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6.8.2 Load tests shall be conducted in accordance with the monotonic or cyclic procedures in ACI 437.2.

6.8.3 The design professional is permitted to waive the $\ell_t/180$ deflection criteria in ACI 437.2.

6.8.4 If a member fails a cyclic load test, it shall be permitted to retest the member or structure in accordance with ACI 437.2. It shall be permitted to waive the maximum deflection limit ($\ell_t/180$) in ACI 437.2 that precludes a retest.

6.8.5 Model analysis shall be permitted to supplement calculations.

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R6.8.2 If the strength of the structure being evaluated is limited by the strength of concrete, or the expected failure of the structure is controlled by shear or development of the reinforcement, the sustained load applied using the monotonic test allows greater time for widening and propagation of cracks, creep, and slip of reinforcement compared with the cyclic procedure.

R6.8.3 The $\ell_t/180$ deflection limit was included to provide an upper limit on the deflection of a member during a load test. The deflection limit may be waived by the design professional when the tested member is not damaged by large deflections or when the residual deflection criteria is satisfied.

R6.8.4 ACI 437.2 precludes a retest if the member exceeds a maximum deflection limit of $\ell_t/180$ (Section 6.4.4.2 in ACI 437.2). For consistency with the monotonic testing protocol, this $\ell_t/180$ limit is waived.

R6.8.5 This code permits model analysis to be used to supplement structural analysis and design calculations. Model analysis involves the construction and experimental testing of full or scale models of structure components, assemblages, or systems. Documentation of the model tests and subsequent interpretation should be provided with the related calculations. Model analysis should be performed by an individual having experience in this technique. References are provided in Harris and Sabnis (1999) and White (1970).

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CHAPTER 7—DESIGN OF STRUCTURAL REPAIRS

R7—DESIGN OF STRUCTURAL REPAIRS

7.1—General

R7.1—General

Repair and rehabilitation, as defined in Chapter 2, are processes in which deficiencies and damage in a structure or member are corrected. The methods used to correct deficiencies and damage in structures will be the same for both repair and rehabilitation projects. For the purposes of this chapter, design requirements for repair and rehabilitation can be considered to be equivalent.

Durability requirements for repairs are in Chapter 8.

7.1.1 Repaired structural elements and connections within the work area shall have design strengths at all sections at least equal to the required strengths calculated using the applied factored loads and internal forces in such combinations as required by this code.

R7.1.1 Internal forces include those from externally applied loads and those from imposed deformations, from such actions as prestressing, shrinkage of repair materials, temperature changes, creep, unequal settlement of supports, and listing, leaning and tilting displacements.

7.2—Strength and serviceability

7.2.1 Repaired structures shall be designed to meet the strength requirements of the design-basis code.

R7.2—Strength and serviceability

7.2.2 Repair design and construction procedures shall consider loading, internal forces, and deformations in both the existing and repaired structure during the repair process.

R7.2.2 During the repair process, it may not be possible or practical to relieve existing stresses or deformations. Consideration should be given to the in-place internal forces and deformations present in the structure during the repair and the subsequent internal forces from the design loads that the repaired section will resist. Internal forces and deformations caused by existing loads may be locked in by the repair.

Analysis to evaluate the effects of structural modifications should verify that the strength is adequate and that serviceability conditions are met. As an example, creating a large opening in structural slabs may necessitate cutting reinforcement, which can significantly influence the global behavior of the structure. Supplementary strengthening may be required to address force redistribution that can exceed the existing strength of the affected elements. Slab punching shear strength should be evaluated for openings at the intersection of column strips to verify that the slab is adequate. This is especially critical near corner and edge columns where the slab shear stress is typically highest.

7.2.3 When serviceability issues are identified consistent with Section 6.6, repairs to address serviceability issues shall be considered.

R7.2.3 Adequate stiffness needs to be determined on a project-specific basis and is a function of the structure type, the desired performance of the structure, and loading conditions and use.

7.3—Behavior of repaired systems

7.3.1 Repairs to sections, components, reinforcement, connections of members, or systems shall be designed to be integrated with the existing structure, creating a structural system capable of resisting the design loads by sharing and transferring loads between repaired and existing elements.

R7.3—Behavior of repaired systems

7.3.1.1 Repairs incorporating new members shall be designed to be integrated with the existing structure, creating

R7.3.1.1 Repair of a structure may be achieved by improving the global behavior of the structure by adding

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a structural system capable of resisting the design loads by sharing and transferring loads between new members and existing elements.The effect of the new members on the structure shall be evaluated according to the design-basis code.

