

Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349R-01)

Reported by ACI Committee 349

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This commentary discusses some of the considerations of ACI Committee 349 in developing "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01)." This information is provided in the commentary because the Code is written as a legal document and cannot therefore present background details or suggestions for carrying out its requirements.

ACI 349-01 is based on "Building Code Requirements for Structural Concrete (ACI 318-95)," except for Chapter 12, which is based on ACI 318-99. This commentary provides discussion of those requirements in ACI 349 that differ from those of ACI 318.

Keywords: admixtures; aggregates; anchorage (structural); beam-column frame; beams (supports); building codes; cements; cold weather construction; columns (supports); combined stress; composite construction (concrete and steel); composite construction (concrete to concrete); compressive strength; concrete construction; concretes; concrete slabs; construction joints; continuity (structural); cover; curing; deep beams; deflection; earthquake resistant structures; edge beams; embedded service ducts; flexural strength; floors; folded plates; footings; formwork (construction); frames; hot weather construction; inspection; joists; loads (forces); load tests (structural); mixing; mixture proportioning; modulus of elasticity; moments; nuclear power plants; nuclear reactor containment; pipe columns; pipes (tubes); placing; precast concrete;

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prestressed concrete; prestressing steels; quality control; reinforced concrete; reinforcing steels; roofs; safety; serviceability; shear strength; shearwalls; shells (structural forms); spans; specifications; splicing; stability standards; strength; stresses; structural analysis; structural design; T-beams; temperature; tensile strength; torsion; walls; water; welded wire fabric.

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INTRODUCTION

This commentary discusses some of the considerations of Committee 349 in developing the provisions contained in “Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01)” hereinafter called the Code. The Code is based on “Building Code Requirements for Reinforced Concrete (ACI 318-95),” which is hereinafter referred to as the Building Code. This commentary discusses provisions in the Code which differ from the Building Code. In preparing ACI 349-01, the committee has followed the text of the Building Code wherever appropriate.

In the following commentary, all references to the Building Code and its commentary are to the 1995 edition unless specifically noted otherwise. Provisions of ACI 318R apply, except replace the term “building official” with “engineer.”

CHAPTER R1—GENERAL REQUIREMENTS

The commentary on ACI 318 is applicable to this chapter except as described below:

R1.1—Scope

The American Concrete Institute “Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01),” referred to as “the Code,” provides minimum requirements for reinforced concrete design or construction in applications where protection against potential radioactive releases is a concern. The scope of the Code has historically addressed new construction; however, this scope has recently been extended to providing recommendations for the analysis, design, construction, testing, and evaluation of new and existing concrete nuclear structures. While the requirements of this Code pertain primarily to new concrete structures, corresponding recommendations for the evaluation of existing concrete nuclear structures are provided in ACI 349.3R-96.^{1,3}

The Owner is to identify nuclear safety related structures and establish which of them are covered by “Code for Concrete Reactor Vessels and Containments (ACI 359)” and its latest revisions in lieu of this Code.

Some special structures involve unique problems that are not covered by the Code. While the requirements of this Code pertain primarily to new concrete structures, corresponding recommendations for the evaluation of existing concrete nuclear structures have been developed by the committee separately (ACI 349.3R-96).

The Code is applicable to radioactive waste repository structures; however, considerations of thermal loads, load combinations, and long-term durability should be considered.

R1.2—Drawings, specifications, and calculations

The design of plain concrete is not included in this Code. Details of all contraction joints and isolation joints in reinforced-concrete structures, however, are considered important to the as-built condition of the structure.

Guidelines for the preparation and retention of design documents are covered by ANSI/ASME NQA-1.^{1,1} Any documentation that uniquely reflects the as-built condition of the concrete nuclear structure should be considered for retention as a permanent record for the life of the structure.

Drawings and specifications should be prepared under the direction of a licensed (or registered) engineer competent in

the field of design of concrete structures, who is required to sign these documents signifying his approval. This Code requires that the Owner be responsible for drawings and calculations, but does not preclude him from assigning the function of detailed implementation to others.

R1.2.3 Whenever the Building Code uses the term “building official,” it shall be replaced (when using ACI 349R) with “engineer.”

R1.3—Inspection and record keeping

This Code requires that the Owner be responsible for inspection but does not explicitly preclude him from assigning the function of detailed implementation to others.

Inspection personnel should be qualified by the Owner. ANSI/ASME NQA-1^{1,1} or ACI 359 Appendix VII may be used to qualify inspectors.

Requirements for the retention of inspection records should follow ANSI/ASME NQA-1.^{1,1}

R1.4—Approval of special systems of design or construction

New methods of design, new materials, and new uses of materials must undergo a period of development before being specifically covered in a code. Hence, good systems or components might be excluded from use by implication if means were not available to obtain acceptance. This section permits proponents to submit data substantiating the adequacy of their system or component to the Regulatory Authority, which presently is the United States Nuclear Regulatory Commission (USNRC) in the U.S.

R1.5—Quality assurance program

Title 10 CFR 50, Appendix B, requires that the Owner have a quality assurance program approved by the Regulatory Authority and states that the Owner is responsible for the establishment and execution of programs developed by his engineers, construction contractors, and suppliers. More detailed requirements for development and implementation of a quality assurance program are contained in ANSI/ASME NQA-1^{1,1} and ANSI/ASME NQA-2.^{1,2}

References

- 1.1. American National Standards Institute (ANSI) and American Society of Mechanical Engineers (ASME), “Quality Assurance Requirements for Nuclear Facilities,” NQA-1, 1989.
- 1.2 ANSI/ASME, “Quality Assurance Requirements for Nuclear Facility Applications,” NQA-2, 1989.
- 1.3 ACI Committee 349, “Evaluation of Existing Nuclear Safety Related Concrete Structures (ACI 349.3R-96),” American Concrete Institute, Farmington Hills, Mich., 1996, 19 pp.

CHAPTER R2—DEFINITIONS

Definitions in this Code are the same as those in the Building Code (ACI 318) except for a few that are added or modified to meet the structures and materials concerned in this Code.

The definitions that differ from or are not listed in ACI 318 are:

- Cementitious material
- Creep
- Embedment
- Engineer
- Evaluation

Effective prestress
 Load, dead
 Load, live
 Load, sustained
 Massive concrete
 Operating basis earthquake (OBE)
 Operating basis wind
 Owner
 Regulatory Authority
 Safe shutdown earthquake (SSE)
 Shrinkage
 Stress relaxation
 Tendon
 Unbonded tendons
 Wall

In the definition of the term “engineer,” the phrase “other documents” is used. “Other documents” include quality assurance plans, design guidelines, and other similar documents for which the engineer is responsible. The user of ASCE 7-95 for ACI 349 applications should be warned of the differences, such as the need to recreate Table 6.1 for 100 year recurrence using Table C6.5 and its commentary.

