance with 22.5.6.3.2 or 22.5.6.3.3.

R22.5.6.3 Two types of inclined cracking occur in concrete beams: web-shear cracking and flexure-shear cracking. These two types of inclined cracking are illustrated in Fig. R22.5.6.3.

Web-shear cracking begins from an interior point in a member when the principal tensile stresses exceed the tensile strength of the concrete. Flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, the shear stresses in the concrete above the crack are increased. The flexure-shear crack develops when the combined shear and flexural-tensile stress exceeds the tensile strength of the concrete.
22.5.6.3.1 The flexure-shear strength $V_{ci}$ shall be calculated by (a) but need not be taken less than (b) or (c):

(a) $V_{ci} = 0.6\lambda \sqrt{f_{c} b_{w} d} + V_d + \frac{V M_{cre}}{M_{max}}$  \hspace{1cm} (22.5.6.3.1a)

(b) For members with $A_{pfse} < 0.4(A_{pfpu} + A_{fy})$,

$$V_{ci} = 1.7\lambda \sqrt{f_{c} b_{w} d}$$ \hspace{1cm} (22.5.6.3.1b)

(c) For members with $A_{pfse} \geq 0.4(A_{pfpu} + A_{fy})$,

$$V_{ci} = 2\lambda \sqrt{f_{c} b_{w} d}$$ \hspace{1cm} (22.5.6.3.1c)

where $d_p$ need not be taken less than 0.80h, the values of $M_{max}$ and $V_i$ shall be calculated from the load combinations causing maximum factored moment to occur at section considered, and $M_{cre}$ shall be calculated by:

$$M_{cre} = \left(1 - \frac{1}{y_t}\right) \left(6\lambda \sqrt{f_{c}} f_{pu} - f_{d}\right)$$ \hspace{1cm} (22.5.6.3.1d)

The nominal shear strength provided by the concrete, $V_c$, is assumed equal to the lesser of $V_{ci}$ and $V_{cw}$. The derivations of Eq. (22.5.6.3.1a) and Eq. (22.5.6.3.2) are summarized in ACI Committee 318 (1965).

Fig. R22.5.6.3-Types of cracking in concrete beams.

R22.5.6.3.1 In deriving Eq. (22.5.6.3.1a), it was assumed that $V_{ci}$ is the sum of the shear required to cause a flexural crack at the section in question given by:

$$V = \frac{V M_{cre}}{M_{max}}$$ \hspace{1cm} (R22.5.6.3.1a)

plus an additional increment of shear required to change the flexural crack to a flexure-shear crack. The externally applied factored loads, from which $V_i$ and $M_{max}$ are determined, include superimposed dead load and live load. In calculating $M_{cre}$ for substitution into Eq. (22.5.6.3.1a), $I$ and $y_t$ are the properties of the section resisting the externally applied loads.

For a composite member, where part of the dead load is resisted by only a part of the section, appropriate section properties should be used to calculate $f_d$. The shear due to dead loads, $V_d$, and that due to other loads, $V_i$, are separated in this case. $V_d$ is then the total shear force due to unfactored dead load acting on that part of the section resisting the dead loads acting prior to composite action plus the unfactored superimposed dead load acting on the composite member. The terms $V_i$ and $M_{max}$ may be taken as

$$V_i = V_a - V_d$$ \hspace{1cm} (R22.5.6.3.1b)

$$M_{max} = M_u - M_d$$ \hspace{1cm} (R22.5.6.3.1c)

where $V_a$ and $M_u$ are the factored shear and moment due to the total factored loads, and $M_d$ is the moment due to unfactored dead load (the moment corresponding to $f_d$).

The terms $V_i$ and $M_{max}$ may be taken as

$$V_i = V_a - V_d$$ \hspace{1cm} (R22.5.6.3.1b)

$$M_{max} = M_u - M_d$$ \hspace{1cm} (R22.5.6.3.1c)

where $V_a$ and $M_u$ are the factored shear and moment due to the total factored loads, and $M_d$ is the moment due to unfactored dead load (the moment corresponding to $f_d$).

For noncomposite, uniformly loaded beams, the total cross section resists all the shear, and the live and dead load shear force diagrams are similar. In this case, Eq. (22.5.6.3.1a) and Eq. (22.5.6.3.1d) reduce to
22.5.6.3.2 The web-shear strength $V_{cw}$ shall be calculated by:

$$V_{cw} = 0.6\lambda\sqrt{f_c}b_n d + \frac{V f f _b d M}{M_a} \quad \text{(R22.5.6.3.1d)}$$

where

$$M_a = \left( \frac{I}{y} \right) (6\lambda\sqrt{f_c} + f_{pe}) \quad \text{(R22.5.6.3.1e)}$$

The cracking moment $M_a$ in the two preceding equations represents the total moment, including dead load, required to cause cracking at the extreme fiber in tension. This is not the same as $M_{cr}$ in Eq. (22.5.6.3.1a) where the cracking moment is that due to all loads except the dead load. In Eq. (22.5.6.3.1a), the dead load shear is added as a separate term. $M_a$ is the factored moment on the beam at the section under consideration, and $V_a$ is the factored shear force occurring simultaneously with $M_a$. Because the same section properties apply to both dead and live load stresses, there is no need to calculate dead load stresses and shears separately. $M_a$ reflects the total stress change from effective prestress to a tension of $6\lambda\sqrt{f_c}$, assumed to cause flexural cracking.

R22.5.6.3.2 Equation (22.5.6.3.2) is based on the assumption that web-shear cracking occurs at a shear level causing a principal tensile stress of approximately $4\lambda\sqrt{f_c}$ at the centroidal axis of the cross section. $V_p$ is calculated from the effective prestress force without load factors.

22.5.6.3.3 As an alternative to 22.5.6.3.2, it shall be permitted to calculate $V_{cw}$ as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $4\lambda\sqrt{f_c}$ at location (a) or (b):

(a) Where the centroidal axis of the prestressed cross section is in the web, the principal tensile stress shall be calculated at the centroidal axis.

(b) Where the centroidal axis of the prestressed cross section is in the flange, the principal tensile stress shall be calculated at the intersection of the flange and the web.

22.5.6.3.4 In composite members, the principal tensile stress shall be calculated at the location specified in 22.5.6.3.3 for the composite section, considering superposition of stresses calculated cross sections that resist the corresponding loads.

R22.5.6.3.4 Generally, in unshored construction the principal tensile stresses due to dead load are caused before composite action and principal tensile stresses due to live load are caused after composite action is developed in a member. In shored construction the principal tensile stresses due to both the dead load and live load are caused after composite action is...
22.5.7 $V_c$ for pretensioned members in regions of reduced prestress force

22.5.7.1 When calculating $V_c$, the transfer length of prestressed reinforcement, $\ell_{tr}$, shall be assumed to be $50d_b$ for strand and $100d_b$ for wire.

22.5.7.2 If bonding of strands extends to the end of the member, the effective prestress force shall be assumed to vary linearly from zero at the end of the prestressed reinforcement to a maximum at a distance $\ell_{tr}$ from the end of the prestressed reinforcement.

22.5.7.3 At locations corresponding to a reduced effective prestress force in 22.5.7.2, $V_c$ shall be calculated in accordance with (a) through (c):

(a) The reduced effective prestress force shall be used to determine the applicability of 22.5.6.2.
(b) The reduced effective prestress force shall be used to calculate $V_{cw}$ in 22.5.6.3.
(c) The value of $V_c$ calculated using 22.5.6.2 shall not exceed the value of $V_{cw}$ calculated using the reduced effective prestress force.

22.5.7.4 If bonding of strands does not extend to the end of the member, the effective prestress force shall be assumed to vary linearly from zero at the point where bonding commences to a maximum at a distance $\ell_{tr}$ from that point.

22.5.7.5 At locations corresponding to a reduced effective prestress force according to 22.5.7.4, $V_c$ shall be calculated in accordance with (a) through (c):

(a) The reduced effective prestress force shall be used to determine the applicability of 22.5.6.2.
(b) The reduced effective prestress force shall be used to calculate $V_c$ in accordance with 22.5.6.3.
(c) The value of $V_c$ calculated using 22.5.6.2 shall not exceed the value of $V_{cw}$ calculated using the reduced effective prestress force.

