Response to Public Comments on Draft Regulatory Guide (DG)-1283
"Safety Related Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments"
Proposed Revision 3 of Regulatory Guide (RG) 1.142

On April 23, 2019 the NRC published a notice in the *Federal Register* (84 FR 16897) that Draft Regulatory Guide, DG-1283 (Proposed Revision 3 of RG 1.142, was available for public comment. The Public Comment period ended on June 24, 2019. The NRC received comments from the organizations listed below. The NRC has combined the comments and NRC staff responses in the following table.

Comments were received from the following:

Ronald LaVera
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ADAMS Accession No. ML19190A126

Anthony Ponko
ADAMS Accession No. ML19165A044
ADAMS Accession No. ML19165A047
ADAMS Accession No. ML19165A232
ADAMS Accession No. ML19165A159

Kyle Gould
Concerned Citizen, New York State
ADAMS Accession No. ML082190536

Adeola Adediran for ACI 349
ADAMS Accession No. ML19176A439

Commenter	Section of DG-1283	Specifc Comments	NRC Resolution
Ronald LaVera	General Comment	**General Comment** (Radiation shielding) Regulatory Guide (RG) 1.142 Revision 2 contains Regulatory Position 2, which states that: "This position emphasizes the need to evaluate concrete structures for their effectiveness as radiation shields, when they are so intended. Some specific guidance for this purpose may be obtained from ANSI/ANS 6.4-1997. This is the current ANSI standard for radiation shielding."	Agree In response to the comment, a regulatory position was added stating that RG 1.69, "Concrete Radiation Shields and Generic Shield Testing for Nuclear Power Plants," should be used when concrete is a radiation shield for concrete structures covered by this RG. Rev. 1 of RG 1.69 endorses with exceptions a more recent version of the ANSI/ANS standard than suggested by the comment, namely ANSI/ANS-6.4-2006, "Nuclear Analysis and Design of Concrete Radiation Shielding for Nuclear Power Plants" (2006).

		This proposed version of RG 1.142 deleted this statement. Structural concrete is frequently used as radiation shielding to limit dose to members of the public, and plant workers, both during normal operation, as well as during design basis events. The concrete, or other structural materials may not be explicitly identified in the FSAR as required radiation shielding material. However, it may be used as part of the analysis used to demonstrate compliance with 10 CFR 20.1301(e), GDC 4, GDC 19, GDC 23, 10 CFR 50.49(d)(3), 10 CFR 50.49(e)(4), 10 CFR 50.34(f)(2)(vii)(viii), (xxviii) 10 CFR 50.67(b)(2)(iii) and 10 CFR 50.44(b)(3). The fact that these structures provide other protective functions of safety related structures systems and components, and protection of members of the public needs to be an integral part of guidance to users provided by the staff. This position should be included in Revision 3. "This position emphasizes the need to evaluate concrete structures for their effectiveness as radiation shields, when they are so intended. Some specific guidance for this purpose may be obtained from ANSI/ANS 6.4-1997. This is the current ANSI standard for radiation shielding."	

Kyle Gould	General Comment	**General Comment** (Call for stronger safety measures) I'm writing to call for stronger safety measures for concrete structures. This includes more reinforcement and more regular inspections of the concrete.	Agree with this comment. The guidance in this Regulatory Guide is intended to foster safety. It is based on industrial consensus standards that reflect many years of experience and experimental data on design, materials, and long term performance of concrete structures. In addition, those standards are complemented with RG positions where the staff deems necessary.
Anthony Ponko	General Comment	**General Comment** (on welding of reinforcing bars) The regulatory guide should provide guidance on the welding of reinforcing bars in Seismic Category I concrete structures. Specifically, that welding of reinforcing bars should comply with paragraph CC-4334 of ASME B&PV Code, Section III, Division 2 (ACI 359-15). This position would align with the guidance provided in NUREG-0800. Seismic Category I concrete structures inside and outside of containment fall within the scope of NUREG- 0800, Sections 3.8.3, Concrete and Steel Internal Structures of Steel or Concrete Containments, and 3.8.4, Other Seismic Category I Structures, respectively. These structures are typically designed and constructed in accordance with ACI 349. NUREG-0800, Section 3.8.4.I.6(B), Revision 4 September 2013 states: If welding of reinforcing bars is used, it should comply with American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code (Code) Section III, Division 2. Any exception to compliance should be supported with adequate justification. Similar guidance is provided in NUREG-0800, Section 3.8.3.I.6(B). This guidance was formerly provided in Regulatory Position C.4 of Regulatory Guide 1.94, Revision 1. These requirements were subsequently incorporated into ASME NQA-2. ACI 349-13 adopts by reference AWS D1.4:2005 for the welding of reinforcing bars. There is a lot of commonality	Agree with this comment. The following was added to Regulatory Position 4: C.4.2.2 For qualification of welding of reinforcing bars, follow the requirements of CC-4334 of American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code Section III, Division 2 (ACI 359-19). Any exception to compliance should be supported with adequate justification. Strength requirement for a full welded splice should be in accordance with Section 12.14.3.4 of ACI 349-13.