7.3.2 Repairs to members shall account for force transfer at the interface between the member and the repair material or repair system. It shall be permitted to use ACI 318 to design the force transfer mechanism between new and existing concrete.

7.3.3 Structural repairs required for strength or stiffness shall maintain composite behavior under service load. The repaired system shall be designed to mitigate potentially dangerous conditions if bond between the repair and the substrate is lost.

7.4—Interface bond of cementitious repair materials

7.4.1 Repair design shall include an analysis to determine the interface shear and tension stresses across bonded interfaces between cementitious repair materials and the existing substrate. The interface analysis shall use factored loads in

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new structural members that act integrally with the existing structural system or improving the behavior of the existing members. The design of the repair should consider connections of new members to the structure. Connections of new members should be designed to transfer design forces between new members and the structure.

Load sharing and load transfer should exist between the structure and the new members so that the assumed load path and force distribution can occur. The effects of adding new members on the global stiffness and force distribution should be considered.

New members may need to be separated from adjacent members to prevent or minimize interaction that may result in damage to adjacent portions of the structure. Transfer of forces between new and existing members should not compromise the performance of the structural system.

R7.3.2 Induced forces on the repaired member are shared between the existing member and the repair material or system. The repair should be designed to allow for transfer of forces between the two components.

The requirements for composite behavior between the repair and the member may vary depending on the type of repair (structural or nonstructural), the performance criteria at service, and the required strength at the ultimate limit states. While certain designs require composite behavior up to an ultimate limit state, others may be limited to service conditions. Composite behavior can be achieved by chemical bonding, mechanical means, or a combination thereof. The design should specify the repair materials and techniques that will develop the level of composite behavior to achieve the intended performance of the repaired member. Specific reference is made to ACI 318-14, Sections 16.4 and 22.9, for force transfer requirements between new and existing concrete. Techniques other than shear-friction may be acceptable.

Design guidelines for bond of fiber-reinforced polymer (FRP) are provided in ACI 440.1R and 440.2R. Design provisions to achieve composite behavior with structural steel sections are provided in the "Specification for Structural Steel Buildings" (ANSI/AISC 360-16, Chapter I).

R7.3.3 Nonstructural repairs intended to improve durability or aesthetics may not require composite behavior under service loads. To prevent potentially dangerous conditions in the event of bond failure in a repair, the repair should encapsulate existing steel reinforcement. Alternately, the repair systems should be designed to provide redundant attachment of the repair material to the existing structure.

R7.4—Interface bond of cementitious repair materials

R7.4.1 The forces acting on the interface between cementitious repair materials and existing substrate can include tension, shear, or a combination of tension and shear depending on repair geometry and the applied loads. The

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addition to internal forces resulting from restrained volume change to calculate the resultant interface stress demand (v_u) from the transfer of tension and shear.

7.4.1.1 Interface shear stress shall be designed based on

$$v_u \leq \phi v_{ni} \quad (7.4.1.1)$$

where v_{ni} is nominal interface shear stress capacity and ϕ is the strength reduction factor determined in accordance with 5.3.2.

7.4.1.2 Testing requirements for interface bond shall be in accordance with Table 7.4.1.2.

Table 7.4.1.2—Testing requirements where v_u is partially or totally resisted by the concrete

v_u	Testing requirements
Less than or equal to 30 psi	Bond integrity testing
Greater than 30 psi	Quantitative bond strength testing unless design satisfies 7.4.5

7.4.2 If v_u does not exceed 30 psi, interface reinforcement shall not be required. Bond integrity testing as specified in the construction documents shall be performed.

7.4.3 If v_u is between 30 psi and 60 psi, interface reinforcement is not required. Quantitative bond strength testing shall be performed to verify performance. Direct tension pull-off tests (ASTM C1583/C1583M) or other similar quantitative test methods shall be specified. The frequency of tests and acceptance criteria shall be specified, but the number of tests on a project shall be at least three (3).

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tensile and shear demand at an interface between a cementitious repair material and the substrate from applied loads and from volume changes that occur as a result of shrinkage or thermal movement can be calculated using principles of structural mechanics, but these calculations can be complex. Guidance on designing the interface for horizontal shear can be found in Chapter 16 of ACI 318-14, Chapter I of ANSI/AISC 360-16, and Bakhsh (2010).

Where the required nominal interface shear stress is lower than 80 psi, and where good surface preparation, placement, repair materials, and curing techniques are employed, satisfactory composite behavior will likely be achieved without interface reinforcement.

