An *IPRF* Research Report

Innovative Pavement Research Foundation

Airport Concrete Pavement Technology Program

Report IPRF-01-G-002-02-2

Acceptance Criteria of Airfield Concrete Pavement Using Seismic and Maturity Concepts: Appendices

Programs Management Office
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Acceptance Criteria of Airfield Concrete Pavement Using Seismic and Maturity Concepts: Appendices

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Mr. Jeff Rapol	Federal Aviation Administration
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Appendix A

Conventional Methods for Estimating PCC Thickness and Strength

Introduction

Thickness and strength are two important factors that have significant effects on Portland cement concrete (PCC) pavement performance. Accordingly, these parameters are carefully designed or specified, and significant attention is given to their control during the construction process. In general, the different methods currently used to assess PCC pavement thickness and strength can be effective, but these procedures do have certain weaknesses with regard to their basis, accuracy, ease of measurement and ability to represent in situ conditions. Because of these limitations, some new methods and new technologies have been developed with the potential for significant improvements in the characterization of pavement thickness and strength on a construction project. This appendix summarizes the characteristics, advantages and disadvantages of both traditional methods and newer technologies that may be employed for the assessment of PCC pavement thickness and strength.

Pavement Thickness

ASTM C174 Method

Pavement thickness determination is a destructive process involving the extraction of cores (typically 4 in. in diameter) from the constructed pavement. The method of obtaining the drilled cores is covered under ASTM C42, while the actual method of measurement is covered under ASTM C174.

The ASTM C174 method employs a special testing apparatus consisting of a three-point callipering device. The apparatus is designed so that the specimen is held with its axis in a vertical position by three symmetrically placed supports bearing against the lower end. The specimen is placed in the calipers so that the smooth end is placed down. Nine measurements are taken on each specimen, one at the central position and one at eight additional positions spaced at equal intervals along the circumference of the circle. The individual observations are recorded to the nearest 0.05 in., with the average of the nine measurements expressed to the nearest 0.1 in.

Core measurements represent the most accurate means of thickness determination and are used as the basis for calibrating nondestructive techniques. The principal disadvantage of coring is that it is a destructive process that can only be performed after the concrete has set. Furthermore, the thickness of the core is representative of only a small area, and an extremely large number of cores may be required if significant variability exists. Also, the coring operation can be costly and time consuming, and additional time and effort are required to fill the holes after the cores have been extracted.

Thickness Probing

An alternative procedure for the determination of pavement thickness is by probing the fresh PCC pavement and directly measuring the resultant thickness. This procedure is employed by at least one highway agency (Texas DOT, under specification Tex-423-A). The procedure employs a rigid straight steel rod at least 4 in. longer than the thickness of the pavement and a standard tape measure or ruler readable to the nearest 1/16 in. The process requires the steel rod to be inserted full depth into the concrete in a position perpendicular to the pavement surface. The rod is then retracted and the depth of the pavement is measured using the tape measure or ruler. One test point represents the average of three readings taken at points located one-quarter, one-half, and three-quarters across the width of the pavement. No information is provided on how frequently the testing measurements are recorded along the length of a project.

Ground Penetrating Radar

Ground penetrating radar (GPR) is used for locating structural objects and evaluating material properties and layer thicknesses. The technology was originally developed by the military in the 1960s to detect land mines and shallow tunnels (Morey 1998).

GPR has been used as a pavement evaluation tool since the late 1970s. GPR is an attractive tool because of its ability to collect large volumes of continuous data (roughly 200 lane-miles per day by an air-coupled antenna) in a nondestructive manner with limited traffic interruptions. Other applications of GPR on PCC pavements include the location of embedded steel and the detection of voids beneath pavement corners.

ASTM D4748 provides a standard method for determining the thickness of bound pavement layers using GPR. During GPR testing on a pavement, short pulses of radio wave energy are transmitted into the pavement surface by either an air-launched horn or ground-coupled antenna. This energy travels down through the material and echoes are created at boundaries of dissimilar materials; the arrival time and strength of the echoes can be used to determine layer thicknesses (Maser 2000).

Each type of pavement layer material (as well as embedded items such as dowel bars) has a different dielectric constant that affects the amplitude and the wavelength of the reflected signal, with stronger signals being created when there are greater differences in electromagnetic properties (for example, all of the signal is reflected at a metal surface and none is transmitted through the metal surface) (Morey 1998).

The primary components of a GPR system are illustrated in figure A.1 (Morey 1998). The typical transmit/receive unit consists of a transmitter for signal generation, a receiver for signal detection, and timing electronics for synchronizing the transmitter and receiver. The control unit is the operator interface that controls the overall operation of the radar system.

The antenna unit can be a single antenna that transmits and receives radar signals or separate antennas for transmission and reception. These antennas can be either “air-coupled” or “ground coupled,” referring to the location of the antenna relative to the pavement surface. In an air-coupled configuration, the antennas are located about 10 in. above the ground, whereas in a ground-coupled operation, the antenna unit rests on the surface of the pavement (Morey 1998).

Both the air-coupled and the ground-coupled configurations have advantages and disadvantages. The air-coupled configuration can be used at highway speeds (up to about 50 mi/hr), but is less able to distinguish between certain materials. The ground-coupled configuration provides a better signal penetration into the ground, but is limited to much slower test speeds because of its contact with the pavement surface.

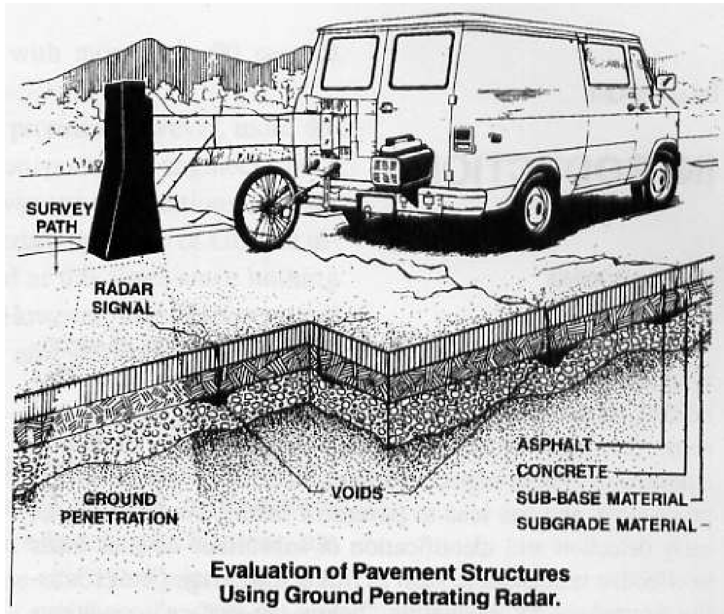


Figure A.1 - Illustrated Example of a Van-Mounted GPR (Morey 1998)

Pavement thickness evaluation using GPR technology is based on the measurement of the time difference between layer reflections and the velocity of propagation within the layers (Morey 1998). The reflections from the interfaces must be sufficiently strong to be monitored and interpreted. Unfortunately, experience has shown that GPR has certain limitations in its ability to assess PCC pavement thickness (Morey 1998; Maser 2000; Wells and Lytton 2001):

1. A PCC layer has very similar dielectric properties as the granular base layer usually located beneath it. Without a large contrast in layer properties, changes in radar waves are slight and difficult to discern. Concrete exhibits a significant electromagnetic attenuation due to its moisture content and dissolved salts. The large electromagnetic attenuation decreases the strength of the radar reflections and makes GPR more difficult to interpret.
2. The presence of reinforcing steel in the concrete pavement also greatly affects the thickness estimates of the GPR, as the steel fully reflects the signal making the interpretation difficult.
3. The range or depth to which GPR is effective is a function of several parameters, such as material conductivity, water content, transmitter pulse width and power output, antenna gain and efficiency, and receiver sensitivity.
4. Experienced operators and regular calibration are required in order to achieve the best results.

Overall, these limitations can translate into some significant measurement error. For example, one study determined a thickness range of $\pm$1.66 in. for concrete layers between 9 and 12 in thick (Willet and Rister 2003). Maser (2000) reports an expected "accuracy" level between 5 and 10 percent for concrete pavements, provided there is an adequate contrast between layer materials.

Impact-Echo Method

The impact-echo method is a nondestructive, seismic-based approach used to analyze the stress waves generated in a solid object after some type of impact load is applied. It is best suited for the determination of pavement thickness, although it has also been used to detect delamination, flaws, and other discontinuities within the PCC pavement. Sansalone and Carino (1991) developed the methodology for testing of concrete structures, and a standard test method (ASTM C1383) is available. Equipment used in impact-echo testing includes an impactor (which can be a hand-held hammer, a small steel bearing, or a mechanically actuated impact device), a receiver (to monitor surface motion and record waveforms), and a data acquisition tool (see figure A.2).

The pavement surface is struck with the impactor during testing, which creates three types of stress waves that propagate in all directions through the medium: P-waves (compression waves), S-waves (shear waves), and R-waves (surface or Rayleigh waves) (Nelson 2003). P-waves are most important to impact-echo testing. P- and S-waves travel together along spherical wavefronts, whereas R-waves propagate along the surface in a circular movement similar to ripples in a pond (see figure A.3).

The P-waves travel down through the pavement and are reflected back from the bottom of the pavement, as shown in figure A.4; the reflection occurs due to the difference in wave velocity and density between the pavement and the base (Infrasense 2003).