		between AWS D1.4 and ASME BP&V Code, Section III, Division 2. In fact, ASME B&PV Code, Section III, Division 2, Mandatory Appendix D2-XIII references AWS D1.4 in some sections. The focus of AWS D1.4, however, is not nuclear safety-related construction. It is a code with a broad range of applicability. When used for a specific application such as the welding of reinforcing bars in Seismic Category I concrete structures, the applicable requirements of AWS D1.4 should be clearly specified and, if necessary, supplemented with other measures. ACI 349, however, does not adequately address the use of AWS D1.4 for the welding of reinforcing bars in Seismic Category I structures. For example, AWS D1.4 does not specify the appropriate types of inspections for a specific application of the code. It would seem reasonable to expect ACI 349 to address this issue, but it does not. This gap can be closed by endorsing ASME BP&V Code, Section III, Division 2 for the welding of reinforcing bar splices in Seismic Category I concrete structures. ASME BP&V Code, Section III, Division 2 requires destructive and nondestructive testing of production splices to verify tensile and weld quality requirements, respectively. These continuing performance tests are comparable to those endorsed for mechanical splices in Regulatory Position 2.2 of DG-1283. There is more potential variability in the quality of welded splices than mechanical splices. The types of inspections performed to verify welded splices meet performance and quality requirements in Seismic Category I concrete structures should at least be equal to those required for mechanical splices. Endorsement of ASME BP&V Code, Section III, Division 2 requirements for the welding of reinforcing bars would also align with NRC staff implementation of NUREG-0800 guidance in	

		the design certification of the Economic Simplified Boiling-Water Reactor (ESBWR) Standard Design. Concerning the use of welded reinforcing bar splices in areas other than containment, NRC staff made the following comment in NUREG-1966, Final Safety Evaluation Report Related to the Certification of the Economic Simplified Boiling-Water Reactor Standard Design: The staff noted that the applicants proposed DCD Tier 2, Section 3.8.4.6 referenced only ACI 349-01 and applicable RGs for splices. SRP Section 3.8.4.I.6 requiresthat the welding of reinforcing bars (splices) comply with the applicable sectionsof ASME Code, Section III, Division 2.The staffs position is that welding of reinforcing bars should comply with all the applicable sections of ASME Code, Section III, Division 2This position applies to all seismic Category I concrete structures inside and outside the containment.	
Anthony Panko	General Comment	**General Comment** (Quality Assurance) The revised regulatory guide should address the quality assurance program requirements for Seismic Category I concrete structures by referencing Regulatory Guide 1.28, Quality Assurance Program Criteria (Design and Construction), Revision 5. ACI 349-13, Commentary Section R1.5-Quality assurance program references ASME NQA-1-2000 for detailed requirements for development and implementation of a quality assurance program. However, this version of NQA-1 is not one that is endorsed in Regulatory Guide 1.28.	Agree. To address the comment on quality assurance, a regulatory position stating that "RG 1.28 should be used for quality assurance in design and construction and for inspection and testing of concrete structures covered by this RG" was added to RG 1.142, Revision 3.