22.5.8 One-way shear reinforcement

22.5.8.1 At each section where $V_u > \phi V_c$, transverse reinforcement shall be provided such that Eq. (22.5.8.1) is satisfied.

$$V_s = \frac{V_u - V_c}{\phi} \quad (22.5.8.1)$$

22.5.8.2 For one-ways members reinforced with transverse reinforcement, $V_s$ shall be calculated in accordance with 22.5.8.5.

R22.5.8 One-way shear reinforcement

R22.5.8.2 Provisions of 22.5.8.5 apply to all types of transverse reinforcement, including stirrups, ties, hoops, crossties, and spirals.
**Code**

22.5.8.3 For one-way members reinforced with bent-up longitudinal bars, \( V_i \) shall be calculated in accordance with 22.5.8.6.

22.5.8.4 If more than one type of shear reinforcement is provided to reinforce the same portion of a member, \( V_i \) shall be the sum of the \( V_i \) values for the various types of shear reinforcement.

22.5.8.5 One-way shear strength provided by transverse reinforcement

22.5.8.5.1 In nonprestressed and prestressed members, shear reinforcement satisfying (a), (b), or (c) shall be permitted:

(a) Stirrups, ties, or hoops perpendicular to longitudinal axis of member
(b) Welded wire reinforcement with wires located perpendicular to longitudinal axis of member
(c) Spiral reinforcement

22.5.8.5.2 Inclined stirrups making an angle of at least 45 degrees with the longitudinal axis of the member and crossing the plane of the potential shear crack shall be permitted to be used as shear reinforcement in nonprestressed members.

22.5.8.5.3 \( V_i \) for shear reinforcement in 22.5.8.5.1 shall be calculated by:

\[
V_i = \frac{A_{f_y} f_{y} d}{s} \quad (22.5.8.5.3)
\]

where \( s \) is the spiral pitch or the longitudinal spacing of the shear reinforcement, and \( A \), is given in 22.5.8.5.5 or 22.5.8.5.6.

22.5.8.5.4 \( V_i \) for shear reinforcement in 22.5.8.5.2 shall be calculated by:

\[
V_i = \frac{A_{f_y} (\sin \alpha + \cos \alpha) d}{s} \quad (22.5.8.5.4)
\]

**Commentary**

R22.5.8.5 One-way shear strength provided by transverse reinforcement

Design of shear reinforcement is based on a modified truss analogy. In the truss analogy, the force in vertical ties is resisted by shear reinforcement. Shear reinforcement needs to be designed to resist only the shear exceeding that which causes inclined cracking, provided the diagonal members in the truss are assumed to be inclined at 45 degrees. The concrete is assumed to contribute to the shear capacity through resistance across the concrete compressive zone, aggregate interlock, and dowel action in an amount equivalent to that which caused inclined cracking.

Equations (22.5.8.5.3), (22.5.8.5.4), and (22.5.8.6.2a) are presented in terms of nominal shear strength provided by shear reinforcement, \( V_i \). Where shear reinforcement perpendicular to the axis of the member is used, the required area of shear reinforcement, \( A \), and its spacing, \( s \), are calculated by

\[
\frac{A}{s} = \frac{(V_i - \phi V_c)}{\phi f_{y} d} \quad (R22.5.8.5)
\]

R22.5.8.5.2 Although inclined stirrups crossing the plane of the potential shear cracks are permitted, their use is not appropriate where the direction of net shear reverses due to changes in transient load.

R22.5.8.5.4 To be effective, it is critical that inclined stirrups cross potential shear cracks. If the inclined stirrups are generally oriented parallel to the potential shear cracks, the stirrups provide no shear strength.
where \( \alpha \) is the angle between the inclined stirrups and the longitudinal axis of the member, \( s \) is measured parallel to the longitudinal reinforcement, and \( A_v \) is given in 22.5.8.5.5.

22.5.8.5.5 For each rectangular tie, stirrup, hoop, or cross tie, \( A_v \) shall be the effective area of all bar legs or wires within spacing \( s \).

22.5.8.5.6 For each circular tie or spiral, \( A_v \) shall be two times the area of the bar or wire within spacing \( s \).

22.5.8.6 One-way shear strength provided by bent-up longitudinal bars

22.5.8.6.1 The center three-fourths of the inclined portion of bent-up longitudinal bars shall be permitted to be used as shear reinforcement in nonprestressed members if the angle \( \alpha \) between the bent-up bars and the longitudinal axis of the member is at least 30 degrees.

22.5.8.6.2 If shear reinforcement consists of a single bar or a single group of parallel bars having an area \( A_v \), all bent the same distance from the support, \( V_s \) shall be the lesser of (a) and (b):

\[
(a) \quad V_s = A_v f_y sin \alpha \quad (22.5.8.6.2a)
\]
\[
(b) \quad V_s = 3\sqrt{f_y b_s d} \quad (22.5.8.6.2b)
\]

where \( \alpha \) is the angle between bent-up reinforcement and longitudinal axis of the member.

22.5.8.6.3 If shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, \( V_s \) shall be calculated by Eq. (22.5.8.5.4).

5.2. Shear reinforcement detailing

7.6—Reinforcement limits

7.6.3 Minimum shear reinforcement

7.6.3.1 A minimum area of shear reinforcement, \( A_{v,min} \), shall be provided in all regions where \( V_u > \phi V_c \). For precast prestressed hollow-core slabs with untopped \( h > 12.5 \) in., \( A_{v,min} \) shall be provided in all regions where \( V_u > 0.5\phi V_c \).

R22.5.8.5.6 Although the transverse reinforcement in a circular section may not consist of straight legs, tests indicate that Eq. (22.5.8.5.3) is conservative if \( d \) is taken as defined in 22.5.2.2 (Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

R22.5.8.6 One-way shear strength provided by bent-up longitudinal bars

To be effective, it is critical that the inclined portion of the bent-up longitudinal bar cross potential shear cracks. If the inclined bars are generally oriented parallel to the potential shear cracks, the bars provide no shear strength.
7.6.3.2 If shown by testing that the required $M_a$ and $V_a$ can be developed, 7.6.3.1 need not be satisfied. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of these effects occurring in service.

7.6.3.3 If shear reinforcement is required, $A_{v,min}$ shall be in accordance with 9.6.3.4.

7.7—Reinforcement detailing
7.7.5 Shear reinforcement
7.7.5.1 If shear reinforcement is required, transverse reinforcement shall be detailed according to 9.7.6.2.

9.6—Reinforcement limits
9.6.3 Minimum shear reinforcement
9.6.3.1 For nonprestressed beams, minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all regions where $V_a > \phi \sqrt{f'_{c} b_w d}$ except for the cases in Table 9.6.3.1. For these cases, at least $A_{v,min}$ shall be provided where $V_a > \phi V_c$.

**Table 9.6.3.1—Cases where $A_{v,min}$ is not required if $V_a \leq \phi V_c$**

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow depth</td>
<td>$h \leq 10$ in.</td>
</tr>
<tr>
<td>Integral with slab</td>
<td>$h \leq$ greater of 2.5$w$ or 0.5$b$, and $h \leq 24$ in.</td>
</tr>
<tr>
<td>Constructed with steel fiber-reinforced normalweight concrete conforming to 26.4.1.5.1(a), 26.4.2.2(i), and 26.12.7.1(a) and with $f'_c \leq 6000$ psi</td>
<td>$h \leq 24$ in. and $V_a \leq 2 \sqrt{f'_c b_w d}$</td>
</tr>
<tr>
<td>One-way joist system</td>
<td>In accordance with 9.8</td>
</tr>
</tbody>
</table>

R7.6.3.2 The basis for the testing-based strength evaluation for one-way slabs is the same as that for beams. Refer to R9.6.3.3 for additional information.

R9.6—Reinforcement limits
R9.6.3 Minimum shear reinforcement
R9.6.3.1 Shear reinforcement restraints the growth of inclined cracking so that ductility of the beam is improved and a warning of failure is provided. In an unreinforced web, the formation of inclined cracking might lead directly to failure without warning. Such reinforcement is of great value if a beam is subjected to an unexpected tensile force or an overload.