CHAPTER R3—MATERIALS

The commentary on ACI 318 is applicable to this chapter except as described below:

R3.1—Tests of materials

R3.1.3 The Owner shall designate the period of retention for all records of tests of materials and of concrete used in concrete nuclear structures. Typically, any documentation that uniquely describes tests of materials and of concrete used in concrete nuclear structures should be retained for the life of the plant.

R3.2—Cements

R3.2.1 Adequate performance of expansive cements should be demonstrated for the particular application before selecting such materials, such as in areas of high temperature and irradiation. Additionally, the user should address potential adverse reactions with dissimilar metals that can exist in certain expansive hydraulic cements.

R3.2.3 The requirement for receipt of certified 7-day mill test materials reports in advance of use is imposed to inform the materials engineer of changes in cement strength to provide guidance in altering concrete mixtures when significant changes in strength occur. Such alterations can both achieve increased assurance against low strengths and reduce the standard deviation of strengths, providing a means of optimizing the cement contents and reducing the heat of hydration effects in these relatively massive structures.

R3.3—Aggregates

R3.3.1 The reference to lightweight aggregates has been deleted. The minimum thickness of most concrete members in nuclear plant construction is based on shielding requirements that are dependent on the density of the concrete. Lightweight aggregates would require larger minimum thicknesses because the sizing of columns and spacing of walls, is usually controlled by live load, seismic, or shielding requirements and

not floor dead loads. There appears to be no advantage in using lightweight aggregates in nuclear structures.

R3.3.3 Minimum testing requirements are specified to assure aggregate quality.

R3.3.3.3 Anytime there is a basic change in aggregate source, the aggregate should be tested to determine suitability for use. Retests for soundness (ASTM C 88), abrasion resistance (ASTM C 131), and potential reactivity (ASTM C 289) are required because they are not included in the routine testing of aggregates.

R3.4—Water

Guidance for water quality may be obtained from Subparagraph CC-2223 of ACI 359-92.

R3.5—Steel reinforcement

R3.5.3 Zinc used in the galvanizing process may negatively react with alkaline materials commonly found in concrete. In addition, potential galvanic corrosion with other embedded metals, as well as hydrogen generation and potential for hydrogen embrittlement suggest that such coatings may be detrimental. Research conducted by Sergi^{3,2} concluded that zinc coatings provide little value in providing long-term protection of reinforcing steel, and cautionary statements in ACI Report 201.2R-92^{3,3} support this position. These industry concerns have prompted the ACI Committee 349 to prohibit use of zinc coatings on reinforcing steel in nuclear safety related structures until adequate data justifying its use can be reviewed.

R3.5.3.1 and **R3.5.4.1** Use of rail and axle steel reinforcing bars is excluded because of inadequate traceability.

R3.5.3.2 Testing requirements for reinforcing steel are given to conform to regulatory guides. A minimum of one tensile test for each 50 tons of each bar size produced from each heat of steel shall be used, whichever governs.

R3.5.3.3, R3.5.3.5, R3.5.3.6, R3.5.3.7, and R3.5.4.2 To limit the sizes of cracks in the relatively massive concrete sections, design yield strengths are limited to a maximum of 60,000 psi for deformed bar reinforcement. For further comments on the 60,000 psi reinforcement limit, refer to R.9.4. For further comments on the design yield strength limitation, refer to R.9.4.

R3.5.3.8 Current industry studies have not quantified the long-term durability of epoxy coatings in areas of elevated temperature and irradiation. Degradation of the epoxy coating, under certain environmental conditions, may adversely affect reinforcing steel performance and anchorage, resulting in splitting effects. Adequate performance of the epoxy-coated reinforcement and concrete should be demonstrated for the particular environmental application before selecting epoxy-coated reinforcement.

R3.6—Admixtures

R3.6.7 Adequate performance of ground-granulated blast-furnace slag should be demonstrated for the particular application before selecting such materials, such as in areas of high temperature and irradiation.

R3.6.10 For quality assurance, all admixtures should be fully tested and verified to be in compliance with the referenced ASTM test methods prior to use.

R3.7—Storage and identification of materials

R3.7.1 Expanded emphasis on the protection and traceability of materials in storage is given to assure that the proper materials are used and to minimize deterioration of these materials during storage.

Certified Material Test Reports (MTR) provided for concrete material, reinforcing system material, prestressing system material, and welding and brazing material, should include the following:

(a) Certified reports of the actual results of all required chemical analyses, physical tests, mechanical tests, examinations (including radiographic film), repairs, and heat treatments (including times and temperatures) performed on the material.

(b) A statement listing any chemical analyses, tests, examinations, and heat treatment required by the material specification, which were not performed.

(c) A statement giving the manner in which the material is identified, including a specific marking.

R3.7.2 Concrete materials should be handled and stored in accordance with Chapter 2 of ACI 304R^{3.1}

R3.7.4 To prevent detrimental corrosion, prestressing systems should normally be stored in a completely enclosed building.

References

3.1. ACI Committee 304, "Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI 304R-89)," American Concrete Institute, Farmington Hills, Mich., 1989, 49 pp. Also, *ACI Manual of Concrete Practice*.

3.2. Sergi, G.; Short, N.; and Page, C., "Corrosion of Galvanized and Galvannealed Steel in Solutions of pH 9.0 to 14.0," National Association of Corrosion Engineers, Nov. 1985.

3.3. ACI Committee 201, "Guide to Durable Concrete (ACI 201.2R-92)," *ACI Materials Journal*, V. 88, No. 5, Sept.-Oct. 1991, pp. 544-582. Also, *ACI Manual of Concrete Practice*.

CHAPTER R4—DURABILITY REQUIREMENTS

The commentary on ACI 318-95 is applicable to this chapter. Certain sections of the Commentary in ACI 318R-95, Chapter 4, have been repeated in this Commentary chapter to maintain correspondence with footnote numbers in the References section. All references to lightweight aggregate concrete have been deleted.

R4.2—Freezing and thawing exposures

In 4.2 on freezing and thawing exposures, the quantity of fly ash and other pozzolans used in the calculation of the water-cementitious materials ratio is subject to the percentage limits in 4.2.3.