Anthony Panko	General Comment	**General Comment** (Inspecting and Testing) ACI 349-13 does not adequately address the inspection and testing of Seismic Category I concrete structures. The measures provided solely in ACI 349 are inadequate to meet the requirements of Appendix B to 10 CFR Part 50. This was discussed in Regulatory Guide 1.142, Revision 2 in the section dealing with the use of ACI 349-97 and other related standards. The measures in ACI 349-13 for the inspection and testing of Seismic Category I concrete structures have not been significantly revised from those in ACI 349-97. The revised regulatory guide should address the inspection and testing of Seismic Category I concrete structures, by referencing and reinforcing the endorsement in Regulatory Guide 1.28 of ASME NQA-1, Part II, Subpart 2.5, Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete, Structural Steel, Soils, and Foundations for Nuclear Facilities. Doing so would be consistent with the endorsement of NQA-2-1983 in Regulatory Guide 1.142, Revision 2.	Agree. To address the comment on inspection and testing, a regulatory position stating that "RG 1.28 should be used for quality assurance in design and construction and for inspection and testing of concrete structures covered by this RG" was added to RG 1.142, Revision 3.
Anthony Ponko	C.2.2	The Regulatory Guide should clarify that Regulatory Position 2.2 applies to mechanically headed deformed bar systems as well as mechanical splices.	Agree. (RG 1.142 was renumbered and C.2.2 is now C.4.2.1) To address the comment C.4.2.1 was revised to include mechanically headed deformed bar systems. In addition, requirments for slip testing have been deleted from Regulatory Position 2.2 because Article CC-4333 of American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code (Code) Section III, Division 2, of 2019 edition, joint committee

			with ACI 359-19 was updated to reflect these splice qualification requirements and published in July 2019. The following text and table from Regulatory Position 2.2 of the draft guide was deleted: *"Conduct the slip test for two mechanical splice samples in accordance with Section 10.7 of ASTM A1034 to a predetermined load equal to one-half the specified yield strength ($0.5F_y$) of the steel reinforcing bar. Ensure that the measured slip does not exceed the values in Table 1 of this RG. The table was derived from California Department of Transportation "Authorization and Acceptance Criteria for Mechanical Couplers on ASTM A706 and ASTM A615 Reinforcing Steel" (Ref. 22), and "California Test 670 – Methods of Tests for Mechanical and Welded Reinforcing Steel Splices" (Ref. 23). Should either of the two mechanical splice samples not meet the slip acceptance criteria in Table 1, conduct a retest in which all remaining static tensile test specimens are evaluated for slip before static tensile testing and meet the slip requirements; if the specimens do not meet the slip requirements, the splices should be rejected."* **Table 1: Total Slip Acceptance Criteria was also deleteted for the reason given above.** In addition, the requirement to follow the splice qualification requirements of American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code (Code) Section III, Division 2, Article CC-4333 was updated to reflect the more recent version of the code. (ACI 359-19).
Adeola Adediran for the ACI 349 Subcommittee		**General Comment** There are seven comments provided in the attached file named NRC_PC_DG-1283_NRC-2019-0100_v2. These represent the comments from the leadership of ACI 349. These are private opinions of these ladies and gentlemen. However, due to time constraints, we are providing preliminary comments and request the permission to issue a final more complete set of comments in a week's time. (Submitted 6/25/19)	