The exception for beams constructed using steel fiber-reinforced concrete is intended to provide a design alternative to the use of shear reinforcement, as defined in 22.5.8.5, for beams with longitudinal flexural reinforcement in which $V_a$ does not exceed $\phi \sqrt{f'_c b_w d}$. Chapter 26 specifies design information and compliance requirements that need to be incorporated into the construction documents when steel fiber-reinforced concrete is used for this purpose. Fiber-reinforced concrete beams with hooked or crimped steel fibers, in dosages as required by 26.4.2.2(i), have been shown through laboratory tests to exhibit shear strengths greater than $3.5 \sqrt{f'_c b_w d}$ (Parra-Montesinos 2006). There are no

al. 2006) has shown that deep, lightly reinforced one-way slabs, particularly if constructed with high-strength concrete or concrete having a small coarse aggregate size, may fail at shear less than $V_c$ calculated from Eq. (22.5.5.1). One-way slabs subjected to concentrated loads are more likely to exhibit this vulnerability.

Results of tests on precast, prestressed hollow-core units (Becker and Buettner 1985; Anderson 1978) with $h \leq 12.5$ in. have shown shear strengths greater than those calculated by Eq. (22.5.6.3.1a) and Eq. (22.5.6.3.2). Results of tests on hollow-core units with $h > 12.5$ in. have shown that web-shear strengths in end regions can be less than strengths calculated by Eq. (22.5.6.3.2). In contrast, flexure-shear strengths in the deeper hollow-core units equaled or exceeded strengths calculated by Eq. (22.5.6.3.1a).
For prestressed beams, a minimum area of shear reinforcement, $A_{v,\text{min}}$, shall be provided in all regions where $V_u > 0.5\phi V_c$ except for the cases in Table 9.6.3.1. For these cases, at least $A_{v,\text{min}}$ shall be provided where $V_u > \phi V_c$.

If shown by testing that the required $M_n$ and $V_n$ can be developed, 9.6.3.1 and 9.6.3.2 need not be satisfied. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of these effects occurring in service.

When a beam is tested to demonstrate that its shear and flexural strengths are adequate, the actual beam dimensions and material strengths are known. Therefore, the test strengths are considered the nominal strengths $V_n$ and $M_n$. Considering these strengths as nominal values ensures that if the actual material strengths in the field were less than 

\[\phi \cdot \sqrt{f_c b_d} \]
9.6.3.4 If shear reinforcement is required and torsional effects can be neglected according to 9.5.4.1, \( A_{v,\text{min}} \) shall be in accordance with Table 9.6.3.4.

Table 9.6.3.4—Required \( A_{v,\text{min}} \)

<table>
<thead>
<tr>
<th>Beam type</th>
<th>( A_{v,\text{min}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonprestressed and prestressed with ( A_{v,fc} &lt; 0.4(A_{v,fc} + A_{f}) )</td>
<td>Greater of:</td>
</tr>
<tr>
<td></td>
<td>( 0.75 \sqrt{f_{y} \frac{b_{v}}{f_{y}}} )</td>
</tr>
<tr>
<td></td>
<td>( \frac{50 b_{v}}{f_{y}} )</td>
</tr>
<tr>
<td>Prestressed with ( A_{v,fc} \geq 0.4(A_{v,fc} + A_{f}) )</td>
<td>Lesser of:</td>
</tr>
<tr>
<td></td>
<td>( 0.75 \sqrt{f_{y} \frac{b_{v}}{f_{y}}} )</td>
</tr>
<tr>
<td></td>
<td>( \frac{50 b_{v}}{f_{y}} )</td>
</tr>
<tr>
<td></td>
<td>( \frac{A_{v,fc}}{80 f_{y,d}} \sqrt{f_{y}} )</td>
</tr>
</tbody>
</table>

9.6.4 Minimum torsional reinforcement
9.6.4.1 A minimum area of torsional reinforcement shall be provided in all regions where \( T_u \geq \phi T_{th} \) in accordance with 22.7.

9.6.4.2 If torsional reinforcement is required, minimum transverse reinforcement \( A_{t} \) shall be the greater of (a) and (b):

(a) \( 0.75 \sqrt{f_{y} \frac{b_{v}}{f_{y}}} \)

(b) \( \frac{50 b_{v}}{f_{y}} \)

9.6.4.3 If torsional reinforcement is required, minimum area of longitudinal reinforcement \( A_{l,\text{min}} \) shall be the lesser of (a) and (b):

(a) \( \frac{5 \sqrt{f_{c}} A_{p}}{f_{c}} - \left( \frac{A_{t}}{s} \right) p_{h} \frac{f_{y}}{f_{c}} \)

R9.6.4.2 The differences in the definitions of \( A_{v} \) and \( A_{t} \) should be noted: \( A_{v} \) is the area of two legs of a closed stirrup, whereas \( A_{t} \) is the area of only one leg of a closed stirrup. If a stirrup group has more than two legs, only the legs adjacent to the sides of the beam are considered, as discussed in R9.5.4.3.

Tests (Roller and Russell 1990) of high-strength reinforced concrete beams have indicated the need to increase the minimum area of shear reinforcement to prevent shear failures when inclined cracking occurs. Although there are a limited number of tests of high-strength concrete beams in torsion, the equation for the minimum area of transverse closed stirrups has been made consistent with calculations required for minimum shear reinforcement.

R9.6.4.3 Under combined torsion and shear, the torsional cracking moment decreases with applied shear, which leads to a reduction in torsional reinforcement required to prevent brittle failure immediately after cracking. When subjected to pure torsion, reinforced concrete beam specimens with less than 1 percent torsional reinforcement by volume have failed at first torsional cracking (MacGregor and Ghoneim 1995). Equation 9.6.4.3(a) specified, or the member dimensions were in error such as to result in a reduced member strength, a satisfactory margin of safety will be retained due to the strength reduction factor \( \phi \).
9.7—Reinforcement detailing
9.7.6 Transverse reinforcement
9.7.6.1 General
9.7.6.1.1 Transverse reinforcement shall be in accordance with this section. The most restrictive requirements shall apply.

9.7.6.1.2 Details of transverse reinforcement shall be in accordance with 25.7.

9.7.6.2 Shear
9.7.6.2.1 If required, shear reinforcement shall be provided using stirrups, hoops, or longitudinal bent bars.

9.7.6.2.2 Maximum spacing of legs of shear reinforcement along the length of the member and across the width of the member shall be in accordance with Table 9.7.6.2.2.

is based on a 2:1 ratio of torsion stress to shear stress and results in a torsional reinforcement volumetric ratio of approximately 0.5 percent (Hsu 1968). Tests of prestressed concrete beams have shown that a similar amount of longitudinal reinforcement is required.

R9.7—Reinforcement detailing
R9.7.6 Transverse reinforcement

R9.7.6.2 Shear
R9.7.6.2.1 If a reinforced concrete beam is cast monolithically with a supporting beam and intersects one or both side faces of a supporting beam, the soffit of the supporting beam may be subject to premature failure unless additional transverse reinforcement, commonly referred to as hanger reinforcement, is provided (Mattock and Shen 1992). The hanger reinforcement (Fig. R9.7.6.2.1), placed in addition to other transverse reinforcement, is provided to transfer shear from the end of the supported beam. Research indicates that if the bottom of the supported beam is at or above mid-depth of the supporting beam or if the factored shear transferred from the supported beam is less than \[ 3\sqrt{f_y b_n d} \], hanger reinforcement is not required.

Fig. R9.7.6.2.1-Hanger reinforcement for shear transfer.

R9.7.6.2.2 Reduced stirrup spacing across the beam width provides a more uniform transfer of diagonal compression across the beam web, enhancing shear capacity. Laboratory tests (Leonhardt and Walther 1964; Anderson and Ramirez 1989; Lubell et al. 2009) of wide members with large spacing of legs of shear reinforcement across the member width indicate that the nominal shear capacity is not always
Table 9.7.6.2.2—Maximum spacing of legs of shear reinforcement

<table>
<thead>
<tr>
<th>Required $V_c$</th>
<th>Maximum s, in.</th>
<th>Nonprestressed beam</th>
<th>Prestressed beam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Along length</td>
<td>Across width</td>
</tr>
<tr>
<td>$\leq 4\sqrt{f_y b_d}$</td>
<td>Lesser of:</td>
<td>$d/2$</td>
<td>$d$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24 in.</td>
<td></td>
</tr>
<tr>
<td>$&gt;4\sqrt{f_y b_d}$</td>
<td>Lesser of:</td>
<td>$d/4$</td>
<td>$d/2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12 in.</td>
<td></td>
</tr>
</tbody>
</table>

9.7.6.2.3 Inclined stirrups and longitudinal bars bent to act as shear reinforcement shall be spaced so that every 45-degree line, extending $d/2$ toward the reaction from mid-depth of member to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

9.7.6.2.4 Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall be continuous with longitudinal reinforcement and, if extended into a region of compression, shall be anchored $d/2$ beyond mid-depth of member.

