3.4 Inadequate Aggregate

Description:

Aggregates occupy 60-75% of concrete volume and therefore have strong influence on its physical, chemical, and thermal properties. Aggregates also determine the concrete's mix proportions and economy since it is the least expensive component.

The aggregate's gradation, strength, porosity, mineral composition, hardness, and chemical stability are evaluated through a set of ASTM standards which determine its suitability for use in concrete. Aggregate with deficiencies in any of these properties may still make good concrete when combined with proper ingredients in a proper mix design. Therefore, failure in any of these tests does not automatically disqualify the aggregate for use in concrete. The aggregates used at CR3 were accepted, after failing to meet some requirements, based on their ability to produce concrete with the specified strength and on long term local experience.

A complete discussion of aggregate properties and use is beyond the scope of this document. We will review the original requirements, tests, and choices made in the selection of aggregates, and how these aggregates affected the properties of the concrete in the containment structure.

Data to be collected and Analyzed:

1. Original project specifications (FM 3.4 Exhibit 1)
2. Reports on testing and evaluation of the aggregates during construction (FM 3.4 Exhibit 2 is Law Engineering report)
3. Aggregate Quality Control (QC) program at the batch plant (FM 3.4 Exhibit 11 is a summary followed by original detailed test records of aggregates)
4. Available information on the Florida aggregates (FM 3.4 Exhibit 12).
5. Physical properties of the concrete
6. Chemical properties of the concrete
7. Petrographic analysis reports (FM 3.4 Exhibit 3, 4, 5, 6, 7)

Verified Supporting Evidence:

a. Aggregate grading was deficient, causing modifications to the mix design and an increase of 18% in cement used relative to the original approved mix-design (FM 3.4 Exhibit 11 is a summary and details of grading reports during construction; details of mix-design modifications are discussed in another (FM 2.6) Failure Mode).
b. The coarse aggregate included up to 50% softer particles that reduce the concrete's modulus of elasticity.
c. Analysis of concrete strength records lead to the conclusion that aggregate properties limited concrete strength development. It was observed that the seven (7) and twenty-eight (28) day strengths were higher for concrete mixes that had an addition of

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cement and reduction in water/cement (W/C) ratio compared to the original mix-design. However, by ninety (90) days the strength difference between concrete mixes almost disappeared and core tests do not show a significant difference between mixes with different cement content and W/C ratios. Tests of grout cores (Concrete without the coarse aggregate used to start each pour) were significantly stronger than concrete from the same pour.

Verified Refuting Evidence:

a. Project specifications required aggregate compliance with ACI 301 and ASTM C 33 and concrete strength of 5000 psi at 28 days.
b. Original testing program (Law Engineering FM 3.4 Exhibit 2) confirmed that the aggregate complied with ASTM C 33 requirements and project specifications, and that the concrete met project specifications.
c. Aggregates were tested for Alkali Reactivity and passed the ASTM C-277 mortar bar test. Subsequent Petrographic analysis certified the aggregate as innocuous.

Discussion:

a. Aggregate test data from the time of original construction is very limited. It was therefore necessary to use industry reference report (FM 3.4 Exhibit 12) and current tests in the evaluation process.
b. Multiple Petrographic reports over the life of the structure provided the following:
   i. Petrography identified the coarse aggregates as crushed limestone of various types, mainly microcrystalline and fossiliferous, many of which exhibit high porosity and permeability. (FM 3.4 Exhibit 3, 4, 5, 6, 7)
   ii. FM 3.4 Exhibit 4 indicated that about half of the coarse aggregate (CA) particles were dense and firm while the rest were porous with chalky texture and contained shell fragments and fossils.
   iii. Mactec report (FM 3.4 Exhibit 7 & 7a) included Mohs hardness test results of approximately 3 – a low value corresponding to soft material that is about 10% the hardness of the fine aggregate (Quartz).
   iv. There was no evidence that the aggregate had been either chemically or physically unsound (FM 3.4 Exhibit 3, 4). PCA report (FM 3.4 Exhibit 5) identified one CA particle similar to those causing ASR reactions in Florida aggregates; however, no actual reaction was reported. CTL report (FM 3.4 Exhibit 6) agrees that there was no evidence of deleterious chemical reactions. Mactec report (FM 3.4 Exhibit 7 & 7a) identified CA pieces that retained moisture longer than other portions of a sample and exhibited localized evidence of alkali silica reaction (ASR).
   v. The aggregate contained less than 0.1% clay lumps (FM 3.4 Exhibit 4) while ASTM C 33 allows up to 5.0%; these soft and friable particles are expected to degrade during mixing and do not pose a problem.

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   vi. Aggregate grading was deficient in the finer sizes of the coarse aggregate and coarser sizes of the fine aggregate (FM 3.4 Exhibit 3).
   vii. According to CTL report (FM 3.4 Exhibit 6) the aggregate/paste bond was tight and there was a "lack of major cracks and microcracks.