R4.2.1—A table of required air contents for frost-resistant concrete is included in the Code, based on "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete" (ACI 211.1).^{4.3} Values are provided for both severe and moderate exposures depending on the exposure to moisture or deicing salts. Entrained air will not protect concrete containing coarse aggregates that undergo disruptive volume changes when frozen in a saturated condition. In Table 4.2.1, a severe exposure is where the concrete in a cold climate may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, parking garages, and water tanks. A moderate exposure is where the concrete in a cold climate will be only occasionally exposed to moisture prior to freezing, and where no deicing salts are used.

Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil. Section 4.2.1 permits 1% lower air content for concrete with f'_c greater than 5000 psi. Such high-strength concretes will have lower water-cementitious materials ratios and porosity and, therefore, improved frost resistance. References to lightweight aggregates and lightweight concrete have been omitted from these Code and Commentary discussions.

R4.2.2 For normalweight aggregate concrete, use of both minimum strength and maximum water-cementitious materials ratio provide additional assurance that a high-quality cement paste will be obtained.

Table 4.2.2 and Code Section 4.2.3 establish limitations on the amount of fly ash and other pozzolans that can be included in the calculation of water-cementitious materials ratios for concrete exposed to deicing chemicals.^{4.4-4.6} Recent research has demonstrated that the use of fly ash and silica fume produce concrete with a finer pore structure and, therefore, lower permeability.^{4.7-4.9} For concretes exposed to deicing salts, the inclusion of maximum percentages of fly ash in the calculation of water-cementitious materials ratio is considered preferable to inclusion of minimum cement content required by the Code.

R4.2.3 The limitations of this section apply only to concrete exposed to deicing chemicals and are intended to provide protection against deicer scaling in the presence of freezing and thawing.

R4.3—Sulfate exposures

The commentary for this section on ACI 318-95 is applicable to this section. All references to lightweight aggregate concrete have been deleted. Further, in 4.3 for sulfate exposures, the pozzolan should be Class F by ASTM C 618,^{4.1} or have been tested by ASTM 1012^{4.2} or determined by service record to improve sulfate resistance.

R4.4—Corrosion protection of reinforcement

Additional information on the effects of chlorides on the corrosion of reinforcing steel is given in "Guide to Durable Concrete" reported by ACI Committee 201^{4.10} and "Corrosion of Metals in Concrete" reported by ACI Committee 222.^{4.11} Test procedures must conform to those given in ASTM C 1218. The FHWA test referenced^{4.15} is also applicable for determining chloride ion content in concrete. An initial evaluation may be obtained by testing individual concrete ingredients for total chloride ion content. If total chloride ion content, calculated on the basis of concrete proportions, exceeds those permitted in Table 4.4.1, it may be necessary to test samples of the hardened concrete for water soluble chloride ion content described in the guide. Some of the total chloride ions present in the ingredients will either be insoluble or will react with the cement during hydration and become insoluble under the test procedures described.

When concretes are tested for soluble chloride ion content the tests should be made at an age of 28 to 42 days. The limits in Table 4.4.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.

The chloride ion limits in Table 4.4.1 differ from those recommended in ACI 318, ACI 201.2R, and ACI 222R. The

limits for reinforced and prestressed concrete of 0.06 and 0.15%, respectively, represent the lowest value presented in these documents. For simplicity and to reflect the more critical nature of safety related structures, more restrictive limits have been adopted by the committee.

R4.4.2 When concretes are exposed to external sources of chlorides, the water-cementitious materials ratio and specified compressive strength f'_c of 4.2.2 are the minimum requirements that must be considered. The designer should evaluate conditions in structures where chlorides may be applied or in structures near seawater. Epoxy-coated bars or cover greater than the minimum required in 7.7 may be desirable. Use of fly ash meeting ASTM C 618 and increased levels of specified strength provide increased protection. Silica fume, conforming to ASTM C 1240, when combined with an appropriate high-range water reducer (ASTM C 494, Types F and G, or ASTM C 1017), can also provide additional protection.^{4,13} Performance tests for chloride permeability by AASHTO T 277^{4,14} or ASTM C 1202 of concrete mixtures prior to use will also provide additional assurance.

References

- 4.1. Dikeou, J. T., "Fly Ash Increases Resistance of Concrete to Sulfate Attack," *Research Report* No. C-1224, Concrete and Structures Branch, Division of Research, U.S. Bureau of Reclamation, Jan. 1967, 25 pp.
- 4.2. ASTM C 1012-89, "Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution," *ASTM Book of Standards*, Part 04.01, ASTM, West Conshohocken, Pa., p. 5.
- 4.3. ACI Committee 211, "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete (ACI 211.1-91)," American Concrete Institute, Farmington Hills, Mich., 1991, 38 pp. Also, *ACI Manual of Concrete Practice*, Part 1.
- 4.4. Drahushak-Crow, Roselle, "Freeze-Thaw Durability of Fly Ash Concrete," *EPRI Proceedings*, Eighth International Ash Utilization Symposium, V. 2, Oct. 1987, p. 37-1.
- 4.5. Sivasundaram, V.; Carrette, G. G.; and Malhotra, V. M., "Properties of Concrete Incorporating Low Quantity of Cement and High Volumes of Low-Calcium Fly Ash," *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, SP-114, V. 1, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, Mich., 1989, pp. 45-71.
- 4.6. Whiting, D., "Deicer Scaling and Resistance of Lean Concrete Containing Fly Ash," *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, SP-114, V. 1, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, Mich., 1989, pp. 349-372.
- 4.7. Rosenberg, A., and Hanson, C. M., "Mechanisms of Corrosion of Steel in Concrete," *Materials Science in Concrete I*, American Ceramic Society, Westerville, Ohio, 1989, p. 285.
- 4.8. Berry, E. E., and Malhotra, V. M., *Fly Ash in Concrete*, CANMET, Ottawa, 1985.
- 4.9. Li, S., and Roy, D. M., "Investigation of Relations between Porosity, Pore Structure and CL Diffusion of Fly Ash and Blended Cement Pastes," *Cement and Concrete Research*, V. 16, No. 5, Sept. 1986, pp. 749-759.
- 4.10. ACI Committee 201, "Guide to Durable Concrete (ACI 201.2R-92)," *ACI Materials Journal*, V. 88, No. 5, Sept.-Oct. 1991, 38 pp. Also, *ACI Manual of Concrete Practice*, Part 1.
- 4.11. ACI Committee 222, "Corrosion of Metals in Concrete (ACI 222R-89)," American Concrete Institute, Farmington Hills, Mich., 1989, 30 pp. Also, *ACI Manual of Concrete Practice*, Part 1.
- 4.12. ASTM C 1218.
- 4.13. Ozyildirim, C., and Halstead, W., "Resistance to Chloride Ion Penetration of Concretes Containing Fly Ash, Silica Fume, or Slag," *Permeability of Concrete*, SP-108, D. Whiting and A. Walitt, eds., American Concrete Institute, Farmington Hills, Mich., 1988, pp. 35-61.
- 4.14. AASHTO T 277-83, "Rapid Determination of the Chloride Permeability of Concrete," American Association of State Highway and Transportation Officials, Washington, D.C.
- 4.15. FHWA-RD-77-85, "Sampling and Testing for Chloride Ion in Concrete," Federal Highway Administration, Washington, D.C.