9.7.6.3 Torsion
9.7.6.3.1 If required, transverse torsional reinforcement shall be closed stirrups satisfying 25.7.1.6 or hoops.

9.7.6.3.2 Transverse torsional reinforcement shall extend a distance of at least $(b_h + d)$ beyond the point required by analysis.

9.7.6.3.3 Spacing of transverse torsional reinforcement shall not exceed the lesser of $p_h/8$ and 12 in.

9.7.6.3.4 For hollow sections, the distance from the centerline of the transverse torsional reinforcement to the inside face of the wall of the hollow section shall be at least $0.5A_{cd}/p_h$.

R9.7.6.3 Torsion
R9.7.6.3.1 The stirrups are required to be closed because inclined cracking due to torsion may occur on all faces of a member. In the case of sections subjected primarily to torsion, the concrete side cover over the stirrups spalls off at high torsional moments (Mitchell and Collins 1976). This renders lap-spliced stirrups ineffective, leading to a premature torsional failure (Behera and Rajagopalan 1969). Therefore, closed stirrups should not be made up of pairs of U-stirrups lapping one another.

R9.7.6.3.2 The distance $(b_h + d)$ beyond the point at which transverse torsional reinforcement is calculated to be no longer required is greater than that used for shear and flexural reinforcement because torsional diagonal tension cracks develop in a helical form. The same distance is required by 9.7.5.3 for longitudinal torsional reinforcement.

R9.7.6.3.3 Spacing of the transverse torsional reinforcement is limited to ensure development of the torsional strength of the beam, prevent excessive loss of torsional stiffness after cracking, and control crack widths. For a square cross section, the $p_h/8$ limitation requires stirrups at approximately $d/2$, which corresponds to 9.7.6.2.

R9.7.6.3.4 The transverse torsional reinforcement in a hollow section should be located in the outer half of the wall thickness effective for torsion where the...
wall thickness can be taken as $A_{oo}/p_h$. 
PART VI. TWO-WAY SHEAR DESIGN

6.1. Two-way shear stress demand

8.4—Required strength
8.4.2 Factored moment
8.4.2.1 For slabs built integrally with supports, $M_s$ at the support shall be permitted to be calculated at the face of support.

8.4.2.2 Factored slab moment resisted by the column

8.4.2.2.1 If gravity, wind, earthquake, or other loads cause a transfer of moment between the slab and column, a fraction of $M_s$, the factored slab moment resisted by the column at a joint, shall be transferred by flexure in accordance with 8.4.2.2 through 8.4.2.2.5.

8.4.2.2.2 The fraction of factored slab moment resisted by the column, $\gamma M_s$, shall be assumed to be transferred by flexure, where $\gamma$ shall be calculated by:

$$\gamma = \frac{1}{1 + \left(\frac{2}{3}\right) \sqrt{\frac{h}{b_2}}}$$  \hspace{1cm} (8.4.2.2.2)

8.4.2.2.3 The effective slab width $b_{slab}$ for resisting $\gamma M_s$ shall be the width of column or capital plus a distance on each side in accordance with Table 8.4.2.2.3.

Table 8.4.2.2.3—Dimensional limits for effective slab width

<table>
<thead>
<tr>
<th>Distance on each side of column or capital</th>
<th>Without drop panel or shear cap</th>
<th>With drop panel or shear cap</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lesser</td>
<td>$1.5h$ of slab</td>
<td>$3.5h$ of drop or cap</td>
</tr>
<tr>
<td>Lesser</td>
<td>Distance to edge of slab</td>
<td>Distance to edge of the drop or cap plus $1.5h$ of slab</td>
</tr>
</tbody>
</table>

8.4.2.2.4 For nonprestressed slabs, where the limitations on $v_u$ and $\varepsilon$ in Table 8.4.2.2.4 are satisfied, $\gamma$ shall be permitted to be increased to the maximum modified values provided in Table 8.4.2.2.4, where $v$ is calculated in accordance with 22.6.5.

8.4.2.2.4 Some flexibility in distribution of $M_s$ transferred by shear and flexure at both exterior and interior columns is possible. Interior, exterior, and corner columns refer to slab-column connections for which the critical perimeter for rectangular columns has four, three, and two sides, respectively.

At exterior columns, for $M_s$ resisted about an axis parallel to the edge, the portion of moment transferred by eccentricity of shear $\gamma M_s$ may be reduced, provided that the factored shear at the column (excluding the shear produced by moment transfer) does not exceed 75 percent of the shear strength $\phi v$ as defined in 22.6.5.1 for edge columns,
or 50 percent for corner columns. Tests (Moehle 1988; ACI 352.1R) indicate that there is no significant interaction between shear and $M_{se}$ at the exterior column in such cases. Note that as $\gamma_fM_{se}$ is decreased, $\gamma_fM_{sc}$ is increased.

At interior columns, some flexibility in distributing $M_{se}$ transferred by shear and flexure is possible, but with more severe limitations than for exterior columns. For interior columns, $M_{se}$ transferred by flexure is permitted to be increased up to 25 percent, provided that the factored shear (excluding the shear caused by the moment transfer) at the interior columns does not exceed 40 percent of the shear strength $\phi v$, as defined in 22.6.5.1.

If the factored shear for a slab-column connection is large, the slab-column joint cannot always develop all of the reinforcement provided in the effective width. The modifications for interior slab-column connections in this provision are permitted only where the reinforcement required to develop $\gamma_fM_{se}$ within the effective width has a net tensile strain $\varepsilon_t$ not less than $\varepsilon_{ty} + 0.008$, where the value of $\varepsilon_{ty}$ is determined in 21.2.2. The use of Eq. (8.4.2.2.2) without the modification permitted in this provision will generally indicate overstress conditions on the joint. This provision is intended to improve ductile behavior of the slab-column joint. If reversal of moments occurs at opposite faces of an interior column, both top and bottom reinforcement should be concentrated within the effective width. A ratio of top-to-bottom reinforcement of approximately 2 has been observed to be appropriate.

Before the 2019 Code, the strain limits on $\varepsilon_t$ in Table 8.4.2.2.4 were constants of 0.004 and 0.010. Beginning with the 2019 Code, to accommodate nonprestressed reinforcement of higher grades, these limits are replaced by the expressions $\varepsilon_{ty} + 0.003$ and $\varepsilon_{ty} + 0.008$, respectively. The first expression is the same expression as used for the limit on $\varepsilon_t$ for classification of tension-controlled members in Table 21.2.2; this expression is further described in Commentary R21.2.2. The second expression provides a limit on $\varepsilon_t$ with Grade 60 reinforcement that is approximately the same value as the former constant of 0.010.

### Table 8.4.2.2.4—Maximum modified values of $\gamma_f$ for nonprestressed two-way slabs

<table>
<thead>
<tr>
<th>Column location</th>
<th>Span direction</th>
<th>$v_{se}$</th>
<th>$\varepsilon_t$ (within $b_{se}$)</th>
<th>Maximum modified $\gamma_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner column</td>
<td>Either direction</td>
<td>$\leq 0.5\phi v$</td>
<td>$\geq \varepsilon_{ty} + 0.003$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Perpendicular to the edge</td>
<td>$\leq 0.75\phi v$</td>
<td>$\geq \varepsilon_{ty} + 0.003$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Parallel to the edge</td>
<td>$\leq 0.4\phi v$</td>
<td>$\geq \varepsilon_{ty} + 0.008$</td>
<td>$\frac{1.25}{1 + \left(\frac{2}{3}\right)\frac{P}{P_c}} \leq 1.0$</td>
</tr>
<tr>
<td>Edge column</td>
<td>Either direction</td>
<td>$\leq 0.4\phi v$</td>
<td>$\geq \varepsilon_{ty} + 0.008$</td>
<td>$\frac{1.25}{1 + \left(\frac{2}{3}\right)\frac{P}{P_c}} \leq 1.0$</td>
</tr>
</tbody>
</table>
8.4.2.2.5 Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in 8.4.2.2.2 and 8.4.2.2.3.

