

Dispute in Heavy Duty Concrete Pavement Construction: Significance of Standards

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Abstract

This paper is an essay on the effect of standards and on problems and disputes that commonly develop in construction of heavy duty airfield pavements. It is intended to call attention to the need for improved standards for which research is needed to develop solutions to problems that develop in concrete airfield paving and tend to lead to owner-contractor disputes. Four construction scenarios that appear to suffer from insufficient specifications and consequently result in disputes are presented and discussed. Recommendations for potential improvements are made for each.

Introduction

Conditions sometimes develop during construction concrete air field pavements that are not apparent during construction, but which result in defective concrete that becomes evident at a later time - sometimes many years later. If these deficiencies occur before the expected service life of the pavement, then they can result in claims by the owner on the basis of latent defects. These disputes are costly and cause significant distraction and also have insidious effects for both the owner and the contractor.

The thesis of this essay is that some of these conditions are traceable, at least in part, to deficiencies in specifications and test methods in the form of absence of quantitative and verifiable standards for critical parts of construction.

The objective of this essay is to explore four examples of such effects in construction of heavy-duty airfield pavements and to comment on the plausibility that improvements in specifications and test methods could be helpful. The conditions presented are mortar-rich surface, consolidation, curing, and surface grinding.

Military airfield concrete paving is covered by guide specification UFGS 32 13 11, Nov 2012 (henceforth 32 13 11). Civil airfields constructed under Federal Aviation Administration (FAA) requirements are covered by guide specification FAA P-501, July 2014 (henceforth P-501). These two guide specifications comprise the basis from which project specifications are developed, and will be the focus of the topics covered in this paper.

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Mortar-Rich Surface

As defined in 32 13 11 a mortar-rich surface is a layer concrete at the finished surface of the pavement that is devoid of nominal-size coarse aggregate ≥ 4.75 mm in diameter. Both 32 13 11 and P-501 limit the thickness of a mortar-rich later to ≤ 3 mm (1/8 inch). Figure 1 shows an example of a rather significant mortar-rich layer.

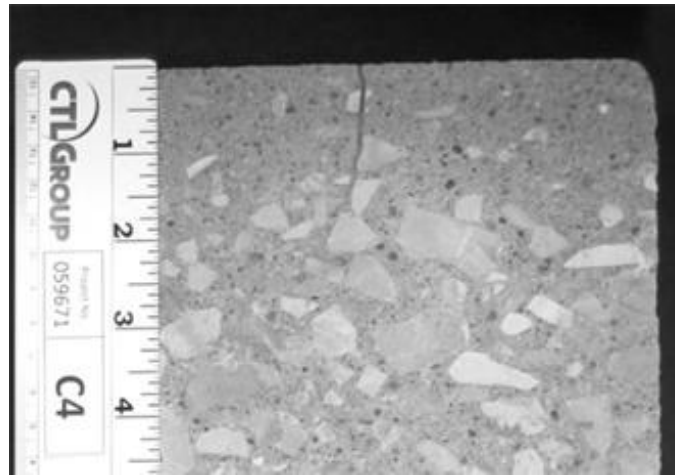


Figure 1. Example of a significant mortar-rich layer. Shrinkage crack is evident near the center of the core surface.

The rationale for the limit on mortar-rich surfaces is that mortars tend to be enriched in cementitious materials relative to the underlying concrete. The underlying concrete tends to represent proportions of the applicable concrete mixture. Hydrated the cementitious material is the fraction of concrete particularly prone to shrinkage cracking. This cracking can eventually lead to deterioration resulting in foreign object debris (FOD) and possibly a reduced surface life of the pavement. The mortar-rich layer shown in Figure 1 also shows such a crack that developed relatively soon after placing. The applicable sections of UFGS 32 13 11 are as follows:

Section 3.6. “Any operations which produce more than **3 mm (1/8 inch)** of mortar-rich surface defined as deficient in plus **4.75 mm U.S. No. 4 sieve** size aggregate) shall be halted immediately and the equipment, mixture, or procedures modified as necessary.”

