UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

JAN 0 7 1994

Docket No. WM-107

Mr. William B. House
Corporate Director of Licensing
Chem-Nuclear Systems, Inc.
140 Stoneridge Drive
Columbia, South Carolina 210

Dear Mr. House:

SUBJECT: REQUEST FOR ADDITIONAL INFORMATION NO 1. (RAI-1) ON THE TOPICAL REPORT ENTITLED, "MULTI-USE CONTAINER HIGH INTEGRITY CONTAINER," CHEM-NUCLEAR SYSTEMS, INC., DATED JULY 23, 1992 (DOCKET WM-107)

We have completed the review of the remaining sections of your topical report on the "Multi-Use Container High Integrity Container," dated July 23, 1992, that was submitted by a letter dated July 22, 1992. Based on that review, we have identified several questions or areas where additional information or clarification is required. This is a supplement to the request for information, dated July 27, 1993, that contained Items 1 through 37. The areas addressed herein are the remainder of Section 3 and Sections 4 through 7. Each of the numbered questions, inquiries, or needed clarifications identifies the page and section, table, or figure from which the item was generated and begins with Item 38.

Your responses to the RAI-1, sent on July 27, 1993, has been received and is being reviewed. In addition, replacement pages for Attachment V to that November 30, 1993, response, were received. Those replacement pages were dated December 10, 1993.

If, as you review this request, there are questions, please feel free to contact the Project Manager, Mr. Robert Shewmaker, for assistance. In addition, it may be advantageous to have a meeting after you have completed your initial review of the request for information. We will be available to meet with you at a mutually agreeable date and time to discuss any questions you might have pertaining to our request.

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If there are any questions, please contact me on (301) 504-3450 or Robert Shewmaker on (301) 504-2596.

Sincerely,

(Original Signed by ________)

John O. Thoma, Section Leader
Technical and Special Issues Section
Low-Level Waste Management Branch
Division of Low-Level Waste Management
and Decommissioning
Office of Nuclear Material Safety
Sincerely,

Enclosure: As stated

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REQUEST FOR ADDITIONAL INFORMATION
JANUARY 6, 1994

38. On p. 3-1 it is stated that the MUCs are to be made with three different dimensional variations to accommodate a range of disposal conditions. Table 3.1-1 and Table 3.3-1 also indicate the three variations. One of these conditions is for direct trench burial where the MUC would be in direct soil contact and the burial depth could be as deep as 55 feet. On p. 2-8 in listing the operational limits it is stated that, "Disposal depth in a shallow land burial facility is a maximum of 25 feet." Please resolve this apparent conflict within the document.

39. In Section 3.4 on p. 3-5 there is a discussion concerning the design basis of the MUCs. It is stated that only one loading combination from ACI 349-85 is applicable and that is combination No. 1. Other loading combinations or derivatives of loading combinations should also be considered since they address expected conditions that may be imposed on the MUCs. These may not necessarily control the design of the MUCs, but they should be identified as having been addressed by the design. For example, a derived loading combination from ACI 349 combination No. 1 that should be part of the design bases would be:

$$U = 1.4D + 1.7L$$

This should be used to consider the cases where the MUC is not under lateral external burial loads, but is subjected to the internal loads when the MUC is at rest as well as during handling with either a forklift from the bottom or an overhead crane utilizing the four anchors for the suspension apparatus.

Other examples of combinations that should be considered would be combinations Nos. 3, 4 and 5. These could be defined as follows:

$$U = 1.4D + 1.7L + 1.7W$$

$$U = 1.0D + 1.0L + 1.0E$$

$$U = 1.0D + 1.0L + 1.0W_t$$

Provide justification for the omission of combinations that address normal handling load, wind, tornado loads, and seismic loads for the MUCs that may experience these environments.

Enclosure

40. On p. 3-5 it is stated that the material specifications and properties are provided in Appendix 5. That appendix is entitled, "FRC Durability Assessment." Is this reference correct, or should the reference be to Appendix 3, "FRC Constituents" ?

41. In Section 3.2 on p. 3-3, one of the basic assumptions for the design of the MUCs is stated as, "Only the fiber reinforced concrete provides the structural strength to the MUCs." In Section 3.6.1, addressing the design strength of sections, it is stated that the recommendations and guidelines of ACI 544.1R-82 and ACI 544.4R-88 were used. The following is a quotation from ACI 544.4R-88, Chapter 1, Introduction.

 "Generally, for structural applications, steel fibers should be used in a role supplementary to reinforcing bars. Steel fibers can reliably inhibit cracking and improve resistance to material deterioration as a result of fatigue, impact, and shrinkage, or thermal stresses. A conservative but justifiable approach in structural members where flexural or tensile loads occur, such as in beams, columns, or elevated slabs (i.e., roofs, floors or slabs not on grade), is that reinforcing bar must be used to support the total tensile load. This is because the variability of the fiber distribution may be such that low fiber content in critical areas could lead to unacceptable reduction in strength.

 In applications where the presence of continuous reinforcement is not essential to the safety and integrity of the structure, e.g., floors on grade, pavements, overlays, and shotcrete linings; the improvements in flexural strength, impact resistance, and fatigue performance associated with the fibers can be used to reduce section thickness, improve performance, or both."

 ACI 544.1R-88 cites some limited examples of instances where the primary loads were fully resisted by the steel fibers. In those cases, the recommendation is that the reliability of such a member should be demonstrated by full-scale load tests, even when the fabrication process is under rigid quality control processes. It should also be noted that the cited applications have been for structures with a normal life span in the 40 to 60 year range.

 Please provide a discussion regarding the design philosophy for the MUC in comparison with that of the cited ACI 544.4R document. Also provide a discussion of other known applications or projects where there was full reliance on the strength of the fiber-reinforced concrete without conventional reinforcing steel. The design life for these applications should also be discussed.

42. On p. 3-5 under Section 3.5, Material Specifications, it is stated that a summary of the key mechanical properties follows. Upon examination of the listing for the fiber-reinforced concrete it is noted that the first-crack strength and tensile creep testing are not mentioned. In addition, the flexural strength of the concrete matrix without the steel fiber reinforcing should be determined. As pointed out in both ACI 544.1R and ACI 544.4R, the shape of the

curve representing load vs. flexural stress or load vs. deflection is important to understand the behavior of the material as well as to understand the impact of variations in the fiber reinforced concrete mixture. Since the intent of the design is to produce structural elements (HICs) with the capability to carry load and thus remain stable for a long period of time (for hundreds of years), the tensile creep characteristics of the material that can be influenced by the long-term performance of the bond between the steel fibers and the concrete matrix is important.

Indicate how these issues are addressed in this design concept.

43. In Section 3.6.1 on p. 3-7, in the discussion of the design strength of sections under various internal loads, there is a reference to ASTM C-78 and ASTM C-1018 and the tests that were performed. Appendix 4 of the topical report entitled, "FRC Test Data," does not however, contain a reference to, or the results of testing under these standards. Consequently, it is not possible to be certain how the results have been used to establish design values to be used in the design analysis. It is stated on p. 3-7 that a load-deflection curve was established as prescribed by ASTM C-1018 and it is stated that the modulus of flexural strength (modulus of rupture) was determined as prescribed by ASTM C-78. On p. 3-8 it is stated that tests were performed and the lowest value was noted as 1131 psi and that a conservative number for design of 1000 psi is assumed. The 1000 psi is then noted as being the flexural modulus at the first crack.

Based on the information provided, it is not apparent that 1000 psi represents the first-crack strength of the proposed FRC. The first-crack strength should be determined from the load-deflection curves established under the procedures outlined in ASTM C-1018, not from the ASTM C-78 testing. Guidance contained in Section 5.1 of ASTM C-1018 attempts to indicate the significance and use of the load-deflection curve established by the test. It is stated that, "The first-crack strength characterizes the behavior of the fiber-reinforced concrete up to the onset of cracking in the matrix, while the toughness indices characterize the toughness thereafter up to specified end-point deflections.... The importance of each depends on the nature of the proposed application and the level of serviceability required in terms of cracking and deflection."