ACI 349 1	Pg. 4. Discussion Section 2.3	Crack control in nuclear safety-related structures is primarily a concern associated with shear, not flexure. ACI 349-13 does not allow grade 75 or grade 80 steel to resist shear or torsion. ACI 349 permits high strength reinforcing to be used only for flexure, where the issue of crack control for these types of structures is generally much less pronounced. The adoption of grade 75/80 reinforcement by the code committee was solve the recurring problem of rebar congestion and constructability issues in safety- related structures. High strength reinforcing has been permitted in European nuclear construction for over a decade and there is an extensive body of research on its use. ACI 318 has recently adopted the use of A706 Grade 80 reinforcing for earthquake-resisting construction, having addressed concerns related to bond and development. These concerns are also addressed in ACI 349-13 by the introduction of the 1.2 factor on development length above that which is ACI 318-08 development length equation. ACI report ACI ITG-6R-10, "Design Guide for the use of ASTM A1035/A1035M Grade 100 (690) Steel Bars for Structural Concrete", 2010, Paul Zia Chair, addressed the issue of cracking associated with the use of 100 ksi rebar. While noting that some cracking issues may occur for the 100 ksi rebar in a commercial application, it is not likely it would occur for an 80 ksi rebar in the nuclear industry. This is because the largest loads in nuclear structures is typically earthquake for which cracking is not a significant issue. In any case, the concerns expressed regarding deflection control, ductility and overstrength are not particularly relevant to the design basis for most safety-related nuclear structures. We urge NCR to rethink its exception to the use of grade 75/80 reinforcement for flexure as allowed by ACI 349.	Disagree with the comment. (RG 1.142 was renumbered and Regulatory Position C.2.3 is now C.4.3.) Regulatory position C.4.3 and the associated discussion in Part B of the draft guide take exception to the general use of high strength reinforcement (Grade 75 and 80). The discussion of C.4.3 in Part B (page 4) lists several design aspects for which research, development and demonstrated performance by use are still ongoing. The comment addresses crack control, which is one of those several design aspects. The comment says that crack control is primarily an issue with shear and not flexure. The general condition in nuclear structures can be more complex and the same reinforcement can, for example, resist both shear and flexure as well as other demands. As an example, earthquake loads in nuclear structures are most often carried by shear walls in which the same reinforcement may resist both shear and flexure. In relation to use of high strength reinforcement, only in 2019 did the ACI 318, the parent standard for ACI 349, adopted the use of A706 Grade 80 reinforcing for earthquake-resisting construction and with numerous specific requirements for its use. Therefore, at this time, the staff guidance in Regulatory Position C.4.3 is to take exception to the general use of high strength reinforcement as described in ACI 349-13. However, the staff guidance in that regulatory position accepts the use of high strength reinforcement for specific use when its adequacy can be demonstrated by an appropriate combination of testing, analysis and performance evaluation. The staff will continue to engage with the code committee, review research and development as well as use for the development of more general guidance.

ACI 349 2	Pg. 5 Discussion Section 2.6	The reasoning for the one-third rule is provided in ACI 318-08 R10.5.3, where it is explained that the use of the minimum area of steel in a thick slab, wall or shell may result in severe congestion of reinforcing steel. ACI 349-13 uses the commentary in ACI 318-08 unless modified. We would request that NRC review the background provided in the 318 commentary as the minimum reinforcing requirement affects the constructability of many nuclear structures. Please note that this provision has been in the code for many years and is not a recent addition.	Disagree with the comment. This Regulatory Position ensures the minimum reinforcment for slabs and footings per ACI 318-08 R10.5.4. Note that ACI 318-19 doesn't include the provision for the one-third rule for slabs and foundations in Section 13.3.4.4, 7.6.1.1, 8.6.1.1. However, in ACI 318-19, the one-third rule is applicable for beams in section 9.6.1.3.
ACI 349 3	Pg. 5. Discussion Section 3.1	When To is combined with Dead and Live in normal loading conditions, i.e. load combinations 9-1 thru 9-4, the concrete sections are most likely taken as uncracked and gross member properties are used. The cracked properties for the seismic load cases are used (ASCE 4 and 43) for the accident and abnormal loading conditions, including thermal loading. Combining operational temperature loading and uncracked member properties is conservative (since cracking relieves these stresses), and applying the 1.6 factor as requested by the NRC position increases the conservatism to a degree that we believe is unwarranted. The resulting effects of applying 1.6 to To is also undesirable because this results in more reinforcement required in the member which in turn increases the stiffness of the member and hence making it attract greater thermal stresses. This is counter-productive to the reinforced concrete member.	Disagree with the comment. In the design of nuclear power plant concrete structures, the operational temperature loading, T_o, is considered as a live load. Though extremes of anticipated temperatures are considered for this purpose, the computational methods of cracked section analysis and the extent of cracking do not lend themselves to the same degree of confidence in assessing its effect as those for a dead load computation. The NRC staff position is to use a load factor of 1.6 for T_o. There is no change in this position from RG 1.142 Revision 2.
ACI 349 4	Pg. 6. Discussion Section 3.1	In as much as Ro is computed mainly from thermally-induced elongation of piping, it is not clear why this should be associated with enhanced uncertainty as stated in the NRC position. Note also that there is already significant conservatism associated with the use of an envelope of temperatures for these cases. Please note that the nuclear industry has long struggled with the difficulty of dealing with temperature loads on nuclear structures. The self-relieving nature of the temperature load makes it less critical than other loads. Adding	Disagree with the comment. According to Section 2.1 of ACI 349-13, R_o does not include dead load and earthquake reactions (those components of R_o are included in D and E_o/E_{ss}). Assessing R_o is associated with larger uncertainty than dead load. ACI 349-13, Appendix C, applies a load factor of 1.7 to R_o, which is similar to a live load. In Regulatory Position 3.1, the staff modified the load factor for R_o to treat it as a live load in load combinations (9-2), (9-3), and (9-4). There is no change in this position from RG 1.142 Revision 2.