Figure A.2 - Impact-Echo Testing Equipment (FHWA 2003)

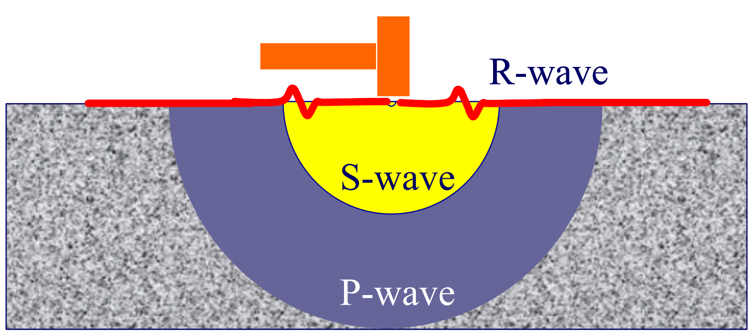


Figure A.3 - Stress Waves Occurring in Concrete Pavement upon Impact (FHWA 2003)

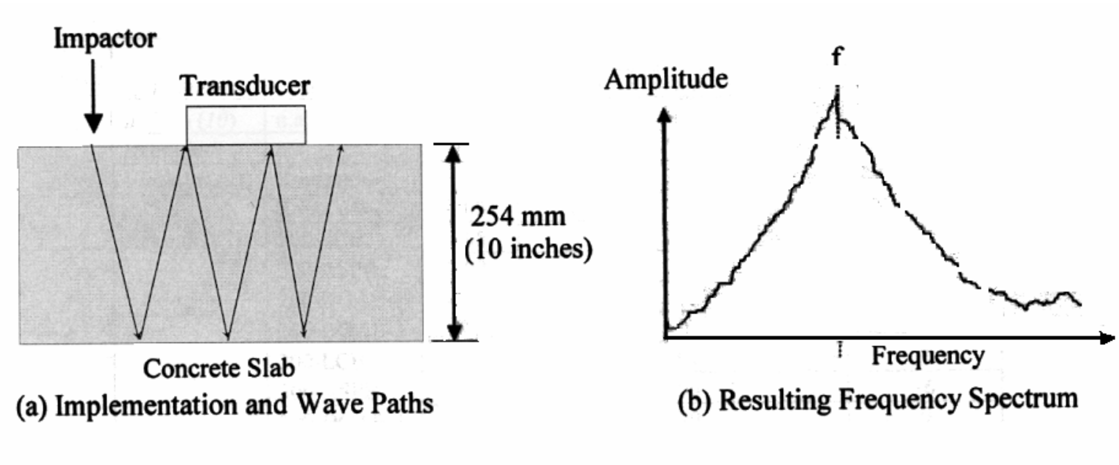


Figure A.4 - Impact-Echo Testing (Infrasense 2003)

The P-wave travels twice the thickness of the pavement before returning to the surface as indicated in figure A.4. Thus, the initial relationship between pavement thickness (h), P-wave velocity (V_p), and travel time (t) is (Sansalone, Lin, and Streett 1997; Infrasense 2003):

$$h = V_p\,(t / 2) \qquad \text{(A.1)}$$

Rather than measuring travel time, a more effective technique is to measure the frequency spectrum of the reflected signal. In the graph of wave amplitude versus frequency in figure A.4b, the frequency peak (f), or "thickness resonance," represents the repetition of reflected arrivals (in arrivals per second). The inverse of f is travel time, so equation A.1 then becomes:

$$h = V_p / 2\,f \qquad \text{(A.2)}$$

The ASTM specification for this method calls for a 0.96 correction factor to this equation to account for the "plate effect" on the P-wave velocity. The P-wave velocity required for this calculation needs to be determined independently (Infrasense 2003). One method for determining V_p involves the use of two transducers arrange in a linear fashion along the surface of a PCC pavement. An impact is made near one of the transducers and the arrival time of the P-waves at both transducers is determined. Knowing the distance between the transducers and the difference in arrival time, the speed of the P-wave can be determined. The problem with this approach, however, is that the determination of P-wave arrival times at the two transducers is often difficult to interpret (Infrasense 2003).

A second method for determining V_p is to use calibration cores (Infrasense 2003). By measuring the travel time of the P-waves propagating in a core of known thickness, V_p for the concrete around the coring location can be calculated using equation A-1. To account for the heterogeneity of concrete, however, several representative cores should be used to obtain an average V_p for the calibration process.

Infrasense (2003) describes an alternative impact-echo technique that involves the use of multiple receivers. The multiple receiver technique (MRT) still relies on P-waves reflected off the bottom of the pavement to estimate its thickness; however, because of its ability to interpret information from the additional receivers, it does not require external calibration or concrete property assumptions. Unlike the standard one-receiver technique, however, experience with this technique is limited.

The impact-echo method is conducted on a point-by-point basis, with each reading taking less than 20 seconds to acquire and process. Accuracies within 3 to 5 percent are reported, provided that clear readings are obtained and there is sufficient differentiation between the PCC pavement and the underlying base course (FHWA 2003). Infrasense (2003) indicated that the impact-echo method consistently underestimated concrete thickness, for tests conducted on slabs at the FAA Technical Center. The findings suggest that the use of the 0.96 correction factor (as recommended by ASTM C1383) may lead to systematic errors in some circumstances.

The limitations to the use of impact-echo method include (Sansalone, Lin, and Streett 1997; FHWA 2003; Nelson 2003):

- Interpretation of the test results can be difficult, and an experienced operator is needed.
- The presence of a lean concrete base (with similar mechanical properties as the PCC pavement) can mask the interlayer and make it difficult to discern the pavement thickness.
- The impactor can significantly affect the results, so it is important to select an appropriate impactor for measuring the thickness of pavement.

<u>Concrete Thickness Gauge</u>

The concrete thickness gauge (CTG) is an automated equipment that operates on the impact-echo technology as governed by ASTM standard C1383. The CTG is a proprietary product developed by Olson Engineering in 2000, and is marketed for quality assurance applications.

The CTG consists of a test head that is connected to the main body by a hardware connection (see figure A.5). An extension pole is available to allow users to stand up during the testing process. The testing procedure and concepts are similar to the impact-echo technology. The test head impacts the pavement surface, generating a stress wave that travels into the pavement and gives rise to the transient resonance (Nelson 2003). The main body receives the signal and analyzes the data to produce the resonant spectrum that represents the concrete thickness.

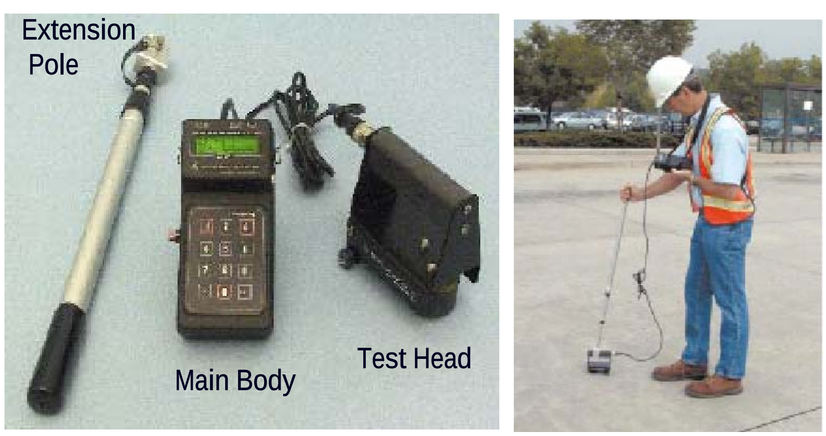


Figure A.5 - CTG Equipment and Testing (FHWA 2003)

The CTG was designed to be simple to operate, with about 10 minutes of training (Nelson 2003). The testing is very rapid, with typically 60 tests being performed in one hour. The device provides a direct readout of the pavement thickness, so there is no interpretation required. Reported accuracy is 2 to 5 percent of the actual thickness, provided that the device has been properly calibrated (Nelson 2003).

PCC Strength

<u>Standard ASTM Methods</u>

ASTM has published standards for the assessment of compressive, flexural, or split tensile strength. Compressive testing is the most common measure of PCC strength, although in pavement applications the flexural strength is of critical importance because of its relation to structural cracking of the pavement under bending. In this section, the standard ASTM procedures for measuring and expressing PCC strength in failure modes are described.

Compressive Strength

The standard method of determining the PCC compressive strength involves sampling, specimen preparation, curing, and compression testing of cylindrical specimens obtained either from molded cylinders (made from fresh concrete sampled at the job site) or cores retrieved from hardened PCC. Specimens should be at least 4 in. in diameter and should have a length-to-diameter ratio of about 2. The standard practice for sampling fresh concrete for molded cylindrical specimens is covered by ASTM C172; the cylinders are prepared in the field, transported, and cured in a temperature and moisture controlled environment in accordance with ASTM C31. The standard practice for retrieving drilled cylindrical core samples is described under ASTM C42.

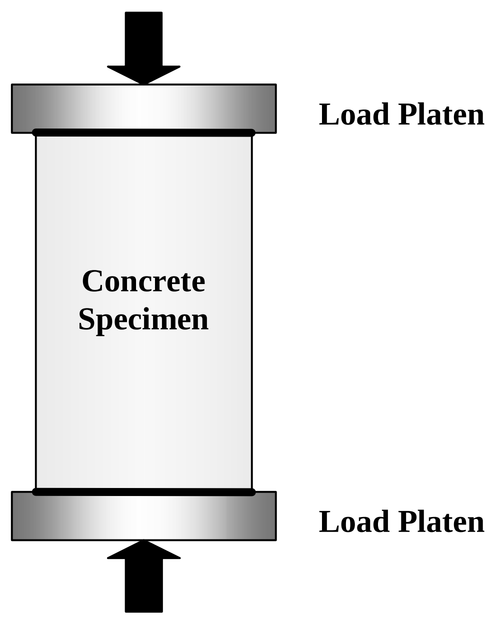


Figure A.6 - Schematic of Compressive Strength Test

The standard method of compression testing is ASTM C39. In this test, the specimen is first capped to provide a uniform top and bottom surface and then placed into a universal testing machine that loads the specimen in uniaxial compression (see figure A.6). The load is applied at the rate of 20 to 50 psi per second until the specimen fails. The maximum load sustained by the specimen is used to calculate its compressive strength.

A concrete cylinder may be tested at different times during the curing process. For PCC pavements, the times are usually determined based upon the anticipated time before opening to traffic and range between 2 hours for fast-track mixes to 28 days for less critical mixes.

This method is a well-established test with a long history of use for concrete strength assessment. The primary disadvantages include the effort required to prepare, transport, and test the specimens and the error associated with differences between curing of the specimen and the in situ pavement. Furthermore, there is concern about the damage caused by the drilling operation to the cored specimen for testing.

Flexural Strength

Flexural strength is a measure of the extreme fiber stress developed under slab bending, and is an important parameter in PCC pavement design. The standard method of determining the flexural strength of concrete involves sampling, specimen preparation, curing, and flexural testing of rectangular beams that are 6 in. wide and 6 in. high and at least 21 in. long. These rectangular beams can be made from the fresh concrete sampled from the job site (obtained in accordance with ASTM C172 and prepared, transported and cured in a temperature and moisture controlled environment according to ASTM C31) or may be sawed from an existing pavement (obtained in accordance with ASTM C42).