8.4.2.2.6 The fraction of $M_{ce}$ not calculated to be resisted by flexure shall be assumed to be resisted by eccentricity of shear in accordance with 8.4.4.2.

8.4.3 Factored one-way shear
8.4.3.1 For slabs built integrally with supports, $V_u$ at the support shall be permitted to be calculated at the face of support.

8.4.3.2 Sections between the face of support and a critical section located $d$ from the face of support for nonprestressed slabs and $h/2$ from the face of support for prestressed slabs shall be permitted to be designed for $V_u$ at that critical section if (a) through (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end regions of the slab.
(b) Loads are applied at or near the top surface of the slab.
(c) No concentrated load occurs between the face of support and critical section.

8.4.4 Factored two-way shear
8.4.4.1 Critical section
8.4.4.1.1 Slabs shall be evaluated for two-way shear in the vicinity of columns, concentrated loads, and reaction areas at critical sections in accordance with 22.6.4.

8.4.4.1.2 Slabs reinforced with stirrups or headed shear stud reinforcement shall be evaluated for two-way shear at critical sections in accordance with 22.6.4.2.

8.4.4.2 Factored two-way shear stress due to shear and factored slab moment resisted by the column
8.4.4.2.1 For two-way shear with factored slab moment resisted by the column, factored shear stress $v_u$ shall be calculated at critical sections in accordance with 8.4.4.1. Factored shear stress $v_u$ corresponds to a combination of $v_{ue}$ and the shear stress produced by $\gamma_{v}M_{ce}$, where $\gamma_{v}$ is given in 8.4.4.2.2 and $M_{ce}$ is given in 8.4.2.2.1.

8.4.4.2.2 The fraction of $M_{ce}$ transferred by eccentricity of shear, $\gamma_{v}M_{ce}$, shall be applied at the centroid of the critical section in accordance with 8.4.4.1, where:

$$\gamma_{v} = 1 - \gamma_f$$  \hspace{1cm} (8.4.4.2.2)

R8.4.4 Factored two-way shear
The calculated shear stresses in the slab around the column are required to conform to the requirements of 22.6.

R8.4.4.2 Factored two-way shear stress due to shear and factored slab moment resisted by the column
Hanson and Hanson (1968) found that where moment is transferred between a column and a slab, 60 percent of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 22.6.4.1, and 40 percent by eccentricity of the shear about the centroid of the critical section. For rectangular columns, the portion of the moment transferred by flexure increases as the
8.4.4.2.3 The factored shear stress resulting from $\gamma M_{sc}$ shall be assumed to vary linearly about the centroid of the critical section in accordance with 8.4.4.1.

R8.4.4.2.3 The stress distribution is assumed as illustrated in Fig. R8.4.4.2.3 for an interior or exterior column. The perimeter of the critical section, $ABCD$, is determined in accordance with 22.6.4.1. The factored shear stress $v_u$ and factored slab moment resisted by the column $M_{sc}$ are determined at the centroidal axis $c-c$ of the critical section. The maximum factored shear stress may be calculated from:

$$v_{scAB} = v_u + \frac{\gamma M_{sc} c_{AB}}{J_c}$$

or

$$v_{scCD} = v_u + \frac{\gamma M_{sc} c_{CD}}{J_c}$$

where $\gamma$ is given by Eq. (8.4.4.2.2).

For an interior column, $J_c$ may be calculated by:

$$J_c = \text{property of assumed critical section analogous to polar moment of inertia}$$

$$= \frac{d (c_1 + d)^3}{6} + \frac{(c_1 + d)^3}{6} + \frac{d (c_2 + d)(c_1 + d)^2}{2}$$

Similar equations may be developed for $J_c$ for columns located at the edge or corner of a slab.

The fraction of $M_{sc}$ not transferred by eccentricity of the shear should be transferred by flexure in accordance with 8.4.2.2. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 8.4.2.2.3. Often, column strip reinforcement is concentrated near the column to accommodate $M_{sc}$. Available test data (Hanson and Hanson 1968) seem to indicate that this practice does not increase shear strength but may be desirable to increase the stiffness of the slab-column junction.

Test data (Hawkins 1981) indicate that the moment transfer strength of a prestressed slab-to-column connection can be calculated using the procedures of 8.4.2.2 and 8.4.4.2.

Where shear reinforcement has been used, the critical section beyond the shear reinforcement generally has a polygonal shape (Fig. R8.7.6(d) and (e)). Equations for calculating shear stresses on such sections are given in ACI 421.1R.
6.2. Two-way shear strength

8.5—Design strength
8.5.3 Shear
8.5.3.1 Design shear strength of slabs in the vicinity of columns, concentrated loads, or reaction areas shall be the more severe of 8.5.3.1.1 and 8.5.3.1.2.

8.5.3.1.1 For one-way shear, where each critical section to be investigated extends in a plane across the entire slab width, $V_n$ shall be calculated in accordance with 22.5.

8.5.3.1.2 For two-way shear, $v_n$ shall be calculated in accordance with 22.6.

8.5.3.2 For composite concrete slabs, horizontal shear strength $V_{nh}$ shall be calculated in accordance with 16.4.

8.5.4 Openings in slab systems
8.5.4.1 Openings of any size shall be permitted in slab systems if shown by analysis that all strength and serviceability requirements, including the limits on deflections, are satisfied.

8.5.4.1 Openings of any size shall be permitted in
slab systems if shown by analysis that all strength and serviceability requirements, including the limits on deflections, are satisfied.

(a) Openings of any size shall be permitted in the area common to intersecting middle strips, but the total quantity of reinforcement in the panel shall be at least that required for the panel without the opening.

(b) At two intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings. A quantity of reinforcement at least equal to that interrupted by an opening shall be added on the sides of the opening.

(c) At the intersection of one column strip and one middle strip, not more than one-fourth of the reinforcement in either strip shall be interrupted by openings. A quantity of reinforcement at least equal to that interrupted by an opening shall be added on the sides of the opening.

(d) If an opening is located closer than $4h$ from the periphery of a column, concentrated load or reaction area, 22.6.4.3 shall be satisfied.

22.6—Two-way shear strength

R22.6—Two-way shear strength

Factored shear stress in two-way members due to shear and moment transfer is calculated in accordance with the requirements of 8.4.4. Section 22.6 provides requirements for determining nominal shear strength, either without shear reinforcement or with shear reinforcement in the form of stirrups or headed shear studs. Factored shear demand and strength are calculated in terms of stress, permitting superposition of effects from direct shear and moment transfer.

Design provisions for shearheads have been eliminated from the Code because this type of shear reinforcement is seldom used in current practice. Shearheads may be designed following the provisions of ACI 318-14.

22.6.1 General

22.6.1.1 Provisions 22.6.1 through 22.6.8 apply to the nominal shear strength of two-way members with and without shear reinforcement.

22.6.1.2 Nominal shear strength for two-way members without shear reinforcement shall be calculated by

$$v_n = v_r$$

(22.6.1.2)

22.6.1.3 Nominal shear strength for two-way members with shear reinforcement shall be calculated by

$$v_n = v_r + v_s$$

(22.6.1.3)

22.6.1.4 Two-way shear shall be resisted by a section
with a depth $d$ and an assumed critical perimeter $b_o$ as defined in 22.6.4.

22.6.1.5 $v_c$ for two-way shear shall be calculated in accordance with 22.6.5. For two-way members with shear reinforcement, $v_c$ shall not exceed the limits in 22.6.6.1.

22.6.1.6 For calculation of $v_c$, $\lambda$ shall be in accordance with 19.2.4.

22.6.1.7 For two-way members reinforced with single- or multiple-leg stirrups, $v_t$ shall be calculated in accordance with 22.6.7.

22.6.1.8 For two-way members reinforced with headed shear stud reinforcement, $v_s$ shall be calculated in accordance with 22.6.8.

