passed over the area on September 10. Panel placement typically began after midnight after the base and bedding layer work was completed. The work area was opened to traffic the next morning by about 6 am. Figure 7 shows the panel being installed. Figure 8 shows the use of leveling bolts to set the panel elevation.

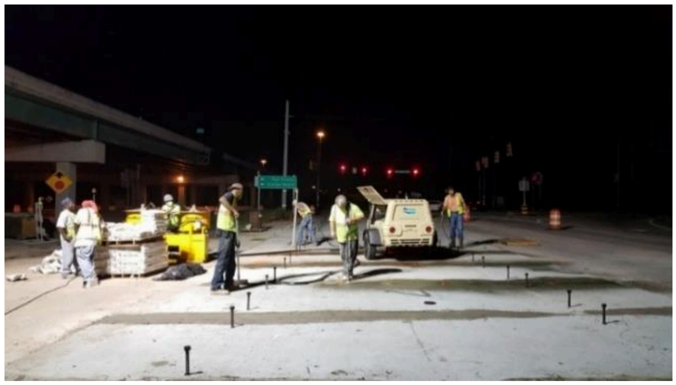


Figure 8. Using leveling bolts to set panel elevation.

Figure 9 shows the completed precast pavement installation. The small gaps at the beginning and end of the precast concrete pavement section were filled with hot-mixed AC material. The longitudinal gap between the precast pavement and the adjacent AC lane was also filled with hot-mixed AC. The shoulder was reconstructed with hot-mixed AC.

Figure 9. Completed precast pavement installation.

CONNECTICUT PROJECT

The Connecticut DOT (CTDOT) chose use PCP for rehabilitating two rutted HMA pavement bus pads along a section of CTfastrak, a bus rapid transit system. Shortly after CTfastrak became operational on March 28, 2015, about 110 ft (33.5 m) of the HMA pavement at each of the two bus pads at its East Main Street Station in New Britain, CT, exhibited rutting. The rutting was most severe along the bus wheel path adjacent to the platforms, and it was deep enough to prevent opening of the bus doors at the platforms. After several patches had been made, CTtransit consulted CTDOT for a permanent solution. CTDOT recommended using PCP to allow the rehabilitation work to be performed over a weekend, thus avoiding disturbing weekday commuter traffic (Tayabji 2016-1). The project specification required the use of a grout-supported PCP system. The contractor elected to use bottom slot panels fitted with leveling lifts that were also used as panel lifting inserts. Figure 10 shows the precast panel layouts for the two bus pads.

Precast Pavement Details

Each station was repaired using 10 PCP panels (typical panels were 9 ft [2.7 m] long), and two 8 ft (2.4 m) long transition panels at the ends. The PCP panels were 10 in. (254 mm) thick and 15 ft (4.6 m) wide. The existing pavement comprised of 10.5 in. (267 mm) thick HMA pavement on a 6 in. (152 mm) thick aggregate base. The specification allowed the contractor to regrade and compact the existing base, supplementing it with new base material. Panels were set at the desired elevation using leveling lifts, and rapid-setting grout was pumped through ports in the panels to create a bedding layer between the base and the panels. The bedding grout was required to reach minimum compressive strengths of 500 psi (3.4 MPa) before the panels could be opened to traffic, and it was required to reach 4000 psi (27.6 MPa) at 28 days.