Several issues related to these topics need to be addressed.

a. The formula developed for the design flexural strength of the FRC material that is presented on p. 3-8 utilizes the term "R" that is defined as the flexural modulus at the first crack. It appears that a design procedure would consider the MUC under this state of stress. A question remains as to whether the value of "R" is 1000 psi, or not. Please provide representative test data in the form of the load-deflection curves under ASTM C-1018 and the evaluation of this data to establish the first cracking strength.

b. Data should also be made available for ASTM C-78 testing on the concrete without the fiber reinforcement so that the impact of the fiber reinforcing can be understood. Please provide relevant data on this material property.

c. Another issue that appears to need to be examined is the consideration of what loading conditions or combinations should be used in conjunction with the first cracking strength. It would appear that this issue could be approached by considering load combinations for service conditions as a working stress approach. In order to approach this issue there is a need to clearly define what is being proposed as being an acceptable design with respect to cracking as computed from actual loads. Please define loading conditions with respect to first-cracking strength. Also consider defining loading conditions for use in a service load approach with working stresses.

d. For the design analysis for stress conditions beyond the first-cracking strength; has the effect of cracking been considered?

e. No discussion or limits appear to be contained in the topical report relative to the permitted deformations under the loading combinations. Please address the issue of permitted deformations.

44. The shear strength design concept presented on p. 3-9 is related to the use of an equation that is noted in the topical report as defining the shear strength of unreinforced concrete. The value utilized in the formula, namely 5000 psi, represents a value used in the analysis whereas the specified target strength is 7250 psi. Since the material is reinforced with the steel fiber, the use of a compressive strength for the FRC material should perhaps not be used. ACI 544.4R, which was cited as a guidance document for the topical report, states the following with respect to shear in beams (the faces of the MUC may be considered as a series of beam strips).

"It is evident from a number of tests that stirrup and fiber reinforcement can be used effectively in combination. However, although the increase in shear capacity has been quantified in several investigations, it has not yet been used in practical applications."

Please provide data on the compressive strength of the concrete matrix without the FRC reinforcing and discuss the issue of the appropriate value for the design shear based on f'_c without fiber reinforcing.

45. On p. 3-8 where the formula for the flexural moment capacity of a section is developed in terms of the depth of the section, the numerical value of 7.75 inches is not needed and should be deleted from the listed parameters.

46. On p. 3-10 in discussing the loading conditions for the MUC for disposal it is indicated that, "The MUCs may be disposed-off (sic) either in an above-grade engineered facility or buried..." The intent was apparently to use the words, "disposed of," so the text needs to be corrected. In addition, it appears that Chem-Nuclear has a specific above-grade engineered facility defined in addressing the detailed design conditions which would exist for the MUCs. Apparently the vertical space on the interior of such a vault is limited to 30 feet; the vault has a 3 foot thick roof slab with 10 feet of soil cover on top of the roof. There are also apparently no other potential loads that have been envisioned. In order to meet the requirements of 10 CFR 61.52(a), the near-surface disposal facility must have waste emplaced in a manner that minimizes the void spaces between waste packages and permits the void spaces to be filled. For a disposal facility other than near-surface, the operation and closure requirements have not been defined in regulations, but 10 CFR 61.52(b) has been reserved for those provisions if regulations are developed.

    It appears that it would be expected that an above-grade engineered facility would also need to be operated and closed in a manner that minimizes the void spaces between packages and permits the void spaces to be filled. Consequently, there would be lateral pressures exerted on the MUC units during the static condition. During a seismic event there will be additional lateral loads that should be addressed (See RAI-39).

    Please consider the issue of the lateral loading of the MUC units, both as single units resting on their base, as well as stacked in the various configurations that can be permitted in the engineered facility. Provide additional discussion and information on the design capability of the MUCs to remain stable under such loading conditions. These loads may not control the design, but each should be addressed.

47. Section 3.6.4 beginning on p. 3-14 presents information on the calculations for the 68-inch MUC. The various figures associated with the calculations do not contain an identification of the coordinate axes (Figures 3.6-3 through 3.6-7), yet Figures 3.6-6 and 3.6-7 are representing stresses in the x and the y-directions. In the text various internal forces or stresses are discussed, but the direction and sign conventions are not defined. For example,

it is stated that the maximum bending moment is at the edge and is 2,239 in-lb/in, but it is not defined whether it is an Mx, My or Mz moment (at a vertical corner or the side to bottom edge). In Figure 3.6-4 through Figure 3.6-7 there are numerous marginal entries associated with the figures that are undefined. Please provide clarification on the coordinate axes identification, the sign conventions and the identification of marginal entries.

48. Figure 3.6-8 presents a curve of the moment distribution across a centerline of the MUC wall, but the direction of the moments are not defined. This figure also seems to indicate that the plot would represent a structure with a dimension of 2 times approximately 28.5 inches (defined by the end of the curve at the wall centerline), or 57 inches, not 68 inches, or is the analytical model representing a middle-depth surface? Please provide additional clarification on these items.

49. Several items of clarification related to the analysis described in Sections 3.6.3 and 3.6.4, beginning on p. 3-12, need to be provided.

    a. Reference 9 relative to the ANSYS finite element computer software and the related technical formulations are not available in our current library. Please provide the technical basis for the element described as STIF63 and STIF45.

    b. Figure 3.6-4 illustrates the boundary conditions utilized on the symmetric one-quarter of the MUC, but no explanation of the symbols used in the figure are provided. Please provide this information.

    c. On p. 3-20 the reference to Table 26 of Reference 13 is apparently in error. Should this be Reference 14? Please clarify.

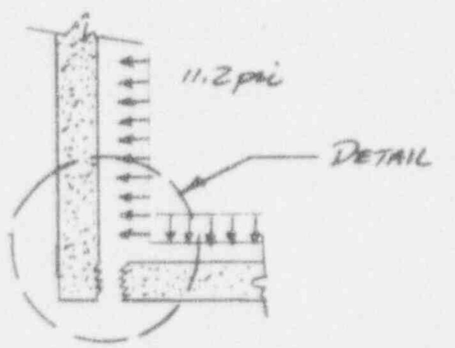
    d. Within these sections there is no specific information on the loading input on the finite element model. All input data for the 68-inch MUC should be provided and its use described.

50. On p. 3-21 in addressing the membrane (uniform) tensile stresses it is stated that the ACI code, meaning ACI 349, does not have any requirements for that state of stress. This is literally correct in that no limits are provided for membrane tensile stress, but it is clear from the Code, that is for application to conventionally reinforced concrete, no membrane tension is permitted. Even the theory of reinforced concrete design for flexure neglects any contribution from the concrete.

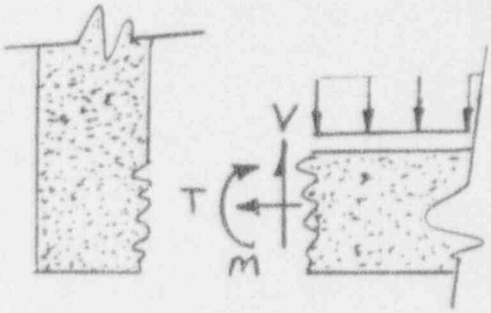
As stated in Section 10.2.5 of ACI 349, "Tensile strength of concrete shall be neglected in flexural calculations of reinforced concrete, except when meeting the requirements of Section 18.4." It follows that membrane tension is not permissible since there is no permitted stress or ultimate load capability defined. Where direct tension occurs, reinforcing steel must resist the load such as is provided for in Section 12.15.5 addressing tension tie members.

Please clarify the discussion related to Code requirements for membrane tension not being in existence.

51. On p. 3-21 the specifics of a calculation to determine the stress levels for combined flexural and axial loads are provided. The membrane tension was determined by a simple hand calculation. While this is an acceptable calculation, it is a value that would normally also be output data from the finite element analysis once a decision was made to develop and analyze such a model. Please explain why the values of the membrane tension were not also available from the analysis of the finite element model. In addition, the discussion of the results of the analysis for internal pressure (reduced external pressure) either from the finite element analysis or some other method, does not address the resulting shear stresses, nor the implications of combined shear, moment and membrane tension. Consider the wall section shown below and indicate how the analysis has considered these internal stresses, and what are the predicted values?



VERTICAL
SECTION AT SIDE WALL-BASE INTERSECTION



DETAIL

52. On p. 3-22 the combined value of the membrane tension stress and flexural tension stress with the appropriate load factor and strength reduction factor is compared to the flexural strength of the material which has been taken as 1000 psi. As discussed in RAI 43, the test data cited apparently showed an average modulus of rupture of greater than the lowest reported value of 1131 psi, but the first cracking strength was not determined. If the first cracking strength is exceeded anywhere in the MUC analysis, how is the change in the section properties reflected in the analysis? Such cracking will change the stiffness of the MUC, its

deformations and crack widths and depths. Please address this issue.