		larger load factors sends a wrong message to the designers that the way to deal with temperature is to make the structure stronger. This again is counter-productive to a rational design. Furthermore, the codes recognize the cumulative approach contained in ASCE 7, which holds that as an increasing number of loading types are combined, the less likely it is that the peaks of these loads will occur concurrently. Ro is consistently addressed in this regard in ASCE 43, ACI 349 and AISC N690.	
ACI 349 5	Pg. 6 Discussion Section 3.1	We disagree with the NRC position to require a load factor of 1.0 for live load. The load factors in Chapter 9 are associated with lower strength (phi) factors; the Appendix C load factors are used with higher strength (phi) factors. These factors cannot be mixed. Thus, increasing load factors in Chapter 9 to match those of Appendix C erroneously alters the global safety factor. We note also that ACI 318 allows live load reductions that result in an equivalent load factor of 0.5L. These reductions are not permitted in nuclear safety-related construction. We strongly recommend that the NRC review their position in this regard. Regarding the load factor in ACI 349-13 for live load, as explained in the commentary of ASCE 7 Section C2.3, the loads used in design account for the maximum lifetime value as well as arbitrary point-in-time values, with the maximum lifetime value always controlling. When many different types of loads are superimposed in a load combination, as is the case for abnormal or extreme load combinations, the arbitrary point-in-time value or the mean value of the load (accounting for industry variation) should be used. The mean value varies between 0.5 to 0.8 of the maximum lifetime value. The value of 0.8L is used for load combination 9-5 to 9-9 on this basis.	Disagree with the comment The comment refers to ACI 318, the parent code of ACI 349, and the ASCE 7 standard. Both the ACI 318 and the ASCE 7-16 use default load factors of 1.0 for the live load in the load combinations involving seismic loads or winds as the principal loads, which are the load combinations analogous to those in the comment. This is consistent with the staff guidance to use a live load factor of 1.0. The ACI 318 and the ASCE 7 allow the use of a live load factor less than 1.0 for load combinations involving seismic and wind loads for specific conditions and only when the nominal live loads are less than 100 pounds-per-square-foot. Therefore, while the staff position is consistent with the default provisions in the references cited in the comment, the generic use of a load factor of 0.8 in ACI 349 deviates from those references. A technical basis derived from surveys and data collection in conjunction with research is still necessary for the generic use of a live load factor of 0.8 in the extreme and abnormal load combinations. In the absence of that technical basis, the staff guidance has been to use a default load factor of 1.0.
ACI 349 6	Pg. 6. Discussion Section 3.7	We strongly recommend that the NRC reconsiders their recommendation to use a phi factor of 0.6 for shear critical walls for both load combinations from Chapter 9 of ACI 349 as well as for load combinations from Appendix C of ACI 349. It is an error to use a phi of 0.6 for load combinations in Appendix C. ACI 318 requires 0.75 for shear critical walls using Appendix C	Partially agree. The use of a higher value of phi (0.75) for shear walls in safety-related nuclear facilities as compared to the value of phi (0.60) for shear-critical shear walls in commercial buildings needs to be justified on a case-by-case basis. It shall be up to the applicant to establish whether specific shear walls in their structure are governed by ductile limit states (such as