Section 3.9.5 directs that mortar rich surfaces $< 6\text{mm}$ thick are allowed to be ground (within the limits of Section) to bring them to within the ≤ 3 mm requirement. Panels with mortar rich surfaces $> 6\text{mm}$ thick shall be removed and replaced.

Section 1.3.5 (Thickness) is also pertinent. This section is primarily directed to determining thickness of cores. However, it is also directs that thickness cores be examined for evidence of a mortar-rich layer.

FAA P-501-4.11 (Finishing) essentially duplicates the UFGS requirement. However, there are no requirements, as there are in 32 13 11 3.9.5, explicitly requiring that panels be removed because of excessive surface mortar.

The concept behind limiting a mortar-rich layer appears to be sound. Determination of the tendency of a mortar-rich layer to develop because of basic problems materials, mixture design, and/or paving equipment and operations can normally be investigated during placement of a preconstruction test strip, if required. If not required, then it is unlikely that it will occur due to the reality of the economics of competitive bidding among contractors. Even if a test strip is executed and sufficiently analyzed for this defect, this one-time verification may not be sufficient to carry through the remainder of the construction. The more difficult problem is the detection of a mortar-rich layer during or soon after paving operations before the need to remove and replace concrete becomes a major economic issue.

In concept the determination of the presence of a mortar rich layer >3-mm thick would seem to be simple. However, the detection of such an event during construction is apparently difficult since there is no standard test method for this purpose. The development of a mortar-rich layer during construction may not be immediately detectable on visual inspection, even on the exposed edge of the pavement such as exists in placement of a pilot lane.

This problem is analogous to the edge slump problem. However, for the edge slump problem, there is considerable detail included in specifications on test methods for verifying its presence soon after concrete placing and finishing.

A test method based on the concept of depth of surface penetration of pavement structure soon after placement with a needle (similar to a Vicat needle), or maybe the penetration of a thin straight-edge type device would be effective in detecting the transition between mortar and concrete in the near surface of a newly placed pavement.

In the absence of a method for analysis of newly placed pavement, use of thickness cores for detection of a mortar-rich layer, as in 1.3.5 of 32 13 11, is a plausible approach. These are normally taken at a rate of 8 cores per lot of concrete placed. The problem of course is that the needed information would be available only after several days of placing of the relevant section of pavement. This would be too late to save that section of pavement, although better than not discovering the problem until much later.

Measuring the thickness of a mortar-rich layer at the surface of a hardened-concrete core is more complicated than would first appear to be the case. The beginning of the mortar rich layer is not necessarily represented by a simple boundary between the presence and absence of coarse aggregate. The depth of the coarse particles can vary considerably relative to the 3 mm limit. As illustrated in Figure 2, the perimeter at the top edge of an extracted core can reveal segments where mortar is

considerably more than 3mm thick, while in other places it will be less than 3 mm. In neither of these cases would a purely visual judgement conclude that there was a serious case of mortar-rich layer, but a decision to remove and replace – or not – should not be based on a visual impression.