CHAPTER R5—CONCRETE QUALITY, MIXING, AND PLACING

The commentary on ACI 318 is applicable to this chapter except as described below. Certain sections of the Commentary in ACI 318R-95, Chapter 5, have been repeated in this Commentary chapter to maintain correspondence with footnote numbers in the References section.

R5.0—Notation

The splitting tensile strength requirements have been eliminated since they only apply to lightweight aggregates, which also have been eliminated.

R5.1—General

R5.1.3 Concrete members 24 in. or more thick will retain sufficient moisture throughout the first 12 months to assure continuous curing and hydration of the major portion of the cross section. These large members rarely receive full service loads for many months and, therefore, the test age designated for the determination of compliance with the specified strength may be later than the usual 28 days. The use of such later age strength requirements may permit the use of a lower cement content and, therefore, help limit or control temperature rise due to hydration and the danger of cracking that may occur as these large members cool to ambient temperature levels. The need to control early temperature rise increases in proportion to the minimum thickness of the section.

R5.1.4 Splitting tensile strength tests are associated with lightweight aggregate concrete of ACI 318-95, which has been omitted from discussion in this Code and commentary.

R5.1.5 See R1.2.1.

R5.4—Proportioning by water-cementitious materials ratio

Figure R5.3 is a flow chart outlining the mixture selection and documentation procedure.

R5.4.1 Proportioning mixtures by water-cementitious materials ratio is deleted as an option because the size of the concrete operation warrants trial mixture investigation, and the resulting higher cement contents with use of the table is inconsistent with the need to control hydration heat in these structures.

R5.6—Evaluation and acceptance of concrete

R5.6.1 The frequency of testing has little influence on the accuracy of calculating standard deviations after 25 or more tests of one mixture have been performed on a given class of concrete. It, therefore, has little effect on the level of concrete production required to assure strength requirements of Section 5.3.3. When the calculated standard deviation for 30 tests indicates better than normal control, then the frequency of tests has even less effect on the predictability of the strengths. The reduced testing frequency is, therefore, given to reward good control and lessen the requirements where lower concrete strength allowables are used to reduce hydration heat in the thick concrete members. In such areas, concrete strength generally has little effect on design strength.

R5.6.3.1 Field-cured cylinders do not represent the in-place strength of typical concrete members used in nuclear

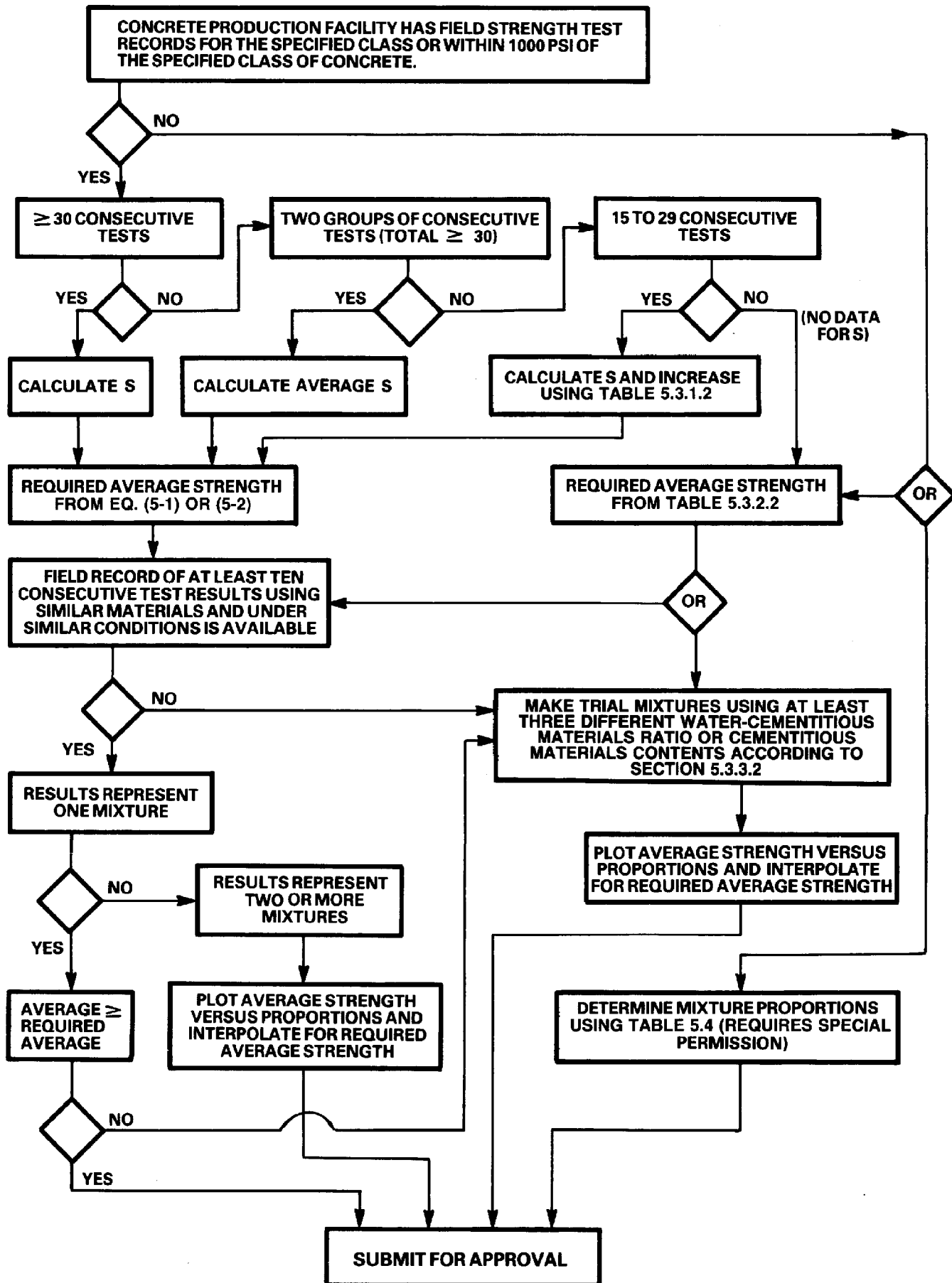


Fig. R5.3—Flowchart for selection and documentation of concrete proportions.