22.6.2 Effective depth
22.6.2.1 For calculation of $v_c$ and $v_s$ for two-way shear, $d$ shall be the average of the effective depths in the two orthogonal directions.

22.6.2.2 For prestressed, two-way members, $d$ need not be taken less than 0.8$h$.

22.6.3 Limiting material strengths
22.6.3.1 The value of $\sqrt{f_{ct}}$ used to calculate $v_c$ for two-way shear shall not exceed 100 psi.

22.6.3.2 The value of $f_{yt}$ used to calculate $v_s$ shall not exceed the limits in 20.2.2.4.

22.6.4 Critical sections for two-way members
22.6.4.1 For two-way shear, critical sections shall be located so that the perimeter $b_o$ is a minimum but need not be closer than $d/2$ to (a) and (b):

(a) Edges or corners of columns, concentrated loads, or reaction areas
(b) Changes in slab or footing thickness, such as edges of capitals, drop panels, or shear caps

22.6.4.1.1 For square or rectangular columns, concentrated loads, or reaction areas, critical sections for two-way shear in accordance with 22.6.4.1(a) and (b) shall be permitted to be defined assuming straight sides.

22.6.4.1.2 For a circular or regular polygon-shaped column, critical sections for two-way shear in accordance with 22.6.4.1(a) and (b) shall be

R22.6.1.4 The critical section perimeter $b_0$ is defined in 22.6.4.

R22.6.3 Limiting material strengths
R22.6.3.1 There are limited test data on the two-way shear strength of high-strength concrete slabs. Until more experience is obtained for two-way slabs constructed with concretes that have compressive strengths greater than 10,000 psi, it is prudent to limit $\sqrt{f_{ct}}$ to 100 psi for the calculation of shear strength.

R22.6.3.2 The upper limit of 60,000 psi on the value of $f_{yt}$ used in design is intended to control cracking.

R22.6.4 Critical sections for two-way members
The critical section defined in 22.6.4.1(a) for shear in slabs and footings subjected to bending in two directions follows the perimeter at the edge of the loaded area (Joint ACI-ASCE Committee 326 1962). Loaded area for shear in two-way slabs and footings includes columns, concentrated loads, and reaction areas. An idealized critical section located a distance $d/2$ from the periphery of the loaded area is considered.

For members of uniform thickness without shear reinforcement, it is sufficient to check shear using one section. For slabs with changes in thickness or with shear reinforcement, it is necessary to check shear at multiple sections as defined in 22.6.4.1(a) and (b) and 22.6.4.2.

For columns near an edge or corner, the critical perimeter may extend to the edge of the slab.
permitted to be defined assuming a square column of equivalent area.

**22.6.4.2** For two-way members reinforced with headed shear reinforcement or single- or multi-leg stirrups, a critical section with perimeter \( b_o \) located \( d/2 \) beyond the outermost peripheral line of shear reinforcement shall also be considered. The shape of this critical section shall be a polygon selected to minimize \( b_o \).

**R22.6.4.2** For two-way members with stirrup or headed stud shear reinforcement, it is required to check shear stress in concrete at a critical section located a distance \( d/2 \) beyond the point where shear reinforcement is discontinued. Calculated shear stress at this section must not exceed the limits given in expressions (b) and (d) in Table 22.6.6.1. The shape of this outermost critical section should correspond to the minimum value of \( b_o \), as depicted in Fig. R22.6.4.2a, b, and c. Note that these figures depict slabs reinforced with stirrups. The shape of the outermost critical section is similar for slabs with headed shear reinforcement. The square or rectangular critical sections described in 22.6.4.1.1 will not result in the minimum value of \( b_o \) for the cases depicted in these figures. Additional critical section checks are required at a distance \( d/2 \) beyond any point where variations in shear reinforcement occur, such as changes in size, spacing, or configuration.

![Figure R22.6.4.2a-Critical sections for two-way shear in slab with shear reinforcement at interior column.](image-url)
If an opening is located closer than $4h$ from the periphery of a column, concentrated load, or reaction area, the portion of $b_o$ enclosed by straight lines projecting from the centroid of the column, concentrated load or reaction area and tangent to the boundaries of the opening shall be considered ineffective.

Provisions for design of openings in slabs (and footings) were developed in Joint ACI-ASCE Committee 326 (1962). The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in Fig. R22.6.4.3. Research (Joint ACI-ASCE Committee 426 1974) has confirmed that these provisions are conservative.

Research (Genikomsou and Polak 2017) has shown that when openings are located at distances greater than $4d$ from the periphery of a column, the punching shear strength is the same as that for a slab without openings.
22.6.5 Two-way shear strength provided by concrete in members without shear reinforcement

22.6.5.1 For nonprestressed two-way members, $v_c$ shall be calculated in accordance with 22.6.5.2. For prestressed two-way members, $v_c$ shall be calculated in accordance with (a) or (b):

(a) 22.6.5.2

(b) 22.6.5.5, if the conditions of 22.6.5.4 are satisfied

22.6.5.2 $v_c$ shall be calculated in accordance with Table 22.6.5.2.

Table 22.6.5.2 — $v_c$ for two-way members without shear reinforcement

| $v_c$ | 4$\lambda_s$,$\lambda$, from Table 22.5.2.
| Least of (a), (b), and (c): |

- a. $2 + \frac{4}{\beta}$ $\lambda_s$, from Table 22.5.2.
- b. $2 + \frac{\alpha_d}{\beta}$, from Table 22.5.2.
- c. $2 + \frac{\alpha_d}{\beta}$, from Table 22.5.2.

Notes:
(i) $\lambda_s$ is the size effect factor given in 22.5.5.1.3.
(ii) $\beta$ is the ratio of long to short sides of the column, concentrated load, or reaction area.
(iii) $\alpha_d$ is given in 22.6.5.3.

R22.6.5 Two-way shear strength provided by concrete in members without shear reinforcement

R22.6.5.2 Experimental evidence indicates that the measured concrete shear strength of two-way members without shear reinforcement does not increase in direct proportion with member depth. This phenomenon is referred to as the “size effect.” The modification factor $\lambda_s$ accounts for the dependence of two-way shear strength of slabs on effective depth.

For nonprestressed two-way slabs without a minimum amount of shear reinforcement and with $d > 10$ in., the size effect specified in 22.5.5.1.3 reduces the shear strength of two-way slabs below $4\sqrt{f_{ct,b,d}}$ (Hawkins and Ospina 2017; Dönmez and Bažant 2017).

For square columns, the stress corresponding to the nominal two-way shear strength provided by concrete in slabs subjected to bending in two directions is limited to $4\sqrt{f_{ct,b,d}}$. However, tests (Joint ACI-ASCE Committee 426 1974) have
The value of \( \alpha \) is 40 for interior columns, 30 for edge columns, and 20 for corner columns.

For prestressed, two-way members, it shall be permitted to calculate \( v_c \) using 22.6.5.5, provided that (a) through (c) are satisfied:

(a) Bonded reinforcement is provided in accordance with 8.6.2.3 and 8.7.5.3.

The terms “interior columns,” “edge columns,” and “corner columns” in this provision refer to critical sections with a continuous slab on four, three, and two sides, respectively.

For prestressed two-way members, modified forms of expressions (b) and (c) in Table 22.6.5.2 are specified. Research (ACI 423.3R) indicates that the shear strength of two-way prestressed slabs around interior columns is conservatively calculated by the expressions in 22.6.5.5, where \( v_c \) corresponds to a diagonal tension failure of the concrete initiating at the critical section.
(b) No portion of the column cross section is closer to a discontinuous edge than four times the slab thickness \( h \)
(c) Effective prestress \( f_{pc} \) in each direction is not less than 125 psi

22.6.5.5 For prestressed, two-way members conforming to 22.6.5.4, \( v_c \) shall be permitted to be the lesser of (a) and (b)

(a) \[ v_c = 3.5\lambda \sqrt{f_c} + 0.3f_{pc} + \frac{V_p}{b_d} \] (22.6.5.5a)

(b) \[ v_c = \left(1.5 + \frac{\alpha_s a}{b_y}\right)\lambda \sqrt{f_c} + 0.3f_{pc} + \frac{V_p}{b_d} \] (22.6.5.5b)

where \( \alpha_s \) is given in 22.6.5.3; the value of \( f_{pc} \) is the average of \( f_{pc} \) in the two directions and shall not exceed 500 psi; \( V_p \) is the vertical component of all effective prestress forces crossing the critical section; and the value of \( \sqrt{f_c} \) shall not exceed 70 psi.