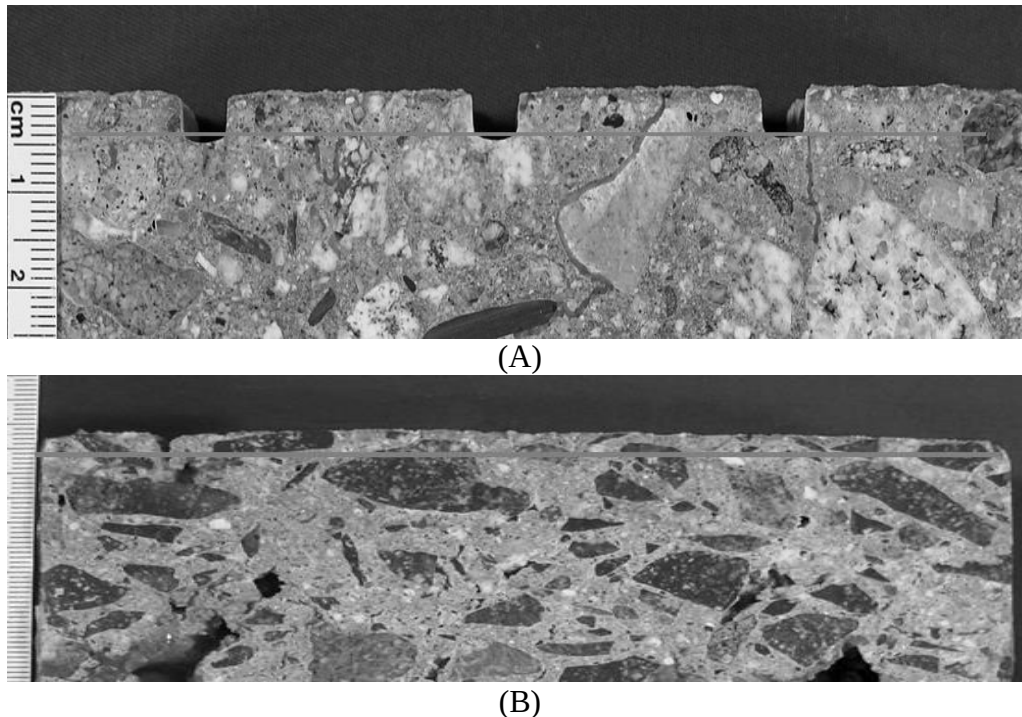


Figure 2. Examples of typical aggregate presentation at concrete pavement surface
Gray line indicates approximate 3 mm depth.

What is needed is a test method that gives precise instructions on how to work with the variation in the mortar-concrete interface and how to determine a test result for a sample. A potential model for such a test method would be ASTM C174 for the measurement of pavement thickness from cores of hardened concrete. This test method requires multiple points of measurement of thickness at prescribed intervals around the perimeter and at the center of the cores. An average is calculated and compared to the acceptance criterion. To be applicable to the mortar-rich condition, some kind of correction would have to be developed to account for the fact that the diameter of an individual piece of coarse aggregate is not exactly determined from the diameter exposed on a cut surface of concrete.

Consolidation

No detailed definition of consolidation is given in either 32.13.11 or P-501, however it is generally understood that consolidation is a process by which voids are removed from concrete. Definitions sometimes use terminology describing the process of increasing the density of freshly placed concrete. Voids in concrete have a negative impact on the strength of a structure because they represent zone in the structure that has no strength.

Except in exceptional cases concrete will always contain some voids. The practical objective is to minimize these to the point that the service of the concrete is not significantly impacted. Consolidation is covered in sections 3.5.2 of 32 13 11 and in sections 4.1.d and 4.8.c of P-501.

Both guide specifications emphasize a prescriptive approach to assuring adequate consolidation. This is based on adjusting the number, spacing, frequency, amplitude, depth of penetration, and angle of vibrators such that complete consolidation of the full depth of the concrete placement is achieved. Verification of these settings is required at prescribed intervals in the QC requirements of the project specification.

Like the problem with mortar-rich layers described previously, poor consolidation may be a property not always detectable during paving. Major consolidation flaws are sometimes detectable by surface features or, in the case of slip-form paving, by examination of a free edge of pavement after the paver passes (Figure 3). Neither of these is definitive as to the extent of the problem in the pavement.

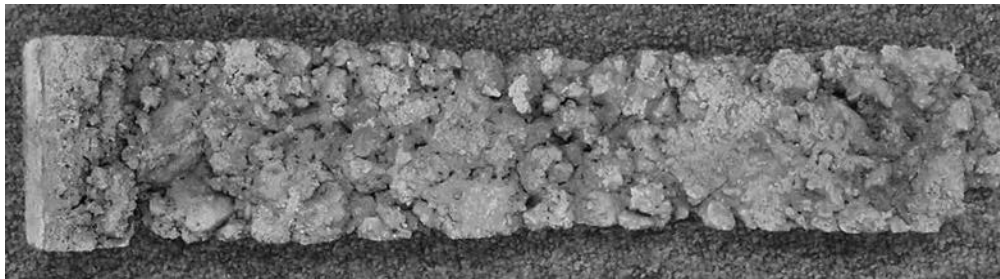


Figure 3. Evidence consolidation issue on an exposed pavement edge.