plant construction, particularly during the first 24 h following placement, because of the effects of hydration heat. The in-place strengths may be several times that of the field-cured cylinders, particularly during cold weather placements. Nondestructive testing, using methods such as sonic methods or other similar techniques, may be used by the engineer to better understand the in-place strength and concrete quality in the structure.

R5.6.4—Investigation of low-strength test results

Instructions are provided concerning the procedure to be followed when strength tests have failed to meet the specified acceptance criteria. For obvious reasons, these instructions cannot be dogmatic. The engineer must apply judgment as to the true significance of low test results and whether they indicate need for concern. If further investigation is deemed necessary, such investigation may include nondestructive tests, or in extreme cases, strength tests of cores taken from the structure.

Nondestructive tests of the concrete in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity, or pullout may be useful in determining whether or not a portion of the structure actually contains low-strength concrete. Such tests are of value primarily for comparisons within the same job rather than as quantitative measures of strength. For cores, if required, conservatively safe acceptance criteria are provided that should assure structural adequacy for virtually any type of construction.^{5.4-5.7} Lower strength may, of course, be tolerated under many circumstances, but this again becomes a matter of judgment on the part of the engineer. When the core tests fail to provide assurance of structural adequacy, it may be practical, particularly in the case of floor or roof systems, for the engineer to require a load test (Chapter 20). Before load testing, if time and conditions permit, an effort may be made to improve the strength of the concrete in place by supplemental wet curing. Effectiveness of such a treatment must be verified by further strength evaluation using procedures previously discussed.

It should be noted that core tests having an average of 85% of the specified strength are entirely realistic. To expect core tests to be equal to f'_c is not realistic because differences in the size of specimens, conditions of obtaining samples, and procedures for curing do not permit equal values to be obtained.

The Code, as stated, concerns itself with assuring structural safety, and the instructions in 5.6 are aimed at that objective. It is not the function of the Code to assign responsibility for strength deficiencies, whether or not they are such as to require corrective measures.

Under the requirements of this section, cores taken to confirm structural adequacy will usually be taken at ages later than those specified for determination of f'_c .

R5.7—Preparation of equipment and place of deposit

Code Sections 5.7.1(f) and 5.7.1(g) requires free water deeper than 1/4 in. in hollows be removed from the forms. It is not necessary that all water be removed. The requirement for specifying the method for cleaning joints in construction

specifications was added to ensure that only an approved method or methods of joint cleanup would be used.

R5.9—Conveying

R5.9.3 Conveyance of concrete by pumping through aluminum pipe is not permitted because hydrogen gas bubbles are produced by the reaction of aluminum abraded from the pipe with the alkalis in the concrete. These gas bubbles are retained in the hardened concrete and reduce concrete strength.^{5.1, 5.2}

R5.10—Depositing

R5.10.4 The wording has been changed from ACI 318 to be more specific and to exclude the use of retempered concrete.

R5.10.9 Section 5.4.4 of the 1971 Building Code contained the requirement that “where conditions make consolidation difficult or where reinforcement is congested, batches of mortar containing the same proportions of cement, sand, and water as used in the concrete, shall first be deposited in the forms to a depth of at least 1 in.” The requirement has been deleted from the 1977 Code because the conditions for which it was applicable could not be defined precisely enough to justify its inclusion as a code requirement. The practice, however, has merit and is retained since the 1980 edition of ACI 349. The use of reportioned batches will aid in preventing honeycomb and poor bonding of the concrete with the reinforcement. The reportioned concrete or mortar should be placed immediately before depositing the concrete containing larger aggregate and must be plastic (neither stiff nor fluid) when the concrete is placed.

R5.11—Curing

R5.11.4 In many areas, protective coatings are required, and no curing procedure should be used that may be incompatible with those coatings.

R5.11.5 The method or methods of curing should not be left to the discretion of the contractor or to arbitration. Acceptable methods should be clearly stated in the construction specifications.

R5.12—Cold weather requirements

Recommendations for cold weather concreting are given in detail in “Cold Weather Concreting” reported by ACI Committee 306.^{5.3} (Present requirements and methods for producing satisfactory concrete during cold weather.)

R5.13—Hot weather requirements

The method or methods of curing should not be left to the discretion of the contractor or to arbitration. Acceptable methods should be clearly stated in the construction specifications.

The designer should also consult ACI Committee 207 report “Effect of Restraint, Volume Change, and Reinforcement of Cracking of Massive Concrete”^{5.5} for additional information on crack control.

References

5.1. Newlon, H., Jr., and Ozol, A., “Delayed Expansion of Concrete Delivered by Pumping Through Aluminum Pipe Line,” *Concrete Case Study No. 20*, Virginia Highway Research Council, Oct. 1969, 39 pp.

5.2. Fowler, E. E., and Holmgren, E. F., “Expansion of Concrete Pumped Through Aluminum Pipeline,” *ACI JOURNAL*, *Proceedings*, Vol. 69, June 1971, pp. 950-953; and Discussion, *ACI JOURNAL*, *Proceedings*, Vol. 69, June 1972, pp. 357-360.

5.3. ACI Committee 306, "Cold Weather Concreting (ACI 306R-88)," American Concrete Institute, Farmington Hills, Mich. Also, *ACI Manual of Concrete Practice*, Part II.

5.4. ACI Committee 305, "Hot Weather Concreting (ACI 305R-89)," American Concrete Institute, Farmington Hills, Mich. Also, *ACI Manual of Concrete Practice*, Part II.

5.5. ACI Committee 207, "Effect of Restraint, Volume Change and Reinforcement on Cracking of Massive Concrete (ACI 207.2R-73) (Reapproved 1986)," *ACI JOURNAL*, *Proceedings* V. 70, Jul. 1973, pp. 445-470. Also *ACI Manual of Concrete Practice*.