53. With respect to the model used for the finite element analysis (Figure 3.6-4), as well as the cases of boundary conditions used for the theory of elasticity closed form solutions for bounding values (p. 3-20), it does not appear that the correct condition for the top of the MUC was properly represented. The top of a MUC is cast after the other five sides of the basic cubic structure. Therefore, there will be no other mechanism to transfer stresses across the joint interface other than bearing, bond, or friction. For example, how would membrane tension in the top of the MUC be carried across the interface? It does not appear that the analytical approaches have addressed this condition. Using Reference 14, boundary condition cases other than Case 1a and Case 8a would appear to also be appropriate for consideration. Because of the condition at the junction of the top of the MUC and the top edge of the four sidewalls, Cases 2a, 9a and 10a, appear to be conditions that should also be considered in bounding calculations for maximum flexural moment. These cases consider a plate simply supported on three sides and free on one side, a plate fixed on three sides and simply supported on one side, and a plate fixed on three sides and free on one side. The maximum moments generated from these boundary conditions for the 11.2 psi internal pressure are 4730 in-lb/in, 2420 in-lb/in, and 3610 in-lb/in respectively. These are all greater than the 2239 in-lb/in used in the sample calculation.

Please provide additional information regarding the analysis of the behavior of the sidewalls at the top of the MUC in view of the configuration of the top to sidewall interface.

54. On p. 3-22 under the discussion of external pressure loads, it is stated that the walls of the MUC respond as flat plates with the edge conditions being similar to fixed-edge boundary conditions. As was noted in RAI-53, because of the discontinuity at the sidewall to the top joint of the MUC, the assumed conditions do not truly represent the actual conditions in 4a conservative manner. It should be noted that the finite element model in Figure 3.6-4 assumes that there is material continuity at the intersection of the sidewall and the top of the MUC. It is also indicated on p. 3-22 that the external pressure of 5.3 psi was identified as BTP Item 4j. instead of Item 4i. Please amend the statements on p. 3-22 based on the response to RAI-53.

55. On p. 3-22 in the discussion regarding the maximum payload, there is reference to Table 3.3-1 which addresses the Minimum Margin of Safety. Should the reference be to Table 3.1-1?

56. On p. 3-23, the results of the finite element analysis for the base of the MUC, including the forklift notch, are discussed. The model that was analyzed is presented in Figure 3.6-3, but the

boundary conditions used for the detailed breakout analysis of the base are not provided. Please provide a description or identification of the boundary conditions assumed in the analysis and provide an explanation of any symbols used (See RAI- 49b.).

57. On p. 3-26 the results of the analysis for sling handling are discussed. The model that was apparently analyzed is presented in Figure 3.6-3, but the boundary conditions used for the detailed breakout analysis of the base are not included. Please provide a description or identification of the boundary conditions assumed in the analysis and provide an explanation of any symbols used (See RAI-56).

58. On p. 3-26 a calculation is made to check the shear load capability of the base of a loaded MUC. The payload is listed as 1100 lbs. instead of 11,100 lbs.; however the resulting answer is correct. Please correct this typographical error.

59. On p. 3-29 and in Appendix 2 the prototype testing program is described. Section 2.0 of Appendix 2 states that the nominal bottom thickness of the test model was 6 inches and the top lid was 4 inches. This was a model of the 68 inch MUC intended for vault disposal which on Drawing Number C-110-D-12416-001, Rev.0, that has values of 8 inches and 5.5 inches respectively. Please clarify.

60. On p. 3-29 there is a discussion regarding the worst orientation of a MUC for the free drop test on the flat, essentially unyielding horizontal surface and reference is made to Figure 3.6-15. Equations are developed for the potential energy associated with a free drop of equal heights for the three orientations. While the relative value of the resulting energy calculations would not change the order of the highest potential energy, the equations do not appear to be consistent with the figure. For a three foot free fall the distance the center of gravity would fall would be as follows:

Flat face	3 + 0.5L
Edge	3 + 0.707L
Corner	3 + 0.867

The figure illustrates the center of gravity for the flat face orientation to be at 4 feet above the surface. Please clarify the information in the text and on the figure.

61. The data provided in Appendix 2 presenting the prototype testing program results include sheets addressing the data and quality of the FRC shell of the MUC. These are all presented in French as the control documents for the units by serial number. Please

provide that same data translated into English. This data could also be provided on a form that will be used for the units to be manufactured in the U.S.

62. Within Appendix 2, the three page text summary of the free drop tests appears to contain a typographical error in the serial number of the Test #1 polyethylene container that is listed as C-469662-K, whereas the data sheet for that polyethylene container shows the number to be C469262-K. Please clarify this discrepancy.

63. The three page summary of the free drop tests included with-in Appendix 2 also describes the observations of damage after each of the tests. For Test #2, the test judged to be acceptable, it is indicated that "cracking up to 1-1/4 inches one side with cracking up to 1/4 inch on the adjacent side," was observed. It is assumed that this is a description of the width of the cracks observed and that the cracks were through-wall cracks. Please verify this assumption.

64. Appendix 2 contains a series of ten (10) photographs related to the prototype testing program. Only two of the photographs allow for the positive identification of which test or test specimen is addressed in each photograph. This is only possible from the text summary and the mention of Serial Nos. of the components that can be seen in those two photographs. Please provide for the identification of the information presented in the photographs.

65. Appendix 2 contains the Engineering Test Instructions for the Prototype Testing of CNSI's MUC HIC, ETI-92-008, Rev. 1, dated 6/11/92, and Engineering Test Instruction for Free Drop Testing CNSI's MUC HIC, ETI-92-012, dated 6/12/92. ETI-92-008 in Section 5.2 states that CNSI intends to drop the MUC HIC from a height of 3 feet and ETI-92-012 in Section 6.3.3 states that the container shall be dropped from a height of 0.9 meters, which is approximately 3 feet. It is noted that drop test #3 became an extra test based on CNSI's conclusion that drop test #2 had met the test acceptance criteria. The actual test data indicate that the test was performed by a drop from 1.2 meters, or approximately 4 feet. The three page summary of the testing that was completed that is included in Appendix 2 indicates that a decision was made to do an additional test on the remaining container that represented the 8mm wire reinforcing and to drop from 4 feet instead of 3 feet. Please provide a discussion regarding any conclusions that were made from this test that add to the data from drop test #2.

66. On p. 3-35 the sample calculations for the MUC(68") address the loadings for the direct burial disposal method. The finite element model and the assumed boundary conditions are apparently the same as those used for the wall pressure loadings. The questions related to that model and the boundary conditions as

well as the cases reviewed for classical theory of elasticity closed form solutions for uniformly loaded flat plates presented in RAI-53 should also be addressed for this loading condition.

67. Technical Note, 1000.NT.001 of Appendix 4, describes the manufacture and storage of FRC test specimens. In Section 3.2 of that document it is stated that after 24 hours in an air conditioned room after casting, the test specimens are placed in a temperature controlled underwater environment unless the specimen is to be used for shrinkage or mass loss testing. Are the test specimens maintained in this curing environment until testing is done which might be at different times, or is this storage environment only maintained for a specific length of time such as 7 days or 14 days?

68. Test Report, 1000.RE.001 of Appendix 4, provides the test results of compression tests on the FRC material for the characterization or development tests and the average of 190 tests noted as industrial results. What is the minimum number of tests that would be required for characterization for the startup of a new production facility with new material sources? How many tests were run during the development of the basic FRC material, shown with a 28-day strength of 57 MPa, once the material sources were established for the Sogefibre plant in Valognes, France? If there are data available on compressive strengths determined from cores taken from production containers that have been correlated with production cast cylinders, please provide a summary of that information. What are the statistics on the 190 tests, including the range of values, standard deviation and coefficient of variation?

69. Test Report, 1000.RE.002 of Appendix 4, provides the test results of splitting tension tests on the FRC material for the characterization or development tests and the average of 190 tests noted as industrial results. Provide information for the split tension tests similar to that requested for compression tests (RAI-68).