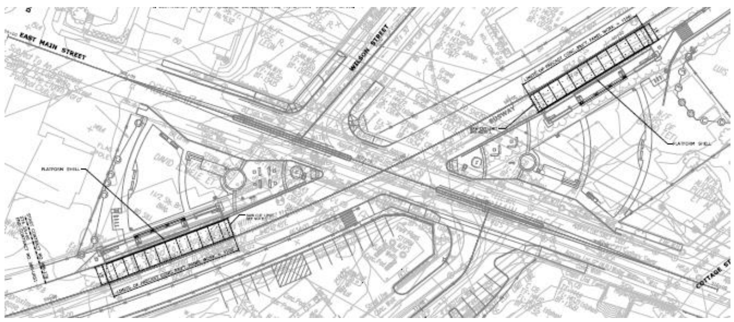
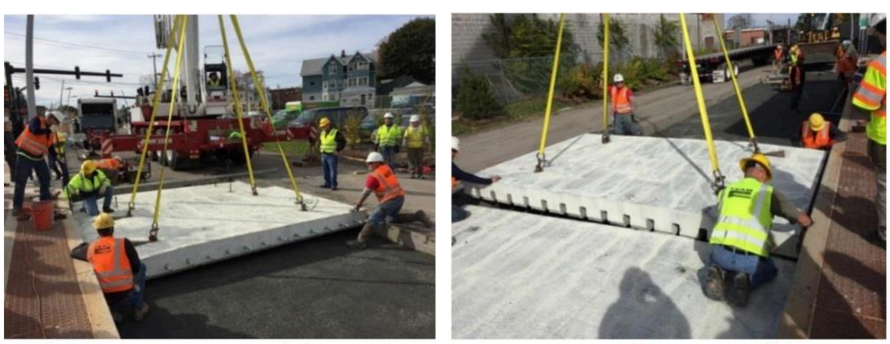


Figure 10. Panel layout of the two bus pads.

The PCP panel's bottom slots were filled with a rapid-setting grout. The grout was required to reach a strength of 2500 psi (17.2 MPa) before the panels could be opened to traffic.

Panel Fabrication

The 24 panels were fabricated at the precast plant in Schuylerville, NY, about 170 miles (274 km) from the project site. The panels were delivered to a staging area near the project site a few days before panel placement. The standard panels were provided with dowel bars along one end (Figure 11(a)) and corresponding dowel bar slots along the other end (Figure 11(b)).



(a) **(b)**

Figure 11. Precast panels used: (a) dowel bars embedded at one transverse side; and (b) dowel bar slots along the opposite side.

Site Preparation and Panel Installation

The work started on a Thursday evening and was substantially complete before rush hour on Monday morning. The work included:

- Removal of the existing HMA pavement at each pad.
- Grading and compaction of the base, including addition of new crushed stone material to correct for any over-excavation of the existing base.
- Placement of 12 PCP panels at each bus pad.
- Adjusting a total of 48 leveling bolts to match PCP panel elevations with the adjacent HMA pavement.
- Grouting the dowel bar slots and filling joint gaps.
- Removing leveling bolts as soon as the dowel bar slot grout had hardened.
- Pumping bedding grout beneath the panels.
- Milling about 3 in. (76 mm) of the HMA surface for a distance of about 10 ft (3 m) from each end of each pad and placing HMA surfacing over of the milled surface and the PCP transition panels, and
- Sawing and sealing joints.

Removal of the distressed HMA pavement was intiated during the night of Thursday, October 27, 2016, when the section designated for removal was sawcut over its full depth into 7 by 5 ft (2.1 by 1.5 m) segments. The HMA pavement sections were removed starting Friday evening, and most of the panel installation activities were completed by early morning on Monday, October 30, 2016. Bus traffic was allowed to use the completed bus pads at 4:00 am, the start of Monday morning's CTfastrak operation. Figure 12 shows a view of the north bus pad with all panels in place and set at the desired elevation. Figure 13 shows a view of the completed north bus pad in operation.

Although a few issues (related to base preparation and panel installation) had to be resolved at the work site, the use of the PCP technology on a production pavement rehabilitation project was an important step for CTDOT. CTDOT was able to complete its first PCP installation in a challenging setting, and it expects to use the findings from this demonstration project to refine specifications and plans for use of PCP technology at future pavement repair and rehabilitation projects along roadways with high traffic volumes.

Figure 12. The north bus pad rehabilitation - panels with leveling lifts engaged.

TEXAS PROJECT

In early 2015, the Texas Department of Transportation (TxDOT) was awarded $300,000 to help offset the cost of implementing PCP technology. TxDOT chose to apply the funds toward rehabilitating the SH 97 and SH 72 intersection (Tayabji 2016-2). Located near Fowlerton, TX, the intersection's HMA pavement was exhibiting extensive rutting—a combination of high temperatures and heavy truck traffic associated with the local energy industry. The SH 97 and SH 72 intersection is T-shaped, with SH 97 as the through highway and SH 72 meeting SH 97 at a right angle, as shown in Figure 14.

Figure 13. The north bus pad in operation.

TxDOT developed generic plans and specifications for use of PCP at the intersection. The plans called for the use of pretensioned PCP panels. Figure 15 shows the panel layout and panel placement stages.