R7.4.2 The 30 psi bond stress specified by this code is based on half of a nominal shear stress of 80 psi multiplied by the strength reduction factor in 5.3.2.

A properly prepared substrate is achieved by removing existing deteriorated, damaged, or contaminated concrete. The exposed sound concrete is then roughened and cleaned to allow for adequate bond of a repair material. ICRI Guideline No. 210.3 presents a discussion of achievable tensile bond strengths, suggests a minimum value of 100 psi for less critical applications, and indicates that tensile bond test values less than 175 psi that fail at the bond interface or superficially within the existing concrete substrate may indicate a partially damaged, contaminated, or otherwise inadequate bond surface. BS EN 1504-10 suggests minimum direct tension strengths of 100 psi for nonstructural repair and 175 to 215 psi for structural repairs. Interface reinforcement may be needed if sufficient interface capacity cannot be achieved through bond.

Bond integrity testing can consist of various nondestructive qualitative test methods such as sounding in accordance with ASTM D4580/D4580M, ground-penetrating radar or impact-echo described in ACI 228.2R or ICRI Guideline No. 210.4.

R7.4.3 The 60-psi bond stress is based on a nominal shear stress of 80 psi multiplied by the strength reduction in 5.3.2

On most concrete repair projects, testing to verify the bond of cementitious repair materials to the substrate is recommended as part of a quality assurance program. Quantitative bond strength testing is required when the bond stress exceeds 30 psi and interface reinforcement is not provided. ICRI Guideline No. 210.3 provides guidance on the number

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of tests that should be performed based upon the repair area and acceptance criteria.

Bond capacity has primarily been evaluated using direct tension pull-off tests, as defined in ASTM C1583/C1583M and as described in ICRI Guideline No. 210.3. In some instances, laboratory slant shear tests in accordance with ASTM C882/C882M of cores made in the lab or cores taken from mockups in the field have been used to assist the licensed design professional to make informed design decisions. Slant shear test results typically exceed direct tension pull-off test results, but the slant shear strength is greatly influenced by the compressive stress the test setup introduces across the interface and may not be directly comparable to field conditions. Typically direct shear strengths are larger than direct tension strengths. Comparisons of these tests and other tests, for the purpose of achieving adequate bond is discussed in Bakhsh (2010). It generally is adequate to assume that the repair to substrate bond will resist an interfacial shear equal to the direct tensile pull-off test result.

If failure during direct pull-off testing occurs at the bond line, it may indicate inadequate surface preparation of the concrete substrate or the substrate surface was damaged by the surface preparation method (bruising of the substrate). Modifications to the surface preparation procedures may improve the tensile bond strength. Discussion of proper methods for surface preparation can be found in ACI 546R and ICRI Guideline No. 310.2R.

7.4.4 If v_u exceeds 60 psi, interface reinforcement shall be provided.

7.4.5 If v_u is completely resisted by interface reinforcement, quantitative bond strength testing is not required.

R7.4.5 This provision provides an alternative to bond strength testing.

7.4.6 Interface reinforcement shall be designed in accordance with ACI 318.

R7.4.6 ACI 318 provides design provisions for horizontal shear transfer in composite concrete flexural members. Minimum reinforcement is required between horizontal shear stress of 60 and 375 psi (500 psi multiplied by strength reduction factor of 0.75). Where the required design horizontal shear stress is greater than 375 psi, Section 16.4.4.1 of ACI 318-14 requires design per Section 22.9 of ACI 318-14. For cases where there is a net factored tension across the interface, reinforcement should be provided and designed in accordance with ACI 318.

7.4.7 Construction documents shall specify testing requirements for interface reinforcement in the repair applications.

R7.4.7 Testing to verify the performance of the interface reinforcement to transfer horizontal shear can be performed in accordance with the recommendations contained in ACI 355.2 and 355.4. Specific requirements for testing of ties should be included in a quality assurance plan.

Direct tension testing of post-installed interface reinforcement is recommended to provide verification of the installation. Guidance for determining the number of tests and acceptance criteria of the direct tension testing is similar to principles used in developing direct tension pull-off testing requirements described in ICRI Guideline No. 210.3.

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7.5—Materials

7.5.1 Materials in a structure shall be permitted to remain if such materials are performing satisfactorily.

7.5.2 Except as permitted by this code, materials permitted by the current building code for new construction shall be used. Like materials shall be permitted, provided they do not contain hazardous materials or other materials not permitted by the code for new construction.