70. Test Report, 1000.RE.003 of Appendix 4, provides the test results of shrinkage tests on prismatic bars of the FRC material for the characterization or development tests and the average of 192 tests noted as industrial results. Provide information for the shrinkage tests similar to that requested for compression tests (RAI-68).

71. Test Report, 1000.RE.004 of Appendix 4, states that the test specimens do not begin to undergo the mass loss testing until after a temperature controlled 28-day cure underwater. The referenced French standard NFP 15-433, that describes the weight change measurements that are made during the contraction and expansion tests, indicates the time for the tests are at 3, 7 and 28 days after preparation of the specimens. Is the intent to test

at the age of 31, 35 and 56 days or at 3, 7 and 28 days after molding? For use in France, has ANDRA defined a upper limit of mass loss? The Test Report indicates the average mass loss value based on 192 industrial test results. Provide information for the mass loss tests similar to that requested for compression tests (RAI-68).

72. Test Report, 1000.RE.006 of Appendix 4, indicates that the flow tests reported were the results of tests on an FRC formulation that was different than that used in the production of the units in France. Indicate what those differences were and the reason for the differences. Also explain the apparent changes that were made in the mix, based on the formulation at the time of the characterization test when compared to the results of 199 tests based on industrial results.

73. Test Report, 1000.RE.008 of Appendix 4, states that ANDRA has no requirements on water permeability for the containers yet there is a reference to an ANDRA test specification. Please provide an English translation to ANDRA Test Specification 322 ET 09-02 ind 0, unless the summary of the content of the specification that is included in the test report includes all relevant parts of the specification. Explain how the container is treated in a performance assessment type analysis as a barrier to the inward flow of water or moisture and the outward migration of radionuclides considering the permeability values. Since no values for industrial test results are provided, it is assumed that there are no tests for water permeability conducted on the production containers. Please verify this assumption. How many tests were performed during the characterization phase that were used to derive the value listed in Section 4 and what was the range of test results?

74. Test Report, 1000.RE.009 of Appendix 4, states that ANDRA has specified diffusion coefficients for tritium and cesium in STE 119-581-S and that ANDRA Test Specification 330 ET 09-07 ind B, defines the test requirements. Please provide an English translation to ANDRA Test Specification 330 ET 09-07 ind B, unless the summary of the content of the specification that is included in the test report includes all relevant parts of the specification. It is indicated that the tests reported on were performed on specimens created from cored samples, but it is not clear whether the parent concrete was a cast test cylinder or prism, or whether the cored samples were taken from production containers. Please clarify the origin of the cores. It is noted in the Test Report that the length of time for the diffusion testing is limited to 1 year. Is this intended to be a maximum as stated, or is the minimum time for the test a year? Indicate at what time intervals test data were obtained to produce the reported results, the total length of time over which data were collected and the number of samples used.

75. Test Report, 1000.RE.010 of Appendix 4, presents test results obtained during characterization testing. Indicate the number of samples subjected to the nitrogen permeability testing. It was noted in the test results that there was a leak at the resin/sample housing interface. Was this a defined leak from the outset of the test or did the leak occur above some threshold pressure?

76. Test Report, 1000.RE.017 of Appendix 4, addresses the drop test of a container onto a hard, unyielding surface located 1.2 m below the container. Indicate whether or not the test specimen contained any conventional carbon steel reinforcing in the form of bar or wire fabric. How many specimens have been tested?

77. Test Report, 1000.RE.018 of Appendix 4, addresses a water tightness test that was performed on a FRC container. It is not clear from the information presented exactly how the test was performed other than filling up the container and making a visual survey for leakage over a seven day period. Please clarify the test procedure. Are such tests run on production samples at some prescribed frequency and if so what is that minimum frequency?

78. Technical Note, 1000.NT.004 of Appendix 4, in Section 3 shows the Industrial Formula for the FRC and refers to Reference 2.5, which is Specification 1000.SP.001. There is a discrepancy in the amount of the siliceous sand filler (SC 200) listed in these two documents. The specification shows 60 kg +/- 5% whereas the technical note shows 20 kg +/- 5%. Please clarify this item.

79. Specification, 1000.SP.005 of Appendix 3, indicates the cement is acceptable if it has a compressive strength of at least 35 MPa at 28 days when tested in accordance with NF EN 196-1. Will a cement just meeting this strength produce an FRC that will have a minimum compressive strength of 50 MPa at 28 days which is the minimum strength value for the container? What testing has demonstrated this?

80. Specification, 1000.SP.006 of Appendix 3, indicates in Section 2.2 that the number of fibers per kg can vary by +/- 25%. Were the material properties of the fiber-reinforced concrete, such as the flexural strength and splitting tensile strength, determined, with a condition of 75,000 fibers per kg, by test to meet the minimum strengths that are specified? If this was not established by tests, provide the basis for this range of variance in the number of steel fibers.

81. Specification, 1000.SP.006 of Appendix 3, lists two tests that were performed on the metal fibers to evaluate corrosion resistance. No information is provided on the pH of the test solutions which can be a critical parameter in assessing the long-term performance of the metallic fibers inside the concrete.

Provide any available information on this issue and discuss the impact of pH reductions on the test results.

82. Specification, 1000.SP.006 of Appendix 3, indicates that the average tensile strength of the metallic fibers, based on more than 10 fibers, should equal or exceed 1400 MPa. Based on the available information relative to the description of the failure surfaces from tests for split tension test and flexural tension tests, it is not possible to judge the criticality of the fiber tensile strength. That is to say that it is not evident from the information submitted whether the failures are a result of bond failure of the cementitious matrix to the fiber, or whether the failures are a result of tensile failure of the fibers. Based on the information provided, the computed bond stress between a fiber and the concrete matrix would have to approach 200 psi to force a fiber failure, therefore it is expected that the tensile failure surfaces would not indicate many fractured fibers, but rather protruding fibers that pulled out of one side or the other because of bond failure. This conclusion is based on the belief that the bond failure stress is less than 200 psi. The mode of failure can influence the frequency of the tests that should be performed on the fibers. Please provide additional information on this subject.

83. In Appendix 2, discussing the prototype testing program, it is noted in Section 2.0 that the prototype test containers had a bottom thickness of 6 inches. It was also noted that the containers were production containers and the other dimensions matched those of the 68 inch model except for the bottom thickness. Drawing No. C-110-D-12416-001, Rev.1, dated 7/22/92, Table I, lists the bottom thickness as 8 inches for the 68 inch model. Please clarify this difference.

84. Contained in Appendix 2 is a memo by S. Pearson addressing the free drop tests conducted at the Cogema Test Facility and it is stated that for Test Number 1 the serial number of the polyethylene container was C-469662-K. The data sheet form from ETI-92-008, however, list the number as C-469262-K. Since the other two tests indicate numbers of C-469662-J and C-469662-E, it appears that the data sheet created by the Quality Assurance Section is in error. Please clarify.

85. Several of the data sheets contained in Appendix 2 are in French and need to have an accompanying translation sheet provided. These are sheets from SABLA of Valognes, France, addressing the final acceptance of the prototype unit for testing.

86. In Sections 5.3.2 and 5.3.3 of the report there are discussions of permitted crack widths and it is stated that it is permissible to dispose of MUC-HICs with crack widths of up to 0.004 inches. What would be a expected depth of such a crack and how does such a crack relate to the durability of a container? How would such

cracks impact the assumptions regarding concrete cover over the conventional reinforcing steel? Please provide additional information on this topic.

87. Table 5.3-2 on p. 5-6 refers to References #14 and #15 which seem to be incorrect. Should the references be #23 and #24? Please clarify.

88. On page 6-1 of Section 6.0 of the report the last sentence appears to need editing. Please clarify the sentence.

89. On p. 7-1 of Section 7.0, the indication is that the CNSI quality assurance program, QA-AD-001, will control sealing of the containers. Does this mean that the intent is that these containers will only be used at disposal sites where CNSI is the operator of the disposal facility?

90. Table 4.2-4, p. 4-5, lists the environmental exposure limits of the FRC container to be a pH of 4 through 13. In Specification 1000.SP.006 of Appendix 4, it is stated that the metallic fibers were tested for corrosion resistance in ferric chloride according to ASTM G48-76-A. It is noted that the test was run for 24 hours, whereas the test standard states that a reasonable test period is 72 hours. The test standard also indicates a visual examination and a photographic reproduction of the specimen surfaces, along with the specimen weight losses, are often sufficient to characterize the pitting resistance. An examination of the planar surfaces for pits under low-power magnification (20X) is provided for in the standard. A more detailed examination would include the measurement of maximum pit depth, average pit depth and pit density. The reproductions of the photographs contained in Technical Note 100.NT.002 of Appendix 5 are not legible. Please provide at least one good copy of each. The standard also outlines what should be contained in the test report. The results of the test were not described or presented in the topical report. Please provide additional details on the tests conducted and the results. Also compare the ranges of pH tested to the range anticipated range of use for the containers of 4 to 13.