CHAPTER R6—FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS

The commentary on ACI 318 is applicable to this chapter except as described below:

R6.1—Design of formwork

R6.1.7 The use of steel liners as formwork requires considerations in addition to tolerances.

R6.1.8 and R6.2.5 Form release agents and surface effects of wood type used in the formwork must be compatible with coating systems to assure a durable coating system.

R6.2—Removal of forms, shores, and reshoring

Provisions of ACI 318R apply, except replace the term "building official" with "engineer" whenever it occurs.

R6.3—Conduits and pipes embedded in concrete

R6.3.8 The testing requirements were changed to ensure that there would be no conflict in code applications for the many different piping systems used in nuclear plant construction.

R6.3.9 If an entire pipe system cannot be tested as a unit, a minimum concrete strength of 1000 psi is required for concrete encasing a previously tested length of pipe that has to be filled with liquid, gas, or vapor for testing an added length of the pipe in accordance with Section 6.3.8.

R6.3.13 Piping systems that are embedded in concrete will be inaccessible for normal maintenance. Certain mechanical connections, such as a screwed pipe connection, may not be as reliable as a sealed welded pipe connection and are therefore prohibited from use.

R6.3.14 The design limits specified for concrete temperatures are those in general use in nuclear structures. See, for example, paragraph CC-3440 of ACI 359. Provision is made for the use of higher temperatures provided the effects of higher temperatures on concrete strengths have been established by tests and these effects are incorporated into the design.

R6.4—Construction joints

R6.4.3 Indiscriminate location of construction joints should not be allowed in these structures. The engineer is responsible for all construction joint locations.

CHAPTER 7—DETAILS OF REINFORCEMENT

The commentary on ACI 318 is applicable to this chapter except as described below.

R7.4—Surface conditions of reinforcement

R7.4.1 For an explanation of the service conditions (i.e. temperature and radiation) under which an epoxy coating

may be qualified, see the Section R3.5.3.8 of the commentary in this Code.

R7.4.3 All tendons must be protected against deleterious corrosion from the time they are manufactured to the time they are incorporated in the work (that is, during storage). In the event the tendons are to be grouted (bonded) and coatings or rust inhibitors are used to provide protection against deleterious corrosion, complete removal of the coating or rust inhibitor should be required unless it can be demonstrated that such coatings or inhibitors do not affect the required bond values.

R7.12—Minimum reinforcement

This section has been expanded to include minimum reinforcement for all exposed concrete surfaces. For the purpose of Section 7.12 and as defined in Section 7.12.1, an exposed concrete surface is any concrete surface that is not cast against existing concrete or against rock.

Minimum reinforcement is required to control cracking and to tie the structure together to assure its acting as assumed in the design. Minimum reinforcement is required at each surface in approximately perpendicular directions and the spacing is limited to 18 in. to assist in controlling surface cracking. The section has been expanded by inclusion of the requirements for walls and shells previously identified in Chapters 10, 14, and 19. It also includes requirements for sections having a thickness of 48 in. or greater based on the recommendations made by ACI Committee 207.^{7.1}

R7.12.2 and R7.12.3 If the concrete sections referred to in 7.12.2 and 7.12.3 are classifiable as beams or structural slabs, the minimum reinforcement provisions of Sections 10.5.1 and 10.5.2 (beams), Section 10.5.3 (one-way structural slabs), and Section 13.4.1 (two-way structural slabs) apply in addition to the provisions of Sections 7.12.2 and 7.12.3. Minimum reinforcement required for beams is given directly in Section 10.5.1; whereas that required for structural slabs is given in Section 7.12.5 by way of reference from Section 10.5.3 and Section 13.4.1.

R7.12.3 The reinforcement stress f_s in Section 7.12.3 does not represent the same effects as that in Section 10.6. Section 7.12.3 applies only to massive concrete sections. The quantity $A_{s,min}$ is the minimum amount of reinforcement required to limit the widths of surface shrinkage cracks in these sections. This can be achieved by keeping the stresses in the reinforcement after the concrete cracks well below yield. A value of $0.60 f_y$ was selected for this purpose.

R7.12.4 The application of massive concrete is common in design and construction of concrete nuclear safety structures. The requirements for minimum reinforcement and physical limitation of massive concrete, however, have not been clear to the designers. ACI Committee 207 7.2 has studied and developed much useful information and data in regard to massive concrete structures. It is highly recommended that the designer obtain and study the specific reports by ACI Committee 207 for detailed and up-to-date information.

R7.13—Requirements for structural integrity

The integrity of the overall structure is not affected even if local strengths are exceeded, provided there is no loss of intended function of any safety-related systems as stated in 9.2.6.

References

7.1. ACI Committee 207, "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete (ACI 207.2R-73)," *ACI JOURNAL, Proceedings* V. 70, July 1970, pp. 445-470. Also *ACI Manual of Concrete Practice*.

7.2. ACI Committee 207, "Mass Concrete for Dams and other Massive Structures," *ACI JOURNAL, Proceedings* V. 67, No. 4, Apr. 1973, pp. 273-309.

CHAPTER R8—ANALYSIS AND DESIGN: GENERAL CONSIDERATIONS

R8.0—General

Chapter 8 parallels the general considerations for analysis and design presented in Chapter 8 of ACI 318. Some modifications have been made that reflect particular requirements applicable to concrete nuclear safety structures. Reference to the alternative design method and to the use of lightweight concrete and permanent fillers has not been made in this standard. In addition, the load requirements have been appropriately altered. Chapter 8 of the commentary on ACI 318 should be referenced for concrete nuclear safety structures except as noted herein.

R8.1—Design methods

The strength design method of ACI 318 has been referred to exclusively, deleting all reference to the alternate design method of Section 8.1.2 of ACI 318.

R8.2—Loading

The commentary on ACI 318 is not applicable for concrete nuclear safety structures. Commentary regarding design loads and load combinations is presented in Chapter 9 of this document.

R8.3—Method of analysis

Reference to the alternative design method in the commentary on ACI 318 is not applicable.

R8.3.3(e) This requirement has been added to indicate that approximate analysis coefficients are not appropriate for members with haunches and varying cross sections.

R8.5—Modulus of elasticity

Reference to lightweight concrete in ACI 318 has been deleted.

R8.6—Stiffness

Section 8.6.1 differs from ACI 318 in order to be more specific.

R8.11—Joist construction

Reference to permanent burned clay or concrete tile fillers in ACI 318 has been deleted.