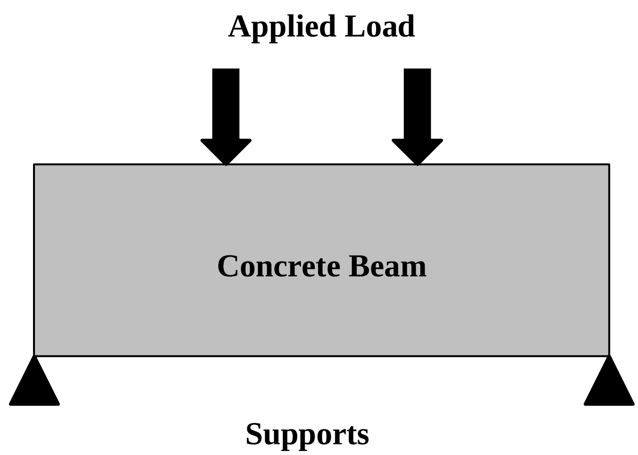


Figure A.7 - Schematic of Third-Point Flexural Testing

Flexural testing can be conducted under either center-point or third-point loading conditions. The third-point loading configuration, described under ASTM C78, is more commonly used in pavement design and provides a more conservative estimate of the flexural strength than the center-point test. In the third-point test, the sample is placed in a special loading device that applies the load at points one-third from each end of the specimen (see figure A.7). This configuration provides a uniform bending moment and uniform maximum tensile stress in the bottom fiber of the middle third of the beam. The load is applied at a rate of 125 to 175 psi per minute until the specimen ruptures. The maximum load sustained by the specimen is used to calculate its flexural strength.

A flexural beam may be tested at different times during the curing process. For PCC pavements, the times are usually determined based upon the anticipated time before opening to traffic and range between 2 hours for fast-track mixes to 28 days for less critical mixes.

The advantages and disadvantages of this method are essentially the same as those for the concrete cylinders. For pavement design purposes, the resulting strengths are believed to be more meaningful than compressive strength values, but the beams are heavier and more difficult to work with. Moreover, recent research conducted by Roesler (1998) indicates that the flexural strength as measured using ASTM C78 is not a unique parameter describing the in situ strength of concrete.

Splitting Tensile Strength

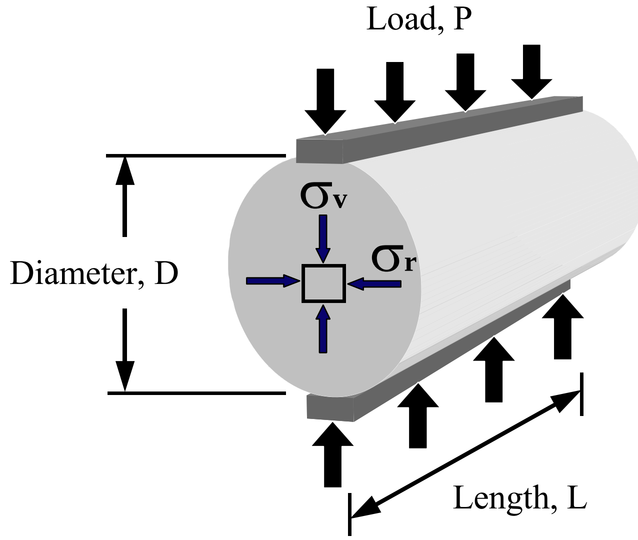


Figure A.8 - Schematic of Splitting Tensile Test

The splitting tensile test, also called the indirect tension test, is used primarily to determine the tensile strength of cores obtained from concrete pavements. The procedure is described in ASTM C496. The test involves applying a vertical load at a constant rate (100 to 200 psi per min) along the length of a cylindrical sample (as shown in figure A.8). The sample will fail in tension along the vertical diameter of the sample and the indirect tensile strength is calculated from the maximum applied load and the dimensions of the specimen.

This test can be performed on the same cores obtained for pavement thickness determination. The advantages of this method include the relative small specimen size, the speed and ease of testing, and, because the specimens are cured "in-place," they are more representative of the in situ pavement. The primary disadvantage of this method is that it is a destructive test that requires patching after sampling.

Maturity Testing

The strength of a given concrete mix, which has been properly placed, consolidated and cured is a function of its age and temperature history (Saul 1951). Longer cure times lead to greater strength, and an increase in temperature during concrete curing can speed up the hydration process and the rate of strength development. The maturity method of testing accounts for this combined effect of time and temperature and provides a basis for estimating the in situ strength gain of concrete by monitoring its temperature over time.

Maturity was developed in the 1950s and ASTM first published its first standard practice for estimating concrete strength using maturity in 1987 (Mohsen 2002). With recent advancements in equipment and technology and more emphasis on high-speed construction, the technology is gaining more widespread use and acceptance, including on many airfield construction projects.

Maturity can be calculated using one of two methods. The first method, the Nurse-Saul maturity relationship, is the most popular means of computing maturity (Crawford 1997). It is the accumulated product of time and temperature:

$$M(t) = \Sigma\,(T_a - T_o)\,\Delta t \qquad \text{(A.3)}$$

where:

$M(t)$ = Maturity at age t.

T_a = Average concrete temperature during time interval.

T_o = Datum temperature.

Δt = Time interval.

Maturity may also be determined using the Arrhenius method, which accounts for nonlinearity in the rate of cement hydration. It involves a slightly more complicated equation, but has gained widespread acceptance in Europe (Crawford 1997). According to Carino (1984), the Arrhenius equation is a better representation of time-temperature function than the Nurse-Saul equation when a wide variation in concrete temperature is expected. Both maturity functions are outlined in ASTM C1074.

Maturity measurement in the field consists primarily of monitoring the internal temperature of the concrete with respect to time. To measure the maturity of concrete in the field, a maturity meter is used to record the concrete temperature as a function of time. Most maturity meters rely upon temperature sensors embedded in the concrete to report mix temperature (see figure A.9), although new sensor technologies (such as the iButton®) are also available (Rasmussen, Cable, and Turner 2003). The maturity meter converts the reported temperatures and time history into a maturity value. The temperature sensors are usually placed at mid-depth of the pavement.

Laboratory testing of the mix must be performed before any field work in order to establish the strength-maturity relationship for a particular mix. Test specimens or slabs, with embedded temperature sensors, are used to develop maturity-strength curves for a given concrete mix. To develop the strength-maturity curve, a sample of the concrete mix planned to be used for the project is prepared. Test specimens are then cast and monitored from this prepared sample. Control of curing to conditions that the pavement will undergo is essential to obtaining accurate maturity results. The maturity of the individual specimens is monitored until they are tested.

The specimens are tested at consistent time intervals and should span a range in strength that includes the opening strength (ACPA 2002). The values obtained from the test specimens are plotted on a strength-maturity graph and a best-fit curve is drawn to represent the strength-maturity relationship for the project.

Figure A.9 - Temperature Sensor in Concrete Pavement Repair Area

An example strength-maturity curve is shown in figure A.10 (ACPA 1994). If the required compressive strength for opening to traffic is 4,000 psi, the corresponding temperature-time factor is approximately 200 degree-days. Thus, when the combination of time and temperature from the data logger or maturity meter indicate a maturity of 200 degree-days, the pavement can be opened to traffic. This correlation curve is valid as long as the mix design (and all mix ingredients) remains constant. If the mix is changed in any way, the constituent that has been changed must be evaluated, and if necessary, a new calibration curve should be developed.

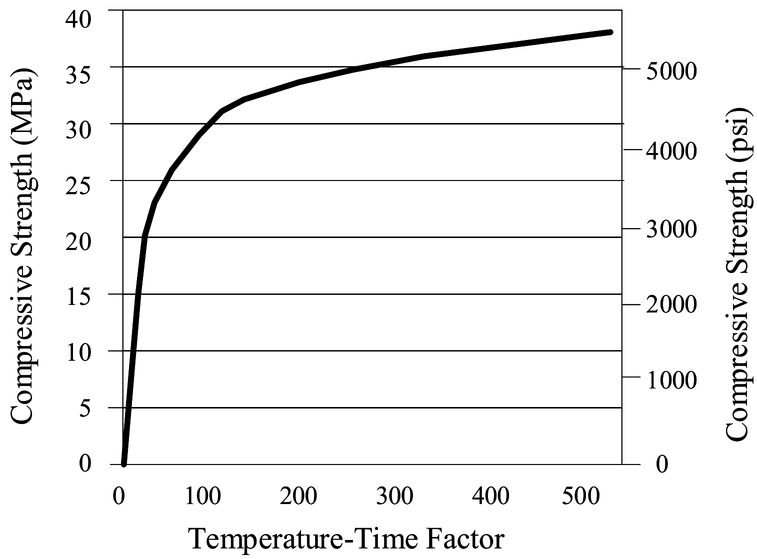


Figure A.10 - Example Strength-Maturity Curve (ACPA 1994)

Many studies have been conducted to test the accuracy and cost effectiveness of the maturity method. A study was conducted in California to determine if maturity concepts could be used to predict the early strength of fast-setting hydraulic cement concrete. The test concluded that the maturity method predicts the early age flexural beam strength with a reasonably high degree of certainty (Mullarky and Wathne 2001). The authors also recommended that maturity testing be implemented for major conventional paving projects where the cost savings associated with reduced testing are significant, and rapid testing information is needed to facilitate early opening to traffic.

A study in Georgia attempted to predict early strength gains of concrete repair slabs using the maturity method. The temperatures were monitored throughout the depth of the test slab for the first eight hours of curing. The study concluded that maturity method was effective (Okamoto and Whiting 1994). The results obtained using these techniques agreed favorably with strengths of cores extracted from repair sections before opening to traffic. The study also pointed out the need to use consistent materials throughout the project for the maturity method to be effective. Changes in mix design or materials during construction can lead to significant predictive errors. Since the use of this method requires a substantial amount of preparation and calibration effort, it is estimated that it will be most cost-effective on large projects (Okamoto and Whiting 1994).

In summary, maturity testing is an effective means of monitoring the early strength gain of concrete pavements. The primary benefit is that it provides a relatively fast, nondestructive means for continuously monitoring concrete strength that can be used to determine when the pavement can be opened to traffic. The primary disadvantages include its inherent assumption that adequate curing is being applied, that the same materials and mix proportions used in the lab are also being used in the field, and its significant up-front effort and costs associated with establishing the maturity curve for a given mix.

Pulse Velocity Test

The standard pulse velocity test is designed to measure the velocity of ultrasonic waves traveling through PCC that are generated as a result of an external pulse. The pulse velocity is proportional to the square root of the elastic modulus and inversely proportional to the square root of the mass density of PCC. Since elastic modulus of PCC is also proportional to the square root of the PCC compressive strength, which may also be correlated to the modulus of rupture, the pulse velocity can be correlated to either the PCC compressive strength or modulus of rupture. The pulse velocity test method has been used successfully to evaluate the quality of concrete for over 50 years and is standardized under ASTM C597.

In the standard pulse velocity method (see figure A.11), an ultrasonic pulse is created at a point on the test object using a high-frequency vibratory transducer, and the time of its travel from that point to another is measured. Knowing the distance between these two points, the velocity of the pulse can be determined. Pulse velocity equipment measures the arrival time of the first (fastest) wave, a compression or P-wave (Crawford 1997). It should be pointed here that there is a potential problem in using ultrasonic pulse velocity (UPV) for concrete testing because of the high frequency and low amplitude of the USW, which make them more susceptible to the non-homogeneity of the concrete, particularly the influence of the embedded coarse aggregate.