91. Table 4.2-1, p. 4-1, lists three categories of chemicals that are not allowed in the waste stream because of the operational limits necessary to protect the integrity of the polyethylene inner liner. What screening tests or techniques are to be used by the waste generators and users of the containers to verify the absence of those compounds?

92. On p. 4-3, it is stated that after a container reaches a disposal facility, the concrete lid is placed that seals the container. Please expand the description to reflect the fact that the void space between the inside of the FRC element and the outside of the polyethylene element is also grouted before disposal.

93. Table 4.2-4 on p. 4-5 indicates that concretes can withstand a pH down to 4 with the reference given to ACI 515.1R-79, "Guide to the Use of Waterproofing, Dampproofing, Protective and Decorative Barrier Systems for Concrete." Please provide the specific reference by section and page number of the source of the statement made in Table 4.2-4. There is a reference to a pH of 4 in Section 3.4.5.1.3 of ACI 515.1R-79, however, Chapter 3 of the guide is devoted to concrete conditioning and surface preparation prior to the placement of a barrier material on the concrete. In that context, the pH of 4 is referred to as the permissible surface contamination left after acid etching of the concrete surface and flushing of the surface. It is not a limit of tolerable pH for long-term contact with concrete.

ACI 201.2R, cited as Reference #18, states the following in Section 2.3, Acid Attack. "In general, portland cement does not have good resistance to acid attack, although weak acids can be tolerated.... No portland cement concrete, regardless of its composition, will long withstand water of high acid concentration. In such cases, an appropriate surface coating or treatment must be used. The ACI Committee 515 Report gives recommendations for barrier coatings to protect concrete from various chemicals."

ACI 515.1R-79 in Table 2.5.2, Effects of Chemicals on Concrete, lists acid waters with a pH of 6.5 or less as resulting in a slow disintegration of concrete. Various specific acids are also addressed in Table 2.5.2 such as nitric acid which at a 2% solution causes rapid disintegration of the concrete.

For the exterior environmental exposure limits relative to acidic conditions to be placed on the MUC-HICs, the attack method generally follows this scenario. When an acidic soil or groundwater is included in the external environment, the effect on the concrete is influenced by several parameters. These include the pH, total acidity, groundwater conditions, and backfill conditions. The acid attack of concrete is an external surface mode unless the acidic material is in solution and the concrete is cracked or otherwise not highly impermeable. In that case, the attack can be internal to the concrete mass itself, but progresses from the surface of the cracks or the pores or other interstitial volumes. In this type attack, the contacting acid is neutralized by the alkalinity of the concrete, and if there is no replenishment of the acid, any further reaction stops. Continuous replenishment of the acid will perpetuate the reaction with the consequences of the acid attack being controlled by the amount of the acidic and basic ingredients available to react and the availability of a transport method which will usually be moisture. Of course, the acid attack may destroy necessary characteristics of the concrete prior to being neutralized, even in cases where there is a sufficient mass of alkalinity to completely neutralize the total acidity.

Please review the following reference, one aspect of which is summarized here relative to actual observed performance of buried concrete. A copy of the reference is enclosed for your information (American Concrete Institute, SP-100, "Concrete Durability," 1987, Paper SP 100-28, "Durability Considerations-Precast Concrete Pipe," M. Bealey).

Based on a ten-year study by the Ohio Department of Transportation on the performance of more than 500 concrete pipe culverts in all areas of the state, a series of curves were developed to relate the slope of the culvert, the pH of the environment to the number of years for the concrete to reach a poor condition. The slope relates to the contact time of the aggressive solutions on the concrete. Based on this data and the empirical results, a concrete pipe placed on a 1-1/2% slope installed in an environment with a pH of 7 would take about 1000 years to reach a poor condition. If the environment presented a pH of 4, the time to reach a poor condition in the concrete pipe reduced to about 100 years. It should be noted that the environmental conditions in Ohio were characterized as relatively neutral and that the soils and groundwater would not contribute to premature deterioration of concrete pipe.

Please re-evaluate the limit of pH of 4 as a minimum value for all models of the MUC-HIC. There does not appear to be adequate justification for this limit.

94. Please provide a copy of Reference #22, "Testing the Influences of Steel Fibre Parameters on Toughness and Cracking of Concrete," J. Kasperkiewicz and A. Skarendahl, Institute of Fundamental Technological Research, Warzowa, Swedish Cement and Concrete Research Institute, Stockholm, 1989.

95. In Technical Note 1000.NT.002 of Appendix 5, in assessing the durability of the FRC against dissolution and leaching resulting in decalcification, the model to simulate the mechanism does not address the cracks that are permitted as described in Section 5.3. It would appear the conclusion that the thickness of FRC degraded over the period of 300 years is 0.24 inches represents a very optimistic view if cracks are to be expected, even though the value of the thickness was increased to 0.39 inches (see RAI-86). In using the French assessment method for durability as shown in Section 4.5.2, a safety factor of 1.7 was applied to the computed thickness of degradation. While the value of such a factor may be justified, it would appear that the factor was chosen from the load factors. This would indicate that the knowledge about the probability of the durability projections for 300 years is the same as that for live loads. Please provide additional discussion to justify the omission of the consideration of cracks when assessing durability and the adequacy of the factor of 1.7.

96. In Technical Note 1000.NT.002, Section 4.1, contained in Appendix 5, the term "during storage" is used. Is this also intended to apply to "during disposal?"

97. On p. 3-35 in discussing the structural adequacy of the MUC-HIC for disposal, no information was presented on the design of the cast-in-place container lid since only the sidewalls of the container are discussed in the sample calculations. Please present information relative to the analysis and design of the cast-in-place container lid. Compare the computed flexural stress at the center of the lid and shear stresses in the lid at the inner edge of the MUC-HIC with the proposed allowable stresses.

98. Figure 3.6-17 on page 3-36, presents a representation of the deformed shape of the MUC-HIC under direct trench disposal. It appears that the lid deflects upward which seems to be counter to what would be expected since the downward loading from the soil pressure will exceed the lateral pressure on the walls of the MUC-HIC. Please verify that the loading condition did, in fact, include the downward load of 55 x 120 pcf (6600 psf).

SP 100-28

Durability Considerations— Precast Concrete Pipe

by M. Bealey

Synopsis: Design methods for buried pipe are fairly well established, but durability, normally, is not given proper consideration. Durability of a pipe is a consideration as significant as its hydraulic and structural functions. The definition of a durable concrete pipe contains three variables that must be evaluated; the required performance, the properties of precast concrete pipe, and the service conditions. This paper discusses these variables, and presents guidelines on how they can be evaluated and on current countermeasures for anticipated aggressive environments.

Keywords: abrasion resistance; acid resistance; cement content; concrete durability; concrete pipes; cover; density (mass/volume); freeze-thaw durability; performance; precast concrete; sulfate resistance; water-cement ratio

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Mike Bealey has been Vice President of Technical Services for the American Concrete Pipe Association for seventeen years. For thirteen prior years he was involved with cement, concrete and concrete pipe.

INTRODUCTION

Durability of a pipe material is as significant as its ability to perform intended structural and hydraulic functions. The capability of the pipe to continue to perform satisfactorily for an economically acceptable period is a fundamental engineering consideration. Unfortunately, predictions of durability cannot be made with the same degree of precision as can structural and hydraulic performance, and, in too many instances, durability is not accorded adequate consideration. Durability is concerned with life expectancy, or the endurance characteristics of a material or structure. In no type of facility is long term integrity more critical than in sewer lines and culverts. Much attention has been directed to the durability of pipe, but the vagaries of climate, soils and geology, fluid impurities, construction materials, and the construction process itself have prevented the development of a systematic and practical theory for predicting performance. The problem has been compounded by the assumed requirement that pipe must last almost indefinitely. The U.S. Bureau of Reclamation (1) defines a durable concrete pipe as **one that will withstand, to a satisfactory degree, the effects of service conditions to which it will be subjected.** This definition contains the three variables that must be evaluated to obtain satisfactory durability; the concrete pipe, the satisfactory degree of performance, and service conditions.