c. Exhaustive search of reports and publications over the years provided information regarding the properties of the Brookville coarse aggregate (FM 3.4 Exhibit 12 is a summary). The highlights of the relevant properties include:
   i. Shrinkage of concrete made with limestone is lower than most other types of aggregate (FM 3.4 Exhibit 12a). However, the Brooksville aggregate used at CR3 exhibited the highest average shrinkage rates when compared to other Florida limestone.
   ii. Thermal coefficient of expansion for limestone is the lowest of all common aggregate types (FM 3.4 Exhibit 12b). Thermal diffusivity is among the highest of all aggregate types, indicating the enhanced ability to transfer heat through the material. There are no specific studies for the Brookville aggregate.
   iii. Aggregate reactivity potential exists in the Brooksville limestone which may include lenses of reactive silica (FM 3.4 Exhibit 12c). However, the reactive silica is encountered in minimal quantities, and tests confirm that the aggregate is innocuous.
   iv. Modulus of Elasticity (Ec) of concrete made with Florida limestone is usually low and may be only 90% of the value calculated from the ACI 318 formula for normal concrete (FM 3.4 Exhibit 12d). Tests performed in 2009-2010 at CR3 on cores from the containment structure reveal much lower values than would be expected from high strength concrete (average measured Ec was 3.5E6 psi compared to calculated values of over 5E6 psi when using either compressive strength or weight in ACI 318 formulas). This deficiency is clearly caused by the coarse aggregate properties.
   v. The aggregate is relatively dense (higher specific gravity) with relatively high absorption (FM 3.4 Exhibit 12e). Its acid-soluble chloride content is very low (less than 0.007% by weight).

d. The coarse aggregate did not meet all the requirements of ASTM C-33 as specified:
   i. According to FM 3.4 Exhibit 8 (appendix A attachment 2) the aggregate failed the ASTM C 88 test for soundness of aggregate (13.47% loss instead the 12% loss allowed) but was accepted based on service record.
   ii. Initially, the coarse aggregate did not meet the ASTM C-117 test for 200 sieve wash loss (2.3% compared to 1% maximum allowed according to FM 3.4 Exhibit 9). The ASTM C-117 test was repeated on a washed sample which passed the test (loss of 0.8%). According to FM 3.4 Exhibit 8 (appendix A attachment 2) the specified limit was raised to 1.5% to account for the fact that the material is crushed aggregate.

e. Deviations from ASTM C 33 grading requirements were made for Fine Aggregate based on satisfactory experience in local applications and the fact that both the US Corp of Engineers and the Florida State Road Department have modified the

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specifications to allow the use of these native aggregates. ASTM C 33 (AASHTO M 6) requirement for Fineness Modulus (FM) range of 2.3-3.1 was changed at CR3 to 2.2-2.4 to match the finer local sands.

f. The aggregate was ultimately accepted based on its local record of use and serviceability as well as acceptable properties of the concrete made with it (as allowed by ASTM C-33).

g. Analysis of aggregate grading (FM 3.4 Exhibit 11) establish the following:

   i. Both coarse aggregate and fine aggregate are lacking in particles in the #8 to #16 size range. As a result, the combined aggregate is "gap graded" and required additional cement to achieve the workability needed (cement quantities ranged from 635 lbs/cu.yd. in the initial mix-design to 682, 705 and 752 lbs/cu.yd as the project progressed. The strength and workability requirements of the concrete were met, but the additional Hardened Cement Paste (HCP) may have contributed to other potential problems such as creep and shrinkage.

   ii. Fineness Modulus (FM) was used as acceptance criteria for fine aggregates and shipments with FM below 2.2 were rejected. The coarse aggregate did not have FM as acceptance criteria and shipments with FM exceeding the recommended 6.5-6.9 range were regularly incorporated into the concrete (see Summary of Quality Control (QC) reports in FM 3.4 Exhibit 11).

h. Analysis of concrete strength records lead to the conclusion that aggregate strength limited concrete strength development. It was observed that seven (7) and twenty-eight (28) day strengths were higher for concrete mixes that had an addition of cement and reduction in water/cement (W/C) ratio compared to original mix-design. However, by ninety (90) days the strength difference between concrete mixes almost disappeared and strength tests of cores do not show a significant difference between mixes with different cement content and W/C ratios.

   Grout cores (Concrete without the coarse aggregate that used to start each pour) tested significantly stronger than concrete from the same pour.

   These observations indicate that the presence of aggregate limits the tested strength of the concrete. This conclusion is further validated based on the observation of core fracture – where all cracks propagate through the aggregate and not around the interface zone.

   Although the concrete at CR3 is in full compliance with design criteria (5000 psi at 28 days) the aggregate limited the concrete's ability to reach the full strength potential of its low W/C and cement paste properties.

i. The relationship between compressive strength (f_c) and modulus of elasticity (E_c) has many different expressions in the literature and codes. Using these empirical relationships, the CR3 concrete should have E_c in the range of 4.75 to 5.25x10^6 psi. However, the measured E_c was substantially lower, averaging 3.4x10^6 psi. This can be explained by the fact that about 50% of the coarse aggregate was softer rock with significant voids – resulting in a lower than normal modulus of elasticity for the aggregate.

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In general, aggregate properties will affect concrete E_c much more than its compressive strength. Accordingly, concrete made with weaker aggregate can meet strength specification while failing to achieve the E_c calculated from design formulas.

j. The aggregate impact on creep/shrinkage properties is governed by two mechanisms – it will either restrain creep by providing a volume of inert hard material, or increase creep by providing a weakened interface zone on its surface. Therefore, for the same aggregate volume, fine aggregate will have more surface area than coarse aggregate and will require more cement paste with a larger effect on the creep properties. At CR3, the aggregates' grading resulted in increase of the finer fractions with the corresponding increase of creep and shrinkage potential.

Conclusion:

Based on the available evidence it is concluded that the aggregate used for concrete at CR3 was deficient and may have been the cause of reduction in creep and shrinkage properties of the concrete. It also produced concrete whose modulus of elasticity was substantially lower than the values predicted by empirical relationships in design manuals and codes. These reductions may have contributed to the delamination at the containment building (Failure Modes 4.1 to 4.6 determine if creep and shrinkage were excessive and contributed to the delamination).