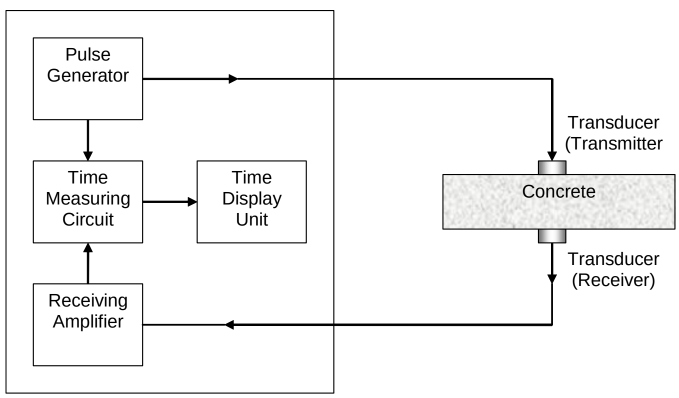


Figure A.11 - Schematic of Pulse Velocity Device (Crawford 1997)

There are three possible ways the transducers can be configured, as shown in figure A.12 (Crawford 1997). The *direct* method has the transmitter on one side of the concrete and the receiver directly opposite on the other. This method gives the most reliable results. The *semi-direct* method has the transmitter and receiver located perpendicular to one another. This method is reliable but the transducers can not be placed so far apart that the signals are attenuated and undetectable. This method has been used to avoid concentrations of steel. The third configuration is called the indirect or surface transmission method and contains both transducers along the same side of the concrete. This method is often used for PCC pavement applications, but it is the least accurate because the amplitude of the received signal is only a small fraction of that associated with the direct transmission method. Also, to determine the pulse velocity, a more complicated procedure involving additional receivers along a certain configuration is necessary. Another disadvantage of this method is that it only samples the waves traveling along the top of the concrete and, therefore, may not be representative of the

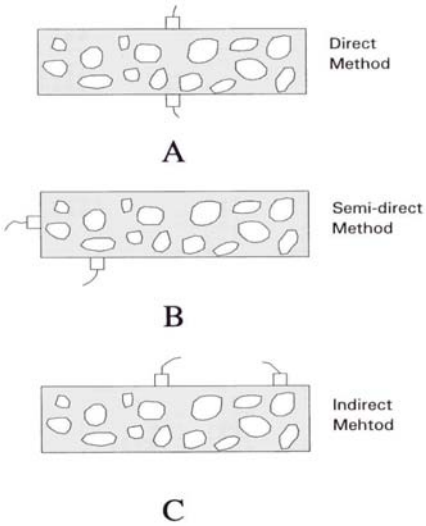


Figure A.12 - Methods of Pulse Velocity Measurement (Crawford 1997)

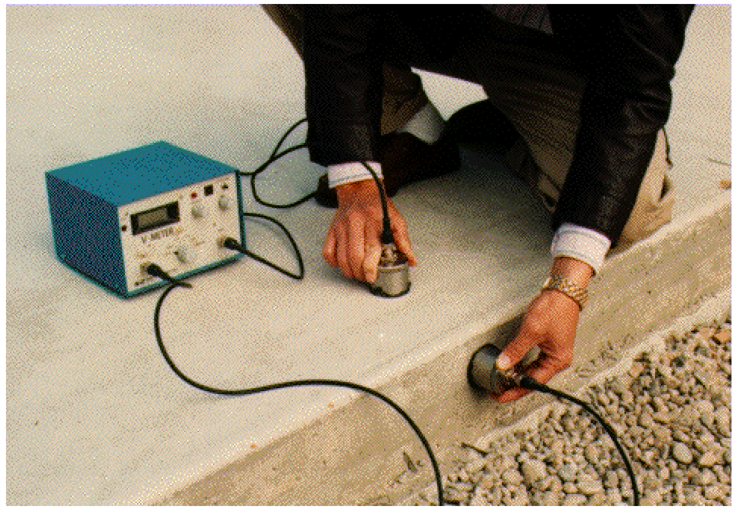


Figure A.13 - Pulse Velocity Equipment in Operation

properties of the entire slab. An example of equipment used to measure pulse velocity is shown in Figure A.13. In this case, the configuration of the transducers is in the semi-direct mode.

According to Naik and Malhotra (1991), the P-wave velocity (V_p) for an infinite, homogeneous, isotropic, elastic medium is:

$$V_p = (KE/D)^{1/2} \tag{A.4}$$

where:

$$K = (1-\nu)g/(1+\nu)(1-2\nu) \tag{A.5}$$

and:

E = Modulus of elasticity.
D = Unit weight.
g = Acceleration due to gravity.
ν = Poisson's ratio.

With these basic formulas, it is possible to determine the modulus of elasticity of concrete, given the measured P-wave wave velocity, the unit weight, and the Poisson's ratio of the material.

As with the maturity method, the pulse velocity method requires laboratory testing of the job mix to establish a correlation between the pulse velocity and the strength of the concrete. A typical plot of pulse velocity versus compressive strength is shown in figure A.14. The correlations are specific to a given mix design, and a new correlation between pulse velocity and strength must be developed for any mix design changes (materials or proportioning).

A study in Georgia attempted to predict early strength gains using the pulse velocity method. The study concluded that pulse velocity techniques can be used to monitor early strength gain during the curing period in pavement repair slabs. The results obtained using these techniques

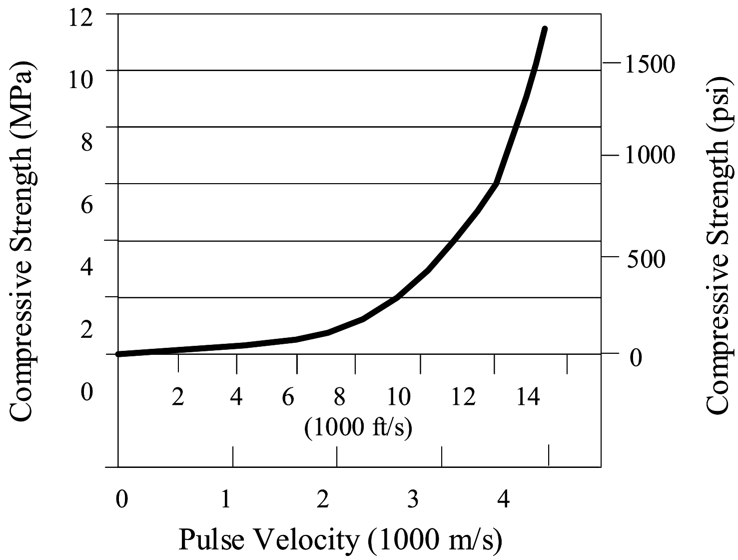


Figure A.14 - Example Pulse Velocity Curve (ACPA 1994)

agreed favorably with strengths of cores extracted from repair sections before opening to traffic (Okamoto and Whiting 1994). The study also pointed out the need to use consistent materials throughout the project for the method to be effective. Changes in mix design or materials during construction can lead to significant predictive errors. Since the use of this method requires a substantial amount of preparation and calibration effort, it is estimated that it will be most cost-effective on large projects (Okamoto and Whiting 1994).

The primary application for the standard method of pulse velocity testing is in evaluating the quality of concrete used in various structures. With its capacity to measure the speed of ultrasonic waves it can be used to detect concrete deterioration due to an aggressive chemical environment, cracking, or changes due to freezing and thawing. The measured wave velocities can also be used to determine the dynamic modulus of elasticity, Poisson’s ratio, PCC thickness, and to estimate the strength of concrete test specimens as well as in-place concrete.

The accuracy of pulse velocity testing depends on the operator’s ability to precisely measure the distance between the transducers, and the equipment’s ability to accurately to measure the transit time (FHWA 2003). If an accuracy of $\pm$2% desired in pulse velocity calculations, then the path length and transit time measurements must be within $\pm$1% accuracy. The accuracy of the readings is also dependent upon the testing configuration, the presence of steel and complex reflections from layer boundaries.

The primary advantages of the pulse velocity approach are that it is an easy, simple, and rapid test. The equipment is portable, and applicable to field and lab specimens regardless of their shape. Primary disadvantages of the equipment include the many variables that can affect pulse velocity measurements (moisture; steel; aggregate type, size, grading, and content; mix non-homogeneity) and the absence of a unique correlation between pulse velocity and concrete properties (mix-specific correlations are required). Moreover, the use of an appropriate transducer configuration (the direct and semi-direct methods are reliable, but the indirect method is prone to error) and ensuring good contact between the transducers and the surface of the concrete are needed in order to provide reliable measurements.

Free-Free Resonant Column (Impact-Resonance) Test

The free-free resonant column test device is particularly suitable for measuring the seismic modulus of concrete in the laboratory. As explained in ASTM C-215, when a cylindrical specimen is subjected to an impulse load at one end, seismic energy over a large range of frequencies will propagate within the specimen (see figure A.15).

Depending on the dimensions and the stiffness of the specimen, energy associated with one or more frequencies is trapped and resonate as they propagate within the specimen. The goal with this test is to determine these resonant frequencies. Since the dimensions of the specimen are known, if one can determine the resonant frequencies, one can readily calculate the modulus of the specimen using principles of wave propagation in a solid rod. Results from a standard cylinder of concrete are shown in figure A.16. Resonant frequencies appear as peaks in a so-called amplitude spectrum. Two peaks are evident, one corresponding to the longitudinal propagation of waves in the specimen, and the other corresponding to the shear mode of vibration. Distinguishing the two peaks is simple, since for typical concrete specimens, the longitudinal resonance occurs at a higher frequency than the shear resonance. Once the longitudinal resonant frequency, f_L, and the length of the specimen, L, are known, laboratory Young's modulus, E_{lab}, can be found from the following relation

$$E_{lab} = \rho\,(2\,f_L\,L)^2. \tag{A.6}$$

where ρ is mass density. Poisson's ratio, ν, is determined from

$$\nu = (0.5\,\alpha - 1) / (\alpha - 1), \qquad \alpha = (f_L / f_S)^2\,C_{L/D} \tag{A.7}$$

with $C_{L/D}$ being a correction factor which is not equal to one when the length-to-diameter ratio of the cylinder differs from 2.