7.5.3 Alternate materials shall be permitted following approval in accordance with 1.4.

7.5.4 Design of the repair system shall consider the properties and installation of the repair materials and systems. These include, but are not limited to: physical properties of the repair materials, type of application, adhesion, volume stability, thermal movement, durability, corrosion resistance, installation methods, curing requirements, and environmental conditions.

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R7.5—Materials

R7.5.2 Hazardous materials include asbestos or other materials specifically prohibited by the current building code.

R7.5.4 Physical properties of repair materials include mechanical, chemical, and electrical properties. Documentation should be obtained for properties of each repair material. The stated properties should be verified that they satisfy the project requirements. ACI and ICRI provide guidelines for the selection of repair materials (ACI 301, ACI 318, ACI 503R, ACI 503.5R, ACI 503.6R, ACI 506R, ACI 546.3R, ACI 549.1R, ICRI Guideline No. 320.2R, ICRI Guideline No. 320.3R, ICRI Guideline No. 330.1, and ICRI Guideline No. 340.1).

The design of a repair should consider the compatibility of the repair materials with the materials of the existing structure. Compatibility of repair materials and systems include volume stability, bond compatibility and durability, mechanical compatibility, and electrochemical and permeability compatibility. Generally, the intent is to use a repair material or repair system that has physical, mechanical, and other properties that are as close as possible to those of the parent material to provide long-term performance.

Individual repair materials may have different properties yet will perform satisfactorily when combined in a repair system. An example of this is where materials with differing thermal coefficients of expansion may be used, provided that the overall performance of the system is not affected by thermal changes.

Volume stability is often estimated as a change in the linear dimensions of the repair and should be considered in the design of a repair system. Autogenous shrinkage, chemical shrinkage, degree of restraint, environmental conditions, drying shrinkage, creep, thermal changes, moisture absorption, and other factors all affect volume stability. Experience has shown that volume change of repair materials has often been the cause of poor performance of repairs. Properties of repair materials should be selected considering volume stability relative to the volume stability of the existing concrete in order to reduce the probability of cracking caused by relative volume changes.

Volume stability is discussed in ACI 209R, ACI 209.1R, ACI 546.3R, and ICRI Guideline No. 320.2R.

Repair materials such as portland-cement concrete, portland-cement mortar, polymer-cement concrete, polymer concrete, shotcrete, fiber-reinforced concrete, resin-based materials, and similar products are commonly used. Repair materials might not necessarily contain portland cement, but should be selected to achieve the necessary service, strength, and durability requirements.

The selection of reinforcement material should consider the durability, performance at elevated temperatures, and ductility. Electrical and chemical reactivity between the reinforcement, the repair material, and the existing reinforcement should also be considered.

Refer to ACI 440.1R for internal FRP reinforcement, ACI 440.2R for externally bonded FRP reinforcement, and ICRI Guideline No. 330.1 and ACI SP-66 for steel reinforcement.

Required properties of the repair reinforcement should be specified in the construction documents. Specified reinforcement properties are dependent on the requirements of the repair and may include physio-chemical (for example, glass transition temperature, and coefficient of thermal expansion) as well as mechanical properties (for example, ultimate strength, tensile modulus, and ultimate elongation).

7.6—Design and detailing considerations

7.6.1 Repair design shall be based upon the member conditions in Chapter 6.

7.6.2 *Concrete*—The in-place properties of the concrete, in accordance with Chapter 6, shall be used in the repair design.

7.6.3 *Reinforcement*

7.6.3.1 Reinforcement that is damaged or corroded shall be permitted to remain. The effective cross-sectional area of remaining reinforcement shall be permitted to be used in the repair design in accordance with the design-basis code. The effect of corrosion damage on development of steel reinforcement shall be considered. Where original deformations are no longer effective, reinforcing bars shall be considered as smooth bars.

7.6.3.2 Repair design shall consider the location and detailing of the reinforcement in accordance with the assessment requirements of Chapter 6.

R7.6—Design and detailing considerations

R7.6.2 The extent and cause of deterioration and the concrete strength and quality should be assessed, including compressive strength, chlorides, carbonation, sulfate attack, alkali-silica reaction, physical damage, corrosion-induced spalling, and cracking.

Chloride penetration can cause corrosion that can lead to cracking and spalling. The depth of a spall reduces the effective area of concrete section. Degradation of the concrete affects the concrete compressive strength.