At the present time, there is no known material completely inert to chemical action and immune to physical deterioration. Concrete is no exception, but, under what might be considered **normal** exposure conditions, it has a very long life. Concrete pipelines have a long history of excellent durability, and it is unlikely this record will change. Pipelines are beneath the ground where temperatures have very little variation, where atmospheric exposure is either not present or greatly reduced, and where the materials in close proximity to the pipe are usually nonaggressive. Laboratory test results, and damage records for cast-in-place concrete that has been exposed to atmospheric conditions, should not be related to buried precast concrete pipe until it is determined that comparable conditions will exist. Improper application of data could lead to overdesign and excessive cost.

This paper reviews the physical and chemical factors which may be aggressive to concrete pipe and their significance; the significance of pertinent service factors; the significance of concrete pipe properties; and concludes with a discussion of durability design and the performance of concrete pipe.

AGGRESSIVE FACTORS AND SIGNIFICANCE

The specific physical and chemical factors which can be aggressive to concrete pipe and which collectively account for practically all of the durability problems that can be encountered in traditional applications of the product include freeze-thaw and weathering, abrasion, acids, sulfates and chlorides. Conditions severe enough to result in durability problems for concrete pipe are, however, quite rare, especially when compared to an estimated installation rate of 10,000 miles per year for the last four decades.

Freeze-Thaw/Weathering

Freeze-thaw damage is caused by water penetrating into concrete interstices and freezing, which generates stresses and disrupts the concrete if it does not have sufficient strength to resist these stresses. Severity of exposure is usually described by the frequency of freeze-thaw cycles. Atmospheric exposure usually accompanies freeze-thaw action, which complicates the situation. Thus, instead of a pure freeze-thaw situation, thermal stresses and evaporative surfaces with concentration effects and crystallization of various soluble salts in the pore structure could combine to provide an accelerated weathering effect.

Normally, concrete pipe is not exposed to this combined set of conditions. When it has been, however, its performance has been excellent, primarily due to the high quality of the concrete. In some circumstances, weathering exposure could be serious enough to require sealing the surface with a protective coating. Such circumstances are not common, but can occur. The high strength, low water-cement ratio concrete of precast concrete pipe inherently has excellent resistance to freeze-thaw forces.

Abrasion

Effluent velocity, by itself, does not create problems for concrete pipe within the ranges normally encountered. Below velocities of 40 feet per second, the severity of velocity-abrasion effects depends upon the characteristics of the bed load. Above velocities of 40 feet per second, cavitation effects can be serious unless the surface is smooth and internal offsets at joints are closely controlled. Bed loads are usually more of an engineering flow problem than a question of pipe abrasion, particularly in a sanitary or storm sewer system, and can be controlled by proper design. Increasing the compressive strength of the concrete, and the specific hardness of the aggregates, increases abrasion resistance.

Acids

Acids will attack most materials. Acid attack of concrete is a surface attack, in which the acid is neutralized by the concrete alkalinity, so that without acid replenishment, the reaction stops. Continuous replenishment of acid with a pH below 5.0 is considered aggressive and below 4.0 is highly aggressive to buried concrete pipe.

Exterior acid attack, although chemically the same as an interior attack, involves a completely different environment. When an acidic soil or groundwater is encountered, its effect on concrete is governed by pH, total acidity, groundwater conditions, and backfill material. Total acidity is the amount of acid available to attack the pipe. As an example, a total acidity of 25 milligrams per gram of soil equivalent with a pH of five would indicate a potentially aggressive situation, and a comprehensive analysis of the site and countermeasure should be required. Such aggressive situations occur very rarely naturally, and are generally manmade, such as sanitary landfills and industrial waste disposal areas. In an installation with no movement or slow movement of groundwater, the acid in contact with the concrete pipe will be neutralized and form a neutral zone which stops further corrosion. For installations with significant groundwater flow, limestone backfill has been successfully used as a neutralizing barrier to prevent corrosion of the concrete pipe; and, also, an impermeable backfill material, such as clay, has proven to be an economical and successful barrier which prevents flow from reaching the concrete pipe.

In the pipe interior, acid attack can occur from two sources. The first source is the hydrogen sulfide cycle which may occur in sanitary sewers. Under proper circumstances, sewage can generate hydrogen sulfide gas which may be converted to sulfuric acid on the unsubmerged crown of the pipe. Several scientific breakthroughs now enable the generation of hydrogen

sulfide to be controlled in existing sewers and predicted in new sewers, and, in new sewers, if the problem cannot be alleviated by proper system design, then the concrete pipe can be designed to be sufficiently resistant to acid attack so as to meet the required project service life (2). Acid attack resulting from sulfuric acid generated by hydrogen sulfide gas in sewers is limited to the unsubmerged interior crown of the pipe, and is affected by a number of factors, including effluent properties and velocity, and total alkalinity of the pipe (2).

Confusion prevails regarding sulfates, sulfides and sulfuric acid. Potentially aggressive sulfates are the soil alkalies found in dry western areas. Sulfates must penetrate the concrete and be concentrated by evaporation to cause disruption. The use of Type II cement is recommended to make cast-in-place concrete more sulfate resistant. Sulfides in sewage do not attack concrete. Hydrogen sulfide gas does not attack concrete, but it does attack iron, steel and other metals, and is toxic and flammable. Under favorable conditions, hydrogen sulfide gas is converted to sulfuric acid on the crown of the sewer pipe. Sulfuric acid attacks the surface of concrete, iron, steel and other materials. Type II cement does not make concrete more resistant to acid attack, although it is erroneously specified as such by some agencies and engineers.

The second source of interior acids is the effluent. Occasionally, an effluent can contain some acid - in culverts, mine acid drainage could be a problem; in sewers, acids can be dumped in from a variety of sources. An acidic effluent will attack most pipe materials, and the area of attack is limited to the pipe invert, or the submerged portion. In any case, in the states, it is illegal to dump acid in a sewer or stream. Pretreatment is required and has successfully alleviated corrosion problems. Acid attack by acidic effluents is limited to the wetted perimeter, and is affected by pH, total acidity, effluent velocity, and total alkalinity of the pipe.

If acids are encountered, and cannot be alleviated by other countermeasures, for either interior or exterior acids, a precast concrete pipe can be produced with a higher total alkalinity, increased concrete cover, a barrier coating or lining, or any combination of these. Additionally, for exterior exposure only, the backfill material can be either of low permeability, so as to inhibit acid replenishment, or calcareous aggregate, so as to neutralize the acid. Table 1 lists evaluation procedures and possible countermeasures for interior and exterior acids. Neither Type II nor Type V cement will increase the resistance of concrete to acids.

Sulfates

Sulfate problems have been almost exclusively limited to exposed cast-in-place concrete structures located in arid areas of North America with alkali soils. The U.S. Bureau of Reclamation has wide experience in these areas, and has developed general criteria for evaluating sulfate environments. The Bureau states, however, it has not found any sulfate problems in buried precast concrete pipe (1). The resistance of precast concrete pipe to sulfate attack is easily understood in view of the Bureau's guidelines for preventing sulfate attack in exposed cast-in-place concrete, and the mechanism of sulfate attack. Besides use of Type II or Type V cement, and fly ash, the Bureau indicates sulfate resistance is increased by accelerated curing, high cement content, and low absorption - exactly the characteristics of precast concrete pipe.

Sodium, magnesium, and calcium sulfates in soil, groundwater, or effluent may be aggressive to concrete, if absorbed and concentrated by an evaporative surface in sufficient quantities within the concrete. The reaction of sulfates with certain concrete constituents results in expansive products which may disrupt the concrete. With respect to buried precast concrete pipe, sulfate problems are inhibited by a lack of the proper mechanism to concentrate sulfates in the concrete, and further inhibited by the high strength, low absorption properties of precast concrete. Table 2 lists the relative severity of various sulfate soil and water conditions. Table 3 lists the exposure conditions which must be present for a sulfate problem to exist and some recommended countermeasures.

Chlorides

The most significant aggressive action of chlorides is corrosion of steel in reinforced concrete. Most problems occur as damage to bridge decks resulting from use of de-icing chemicals. Maintenance problems have also been encountered with reinforced concrete seawater structures, such as pilings and piers, because of chloride-induced corrosion of the reinforcement. Portland cement concrete protects embedded steel against corrosion under conditions that would be highly corrosive to bare steel.