CHAPTER R9—STRENGTH AND SERVICEABILITY REQUIREMENTS

Chapter 9 parallels the requirements for strength and serviceability presented in Chapter 9 of ACI 318. The general requirements and the section on required strength have been completely revised to reflect the requirements regarding loads and load combinations applicable to concrete nuclear safety structures. The maximum specified yield strength of non-prestressed reinforcement f_y has been limited to 60,000 psi. Deflection limitations have been revised. In addition, requirements for the use of lightweight concrete in ACI 318 do not appear in this standard. Chapter 9 of the commentary on

ACI 318 should be referenced for concrete nuclear safety related structures except as noted herein.

R9.0—Notation

In the notation for D , E_o , E_{ss} , R_a , and R_o , "equipment" is to include all items that are permanently attached to the concrete structure, such as cable trays and conduits.

R9.1—General

Even though Section 9.1 of ACI 318 has been completely revised, the commentary on ACI 318 for this section is generally applicable. The loads appropriate for nuclear safety structures are defined in Section 9.0 and categorized in Section 9.1 as follows: normal loads, severe environmental loads, extreme environmental loads, and abnormal loads.

Piping and equipment reactions to be included as R_o are those reactions produced by normal operating temperatures acting on the piping system or equipment; piping reactions generated by normal operation flow transients; and any other reactions occurring during normal operation or shutdown. Note that dead load and earthquake reactions are not included in R_o .

When considering concentrated impactive loads such as those associated with postulated rupture of high energy pipes or missiles, local section strength capacities may be exceeded. Under these concentrated loads, elastoplastic behavior may be assumed with appropriate ductility ratios provided resulting deformation will not result in loss of function of any safety related system.

Live loads associated with elevators, machinery, and craneways should be increased to allow for impact in accordance with the recommendations of ASCE 7.

The discharge of safety relief valves into a suppression pool generates loads that are unique to BWR power plant structures. Specific classification of these loads is not given by the Code at this time due to ongoing efforts by the industry to quantify them.

R9.2—Required strength

Section 9.2 of ACI 318 and the commentary on this section have been completely revised for concrete nuclear safety related structures. The load combinations and load factors of this section reflect consideration of the likelihood of individual and combined-event occurrences as well as possible excess load effects such as variations in loads, assumptions in the structural analysis, and simplifications in the calculations.

Load Combinations 1, 2, and 3 involve normal loads and normal loads in combination with severe environmental loads. Similar to ACI 318, this standard uses load factors of 1.4 and 1.7 for dead and live loads, respectively, in these load combinations. In addition, a load factor of 1.4 was assigned to lateral and vertical liquid pressure, and a load factor of 1.7 was assigned to normal-operation pipe reactions, lateral earth pressure, and the operating basis wind loads. Because the plant could remain operational when subjected to the effects of severe environmental loads such as the operating basis earthquake or operating basis wind, these loads are treated the same as other operating loads. The load factor for the earthquake is 1.7 rather than 1.87 required by ACI 318. ACI 318 assumes a design method similar to that of the Uniform Building Code.

Nuclear safety related concrete structures are designed to significantly more conservative seismic criteria, and the resulting loads are less likely to be exceeded. Hence, the additional factor of 1.1 is considered unnecessary.

Unit load factors are used in Load Combinations 4 and 5 involving extreme environmental loads. This recognizes that the loads caused by the safe shutdown earthquake or the design basis tornado are extreme and are of very low probability.

Load Combinations 6, 7, and 8 are directed towards abnormal loads in combination with normal, severe, and extreme environmental loads, respectively. Abnormal loads are generated by a postulated high-energy, pipe-break accident. This accident could generate differential pressures, thermal loads, pipe and equipment reactions on supports, pipe rupture reaction forces, jet impingement loads, and missile impact effects. Load Combination 8 has unit load factors on all loads because it represents an extremely unlikely combination of events. Load Combination 6 has a load factor of 1.25 on the pressure because, although this is a very unlikely event, it affects a larger portion of the overall structure than the local conditions included in Combination 8. Load Combination 7 is intermediate between 6 and 8 and is included because it has been specified historically. In Combinations 7 and 8, unit load factor is specified for the concentrated effects of pipe rupture. These events are less probable than a differential pressure loading and produce only localized effects.

The load factors for E_o and P_a in Combinations 6 and 7 are lower than the corresponding factors in ACI 359. The reasons for this difference are:

(a) The structures do not function as leakage barriers to contain the effects of the loss of coolant accident;

(b) The pressure loading is a one-time-accident loading. For the corresponding one time loadings of both the safe shutdown earthquake and the tornado the load factor is unity; and

(c) The design pressure is larger than the calculated pressure so that a margin of safety is included therein.

The last three Load Combinations, 9, 10, and 11, are similar to Load Combinations 1, 2, and 3. These Combinations, however, consider normal-operation thermal loads, allowing a reduction of 25% in the required strength. This reduction is in recognition of the fact that such thermal loads tend to be self-relieving. A 25% reduction in required strength for load combinations involving thermal loads is also used in ACI 318.

It should be noted that, in applying the load combinations given in Section 9.2.1 of the Code, due regard should be given to sign because the standard direction of loads may on occasion be of opposing direction, thus producing tensile axial loads, negative reactions, or reverse bending. The provisions of Section 9.2.2 consider the effect of differential settlement, creep, and shrinkage where necessary.

When impact due to operating loads is present, impact effects should be considered and impact loads, if any, included with live loads in the various equations for required strength. As the effects of pipe ruptures P_a , T_a , R_a , Y_r , Y_j , and Y_m may not necessarily occur simultaneously, Section 9.2.5 of the Code permits a time-history analysis to account for the time lag between these loads.

R9.2.7 Apart from the extreme environmental loads generated by the safe shutdown earthquake and by the design basis

tornado, other extreme environmental loads may also be required for the plant design. Examples of such loads are those induced by flood, aircraft impact, or an accidental explosion.

These environmental loads should be treated individually in a manner similar to the loads generated by the design basis tornado in determining the required strength according to the equations in Section 9.2.1. Abnormal loads are not considered concurrently with the above extreme environmental loads.

R9.5—Control of deflections

This section contains different deflection requirements than ACI 318. The deflection requirements in ACI 318 are based on comfort levels for human occupancy, cracks in ceiling, and other considerations applicable to residential buildings. The Building Code also states that if nonstructural elements (such as equipment) are attached to the structure, the deflection requirements are to be satisfied by calculation and it is not sufficient just to use the given l/d ratios.