22.6.6 Two-way shear strength provided by concrete in members with shear reinforcement

R22.6.6 Two-way shear strength provided by concrete in members with shear reinforcement

Critical sections for two-way members with shear reinforcement are defined in 22.6.4.1 for the sections adjacent to the column, concentrated load, or reaction area, and 22.6.4.2 for the section located just beyond the outermost peripheral line of stirrup or headed shear stud reinforcement. Values of maximum \( v_c \) for these critical sections are given in Table 22.6.6.1. Limiting values of \( v_u \) for the critical sections defined in 22.6.4.1 are given in Table 22.6.6.3. The maximum \( v_c \) and limiting value of \( v_u \) at the innermost critical section (defined in 22.6.4.1) are higher where headed shear stud reinforcement is provided than the case where stirrups are provided (refer to R8.7.7). Maximum \( v_c \) values at the critical sections defined in 22.6.4.2 beyond the outermost peripheral line of shear reinforcement are independent of the type of shear reinforcement provided.

R22.6.6.1 For two-way slabs with stirrups, the maximum value of \( v_c \) is taken as \( 2\lambda \sqrt{f_c} \) because the stirrups resist all the shear beyond that at inclined cracking (which occurs at approximately half the
Table 22.6.6.1—v_c for two-way members with shear reinforcement

<table>
<thead>
<tr>
<th>Type of shear reinforcement</th>
<th>Critical sections</th>
<th>v_c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stirrups</td>
<td>All</td>
<td>2\lambda_s \sqrt{f_c}</td>
</tr>
</tbody>
</table>
| Headed shear stud reinforcement | According to 22.6.4.1 | Least of (b), (c), and (d): $3\lambda_s \sqrt{f_c}$,
|                             | According to 22.6.4.2 | $2\lambda_s \sqrt{f_c}$ |

Notes:
(i) $\lambda_s$ is the size effect factor given in 22.5.5.1.3.
(ii) $\beta$ is the ratio of long to short sides of the column, concentrated load, or reaction area.
(iii) $s$ is given in 22.6.5.3.

22.6.6.2 It shall be permitted to take $\lambda_s$ as 1.0 if (a) or (b) is satisfied:

(a) Stirrups are designed and detailed in accordance with 8.7.6 and $A_s / s \geq 2\sqrt{f_c b_s / f_{ct}}$.

(b) Smooth headed shear stud reinforcement with stud shaft length not exceeding 10 in. is designed and detailed in accordance with 8.7.7 and $A_s / s \geq 2\sqrt{f_c b_s / f_{ct}}$.

R22.6.6.2 The size effect in slabs with $d > 10$ in. can be mitigated if a minimum amount of shear reinforcement is provided. The ability of ordinary (smooth) headed shear stud reinforcement to effectively mitigate the size effect on the two-way shear strength of slabs may be compromised if studs longer than 10 in. are used. Until experimental evidence becomes available, it is not permitted to use $\lambda_s$ equal to 1.0 for slabs with $d > 10$ in. without headed shear stud reinforcement with stud shaft length not exceeding 10 in. Stacking or “piggybacking” of headed shear studs, as shown in Fig. R22.6.6.2, introduces an intermediate head that contributes to further anchor the stacked stud.

22.6.6.3 For two-way members with shear reinforcement, effective depth shall be selected such that $v_c$ calculated at critical sections does not exceed the values in Table 22.6.6.3.

Fig. R22.6.6.2—Stacking (piggybacking) of headed shear stud reinforcement.
Table 22.6.3—Maximum $v_s$ for two-way members with shear reinforcement

<table>
<thead>
<tr>
<th>Type of shear reinforcement</th>
<th>Maximum $v_s$ at critical sections defined in 22.6.4.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stirrups</td>
<td>$\phi \sqrt{f_{y}}$ (a)</td>
</tr>
<tr>
<td>Headed shear stud reinforcement</td>
<td>$\phi \sqrt{f_{y}}$ (b)</td>
</tr>
</tbody>
</table>

22.6.7 Two-way shear strength provided by single- or multiple-leg stirrups

22.6.7.1 Single- or multiple-leg stirrups fabricated from bars or wires shall be permitted to be used as shear reinforcement in slabs and footings satisfying (a) and (b):

(a) $d$ is at least 6 in.

(b) $d$ is at least $16d_s$, where $d_s$ is the diameter of the stirrups

22.6.7.2 For two-way members with stirrups, $v_s$ shall be calculated by:

$$v_s = \frac{A_v f_{y}}{b_v s} \quad (22.6.7.2)$$

where $A_v$ is the sum of the area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section, and $s$ is the spacing of the peripheral lines of shear reinforcement in the direction perpendicular to the column face.

22.6.8 Two-way shear strength provided by headed shear stud reinforcement

22.6.8.1 Headed shear stud reinforcement shall be permitted to be used as shear reinforcement in slabs and footings if the placement and geometry of the headed shear stud reinforcement satisfies 8.7.7.

22.6.8.2 For two-way members with headed shear stud reinforcement, $v_s$ shall be calculated by:

$$v_s = \frac{A_v f_{y}}{b_v s} \quad (22.6.8.2)$$

where $A_v$ is the sum of the area of all shear studs on one peripheral line that is geometrically similar to the perimeter of the column section, and $s$ is the spacing of the peripheral lines of headed shear stud reinforcement in the direction perpendicular to the column face.
22.6.8.3 If headed shear stud reinforcement is provided, $A_{vs}$ shall satisfy:

$$\frac{A_{vs}}{s} \geq 2\sqrt{f_y b_s}{f_y}$$

(22.6.8.3)

6.3. Shear reinforcement detailing

8.7—Reinforcement detailing
8.7.6 Shear reinforcement – stirrups
8.7.6.1 Single-leg, simple-U, multiple-U, and closed stirrups shall be permitted as shear reinforcement.

8.7.6.2 Stirrup anchorage and geometry shall be in accordance with 25.7.1.

8.7.6.3 If stirrups are provided, location and spacing shall be in accordance with Table 8.7.6.3.

Table 8.7.6.3—First stirrup location and spacing limits

<table>
<thead>
<tr>
<th>Direction of measurement</th>
<th>Description of measurement</th>
<th>Maximum distance or spacing, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perpendicular to column face</td>
<td>Distance from column face to first stirrup</td>
<td>$d/2$</td>
</tr>
<tr>
<td></td>
<td>Spacing between stirrups</td>
<td>$d/2$</td>
</tr>
<tr>
<td>Parallel to column face</td>
<td>Spacing between vertical legs of stirrups</td>
<td>$2d$</td>
</tr>
</tbody>
</table>

R8.7—Reinforcement detailing
R8.7.6 Shear reinforcement – stirrups

Research (Hawkins 1974; Broms 1990; Yamada et al. 1991; Hawkins et al. 1975; ACI 421.1R) has shown that shear reinforcement consisting of properly anchored bars or wires and single- or multiple-leg stirrups, or closed stirrups, can increase the punching shear resistance of slabs. The spacing limits given in 8.7.6.3 correspond to slab shear reinforcement details that have been shown to be effective. Section 25.7.1 gives anchorage requirements for stirrup-type shear reinforcement that should also be applied for bars or wires used as slab shear reinforcement. It is essential that this shear reinforcement engage longitudinal reinforcement at both the top and bottom of the slab, as shown for typical details in Fig. R8.7.6(a) to (c). Anchorage of shear reinforcement according to the requirements of 25.7.1 is difficult in slabs thinner than 10 in. Shear reinforcement consisting of vertical bars mechanically anchored at each end by a plate or head capable of developing the yield strength of the bars has been used successfully (ACI 421.1R).