R7.6.3 *Reinforcement*

R7.6.3.1 Repair design should consider the in-place condition of the reinforcement, including the effective cross-sectional area of the reinforcing bars. The effective area is calculated using the remaining effective diameter of the reinforcing bar accounting for the loss of section due to corrosion. Further considerations may also include the location of the corroded areas, loss of confinement, the loss of bond, and the effect of corrosion on member strength. If the structure is fire damaged, steel reinforcement may be annealed, and the yield strength reduced. Refer to ACI 216.1 for additional guidance. Durability requirements related to corroded reinforcement are addressed in 8.4 and ACI 364.1R. CRSI (2014) provides information on older reinforcement systems.

R7.6.3.2 The location and detailing includes the horizontal and vertical positions, orientation, geometry of the reinforcement, development of reinforcement, and the presence of hooks and crossties. Field examination to locate reinforcement may be required. Guidance on evaluation techniques for reinforcement location is provided in Chapter 6.

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7.6.3.3 Both existing and new reinforcement shall be adequately developed. Development length shall be permitted to be calculated based upon development in both the existing concrete and new materials and in accordance with the design-basis code.

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R7.6.3.3 Reinforcement development may be inadequate due to corrosion, mechanical damage, insufficient or loss of concrete cover, delaminated concrete, concrete strength, or other conditions. Equations to calculate the development length have also changed over time and current equations may require longer development lengths than earlier equations. The design of the repair should evaluate the required development length. Detailing of the repair should include the proper development of new reinforcement to achieve the design force. ACI 318 provides development equations and requirements for detailing of steel reinforcement. ACI 369.1 (or ASCE/SEI 41) provides an equation when development length of existing reinforcement does not meet ACI 318. ACI 440.1R and ACI 440.2R provide detailing guidance for internal FRP reinforcement and externally bonded FRP reinforcement, respectively. Additional information can be found in *fib* Bulletin No. 10.

7.6.4 *Prestressed structures*

R7.6.4 *Prestressed structures*

7.6.4.1 The effects of prestressing shall be considered in the repair design.

R7.6.4.1 Requirements for repair of structures with bonded and unbonded prestressing are different. Post-tensioned structures (with bonded and unbonded tendons) are often cast-in-place monolithic structures, whereas pretensioned structures (with bonded strands) are often single-span precast structures. Each system is unique and should be individually considered. The repair of prestressed structures requires a condition assessment of the existing tendons. Repair of unbonded tendons may require tendon detensioning. Guidance for analysis, evaluation methods and repair techniques of unbonded post-tensioned structures is provided in ACI 423.4R, ACI 222.2R, ICRI Guideline No. 210.2, PTI DC80.2-10, and PTI DC 80.3/ICRI 320.6.

7.6.4.2 The effects of modifications to existing structure geometry, damage conditions, loss of prestressing force, and repair sequence shall be considered in the repair design.

R7.6.4.2 Analysis to evaluate the effects of structural modifications should verify that strength is adequate and that all serviceability conditions (for example, deflection limits) are satisfied.

Analysis of prestressed structures is required to evaluate the effect of damaged or severed prestressing reinforcement on structural strength and performance. The effect of a severed bonded tendon is typically localized because the severed tendon is effective after a development length is achieved and the full strength of the tendon is reestablished. For structures with bonded tendons, shoring, if necessary, may only be required locally at the repair area.

Review of grouting quality assurance and supervision documents should be performed to evaluate grouted tendons in advance of any repair or rehabilitation of bonded post-tensioning systems. The presence of voids, moisture in ducts, chlorides and the extent of carbonation in the existing grout need to be identified. Methods for evaluation of chloride-ion content are listed in ASTM C1152M, ASTM C1218, and AASHTO T260. Field evaluation of grout may be required even if documentation of the original construction is available.

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Unbonded tendons are designed to be permanently debonded from the member and often extend over multiple spans. As a result, damage or discontinuity of a tendon at one location will reduce the strength for the entire length of the tendon.

If unbonded tendons are severed, the prestressing force is assumed to be lost for the full length of the tendon. Releasing or cutting tendons may affect multiple spans and may require shoring beyond the area where cutting or releasing of tendons occurs. Adjacent spans may require temporary shoring depending on the number of tendons severed at one time and the applied loads. Analysis based on actual loading at the time of the modification may show shoring to be unnecessary.