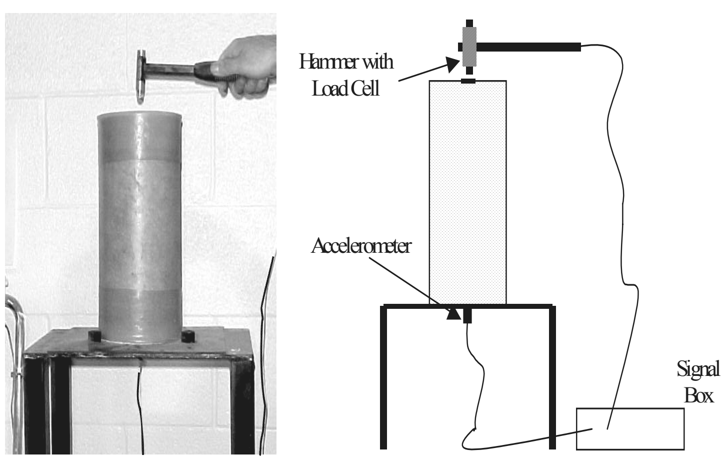


Figure A.15 - Schematic of Impact Resonance Device

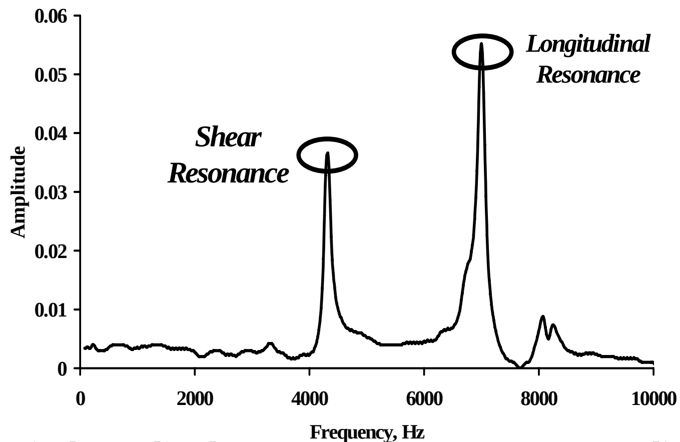


Figure A.16 - Typical Amplitude Spectrum from a Concrete Cylindrical Specimen

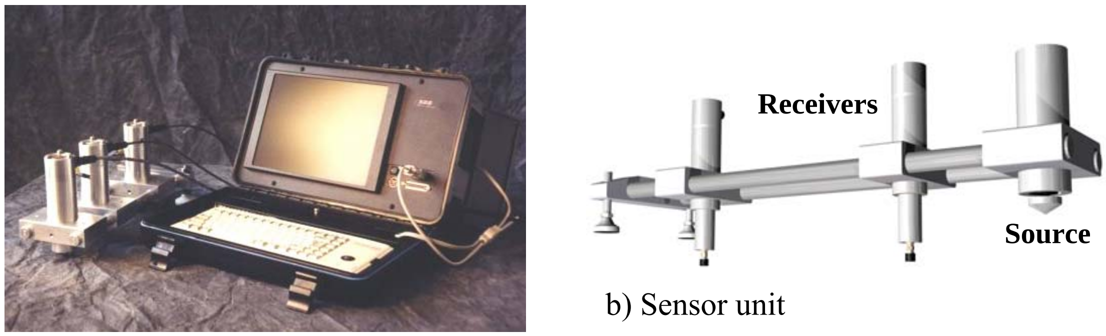
This test, although similar to the ultrasonic pulse velocity (UPV), is more robust and less impacted by the constituents of the concrete. Yuan et al. (2003) have shown that the main parameter that impact the relationship between the modulus obtained with this parameter and strength on the same cylinders is the source of coarse aggregates. Other than this advantage, the other advantages and disadvantages of the UPV method is also applicable to this one. One disadvantage of the impact-resonance tests is that they can only be carried out on lab-prepared specimens or cores and beams extracted from a pavement, and cannot be used as an in situ test.

Seismic Method and Seismic Pavement Analyzer

Developed under the Strategic Highway Research Program, the Seismic Pavement Analyzer (SPA) is nondestructive testing device designed to automate and replace several time-consuming and complex methods of evaluating existing pavements (Nelson 2003). When used on concrete pavements, the SPA can provide information about the quality and thickness of the concrete, the existence of voids or delamination within the concrete, and the presence of voids (or the loss of support) underneath the pavement (Nazarian, Baker, and Crain 1993; McDaniel et al. 2000).

The PSPA (Portable Seismic Pavement Analyzer) is a more suitable version of the SPA for quality acceptance of concrete (see figure A.17). It has been used to evaluate concrete bridge decks and in rigid pavement applications as well (McDaniel et al. 2000; Yuan, Nazarian, and Medichetti 2003). The PSPA can be used within hours of construction for quality control of newly constructed pavements. It provides layer-by-layer estimates of pavement properties, like the seismic moduli, which can be related to other concrete properties such as compressive strength and modulus of rupture. Testing with the PSPA is very rapid, with the collection and preliminary reduction of data at one point taking less than 15 seconds.

The PSPA incorporates the impact echo method with a simplified version of the Spectral-Analysis-of-Surface-Waves (SASW) method, called the ultrasonic surface wave (USW) method, into one device. In that manner, the thickness and the velocity of propagation of waves can be potentially measured simultaneously. As shown in figure A.17, the PSPA consists of a source and two receivers. The source is a computer-controlled impactor that it is capable of generating stress waves at both the sonic and ultrasonic ends of the frequency spectrum. The two receivers are used to monitor the sonic waves generated by the impactor.



b) Sensor unit

a) Complete device

Figure A.17 - Portable Seismic Pavement Analyzer (Yuan et al., 2003)

The ultrasonic-surface-wave (USW) method is an offshoot of the SASW method (Nazarian, Baker, and Crain 1993). The major distinction between these two methods is that in the ultrasonic-surface-wave method the modulus of the top pavement layer can be directly determined without an inversion algorithm. As sketched in figure A.18, at wavelengths less than or equal to the thickness of the uppermost layer, the velocity of propagation is independent of wavelength. Therefore, if one simply generates high-frequency (short-wavelength) waves, and if one assumes that the properties of the uppermost layer are uniform, the modulus of the top layer, E_{field}, can be determined from surface wave velocity of the layer, V_{ph} using

$$E_{field} = 2\,\rho\,[(1.13 - 0.16\nu)\,V_{ph}]^2\,(1 + \nu). \tag{A.8}$$

As a first glance, the USW method described above sounds very similar to the indirect method of ultrasonic pulse velocity (UPV) measurements described in figure A.12c. However, a major distinction exists between the two methods. In the UPV method, the compression wave velocity of the concrete is measured; whereas in the USW method the velocity of propagation of the surface waves is measured. The compression waves measured with the UPV method propagate along a spherical front providing information about the near-surface properties of the material. In the contrary, the surface waves measured with the USW method propagate along a cylindrical front measuring the properties of the material throughout the thickness.

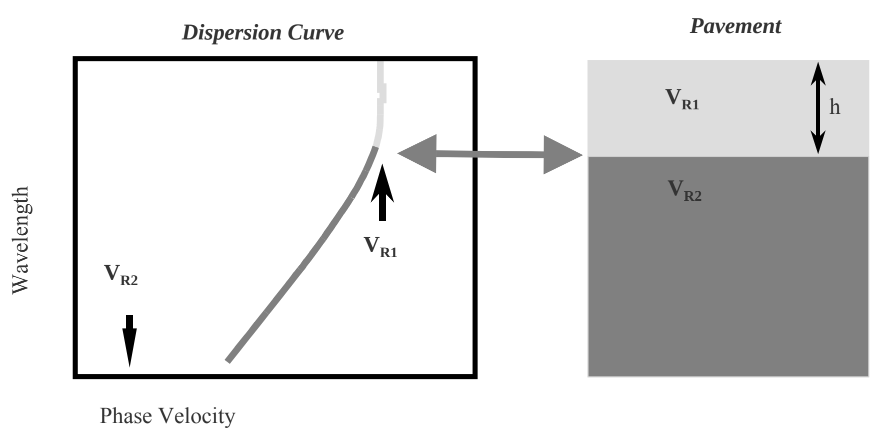


Figure A.18 - Schematic of USW Method

As with the UPV and the maturity method, the measurements with the USW method need to be calibrated for a given mixture. The method of choice is the impact resonance method described above.

Integrated Seismic/Maturity Method

A study conducted for the Texas Department of Transportation (Yuan, Nazarian and Medichetti, 2003) has shown that the combination of maturity and seismic methods complements one another quite nicely. The calibration process for relating strength and maturity can be readily adapted for laboratory seismic testing. In fact, the same specimens can be used for both tests. A proposed protocol that combines the two methodologies is illustrated using an example.

For compressive strength, a total of 15 standard 6 in. (diameter) by 12 in. (length) specimens are prepared. For flexural strength, a similar number of specimens but in the shape of standard beams is poured. During specimen preparation, thermocouples are inserted into 2 beams or 2 cylinders. The specimens are then cured in a water tank.

The protocol consists of four phases: maturity measurement, seismic modulus tests, strength tests and development of the correlations. Each of them is discussed below.

I. Maturity Tests: As usual, the specimens equipped with thermocouples are either connected to a maturity meter or a temperature data-logger. Either device records the variation in temperature with time automatically. The temperature is continuously measured for 28 days. The time and temperature history is converted to the time-temperature factor using Equation A.3.

II. Seismic Tests: Shortly before a specimen is subjected to strength test, the free-free resonant column test will be carried out on it. Since the test is nondestructive, this activity should not impact the results from the strength tests. In this case, the modulus and optionally the Poisson's ratio of the specimen are determined for correlation to strength and maturity.

III. Strength Tests: Standard compression or three point bending tests are performed on 3 cylinders or beams at ages of 1, 3, 7, 14 and 28 days. The average compressive strength or the average flexural strength from the tests is obtained.

IV. Development of Correlations: A plot between the average compressive or flexural strengths and average maturity values at corresponding times is made and a best-fit curve is drawn through the data points. The curve is then used for estimating the strength of concrete based on maturity as it has been traditionally done. Similarly, a plot between the average compressive or flexural strengths and average seismic moduli is developed. A best-fit curve is also drawn through the data points. Based on Equations A.4 and A.8, this relationship can be readily used for predicting the strength of the concrete on the pavement or other structures.

Typical variations in compressive strength from standard cylinders with maturity parameter and flexural strength with maturity parameter from standard beams are shown in figure A.19. The seismic moduli measured at different times are related to the compressive and flexural strengths in figure A.20. The predictive power of he combined methodology in that study, especially at early ages, was better than the maturity alone.