Almost all nuclear safety related concrete structures support nonstructural elements. In addition, because the member sizes used in ACI 349 structures are generally larger than those used in common building structures, it was felt that it is too restrictive to require deflection calculations for all these structural elements.

Table R9.5(a) indicates the deflections that could be expected if no calculations are performed.

Table R9.5(a)—Maximum deflections for unfactored loads

Type	Loading equation number (Section 9.2.1)	Beams	Slabs*
1	Eq. (1), (2), and (3)	$l/400$	$l/320$
2	Eq. (4) and (5)	$l/250$	$l/200$

*For two-way construction l shall be replaced by l_s

Consistent with ACI 349 loading combinations, the table indicates anticipated deflection values for normal, severe environmental, and extreme environmental loading combinations. The Code then states that deflection combinations are to be made only if the requirements given by the manufacturers of the nonstructural elements are more stringent than the anticipated deflections given in Table R9.5(a).

For normal and severe environmental loading conditions, the anticipated deflections are selected to be $l/400$ for beams and ribbed one-way slabs and $l/320$ for two-way slabs and solid one-way slabs. The Building Code also makes a distinction between the appropriate deflection limits for these two groups of structural elements. A more liberal or a more stringent criteria for the anticipated deflection can be chosen, however, the l/d ratio has to be modified accordingly. The deflection limits in the Code will provide a reasonable l/d ratio and at the same time will not require deflection calculations for a large number of structural elements.

The l/d ratios given in the Code were established from:

1. Recommendations by Committee 435, Deflection of Concrete Building Structures; and

2. A review of the minimum member sizes commonly used in ACI 349 structures.

Based on these l/d ratios, the anticipated deflections were computed and given in the table.

In these computations, the following assumptions were made:

1. The stress in the reinforcing steel is $0.6 f_y$, for unfactored service load conditions; and
2. The immediate deflections are multiplied by a factor of 0.5 and 1.5 to obtain the additional long-term deflections due to structural dead load and equipment dead load, respectively. This assumes that the equipment would be placed at least a year after the structure has been placed. Therefore part of the long-term deflection due to structural dead load has already occurred at the time the equipment is placed.

The Code defines the deflections that need to be considered. Immediate deflection is no longer a consideration. Because the minimum l/d ratios are more stringent than ACI 318, the immediate deflection limits of ACI 318 will be satisfied.

The considerations for camber in prestressed concrete members is included in the Code.

A deflection requirement for walls is also included. Because the walls in ACI 349 structures frequently carry axial load and flexure, the walls should satisfy the requirements given for nonprestressed one-way or two-way or composite or prestressed construction, as applicable.

Similar assumptions were made to establish the Building Code deflection requirements. To suit ACI 349 applications, however, slightly different numbers were used. To simplify the deflection computations, a deflection reduction factor γ and moment of inertia modification factor I_g / I_e have been included. These factors can be used to modify the results of the factored load analysis performed for the

strength design of the structure. Reference to ASCE 7 strength reduction factors and load combinations is not applicable to safety-related concrete structures. Thus, a separate analysis for deflection computation is no longer implied.

CHAPTER R10—FLEXURE AND AXIAL LOADS

Chapter 10 is identical to that of ACI 318 except as described below for Sections 10.6.

R10.6—Distribution of flexural reinforcement in beams and one-way slabs

All subsections of Section 10.6 are identical to those of ACI 318 except 10.6.4.

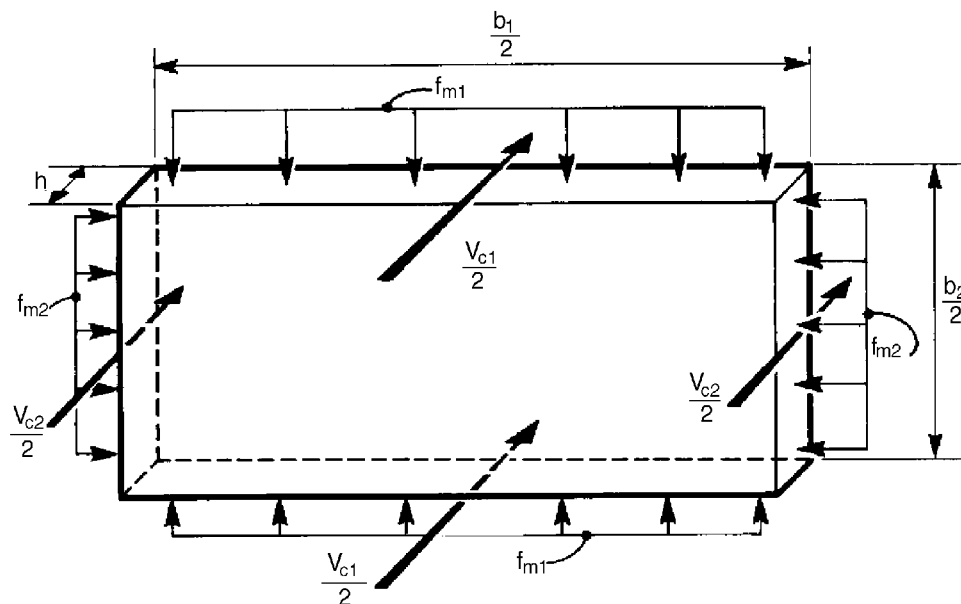
In lieu of stress calculations for f_s , $0.40 f_y$ may be used. This is less than the value of $0.60 f_y$ specified by ACI 318. This difference exists because, unlike ACI 318 structures, the reinforcement design for nuclear safety structures is controlled by load combinations other than those corresponding to a service condition. Consequently, at service loads, the reinforcement stresses for nuclear safety structures are not likely to be as high as those for ACI 318 structures.

CHAPTER R11—SHEAR AND TORSION

The commentary in ACI 318 is applicable to this chapter except as noted herein.

R11.12—Special provisions for slabs and footings

R11.12.2.3 Subsection 11.12.2.3 is added to this section. In concrete nuclear safety structures, slabs or walls may be subject to loads producing punching shear forces that have to be considered in combination with forces in the plane of the slab or wall. The forces in the plane of the



- Concrete shear strengths over perimeters b_1 and b_2 are V_{c1} and V_{c2} , respectively.
- Total concrete shear strength, $V_c = V_{c1} + V_{c2}$.
- f_{m1} and f_{m2} are negative as shown

Fig. R11.1—Concrete shear strength in presence of membrane tensile stresses