Repair and structural modification may require detensioning of prestressing tendons. Unbonded tendons should be detensioned in a controlled manner to ensure performance and safety. Unless not needed based on analysis, unbonded tendons should be reanchored and restressed to restore required structural strength. Cut or damaged unbonded tendons can be restored by splicing or by installing new tendons with anchors at intermediate locations, at the end of the structural member or the edge of any new openings.

The stressing force in a repaired tendon depends on the condition and type of the repaired post-tensioned system and in certain cases this force can be less than the original force if determined to be acceptable by structural analysis. Further discussion of this topic can be found in PTI DC 80.3/ICRI 320.6.

Corrosion on prestressing strands for bonded and unbonded post-tensioned systems may have an effect on strand integrity and strength. Prestressing strands require examination for conditions such as corrosion pitting and hydrogen embrittlement (refer to ICRI Guideline No. 210.2 and ACI 222.2R).

If repairs to prestressed slabs or beams result in increased concrete tensile stress (that is, changing the classification of the prestressed flexural member as defined in ACI 318), impacts of the repair scheme on serviceability should be evaluated.

7.6.4.3 Stresses in remaining section after concrete removal during repair shall not exceed the limits established in the design-basis code.

R7.6.4.3 Removing surface concrete from a prestressed member may cause excessive compressive and tensile stress in the remaining concrete section and may alter secondary forces and moments due to prestressing in indeterminate structures. This condition is more critical for prestressed joists and girders that have a relatively small section and large prestressing force. Slabs are less critical due to the relatively small initial precompression. This change is acceptable as long as durability and strength are addressed as part of the repair design. The impact of removing concrete from a post-tensioned structure is addressed in Scollard and Bartlett (2004). PTI DC 80.3/ICRI 320.6 provides guidance for removing concrete around anchors and splices to prevent catastrophic anchorage failure.

7.6.5 *Anchoring to concrete*—Post-installed anchors shall be designed in accordance with ACI 318 to transfer design

R7.6.5 The design of post-installed anchors requires careful consideration of the loads to be resisted. Anchors

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forces to the substrate considering possible anchor failure modes and the condition of the substrate into which the anchor is installed.

7.6.6 *Repair geometry*—Configuration of repairs shall consider the potential for stress concentrations and cracking in both the existing structure and the repair area.

7.6.7 *Expansion joint materials*—Selection of expansion joint materials shall consider the anticipated movement of the structure and facility maintenance procedures.

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should have adequate strength to transfer design forces across all interfaces and into the existing member. All possible anchor failure modes should be considered to determine the design strength. Anchors should be selected considering the expected concrete substrate cracking condition. For example, post-installed anchors used in the tension zone of concrete members and in structures located in regions of moderate or high seismic hazard should be able to transfer the design seismic forces assuming a cracked concrete condition.

Design of post-installed anchors is provided in ACI 318, which includes provisions that require performance of post-installed anchors in both cracked and uncracked concrete. ACI 355.2 and 355.4 provide the standard required for qualifying post-installed anchors in cracked and uncracked concrete. Specifications for post-installed anchors should include installation, testing, and inspection procedures.

For post-installed expansion or undercut anchors, manufacturer's installation instructions specify procedures for drilling, hole cleaning, installation, torque magnitude, and procedures to engage the anchor.

For adhesive anchors and dowels, hole cleaning and moisture conditions are critically important. Manufacturer's printed installation instructions should specify procedures for drilling, hole cleaning, installation, and the care to be taken until the adhesive has cured.

Testing and inspection of post-installed anchors should be specified in the construction documents. Many building codes require that adhesive anchors be installed under special inspection procedures to ensure that the installation is correctly performed in accordance with the design and manufacturer's procedure. Refer to ACI 318 for specific inspection requirements for post-installed anchors.

R7.6.6 Repair shapes with sharp reentrant corners can cause stress concentrations that may result in cracking. Long, slender (high aspect ratio) repair areas also may result in transverse cracking. The shape of the repair should be considered to reduce stress concentrations and possible cracking. Methods discussed in ICRI Guideline No. 310.1R provide guidance to reduce cracking in concrete repairs including providing a uniform depth of edges and substrate, repair geometry, surface preparation, concrete removal below reinforcement (undercutting) and elimination of feather edge repairs.

R7.6.7 Repairs to expansion joint materials are common, particularly those subjected to snow removal operations.

Design and selection of the expansion joints should consider the total anticipated movement of the expansion joint. Typically, expansion joint capacities listed in manufacturer's literature are based on total movement from minimum installation width to maximum installation width and assume the joint will be installed when the joint is at the midpoint of this movement range. Joints installed in the summer or winter months will experience movement