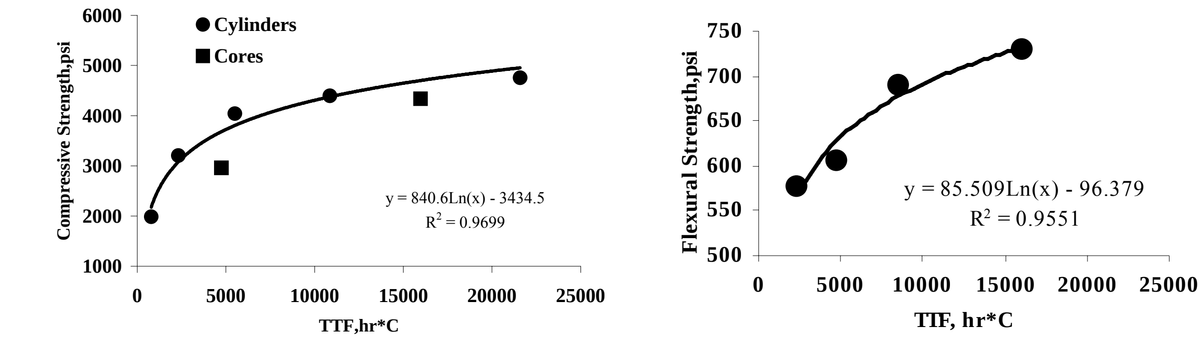


Figure A.19 - Variations in Strength with Maturity Parameter.

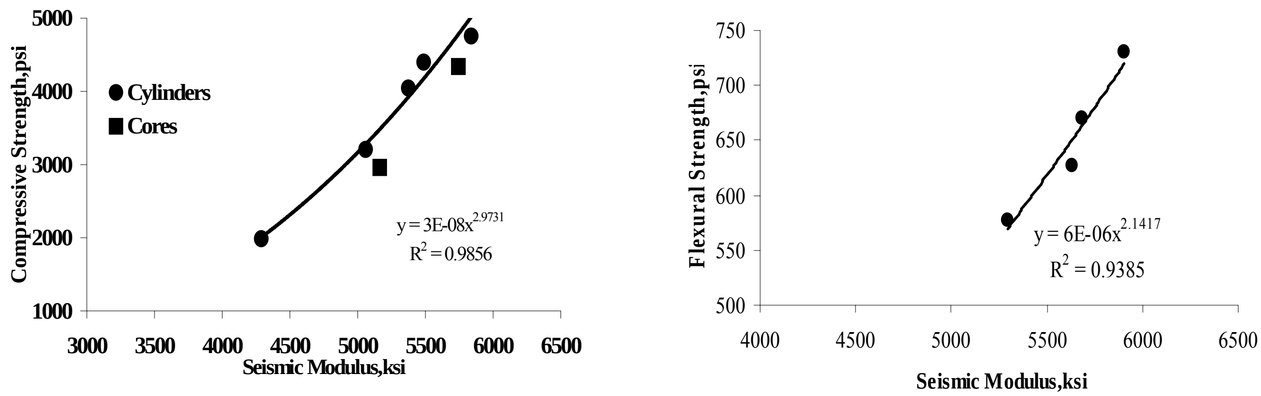


Figure A.20 - Variations in Strength with Seismic Modulus.

Summary

This appendix summarizes some of the more commonly used methods for measuring two critical concrete pavement design elements: pavement thickness and strength. Historically, conventional methods have been destructive tests requiring preparation or retrieval of concrete samples, but more recent methods are nondestructive and are geared to provide more rapid feedback. A summary of the advantages and disadvantages of the different test methods is summarized in table A-1.

Table A.1 - Summary of Tests Used to Determine Pavement Thickness

Test Method	Description	Advantages	Disadvantages	Accuracy
Core Thickness (ASTM C174)	Standard method of pavement thickness determination. Nine measurements are taken on each specimen using a special callipering device.	• Most accurate method of thickness determination. • Can be used to calibrate other test methods.	• Destructive test performed only after the PCC has set. • Time and effort required to patch holes. • Thickness is representative of only a small area.	
Thickness Rodding (e.g., Tex-423-A)	Rigid straight steel rod is inserted full depth into the fresh concrete at specified locations.	• Quick and rapid test. • Greater number of test points possible.	• Less accurate on granular bases or soft soils.	Not established.
Ground Penetrating Radar (ASTM D4748)	Vehicle-mounted equipment is used to transmit and measure the speed of electromagnetic waves which, in turn, are used to determine surface layer thickness.	• Fast, nondestructive, and safe method for estimating surface thickness. • Can be performed at highway speeds. • Provides (almost) continuous coverage.	• Requires experienced operator to interpret results. • Requires project-by-project calibration. • Often ineffective for concrete pavements because of greater electromagnetic attenuation and similar dielectric properties with base course. • Presence of reinforcement makes interpretation difficult.	5 to10% (if adequate contrast exists between layers)
Impact-Echo Method (ASTM C1383)	A nondestructive, seismic-based approach used to measure and analyze the speed of stress waves generated in a solid object after the application of an impact load, which in turn can be related to PCC thickness.	• Equipment is easy to operate and provides rapid results. • Greater number of test points possible. • Can be performed soon after initial concrete set.	• Requires experienced operator to interpret results. • Requires project-by-project calibration. • The presence of a lean concrete base can make it difficult to discern pavement layers. • Impactor and impact contact time are important in determining thickness.	3 to5% (if adequate contrast exists between layers)

Table A.2 - Summary of Tests Used to Determine Concrete Strength

Test Method	Description	Advantages	Disadvantages	Accuracy
Compressive Strength (ASTM C39)	Cylindrical concrete specimen is subjected to axial compressive forces and loaded to failure.	• Standard test method with long history of use in acceptance testing. • Relatively easy test to conduct.	• Test is not representative of the typical stress conditions which cause PCC pavements to deteriorate. • Some effort is required to prepare specimens in the field and transport them to the lab for testing. • Differences between lab and field curing conditions.	--
Flexural Strength (ASTM C78)	Rectangular beam subjected to bending under third-point loading until failure.	• Standard test method with long history of use in PCC pavement design and evaluation. • Test is representative of the typical stress conditions which cause PCC pavements to deteriorate.	• Beam specimens are relatively heavy and bulky. • Significant effort is required to prepare specimens in the field and transport them to the lab for testing. • Differences between lab and field curing conditions.	--
Maturity Method (ASTM C1074)	Accounts for the combined effects of time and temperature on strength gain. Method involves pre-construction testing of the concrete to establish the maturity relationship.	• Fast and simple nondestructive test method. • Accounts for in situ curing conditions.	• Determination of maturity relationship requires significant up-front effort. • Strength-maturity relationship is mix specific.	As yet to be established.
Ultrasonic Pulse Velocity Method (ASTM C597)	A nondestructive, sonic-based approach used to measure and analyze the speed of ultrasonic waves generated in concrete, which can be used to estimate the dynamic modulus of elasticity and strength.	• Fast and simple nondestructive test method. • Accounts for in situ curing conditions.	• Many variables can affect velocity measurements (moisture, steel, aggregate type/size, non-homogeneity of concrete). • Strength-pulse velocity relationship is mix specific. • Appropriate sensor configurations required (direct and semi-direct preferred). • Sensors must have good acoustical contact.	~5%
Seismic Method	Utilizes seismic surface wave velocity to estimate dynamic modulus of elasticity (which can be related to strength).	• Fast and simple nondestructive test method. • Accounts for in situ curing conditions.	• Strength-pulse velocity relationship is mix specific. • Sensors must have good acoustical contact.	~5%
Integrated Seismic-Maturity Method	Incorporates both the concrete maturity and seismic analysis technologies to accurately determine concrete strength and pavement thickness.	• Fast and simple nondestructive test method. • Accounts for in situ curing conditions.	• Strength-pulse velocity relationship is mix specific. • Sensors must have good acoustical contact.	--

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Appendix B

Procedures for Estimating Concrete Strength with Maturity and Seismic Methods Used in This Study

This appendix provides procedures for estimating concrete strength by means of the maturity and seismic methods. The maturity method is based on relating strength gain to temperature and time. The seismic method is in turn based on relating the strength gain to seismic wave velocity and time.

The maturity method consists of three steps:

1. Develop strength-maturity relationship
2. Estimate in-place strength
3. Verify strength-maturity relationship.

The seismic method consists of three steps as well:

1. Develop strength-seismic modulus relationship
2. Estimate in-place strength
3. Verify strength-seismic modulus relationship.

The Nurse-Saul temperature-time factor (TTF) maturity index shall be used. The datum temperature should preferably be determined using the procedure outlined in Annex of ASTM C-1074 using mortar cubes. Alternatively, the approximate values recommended in ASTM C-1074 should be used.

Apparatus

Maturity Testing

- If the maturity meter has input capability for datum temperature, verify that the proper value of the datum temperature has been selected prior to each use.
- Commercial battery-powered maturity meters that automatically compute and display the maturity index in terms of a temperature-time factor, or both a temperature-time factor and an equivalent age, are acceptable.
- The same brand and type of maturity meters shall be used in the field as those used to develop and verify the strength-maturity relationship.
- A minimum of one maturity meter shall be provided for each thermocouple location. A multi-channel meter when several thermocouples are in close proximity can be used.
- Meters shall be protected from excessive moisture, and the LCD display shall be protected from direct sunlight.
- Thermocouple wire grade shall be greater than or equal to 20 awg.

Seismic Testing

- An automated free-free resonant column test device that complies with ASTM C215 shall be used.

Calibration

- Calibration of the maturity device shall be verified prior to use by placing a thermocouple in a controlled-temperature water bath and recording whether the indicated result agrees with the known temperature water bath and recording whether the indicated result agrees with the known temperature of the water bath. At least 3 different temperatures, for example, 5 °C, 25 °C and 40 °C are recommended. The temperature-recording device shall be accurate to within +/- 1 °C.

- For seismic tests, no calibration process is needed. However, to ensure that the device is functioning properly, a calibration specimen provided with the device should be tested prior to the use on a project. If the measured modulus of the calibration specimen differs by more than 2% from those reported, the manufacturer shall be contacted.