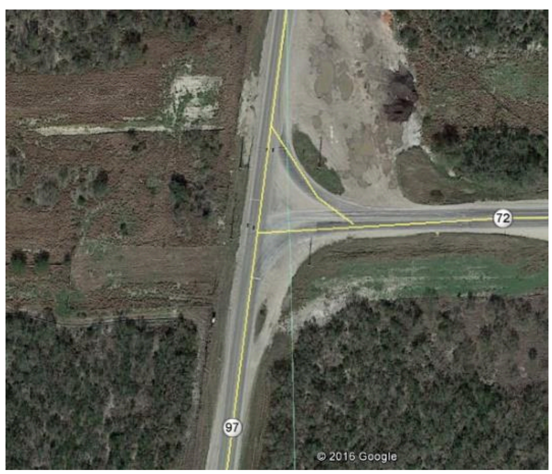


Figure 14. Photo. Layout of the existing AC intersection.

Precast Pavement Details

The project plans called for removal of the existing HMA pavement, cement-treating 6 in. of subgrade, and placing a new 4 in. (102 mm) thick HMA pavement base. The plans called for 12 in. thick PCP panels to be placed directly on the HMA base. Panel elevations were to be adjusted, if necessary, using built-in leveling lifts in the panels. A total of 235 panels were installed. Load transfer across the joints was achieved using 14 in. long dowel bars positioned within dowel bar ducts using mini support chairs. The dowel bars were secured within the ducts using grout. The specified compressive strength for the grout was 3000 psi (20.7 MPa) in 8 hours.

Panel Fabrication and Installation

The panels were fabricated at a precast plant in San Antonio, TX. All panels were 8 ft wide. Two panel types were 18 ft (5.5 m) long, and a third panel type was 12 ft (3.7 m) long. All panels were prestressed along the long dimension. Figure 16 shows a view of a panel with dowel bars ready to be placed in the ducts. The panel installation was performed in three stages (see Figure 15) during the Spring of 2016. Stage 1 included installation of the outer two panel lanes on SH 97 (78 panels) for a total width of 30 ft (9 m). During Stage 2 and Stage 3, panels were installed along SH97 and SH 72 at the northern half of the rest of the intersection (68 panels) and at the remaining southern half of the intersection (89 panels), respectively. The panels were lifted from the staging area at the intersection to the point of placement. Figure 17 shows a panel placement on the HMA base. Dowel bars with chairs were placed in the ducts along the interior side of the panels. Once a pair of adjacent panels had been placed and the elevations adjusted, the dowel bars were manually shifted in the ducts until they were centered at the joint. After all dowel bars were positioned in adjoining ducts, the ducts were filled with rapid-setting grout. Finally, after all panels for each stage were placed and grouting of all dowel-bar ducts and open slots was completed, panel undersealing was performed using grout ports extending the full thickness of the panels. Figure 18 shows the completed intersection in use.

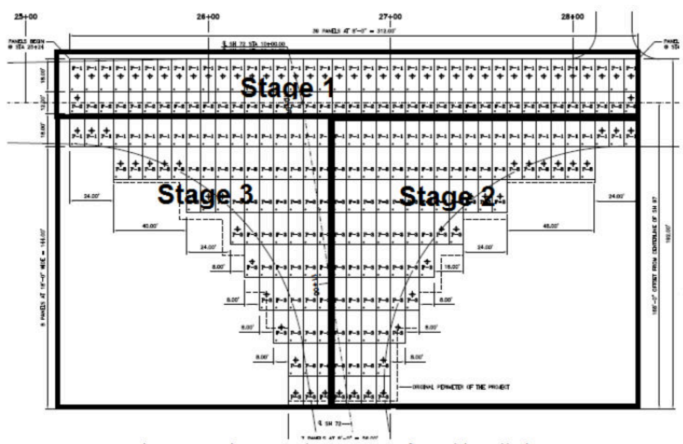


Figure 15. Panel layout and installation stages.

Figure 16. Typical panel with dowel bars ready to be placed in the ducts.

The use of the PCP technology on an actual pavement rehabilitation project was an important step for TxDOT. Although project did not meet the original goal of placing all panels over a weekend (to demonstrate the rapid rehabilitation potential of PCP), TxDOT considers the demonstration project at the SH 97 and SH 72 intersection to be successful.