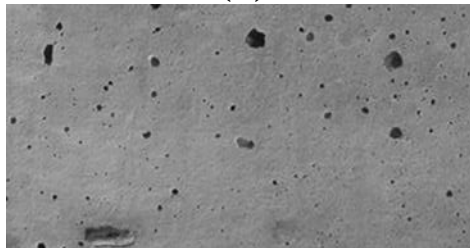
The only way in current practice to definitively determine the extent of any consolidation issues is destructive testing of hardened concrete. In concept use of thickness cores for detection of poor consolidation is a good approach, but results would not be available until after considerable paving has occurred.

One of the problems with using the thickness cores as the basis for the evaluation is that the sampling schemes are not well defined. There is no information on sampling in Section 1.2.5 of 32 13 11. There is no information on how to sample locations. P-501 covers sampling thickness cores in section 8.1 citing ASTM D3665. This standard describes a stratified random scheme in which station number along the length of a pavement lane and horizontal location across the lane at that station are chosen by random sampling. This is the appropriate sampling scheme for this purpose, but some project specifications simply cite “random sampling” without referencing additional detail.

No information was found in 32 13 11 on what constitutes an intolerable number and/or size of voids that would initiate a remove – and – replace requirement. Some individual voids may be so large that some negative impact on properties is expected (Figure 4A) without quantitative assessment. In other cases presence of a large number of small voids may or may not be a problem. No single void or even perhaps a few of them would likely have significant potential for negative impacts on concrete strength (Figure 4B). But a very large number of them would probably have a significant impact the flexural strength of the pavement. The problem this case is measuring the number and size of these smaller voids and determining where the threshold of acceptance should be.



(A)



(B)

Figure 4. Examples of consolidation.

P-501 does contain quantitative guidance on unacceptable consolidation. Section 4.8.c directs that, if poor consolidation is suspected, density of cores taken from the affected area should be tested according to ASTM C642. The acceptance criterion for each sample is that the density of must be $\geq 95\%$ of the theoretical density of the concrete – based on mixture design data and materials properties.

Use of thickness cores and development of PWL requirements for this property as part of QC procedures does not appear to be part of the standard. Also, it should also be noted that the operational principle of C642 is not applicable to the type of poor consolidation shown in Figure 4A.

Research has shown the value of non-destructive techniques for determining void size and distribution of in-place concrete has been conducted in recent years. The author does not know whether these methods could be applied to freshly finished concrete or not. So far these methods do not seem to have become common in field practice.

Curing

Curing can be a highly critical step in PCCP construction, or not so much so, depending on the ambient conditions during and soon after completion of construction. In theory curing is one of the most important aspects of concrete construction given the critical dependence of hardened properties on hydration of the cementitious materials and on the general prevalence of problems associated with shrinkage cracks. ACI 308 (Practice for Concrete Curing) is a good reference for curing and associated problems.

Curing is defined as the management of moisture content and temperature of a concrete structure following placement until sufficient hydration of the cementitious materials has occurred to achieve minimum requirements for physical properties. Inadequately cured concrete is prone to early age cracking and loss of surface strength, which affects abrasion resistance.

The surface of the concrete structure and the concrete within about 50 mm (2 in) of the surface is the most affected part of the structure if improper curing occurs. This is sometimes referred to as the “curing affected zone.” This is particularly important for PCCP structures because of the operational criticality of the surface of the concrete.

Historically, curing protocol for most concrete construction requiring an exposed finished surface was divided into two periods. The first period starts after the placement, consolidation, screeding, and bull floating, and continues until final finishing is complete. Final finishing operations were generally timed to coincide with when concrete was sufficiently set to allow light traffic and so that finishing operation did not result in working paste to the top of the placement. The time of initial setting generally coincided with this timing. This is normally several hours, depending on temperature. This also corresponds with the disappearance of sheen if drying conditions are not too severe.

A somewhat different protocol is generally used in modern slip-form paving. With this construction method the final finishing is complete immediately after the concrete is extruded behind the paver – well before the time of initial setting. Curing in the form of liquid membrane forming curing compound (ASTM C309 or C1315) is then usually applied soon after. If evaporation conditions are severe enough to cause plastic shrinkage cracking the curing compound is commonly applied as soon as possible after the paver. As described below, this practice can have some negative effects on curing compound membrane formation. However this practice was

recommended in a seminal paper on the subject (Lerch 1957) and is commonly used today.