Procedure to Develop Strength-Maturity/Seismic Relationships

Step	Action
1	For every concrete design that will be evaluated by the maturity/seismic method, prepare a minimum of 15 cylinders and/or beams in accordance with ASTM C-31. Additional specimens should be cast to avoid having to repeat the procedure. The mixture proportions and constituents of the concrete shall be the same as those of the concrete whose strength will be estimated using this practice.
2	Fresh concrete testing for each batch shall include concrete placement temperature, slump, and air content in accordance with ASTM C-31.
3	Embed thermocouples in a least two specimens. Thermocouples shall be placed 50-100 mm (2-4 inches) from any surface. Connect the thermocouple to maturity meters. Do not disconnect meters. Data collection must be uninterrupted.
4	Moist cure the specimens in a water bath or in a moist room in accordance with ASTM C-31
5	Perform compression and/or flexural tests at nominal ages of 1, 3, 7, 14 and 28 days in accordance with ASTM C-39 and C-78, as appropriate. Test two specimens at each age and compute the average strength. The specimens with thermocouples are to be tested last. Prior to conducting compression or flexural tests on each specimen, perform free-free resonant column test. If a specimen is obviously defective (for example, out of round, not square, damaged due to handling), the specimen shall be discarded. If the difference in strength between two specimens is greater than 10%, test a third specimen.
6	At each test age, record the individual and average values of maturity, seismic modulus and strength for each batch on a permanent data sheet.
7	Plot the average strengths as a function of the average maturity values, with data points shown. Using a computer spreadsheet program such as Microsoft Excel, calculate a logarithmic best-fit curve through the data. Record the equation of the curve as well as the R^2 value. The resulting curve is the strength-maturity relationship to be used for estimating the strength of the concrete mixture placed in the field. Plot the average strengths as a function of the average seismic values, with data points shown. Using a computer spreadsheet program such as Microsoft Excel, calculate a logarithmic best-fit curve through the data. Record the equation of the curve as well as the R^2 value. The resulting curve is the strength-seismic relationship to be used for estimating the strength of the concrete mixture placed in the field. Plot also the average seismic modulus as a function of the average maturity values, with data points shown. Using a computer spreadsheet program such as Microsoft Excel, calculate a logarithmic best-fit curve through the data. Record the equation of the curve as well as the R^2 value. The resulting curve is the seismic modulus-maturity relationship to be used for estimating the modulus of the concrete mixture placed in the field.

Procedure to Estimate In-Place Strength

Step	Action
1	Prior to concrete placement, install one thermocouple set in the pavement. For each thermocouple set, install a thermocouple about 75 mm (3 inches) from the bottom and another about 75 mm from the top of the pavement.
2	As soon as practical after concrete placement, connect and activate the maturity meter(s). Do not disconnect meters until the required maturity values are achieved. Data collection must be uninterrupted.
3	At age of 1 day, record maturity data on a permanent data sheet. Also perform a PSPA test. Perform 12 PSPA tests at predetermined locations on the pavement.
4	Repeat Step 3 at nominal ages of 3, 7, 14, and 28 days, if desired

Procedure to Verify Strength-Maturity/Seismic Relationships

Step	Action
1	Core or cut a minimum of 2 cylinders and/or beams concurrent with pouring the pavement in accordance with ASTM C-42
2	Perform compression or flexural strength tests, on two beams and or cylinders in accordance with ASTM C-39 and/or ASTM C-78, and compute the average strength of the specimens. Prior to conducting compression or flexural tests on each specimen, perform free-free resonant column test.
3	Record the individual and average values of maturity, individual and average strengths, and seismic modulus established from the specimen breaks on a permanent data sheet. Also record the predicted strength based on the strength-maturity/seismic relationships established for that particular concrete design, and the percent difference between average and predicted values. Compare the average strength determined from the specimen breaks to the strength predicted by the strength-maturity/seismic relationships. The average strength of the specimens shall be within the verification tolerance specified for the item of work.

Appendix C

Procedures for Estimating Concrete Pavement Thickness with Impact-Echo Method Used in This Study

This appendix provides procedures for estimating concrete pavement thickness with the impact-echo method.

Apparatus

An impact-echo device that conforms to ASTM C-1383.

Calibration

- Calibration of the IE device should initially be carried out as per manufacture's instructions and as per ASTM C-1383.

Procedure to Measure Thickness

Step	Action
1	Test a point with known thickness on the pavement for calibration purposes
2	Place the IE device on the twelve points marked on the pavement. Record the time records for analysis. Report thickness at each point. Also report the average and coefficient of variation for the pavement

Procedure to Verify Thickness

Step	Action
1	Core the six points marked on the pavement as per ASTM C-42
2	Measure the thickness of each core using ASTM C-174. Record the individual and average thickness of each core on a permanent data sheet
3	Compare the average thickness determined from the cores to the thickness predicted by the IE method.

Appendix D

Development of PWL-Based Pay Schedule for New Acceptance Criteria

This appendix discusses the development of percent within limit (PWL) pay schedules based on concrete seismic modulus and pavement thickness measured with the Portable Seismic Pavement Analyzer (PSPA). This was accomplished by transforming existing PWL-pay schedules (FAA specifications P501) to equivalent PWL-pay schedules. In addition to the PWL pay schedule transformation methodology and the resulted pay schedules, this appendix also provides overviews of statistical construction specifications as per FAA's P501.

Overview of the Statistical Construction Specification

Statistically based specifications (also called statistical specifications or statistically oriented specifications) are specifications based on random sampling, and in which properties of the desired product or construction are described by appropriate statistical parameters (TRB, 2002). In the P501 Specifications, the statistical parameter is the PWL. Statistical specifications recognize the inherent variability in construction processes and products when determining whether constructed products should be accepted, rejected, or accepted at a reduced payment.

Typically, existing statistical construction specifications consist of the following components (Burati et al., 2004):

- Quality control (QC) — also called process control. Those actions and considerations necessary to assess and adjust production and construction processes so as to control the level of quality being produced in the end product.
- Acceptance — sampling and testing, or inspection, to determine the degree of compliance with contract requirements.
- Quality assurance (QA) — all those planned and systematic actions necessary to provide confidence that a product or facility will perform satisfactorily in service.

The PWL (or its complement percent defective, PD) has been used in recent years to determine the assurance levels that specifications are satisfied because it simultaneously measures both the average level and the variability in a statistically efficient way (Demos et al., 1995). Other quality measures that have been used by some agencies include the average absolute deviation (AAD) and the moving average (Buratti et al., 2004).

Two possible distributions of a quality characteristic that show the importance of considering both the mean and standard deviation in assessing quality are shown in figures D.1 and D.2,. In figure D.1, Lots A and B have the same standard deviations, but Lot A has a higher mean (further above the lower limit). In this case, Lot A would have a higher PWL and thus is expected to perform superior to Lot B. In figure D.2, Lots A and B have the same means, but Lot A has a greater standard deviation. In this case, Lot A would have a lower PWL and thus is expected to perform inferior to Lot B. (note: A and B are not marked in the two figures)

PWL is a statistical procedure for estimating the percentage of the pavement lot that conforms to the specifications based on the sample's mean, standard deviation, and size. Quality Index (Q) is a necessary statistic to compute PWL and is defined as follows:

$$Q_l = \frac{\overline{X} - L}{s} \quad \text{(D.1)}$$

$$Q_u = \frac{U - \overline{X}}{s} \quad \text{(D.2)}$$

where Q = quality index, L and U = lower and upper specifications limits outside of which the quality characteristics is defined as defective, $\overline{X}$ = sample arithmetic mean, s = sample standard deviation. The sample PWL (an estimate of the lot PWL) is obtained from standard statistical tables.

This methodology is referred to as acceptance sampling by variables to control the non-conforming fraction, where the standard deviation is unknown. The Recommended Practice for Acceptance Sampling Plans for Highway Construction (AASHTO R9-90) explains its development and applications (AASHTO, 1990). If the quality characteristic under consideration has a lower limit only, PWL is determined based on Q_l only.

Sampling Plans

A well-defined statistical sampling plan is essential for unbiased specifications. The following steps represent a procedure for establishing a statistically valid sampling plan.

Step 1 - Define a Lot: This is the amount of pavement that maybe accepted with pay adjustment or rejected based on the "as-constructed" quality characteristic. Two major factors should be considered when establishing the lot size:

- Homogeneity: A lot represents the amount of material or pavement produced by essentially the same process, so that the distributions of quality characteristics are more likely to be normally distributed. A reasonable way to define a lot is a one-day production. A new lot should be established if there is any reason to believe that a special cause impacts the process (such as change of weather or change of material) and resulted in a significant shift in the mean or standard deviation of any of the key quality characteristics.
- Economic consequences: This factor concerns the economic consequences of rejecting or erroneously accepting a large quantity of pavement of poor quality. If the lot is large and is rejected, the economic consequences to the contractor may be substantial. If a large lot is erroneously accepted, the future maintenance costs of that lot may also be substantial. If a small lot is rejected or determined to be of poor quality, the contractor has the opportunity of correcting the deficiency before additional pavement of poor quality is constructed.

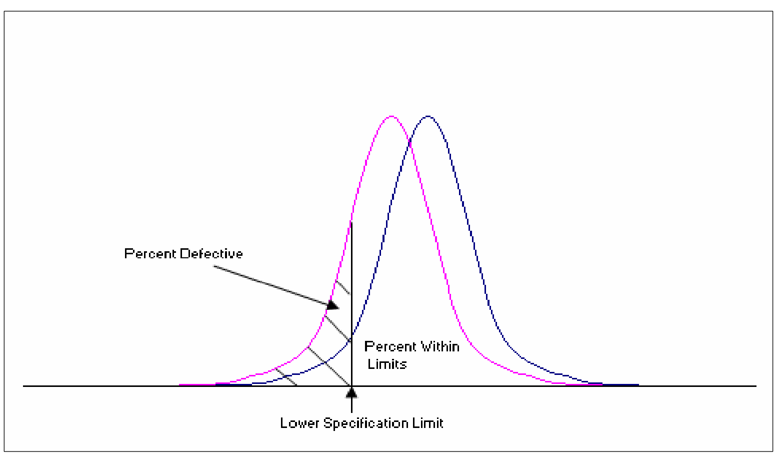

Figure D.1 – Normal Distribution of Lot Population with Different Means

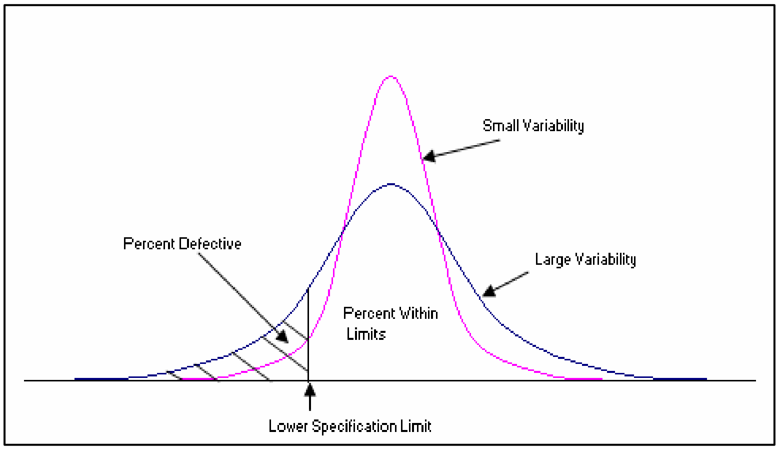

Figure D.2 – Normal Distribution of Lot Population with Different Standard Deviations

Step 2 - Define Acceptable and Rejected Quality Levels: The agency should define the quality levels of acceptable and rejected pavement in terms of the PWL of each quality characteristic in the specifications. The Acceptable Quality Level (AQL) is the degree of conformance measured in PWL at which the agency is willing to pay the full payment for the lot. The Rejected Quality Level (RQL) is the degree of conformance measured in PWL at which the lot is so deficient that replacement or correction action is warranted.