In a slab-column connection for which moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section (Fig. R8.7.6(d)). Spacing limits defined in 8.7.6.3 are also shown in Fig. R8.7.6(d) and (e). At edge columns or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible. Although the average shear stresses on faces $AD$ and $BC$ of the exterior column in Fig. R8.7.6(e) are lower than on face $AB$, the closed stirrups extending from faces $AD$ and $BC$ provide some torsional strength along the edge of the slab.
Fig. R8.7.6(a)-(c) - Single- or multiple-leg stirrup-type slab shear reinforcement.
Fig. R8.7.6(d)-Arrangement of stirrup shear reinforcement, interior column.
8.7.7 Shear reinforcement – headed studs

8.7.7.1 Headed shear stud reinforcement shall be permitted if placed perpendicular to the plane of the slab.

8.7.7.1.1 The overall height of the shear stud assembly shall be at least the thickness of the slab minus the sum of (a) through (c):

(a) Concrete cover on the top flexural reinforcement
(b) Concrete cover on the base rail
(c) One-half the bar diameter of the flexural tension reinforcement

R8.7.7 Shear reinforcement – headed studs

Using headed stud assemblies as shear reinforcement in slabs requires specifying the stud shank diameter, the spacing of the studs, and the height of the assemblies for the particular applications.

Tests (ACI 421.1R) show that vertical studs mechanically anchored as close as possible to the top and bottom of slabs are effective in resisting punching shear. The bounds of the overall specified height achieve this objective while providing a reasonable tolerance in specifying that height, as shown in Fig. R20.5.1.3.5.

Compared with a leg of a stirrup having bends at the ends, a stud head exhibits smaller slip and, thus, results in smaller shear crack widths. The improved performance results in increased limits for shear strength and spacing between peripheral lines of headed shear stud reinforcement. Typical arrangements of headed shear stud reinforcement are shown in Fig. R8.7.7. The critical section beyond the shear reinforcement generally has a polygonal shape. Equations for calculating shear stresses on such sections are given in ACI 421.1R.
8.7.7.1.2 Headed shear stud reinforcement location and spacing shall be in accordance with Table 8.7.1.2.

R8.7.7.1.2 The specified spacings between peripheral lines of shear reinforcement are justified by experiments (ACI 421.1R). The clear spacing between the heads of the studs should be adequate to permit placing of the flexural reinforcement.

Table 8.7.1.2—Shear stud location and spacing limits

<table>
<thead>
<tr>
<th>Direction of measurement</th>
<th>Description of measurement</th>
<th>Condition</th>
<th>Maximum distance or spacing, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perpendicular to column face</td>
<td>Distance from column face to First peripheral line of shear studs</td>
<td>All</td>
<td>( d/2 )</td>
</tr>
<tr>
<td></td>
<td>Constant spacing between peripheral lines of shear studs</td>
<td>Nonprestressed slab with ( v_s \leq \phi \sigma_f / \sqrt{f_c} )</td>
<td>( 3d/4 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nonprestressed slab with ( v_s &gt; \phi \sigma_f / \sqrt{f_c} )</td>
<td>( d/2 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Prestressed slabs conforming to 22.6.5.4</td>
<td>( 3d/4 )</td>
</tr>
<tr>
<td>Parallel to column face</td>
<td>Spacing between adjacent shear studs on peripheral line nearest to column face</td>
<td>All</td>
<td>( 2d )</td>
</tr>
</tbody>
</table>
FA Question:

While the ACI, (PCI and AISC) consider a cast-in-place concrete slab as restrained for the purpose of concrete cover for fire rating, PTI manual #18.3.2 stated that for the selection of the required concrete fire cover to reinforcement in a continuous post-tensioned slab, end spans are “typically” considered to be unrestrained and interior spans are considered restrained. Please clarify if end spans of a cast-in-place PT floor or roof system can be considered as restrained for fire endurance in alignment with ACI?

PTI Response:

Concrete is a non-combustible material with relatively low thermal conductivity thus possess ‘inherent fire resistance’. Due this characteristics, prescriptive engineering codes based on standard fire testing in a controlled testing environment, takes a simplistic approach prescribing minimum overall member dimensions and minimum concrete cover for acceptable fire rating for various structural systems, materials and support conditions. However, fire performance in real buildings of a real fire test on a full-scale, seven-story cast in-place concrete building that was performed at the UK Building Research Establishment (BRE) Cardington test site (Bailey 2002), show that concrete buildings subjected to real fires, rather than a controlled ‘standard’ fires, generally performed well. Structural failure (i.e. local or global collapse) is rare and relaxation of prestressing tendons was observed in both bonded and unbonded post-tensioned construction.

Response of unbonded PT buildings in real fire is more complicated than suggested by the available prescriptive guidance, particularly with respect to cover spalling and prestressed tendon rupture under localized heating. In the ASTM E119, cast-in-place and most precast concrete constructions are considered to be restrained (Fire Resistance of Post-Tensioned Structures, Armand H. Gustafreco, PCI Journal/March-April 1973). The origin of restrained and unrestrained started with the tentative revision of ASTM E119-71, which includes a guide for classifying constructions as restrained or unrestrained. A simply supported determinate structural members is considered as unrestrained. For the fire performance purposes, these unrestrained members with cold-drawn prestressing steel, the fire endurance criterion is based on the time required for the steel to reach a certain temperature limit during a standard fire test. Thus, cover thicknesses for post-tensioned tendons in unrestrained slabs are determined by the elapsed time during a fire test until the tendons reach a critical temperature. For cold-drawn pre-stressing steel that temperature is 800°F.

However, for other restrained members, which are indeterminate structures, irrespective of the end supports conditions, for fire endurance the steel temperature is disregarded and the governing criterion is the structural end point due to the intrinsic structural behavior namely moment redistribution in a continuous indeterminate structures. Unlike unrestrained members, for restrained slabs there are no temperature limitations. The principle involved is that steel temperature at which collapse occurs depends on (1) the stress in the steel, and (2) the type of steel. Even for a continuous slabs and beams with rocker-rollers at the outer supports, due to redistribution of moments a decrease in positive moment means that the positive moment steel can withstand a higher temperature before failure will
occur. Thus the fire endurance of a continuous member is generally significantly longer than that of a simply supported member having the same cover and load intensity.

The IBC 2015 Table 721.1(1) Minimum Protection of Structural Parts Based on Time Periods for Various Noncombustible Insulating Materials provides a prescriptive minimum thickness of insulating material for various fire-resistance periods concrete cover requirement based on fire rating for various structural parts that need to be protected. For example, for an bonded and Bonded or unbonded post-tensioned tendons in pre-stressed concrete a ¾" minimum cover is prescribed if the insulating material used is carbonate, lightweight, sand-lightweight and siliceous aggregate, then for a restrained solid slab members, for a 2 hrs fire rating, while 1 ½” minimum cover would be required for an unrestrained solid slab members. Foot note k states that Interior spans of continuous slabs, beams and girders shall be permitted to be considered restrained. However, for reinforcing and tie rods in floor and roof slabs, carbonate, lightweight and sand-lightweight aggregate concrete would require a ¾” minimum cover with foot note l stating that this requirement is for use with concrete slabs having a comparable fire endurance where members are framed into the structure in such a manner as to provide equivalent performance to that of monolithic concrete construction. Test data (PCI-1973) showed a lower recorded temperature in unbonded prestressing steel, which could be attributed to the fact that the tendons are greased and wrapped (insulation) that might have kept the tendons cooler during fire exposure. Another observation made on fire tests of restrained slabs indicated that slabs with post-tensioned reinforcement behave about the same as reinforced concrete slabs of the same dimensions. Accordingly, the cover for post-tensioned tendons in slabs should be the same as the cover for reinforcing steel in slabs.

ACI 216.1-14, Table 4.3.1- Construction classification: Restrained and unrestrained, classifies cast-in-place floor or roof systems such as beam/slab systems, flat slabs, pan joists and waffle slabs, where the floor or roof system is cast with framing members as restrained structure. Table 4.3.1.1- Minimum cover in concrete floors and roof slabs for restrained condition is prescribed as ¾” for fire resistance 4 hrs or less.

Thus based on the originally intended definition on classification of unrestrained and restrained structural system, and in alignment with ACI 216.1-14, PTI clarifies that the cast-in-place PT floor or roof systems such as beam/slab systems, flat slabs, pan joists and waffle slabs, where the floor or roof system is cast with framing members as restrained structure. Minimum cover to reinforcement in concrete floors and roof slabs for restrained condition is prescribed as ¾” for fire resistance 4 hrs or less.