		load combinations and ACI 349 recommended 0.85. With regard to the deviation from ACI 318, the 349 Committee discussed this issue extensively. Most of the walls in nuclear safety-related structures are squat (low-aspect ratio) flanged walls. Some of these walls could be deemed shear critical. Shear critical walls are walls where the measured ultimate shear strength is less than the value associated with the wall's expected flexural strength. Establishing the flexural strength for squat walls of the type encountered in nuclear construction (considering non-aligned openings) is best addressed experimentally. Numerical simulation is problematic since it involves predicting a three-dimensional yield line and there was general disagreement on how that is done. There was general agreement that the trigger for shear-critical walls as defined in ACI 318 is appropriate for ACI 318-type walls. These rectangular walls have a well- defined B stress region, are relatively short in wall length, are tall in wall height, and generally have aligned openings from top to bottom. In such cases, the wall can be approximated as a 2-D cantilever column that develops a hinge near the foundation and thus the moment Mp can be approximated as well as the associated value of Vp, whereby if Vu>Vp then the wall is deemed shear critical. Walls in nuclear safety-related construction do not generally lend themselves to this type of assessment. Walls often extend over the entire length of the structure because they are also used for shielding. These walls are thick, stocky walls with intersecting walls (flanged walls or barbells) with non-aligned utility openings, comprising exclusively D stress regions better solved using the strut and tie method (because the openings do not align) than the conventional bending model. Computation of a cantilever bending value for such walls would be wholly inaccurate, and thus this method of defining shear-critical walls is not available to the nuclear design community. There were two additional reasons for the deviation from ACI 318. The first is that the committee concluded that the ACI 318 design equation for shear, especially the ones in Chapter 11 of ACI 318-08	flexural yielding) or non-ductile limit states (such as concrete crushing or shear failure). Given the complex load combinations for the design of safety-related nuclear structures, and the complex geometry of such structures, it is the responsibility of the applicant to establish and evaluate the potential limit states for shear walls and establish the governing limit state for different load combinations and scenarios. A higher phi-factor of 0.75 may be used for design when the ductile limit states (such as rebar yielding) govern, and a lower phi factor of 0.60 shall be used when non-ductile limit states (such as concrete crushing or shear failure) govern. Similarly, for ACI 349 Appendix C, Section C.9.3 and RC.9.3.4, a higher value of $\varphi = 0.85$ may be used for design when the ductile limit states (such as rebar yielding) govern behavior, and a lower value of $\varphi = 0.75$ shall be used when non-ductile limit states (such as concrete crushing or shear failure) govern.

		under-predicted the capacity of the ACI 349-type walls. Work by Gulec et al. (ACI Structural Journal, Vol 105, Issue 4, pgs. 488-497) showed that for ACI 318-compliant walls that are shear critical both the Chapter 21 and Chapter 11 equations result in median nominal capacities to measured capacities ratios of 1.08 and 0.86, respectively. Furthermore, it was noted that the standard deviation for shear capacity for the regular ACI 318-compliant squat rectangular walls and those deemed shear critical was approximately the same at 0.53 and 0.54 for the equations in Chapter 21 and 0.38 and 0.42 for the expressions provided in Chapter 11. Note that the shear critical wall data had approximately the same scatter in this research for ACI 318-compliant rectangular walls. The design median strength under-predicts the measured strength by 35% for Chapter 11 equations and 20% for the expressions in Chapter 21. The use of a 0.75 phi factor shifts the 50th percentile nominal strength to an approximately 90th percentile design strength. ASCE 4 predicts seismic demands for DBE shaking at the 80th percentile exceedance probability. ACI 349 design strength equations deliver capacities between 90th and 98th percentile exceedance probability. Taken together they ensure a performance target of 1% or less frequency of unacceptable performance under DBE shaking (Ref. ASCE 43). . Secondly, ACI 318 introduced a phi factor of 0.6 not because of increased scatter associated with the measured shear strength of regular squat walls vs shear-critical walls. The penalty was introduced to offset reductions in seismic load associated with energy dissipation, wherein ductility and degraded shear strength are critical for the expected behavior. This is not the case with walls in nuclear safety-related construction. Nuclear safety- related structures are designed to be subjected to multiple cycles of loading to peak strength in safe shut down DBE shaking. The ductility demands for these walls are intentionally kept low. The structures that we build for these power plants are designed to behave essentially elastic under the design basis earthquake. The accompanying drift associated with the design basis earthquake is very small. The reduced ductility	

		identified by the NRC is deemed acceptable since the ductility demand is indeed very low for nuclear safety-related structures. This is not the case for building structures designed in accordance with ACI 318. We thank you for your attention in this matter.	
ACI 349 7	Pg. 4 Reason for Revision	Why is NRC endorsing a portion of ACI 359-15 in DG-1283 when DG-1283 does not cover Reactors and Containment Structures?	Agree with the comment. It was removed from the Reason for Revision as this was not the reason RG 1.142 was revised. However the guide endorses a portion of ACI 359-15 related to mechanical splices. This is because ACI 349-13 lacks this criteria and ACI 359-15 is considered acceptable to the NRC staff.