[illegible] the passivating effect of [illegible] the reinforcement steel may [illegible] Research has established [illegible] concentration at the [illegible] interface required for corrosion to [illegible] present to support corrosion. Concentration effects which generally occur along with enhanced oxygen availability can produce the critical chloride ion concentration at the steel-concrete interface that is needed to induce corrosion. These effects will more readily occur under the following conditions; low quality concrete of high permeability and porosity, cracks, and the inclusion of calcium chloride in the concrete mix.

A number of conditions can reduce the severity of chloride attack. Increased concrete cover will normally extend service life but will not prevent eventual corrosion under severe exposure conditions. High quality concrete with low permeability, and the absence of cracks and voids, will also extend the life of the pipe under severe exposure conditions but will not prevent eventual corrosion if the mechanism of chloride build-up continues. Under extreme exposure conditions, the use of barrier type coatings is probably the most effective alternative.

Seawater has approximately 20,000 parts per million of chloride. Many concrete pipe installations are completely immersed in seawater and are performing satisfactorily after many years. This is primarily due to low oxygen solubility in high chloride waters plus the extremely low diffusion rate of oxygen through the saturated concrete cover.

As with sulfates, to cause corrosion, chlorides must be in solution, permeate the concrete, be concentrated, and, also, have a ready supply of oxygen. There are no reports nor evidence of any chloride induced corrosion problems with buried precast concrete pipe. Again, this absence of problems is attributed to a lack of the proper mechanism to concentrate chlorides in concrete, a lack of oxygen, and the high strength, low absorption properties of precast concrete pipe.

SERVICE FACTORS AND SIGNIFICANCE

There are a number of purely physical characteristics of the installation which directly and significantly influence the severity of exposure to potentially aggressive factors, including pipe wall hydrokinetics and exposure.

Hydrokinetics

With water at equal pressure on both sides of a pipe wall, the concrete becomes saturated, stability is reached, and there is no water movement through the pipe wall. With a differential pressure, the hydraulic gradient causes movement of water through the wall, along with whatever salts, alkalis, sulfates, and other chemicals are in solution in the water. Direction of flow is highly significant. If the aggressive water were on the side of low pressure, the movement of non-aggressive water through the wall would tend to mitigate any effect. In either case, with no exposure to the atmosphere, there is no concentration effect. With an evaporative surface condition, water movement is due to either hydraulic gradient or capillary action, and there would be a concentration at or near the evaporative surface of whatever chemicals are in solution. These considerations are not relevant to acid environments, since acid attack is essentially confined to the exposed surface. They are significant, however, in evaluating severity of sulfate or chloride exposures.

Full Atmospheric Exposure - Full atmospheric exposure can be a severe condition for concrete pipe. Depending upon climate and location, the exterior of the pipe could be subjected to freeze-thaw cycles, thermal stresses, chlorides in coastal areas, and concentration effects of whatever salts or sulfates are in solution in the effluent.

Partial Burial - Partial burial can be a severe exposure condition. Only a partially evaporative surface is provided, but the concentration effect is more complex since the source of salts or sulfates may be either the effluent or moisture from the ground entering the pipe wall through capillary action and moving toward the evaporative surface.

Full Burial - Buried pipe usually is not exposed to freeze-thaw or thermal stresses, and concentration effects are negligible. When installed above the water table, the hydraulic gradient within the pipe wall of a partially filled pipe is toward the outside, and directly opposite when below the water table. The former condition is more critical for high sulfate or chloride-containing effluents, while the latter is more critical for aggressive groundwater. If the pipe is located between the minimum and maximum groundwater elevations, the hydraulic gradients would reverse on a cyclical basis.

Replenishment

Certain installation characteristics have particular significance in relation to acidic groundwater exposure. The high alkalinity of concrete pipe will almost immediately neutralize acid that comes in contact with it, and the reaction will result in some loss of the concrete surface. For this reaction to continue, there must be replenishment of the acid at the concrete surface. The rate of this replenishment at the external surface of the pipe depends upon the relative permeability of the backfill and bedding material in the pipe zone, the location of the pipe with respect to the water table, and to the fluctuation of the water table. These latter characteristics have been categorized as essentially quiescent, moderately fluctuating, and grossly cyclic. A tightly compacted clay around the pipe will create a relatively impermeable zone, minimizing the potential rate of replenishment. Conversely, a permeable zone around the pipe will not impede the free movement of groundwater and tends to maximize the replenishment potential. The least potential problem with either type of zone exists when the pipe is above the water table. The situation most conducive to replenishment is a permeable zone between the high and low water table, with grossly cyclic groundwater fluctuations, where both horizontal and vertical movement of groundwater can take place. A calcareous backfill, such as limestone, can provide a highly alkaline barrier around the pipe and neutralize the acid before it can contact the pipe wall.

SIGNIFICANCE OF CONCRETE PIPE PROPERTIES

The properties of concrete pipe that may influence its durability are compressive strength, density, absorption, water-cement ratio, cement content and type, aggregates, and total alkalinity. Reinforcement cover, and admixtures, also, may influence the durability of concrete pipe.

Concrete Compressive Strength

Concrete compressive strengths are a function of available aggregates and cement, mix design, inherent characteristics of certain manufacturing processes, and curing procedures. Higher strength usually means overall higher quality, i.e., greater abrasion resistance, lower permeability, and greater resistance to weathering and chemical attack. Minimum concrete compressive strengths of 4,000, 5,000 and 6,000 pounds per square inch are required by ASTM standards. The strengths relate to structural, not durability, considerations and are attained within a short period of time. The 28-day compressive strengths are much higher, often exceeding 8,000 pounds per square inch.

Density

Concrete density of pipe ranges from 135 to 165 pounds per cubic foot. The higher densities are achieved by greater consolidation of the concrete, higher specific gravity aggregates, or by a combination of the two. Higher densities attained exclusively through the use of aggregates with higher specific gravity are not necessarily indicative of an improved level of concrete durability.

Absorption

Absorption is an indicator of the pore structure and is considered by some to be related to the durability of the concrete. Absorption of the cured concrete is influenced significantly by the absorption characteristics of the aggregates and the inherent characteristics of the manufacturing process. Hydration of the cement, which continues under the normally favorable installed pipe environment, further reduces initial absorption values.

Water-Cement Ratio

Precast concrete pipe is produced with a low water-cement ratio concrete. The water-cement ratio is so low for machine-made pipe that the concrete is said to have a negative slump, which means that water would have to be added before any slump would occur. Cast-in-place concrete mixes are designed with much higher water-cement ratios, and placed with slumps ranging from two inches up to the maximum limited by size of the aggregate, resulting in relatively low strength concrete with excessive voids.

Cement Content

A high cement content is normally used by precast concrete pipe manufacturers for a variety of reasons, but mainly because of manufacturing requirements. Other things being equal, increased cement content leads to lower absorption, higher compressive strength and increased resistance to weathering, freeze-thaw, and certain chemical environments. It may also increase the probability of shrinkage cracking, which must be balanced against potential benefits.

Cement Type

Cement used in the manufacture of concrete pipe normally conforms to the requirements of ASTM C 150. Types I, II and V differ primarily in the allowable levels of tricalcium aluminate, C_3A, content. C_3A is the ingredient in cement which is principally involved in the disruptive expansion caused by sulfate reactions. Concrete made with lower C_3A contents provides greater resistance to sulfate attack. Since cements are made from locally available materials, some

Type I cements have less C_3A than allowed by ASTM C 150 for Type V. Unless unusual sulfate resistance is required by the project specifications, or unless the type of cement is otherwise specified, concrete pipe is usually manufactured with Type I cement. Type II and Type V cements, and low C_3A contents, do not increase resistance of concrete to acid attack.

Aggregates

Aggregates used in concrete pipe must meet the requirements in ASTM C 33, except for gradation. Gradation is established by the pipe manufacturer to provide compatibility with a particular manufacturing process, to achieve optimum concrete strength, and to control permeability. Other things being equal, harder and denser aggregates produce concrete with greater abrasion resistance. Aggregates that react with cement are rarely, if ever, a problem with pipe. Aggregate sources are carefully tested and selected by the individual pipe manufacturer, and any problems would be clearly evident in pipe stockpiles.