Strictly speaking, the period just after extrusion from the paver is a period during which concrete is still in bleeding mode. This is often not evident because of the low water-cement ratios used in modern concrete. The slight bleeding that does occur is often evaporated as soon as it forms under even modest drying conditions. If the evaporation of bleed water is stopped by the application of the curing membrane, then bleed water that is still forming will accumulate under the newly applied curing compound application. This can have several outcomes with respect to the curing action in the curing affected zone.

1. No detrimental effect on the finished surface. The bleed water is simply trapped and then reabsorbed a few hours later as cement hydration accelerates.

2. The accumulating bleed water will float the newly applied curing compound off of the surface of the concrete. In this case tears can form in the membrane resulting in less-than-maximal water retention (Figure 5).

3. If the curing compound is applied to a concrete that is dry because the bleed water has evaporated faster than it forms, then curing compound can bind to the concrete surface. Subsequent bleeding then can result in formation of a weak layer just under the concrete surface that will result in a delamination of 1 – 2mm of surface mortar at some later time, usually in the first year of service (Figure 6).

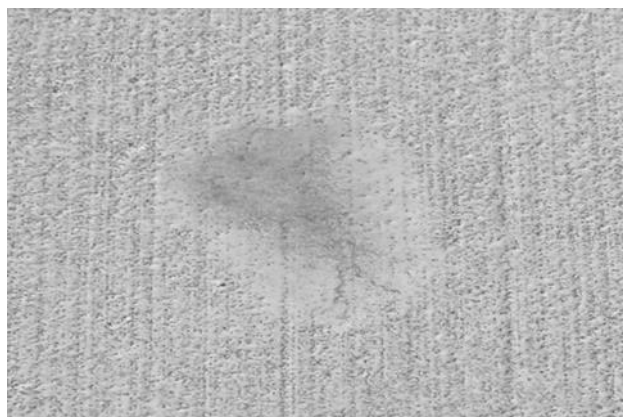


Figure 5. Curing compound blister due to bleeding after application.

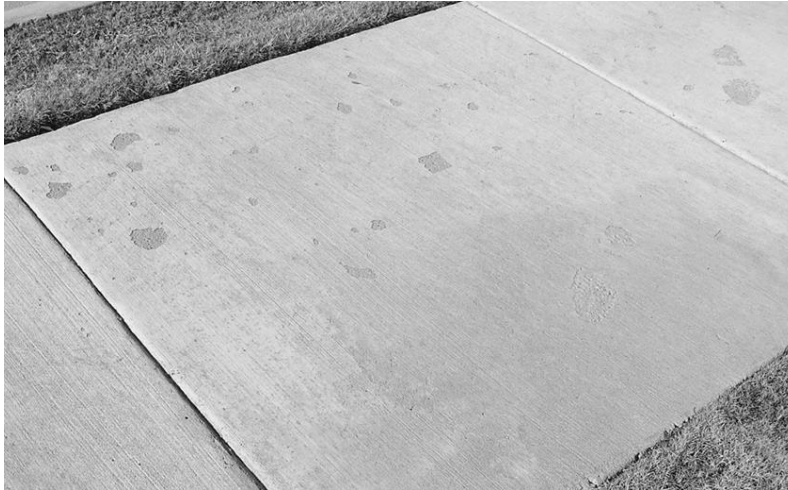


Figure 6. Delamination of surface mortar due to early application of curing compound.

Of course the first scenario is the best one. Based on the low frequency of major problems associated with this phase of the curing, it probably is the most frequent outcome.

The second outcome is not desirable but the damage may not be of major significance since the failure of the membrane is not complete. However, conceivably some weakness in the surface concrete could result from this scenario. This condition is reported from time to time in investigations of concrete surface weakness. The practice sometimes used of application of two light coats of curing compound separated by an interval that allows the first coat to form its membrane may be useful.

With the third outcome the result is mostly cosmetic, but also can result in significant FOD if widespread. This is usually a major concern for military airfields. Loss of a major fraction of surface mortar is never a popular outcome of new construction.