Figure 17. A panel being placed on the HMA base.

Figure 18. Completed intersection in use (view from SH 97).

SUMMARY

PCP technology is gaining wider acceptance in the U.S. Implementation projects in multiple states have demonstrated that PCP systems are constructible during short nighttime lane closures by contractors with no prior PCP experience. For first projects, some initial startup issues develop, but after a few days (nights), the panel placement proceeds smoothly and productively.

Although experience with PCP systems is limited, less than 17 years, well-designed and well-constructed PCP systems can be installed rapidly and can be expected to provide long-term service.

ACKNOWLEDGEMENTS

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Flexural Fatigue Behavior of High Volume Fly Ash Concrete under Constant Amplitude, Compound, and Variable Amplitude Loading

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ABSTRACT

The constituent materials of a concrete pavement have to be optimized with reference to cost effectiveness and sustainability. This paper presents a study on flexural fatigue behavior of high volume fly ash concrete (HFC) under different types of fatigue loading. The different types of fatigue loading used in the study are: constant amplitude, compound, and variable amplitude loading. Mix proportion of HFC, satisfying the strength requirement of pavement concrete, is developed having a cement replacement level of 60% with low calcium fly ash. A total number of 95 beam specimens are tested under constant amplitude fatigue loading and a total number of 24 beam specimens are tested under compound fatigue loading. The three amplitude loading is used as variable amplitude loading. A total number of 10 beam specimens are tested under variable amplitude loading. The relations between the stress level (S) and the fatigue life (N) are established from the results of constant amplitude and compound loading. The results of compound and variable amplitude loading are also used for validating the Miner's hypothesis for cumulative fatigue damage. Probabilty analysis carried out on the pooled data of Miner's sum values showed the normal distribution. The Miner's hypothesis can be accepted conservatively for the thickness design of rigid pavements.

Key words: High volume fly ash concrete; constant amplitude fatigue loading; compound loading; Variable amplitude fatigue loading; Probability distribution; Miner's hypothesis

INTRODUCTION

A concrete having a minimum cement replacement level of 50% with fly ash (Mehta, 2004) is termed as high volume fly ash concrete (HFC). Jelena et al (2014) reviewed the published literature on the fly ash utilization and the properties of HFC. From the reviewed literature it was stated that the environmental releases of constituents of potential concern from fly ash concrete during use is comparable to or lower than concrete without fly ash or are at or below relevant regulatory and health based bench marks for human and ecological receptors. This will promote the greater use of fly ash in concrete production.

Highway and airport pavements and bridges are subjected to dynamic loads. Fatigue strength data of concrete is required for their safe, effective, and economical design. The most commonly used fatigue life prediction model (Hilsdorf et al 1966; Ballinger, 1972; Tepfers et al 1979, IRC58-2011,) is in the form of a relation between the stress level (S) which is defined as the ratio of the maximum stress applied in cyclic loading to static flexural strength, and the number of load cycles to failure (N), termed as fatigue life. From the compiled data of fatigue life results of different researchers (Lee et al, 2004) at different stress levels it is well established that fatigue life is a stochastic function. Oho (1991) and Shi et al (1993) have shown that fatigue life values at different stress levels follow Weibull distribution. Joshi et al (2004) reported that grouped fatigue data follow a log-normal distribution at the stress levels used in the study except at 0.72.

Tse et al (1986) have reported the dependence of compression fatigue behaviour on source of fly ash and cement replacement ratio for HFC mix. Naik et al (1994) reported that a concrete containing 15% Class C fly ash is stronger in both compression and flexural modes of loading compared to concrete containing 50% fly ash. Ramakrishnan et al (2005) have developed S-N relation for HFC with cement replacement level of 58% using flexural fatigue loading at a frequency of 20Hz.