Alkalinity

Total alkalinity of concrete has a greater influence on the ability of concrete to resist acid environments than any other property. All portland cement concrete is alkaline and will react with acid. Total alkalinity is a measure of the total reactivity of any given mass of concrete. A given mass of concrete with a total alkalinity of 40 percent will react with and neutralize twice the volume of any specific acid as would the same mass of concrete with a total alkalinity of 20 percent. Concrete pipe made with an aggregate which is nonreactive with acid, such as granite, will have a total alkalinity of 16 to 24 percent, depending upon cement content. Using a calcareous aggregate, such as dolomite or limestone, can increase the total alkalinity to as much as 100 percent. Suitable sources of calcareous aggregates are not readily available in all geographical areas, and requiring their use could increase the cost of the pipe. All means of increasing durability for a specific aggressive environment should be evaluated by cost/benefit analyses.

Concrete Cover

Minimum concrete cover over the reinforcing steel are specified in ASTM standards. These minimum covers represent a balance between structural efficiency and durability. Assuming both structural adequacy and proper crack control, greater durability is provided against a variety of aggressive conditions by a thicker concrete cover. A modification of cover to increase durability, however, requires re-evaluation of the structural design of the pipe, and possible use of non-standard forms which could lead to significant increases in pipe costs.

Admixtures

Admixtures sometimes used by concrete pipe manufacturers include calcium chloride, air entraining agents and water reducing agents. Air entrainment agents, which are normally used only in wet-cast pipe, increase freeze-thaw and weathering resistance. Water reducing agents are used to provide adequate workability with drier mixes. With the same cement content, water reducing agents can reduce absorption and increase compressive strength. Calcium chloride, while accelerating setting time, tends to reduce resistance to sulfate attack. Chlorides are also related to potential reinforcement corrosion. The use of admixtures should be evaluated as to possible effects on durability performance.

DURABILITY DESIGN

Most aspects of buried pipeline design, from flow determination to loads to structural analysis, are very well established. The aspect of durability, however, is not as well understood by designers, and therefore, generally not given proper nor adequate consideration. When bids are requested on alternate materials, a least cost analysis should be performed (3). Any consideration of durability, and least cost analyses, must begin with definitions of the required project service life and the proven performance of pipe materials.

NCHRP Synthesis No. 50, "Durability of Drainage Pipe," defines service life by the number of years of relatively maintenance-free performance, and states that a high level of maintenance may justify replacement before actual failure occurs (4). As service life guidelines, the synthesis states designers generally are looking for relatively maintenance-free culvert performance for at least 25 years in secondary road facilities; for 40 years or more in primary highway, urban transit, or rail facilities; and longer service life requirements for hard-to-place culverts in key urban locations or under high fills. The synthesis also states that a durability safety factor of at least two should be used to assure that the structure will definitely serve its required life span. Sewers are key facilities and constructed in urban areas where any installation is

costly in terms of both dollars and public inconvenience, and should be designed for a service life at least the same as required for pipe under high type road facilities.

CONCRETE PIPE PERFORMANCE

For all normal, everyday installations, the service life of concrete pipe is virtually unlimited. For example, some of the Roman Aqueducts are still in use after more than 2,000 years, and there is a buried concrete pipeline in Israel that was tentatively dated as 3,000 years old (5). The first known concrete pipe sewer in North America was located, and five sections removed in September, 1982 for inspection and historical purposes (6). Installed in Mohawk, New York in 1842, this six-inch precast concrete pipe is in excellent condition after 140 years, and the sections remaining in service are expected to perform for several more centuries.

A search for precast concrete pipe durability problems indicates very few problems exist, and consequently very few investigations have been conducted and published (4, 7). By 1982, 33 states and numerous researchers had performed culvert surveys and investigated the durability of pipeline materials since 1925, resulting in 131 reports. Since the durability of concrete pipe is so evident, and research money is normally spent only on problems, 63 percent of the reports are concerned primarily with the deterioration and short service life of corrugated metal pipe; 28 percent of the reports cover multiple pipe materials; and five percent of the reports deal with only concrete. In 1982, the Ohio Department of Transportation published a major report on the results of a ten-year study of more than 1,600 culverts in all areas of the state, which included 545 precast concrete pipe installations (8). The environmental conditions in Ohio are relatively neutral, as are most areas of North America, and the soils and water do not possess any characteristics which would contribute to premature deterioration of pipe, except for a few areas with mine acid drainage problems. A look at the overall study indicates the excellent performance of concrete pipe. Of the 519 concrete culverts studied, only nine were rated in poor condition, 33 in fair condition and 477 in good to excellent condition. Of the nine in poor condition, one has been repaired, and repairs are contemplated for the other eight. An equation for predicting service life was developed for precast concrete pipe, which relates pH and pipe slope to the number of years for the pipe to reach a poor condition. With the equation plotted graphically, Figure 1, it is readily apparent that a concrete pipe placed on an average slope of 1-1/2 percent, and installed in an environment with a pH of 7, will take about 1,000 years to reach a poor condition; and, in an aggressive environment with a pH of 4, the concrete pipe will last 100 years, which is adequate for any sewer or high type road facility.

SUMMARY

Precast concrete pipe has served in an impressive fashion for well over 100 years, is being installed at a rate of more than 1,000 miles a month in North America, and has experienced very few problems. These problems have been identified, related to very specific environments, and adequate countermeasures developed to alleviate the problems.

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6. "First Concrete Sewer Pipe Found," Concrete Pipe News, February, 1983.

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TABLE 1: ACID EXPOSURE EVALUATION AND COUNTERMEASURES

EXPOSURE CONDITION	EVALUATION PROCEDURES	COUNTERMEASURES
Interior	1. For potential biochemical problems in sanitary sewers, determine rate of of acid development, if any. 2. For acidic effluents, determine pH, including cyclic variations, as well as continuous or intermittent flow characteristics.	1. Increase total alkalinity by use of calcarious aggregates. 2. Increase concrete cover as sacrificial concrete. 3. Use barrier lining.
Exterior	1. Accurately determine pH and total acidity. 2. Evaluate installation condition from the standpoint of the potential acid replenishment.	1. Increase total alkalinity by use of calcareous aggregates. 2. Increase concrete cover as sacrificial concrete. 3. Use low permeability clay backfill. 4. Use calcareous backfill. 5. Use barrier coating.

TABLE 2: DEGREE OF SULFATE ATTACK

Relative Degree of Sulfate Attack	Percent Water-Soluble Sulfate in Soil Samples	PPM Sulfate in Water Samples
Negligible....	0.00 to 0.10	0 to 150
Positive[1].....	0.10 to 0.20	150 to 1,500
Severe[2].......	0.20 to 2.00	1,500 to 10,000
Very severe[3]..	2.00 or more	10,000 or more

1 Use Type II cement.

2 Use Type V cement, or approved portland-pozzolan cement providing comparable sulfate resistance when used in concrete.

3 Use Type V cement plus approved pozzolan which has been determined by tests to improve sulfate resistance when used in concrete with Type V cement.

TABLE 3: SULFATE EXPOSURE CONDITIONS AND COUNTERMEASURES

Exposure Conditions	1. Sulfates must be in solution. 2. Hydrostatic gradient exists. 3. Evaporative surface to produce concentration effect exists.
Countermeasures	1. Reduce C_3A content of cement. 2. Use accelerated curing. 3. Increase cement content. 4. Use pozzolans. 5. Decrease absorption.

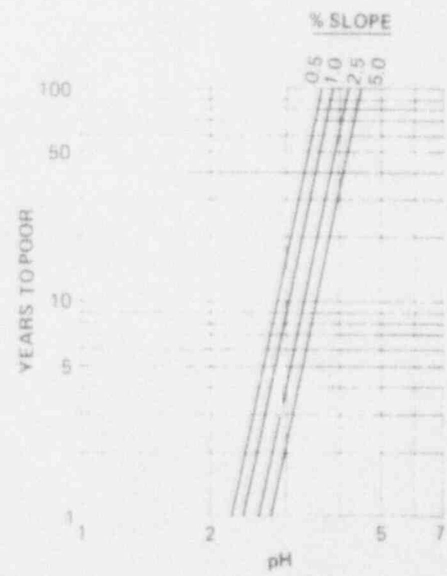

Figure 1 Concrete Pipe Culvert Life