QC and QA procedures for acceptance testing and application of curing compound could probably be improved. Curing compounds for airfield pavements are usually specified by ASTM C309 and tested for acceptance by ASTM C156. The most relevant property is water retention. The limit in C309 is $\leq 0.55 \text{ kg/m}^2$ in 72 h. The standard deviation of that property as tested in C156 is 0.30 kg/m^2 . The standard deviation is rather large given the amount that is being measured. This creates a rather large uncertainty concerning making the correct decision in acceptance testing. Significant improvements in the precision are possible, but funding for this type of research is generally not available.

Determination that the field application of curing compound complies with the project specification is based on the number of containers of curing compound used divided by the surface area on which the application was made. This procedure doesn't account for non-uniformity in application (Figure 7A). Specifications usually call for an application of white-pigmented curing compound at a rate of 5.0 L/m². This is the same application rate applied in C156 testing. C309 requires that the reflectance be at least 60% for this application rate (Figure 7B). Portable field-serviceable reflectometers exist that could potentially be used to verify field applications of white-pigmented curing compounds. This was investigated under laboratory conditions by Poole (2001). The method appears to have potential, but has not been investigated for field use.



(A)



(B)

Figure 7. Examples of poor (A) and good (B) application of white-pigmented curing compound.

Surface Grinding

Surface grinding for purposes of meeting requirements on smoothness and grade are an important part of concrete pavement construction. Until recently 32 13 11 and P-601 contained maximum limits on the amount of a concrete surface area that could be ground before remove-and-replace scenarios develop. Relatively recently guide specifications have been revised to include limits on the maximum depth of grinding.

32 13 11 was modified a few years ago to limit the depth of grinding to ≤ 6 mm ($\frac{1}{4}$ inch). P-501 has a maximum limit of ≤ 13 mm ($\frac{1}{2}$ inch). The rationale for invoking these limits were based some cases in which concrete surfaces ground deeper than these limits showed a tendency for debonding of larger sizes of coarse aggregate particles (Figure 8). This created significant FOD. Petrographic examination (Poole 2005) identified weak bonding between the cement paste and aggregate particles. The logic attributed to the observations was that, in the case of shallow surface grinding, the remnants of coarse aggregate particles near the surface were at least still somewhat encapsulated by mortar, thus inhibiting their debonding. At some deeper grinding depth all of the encapsulating mortar was being removed leaving only the paste-aggregate bond to prevent the remnant from debonding and becoming FOD.

No reasoning was discovered during preparation of this report as to why the military and civil limits differ. Perhaps pertinent is that the military is generally more concerned about FOD issues than are civil agencies because of the differences in types of aircraft and takeoff and landing practices.

Conversations with others in the airfield pavement industry suggest that this has not been a major problem in the past. Apparently the history is that considerable grinding deeper than these limits has occurred without apparent problems. It should be noted that the military is much more concerned with FOD than are civil aviation agencies.

The author is aware of a few instances of contractor-owner disputes over these limits, but does not have a general sense of overall impact of these changes on construction since implementation. It would seem to result in an increased pressure on contractors to the smoothness and grade right on the first pass.

It would be useful to have a test method with which the paste-aggregate bond could be evaluated. This could be the basis for developing performance specification that would be useful for making decisions about whether to allow exceptions to the grinding depth limits.

Petrographers use a semi quantitative procedure for evaluating paste-aggregate interface (ASTM C856). This involves fracturing a sample with a hammer and then examining fragments and counting instances in which fractures followed the paste-aggregate interface or fractured through the aggregate.



Figure 8. Example of coarse aggregate debonding due to deep grinding.

Adaptation of ASTM C1583 for measuring tensile bond strength of concrete overlay materials appears to be a possible approach for establishing quantitative performance limits on a ground surface.

Closing Remarks

That specifications and associated test methods for concrete airfield pavement are not based more on quantitatively verifiable requirements is very likely because these are difficult to develop. Some of the properties are not easy to measure in a timely fashion, and test methods do not exist for others. Also contributing is that research needs on these problems do not match currently available funding very well. The resolution to this problem isn't clear to the author at present.

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