In the case of compound and variable amplitude loading the majority of the literature is available for conventional plain concrete. Miner (1945) developed the concept that the damage in case of variable amplitude loading can be expressed in terms of number of cycles applied divided by the number of cycles to produce failure at a given stress level. According to the Miner's hypothesis when the summation of these "increments of damage" at different stress levels becomes unity, failure occurs. The experimental studies conducted by Hilsdorf et al (1966) on variable amplitude levels showed that when the lower stress level was applied first, the Miner hypothesis was unsafe, whereas when the higher stress level was applied first, the Miner hypothesis was conservative. A large variation in the cumulative damage factor is reported in the literature (Ballinger, 1972). Siemes (1987) investigated the fatigue behavior of plain concrete under variable amplitude compressive loading. Dispersion in the constant amplitude and variable amplitude results was explained from the dispersion in the static strength. The Miner's rule was found to be accurate enough to predict the life of the test. cylinders. Joshi et al (2006) used two stage constant amplitude loading as compound fatigue loading. The cumulative damage factor varied from 0.16 to 5.25.

RESEARCH SIGNIFICANCE AND SCOPE OF THE WORK

In the current study it is aimed to develop the S-N relation for HFC from the test results of constant amplitude and compound fatigue loading, mainly targeting the pavement application. Also it is aimed to verify the validity of Miner's rule using the results of compound and variable amplitude fatigue testing. HFC mix was developed using 60% cement replacement with low calcium fly ash. A total number of 95 beam specimens are tested under constant amplitude loading, whereas a total number of 24 beam specimens are tested under compound loading. For variable amplitude loading a total number of 10 beam specimens are used.

Table 1 Mix proportions of HFC

Ingredients	Weight of ingredient material in kg/m^3 of concrete
Water	132
Cement	176
Fly ash	264
Fine aggregate	858.2
Coarse aggregate	1059
Superplasticizer	3.52

EXPERIMENTAL INVESTIGATIONS

Mix proportions

The minimum grade of concrete specified in the literature (IRC: SP: 62-2004) for pavement application is M30 which results in a minimum flexural strength of 4N/mm^2. Hence the mix proportions for HFC are developed for M35 grade concrete. The cement replacement level with low calcium fly ash was kept at 60%. The mix proportions are shown in Table 1.

Test specimens

Cube specimens of size 150mm×150mm×150mm are used for determining the compressive strength of HFC mix. Beam specimens of size 75mm×100mm×500mm are used for static flexural testing and also for flexural fatigue testing.

TEST METHODOLOGY AND RESULTS

Mechanical properties

Compressive and flexural strength properties of HFC are determined as per the provisions of IS516-1959. The 28 day compressive strength and flexural strength are 40.8MPa and 5.3MPa. Both static strength values are taken as the mean values of six specimens. Authors (2014) have reported that the durability properties of HFC are satisfactory from the criteria of high performance concrete.

Constant amplitude fatigue testing

Cyclic wave (non reversed type) loading with a frequency of 4Hz was used as fatigue loading. Minimum stress level was maintained at 1% of the amplitude. The beam specimens are covered in polythene bags after 28 days of curing for 90 days so as to minimize the effect of strength gain on fatigue results. Typical stress levels of 0.8, 0.75, 0.7, 0.65, 0.6, 0.54 and 0.5 are used in the investigation. At each stress level a minimum number of ten specimens are tested. Since it was not aimed to determine the endurance limit the fatigue testing was stopped after one lakh cycles of loading if specimen did not fail. At stress level of 0.5 none of the specimens failed even after application of one lakh cycles of loading. Authors (2017) have developed S-N curves at different probability of failures and also shown that the fatigue life values at different stress levels follow log normal distribution. In the current work S-N curve is developed conducting regression analysis on the fatigue life data. The S-N curve obtained is shown in Figure 1. The S-N relation is given by equation 1.

$$S = -0.0338 Ln(N) + 0.9389 \qquad (R^2 = 0.8759) \tag{1}$$

Compond fatigue testing

Multiple stage (tyically two, three and four stage) constant amplitude loading is used as comound loading to simulate the variable vehicular traffic on the pavement. A two stage loading used in the investigation is shown in Figure 2.

Authors (2012) have reported that the cumulative damage factor value obtained from compound fatigue test results varies between 0.82 and 2.1. In the current work an attempt has been made to develop the S-N relation from the results of compound fatigue testing.

The S-N relation from compound fatigue loading data was determined assuming the Miner's hypothesis is valid as a failure criteria in case of fatigue loading with varying amplitudes. Damage factor at first stress level (d_1) is calculated using equation 2.