AQL and RQL are currently decided based on experience. However, AQL and RQL are commonly set at the PWL levels of 90% and 50%, respectively. A pavement lot is also rejected if the sample PWL is less than a minimum allowable level (often referred to as M). Traditionally, M is set higher than RQL as an additional reliability of the pavement section. For example, specifications with RQL of 50 PWL and minimum allowable sample PWL of 70% indicate that a sample PWL of 70% is a true estimate of a lot with a PWL of 50%.

Step 3 - Decide Levels of Acceptable Risks for Agency and Contractor: There are two risks involved in making acceptance decisions: the risk of rejecting or assigning a payment reduction (disincentive) to a pavement lot at AQL and the risk of accepting a pavement lot at RQL. The first risk is often referred to as the alpha (α), or seller's risk. The second risk is referred to as the beta (β), or buyer's risk. The seller's risk represents the contractor's risk and the buyer's risk represents the transportation agency's risk. A high buyer's risk may encourage lower bid prices; however, it increases the likelihood for premature failures and higher maintenance costs. This is particularly true for pavements where the failure of a relatively small area (say 10% to 15%) results in major rehabilitation. A large seller's risk may increase the bid prices or design characteristics unnecessarily. AASHTO R9-90 suggests that the risk be balanced between the contractor and the agency, and that it is the agency's responsibility to design acceptance plans that control the risks at suitable levels. It also indicates that these risk levels maybe based on the criticality of the measured property as it affects safety, performance, or durability.

Step 4 - Determine Sample Size: The sample size refers to the number of tests or measurements for each quality characteristic taken randomly from the lot. Obviously, the larger the sample size is, the more reliable the acceptance will become. However, the following fundamental factors should be considered when determining the sample size.

- Cost of sampling and testing: Obviously, the more sampling and testing required, the greater the project cost.
- Buyer's and seller's risks: A larger sample size results in lower buyer's and seller's risks.
- Independence of lot size: In sampling by variables, sample size does not depend on lot size. It is assumed that the lot represents a homogeneous population of pavement produced by essentially the same process, so that the distribution of every quality characteristics is likely to be normally distributed. Thus, the size of this population does not affect the sample size needed to accurately estimate its mean and standard deviation.

Randomness of sampling is a vital assumption upon which the statistical acceptance procedure is based. Random sampling can be defined as a manner of sampling that allows every member of the population (lot) to have an equal opportunity of appearing in the sample (AASHTO, 1990).

Step 5 - Evaluate Sampling Plan using Operating Characteristic (OC) Curves: An OC curve should be constructed for each quality characteristic to determine if the buyer's and seller's risks associated with the sampling plan are acceptable to the agency and the contractor.

A conventional OC curve represents the relationship between PWL and probability of acceptance. However, for quality characteristics with pay adjustment, the probability of acceptance is typically interpreted as the probability of receiving at least the full payment.

Most of the existing construction specifications for pavement (including the FAA P501) recognize that marginal products still have some value, and include payment adjustment schedules instead of requiring complete removal of the pavement. The fundamental reason for assessing payment adjustments is to match payment with serviceability of the product supplied. If, due to construction deficiencies, the pavement is not capable of withstanding the target design loading, it will fail prematurely. The necessity of repairing this pavement at an earlier date will result in an additional expense to the transportation agency. Conversely, a pavement of superior quality that lasts longer than the intended design life will result in a savings in future costs. The appropriate pay adjustment (positive or negative) is considered to reflect expenses or savings expected to occur in the future as the result of a departure from the specified target level of quality. Another reason for pay adjustments is to avoid costly rejection of a product when it is of only slightly lower quality than specified.

Currently, most pay adjustment schedules for pavement construction are established subjectively through engineering judgment and negotiations between the industry and transportation agencies. In performance-related specifications (PRS), pay adjustment factors are determined based on the difference between the total life cycle cost of the as-designed (target) pavement and that of the as-constructed pavement (Hoerner and Darter, 1999).

Overview of Existing Construction Specifications for Airport Concrete Pavement (FAA P501)

This section provides a brief review of the FAA's PCC construction specifications for airport concrete pavements which are documented in Item P-501, Portland Cement Concrete Pavement, of the FAA Advisory Circular 150/5370-10, Standards for Specifying Construction of Airports (FAA, 1999). Hereafter, these specifications are referred to as P501 Specifications.

Quality Characteristics

The acceptance criteria of concrete pavements in Item P501 are based on the following quality characteristics:

a. Flexural Strength
b. Compressive Strength
c. Pavement Thickness

Lot Size

The lot size for a project is specified by the Engineer based on the total quantity and the expected production rate. The lot size is limited to 2,000 yd^3. For projects where the basis for payment is the area of paved surface (in yd^2), the lot size is converted by the Engineer to an equivalent area that contains less than 2,000 yd^3.

Sample Size

Each lot is typically divided into four equal sublots. One sample is taken from each sublot using the plastic concrete delivered to the job site. Sampling locations is typically determined by the Engineer using the random sampling procedures (e.g. ASTM D 3665). The concrete is sampled in accordance with ASTM C 172.

Two specimens are typically prepared from each sample in accordance with ASTM C 31, and the flexural strength or compressive strength of each specimen is determined in accordance with ASTM C 78 or ASTM C 39, respectively. The flexural strength and compressive strength for each sublot is computed by averaging the results of the two test specimens representing that sublot.

For pavement thickness, one core is extracted by the contractor from each sublot from locations determined by the Engineer in accordance with random sampling procedures contained in ASTM D 3665. Areas, such as thickened edges, with planned variable thickness, are excluded from sample locations.

Acceptance Criteria and Specification Tolerance Limits

The design parameters are assumed to meet the designer's intent if they vary one standard deviation on either side of the mean. Assuming that the parameters are normally distributed, one standard deviation on either side of the mean is approximately 68% of the total area under the distribution curve. The area is distributed equally with 34% on each side of the mean. The designer's assumptions are closely related to the AQL, which means the RQL can be expressed in terms of the AQL as follows:

$$RQL = AQL - 34\% \qquad (D.3)$$

The FAA has adopted 90 PWL as the AQL, which implies the RQL can be adopted at (90-34 = 56 PWL), say 55 PWL, and still meet the designer's intent.

The tolerance limit for concrete strength has been set based on one standard deviation, which requires at least 80 PWL. The approximate value of the coefficient of variation from full-scale pavements tested to failure in the 1970’s (using beams made from fresh concrete and beams sawed from hardened concrete) was between 6% and 8%, with an average of 7%. The FAA design curves (1995) were generated from these full scale tests, which imply that the production strength should meet the average strength used to generate the design curves. This allows the lower tolerance limit for strength to be set at 0.93 x Design Strength, which is equivalent to one standard deviation from the average using a 7% COV as a baseline.

Quality lower limits provided in Item P501 specifications are:

- Flexural Strength :0.93 × 600psi = 558 psi
- Compressive Strength: 4140 psi
- Thickness: Lot Plan Thickness in inches − 0.50 in.

Pay Factors

The pay factor for each individual lot shall be calculated in accordance with table D.1 (also presented in figure D.3). This pay schedule incorporates for a bonus pay from 90% PWL to 96% PWL for high quality work and a penalty pay from 75% PWL to 55% PWL for low quality work.

A pay factor is calculated for both strength and thickness. The lot pay factor is the higher of the two values when the individual pay factors for both flexural strength and thickness are 100% or higher. When either strength or thickness pay factor is 100% or higher, the lot pay factor is the product of the two values. The lot pay factor is the lower of the two values when the individual pay factors for both strength and thickness are less than 100%.

Table D.1 - Existing Pay Adjustment Schedule

Percentage of Material Within Specification Limits (PWL)	Lot Pay Factor (Percent of Contract Unit Price)
96 – 100	106
90 – 95	PWL + 10
75 – 90	0.5PWL + 55
55 – 74	1.4PWL – 12
Below 55	Reject

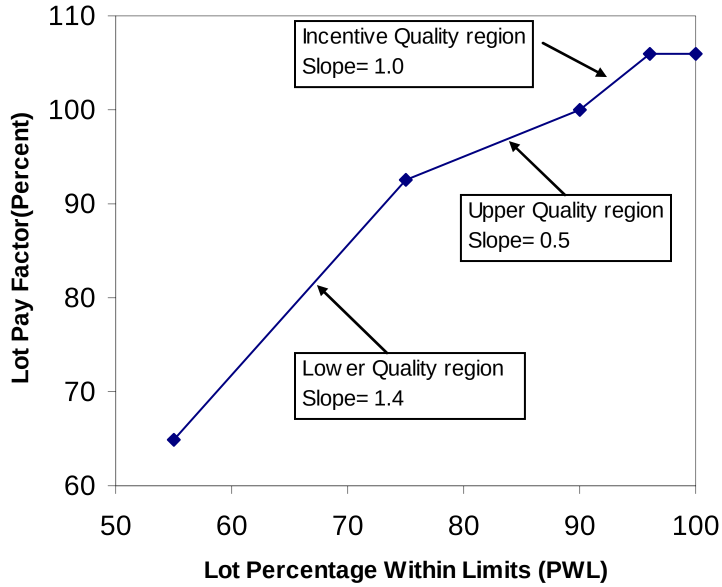

Figure D.3 - Pay Schedule from FAA P 501

Methodology for Transforming Existing Pay Schedules to Pay Schedules Based on Seismic Testing

The establishment of the PWL and pay schedule based on seismic tests can be carried out in two ways. First, a design seismic modulus can be specified, and corresponding test-related variability (in terms of COV) can be established. These values can then be used to establish the PWL and the pay schedule following the process described above. However, as indicated in chapters 4 and 5 in Volume I of this report, the relationship between strength and seismic modulus is nonlinear and depend mainly on the type of coarse aggregate. For example for a flexural strength of 600 psi, the corresponding seismic moduli are 4650 ksi for the SRG, 4800 ksi for the limestone (LS), and 5600 ksi for the granite (GRN). Perhaps an acceptance modulus of 5000 ksi would be a reasonable compromise for this activity. However, it is desirable to gain experience from a larger number of mixes with different types of coarse aggregates before the acceptance modulus can be set.

Since the existing design procedures are based on strength, it would be desirable to develop a methodology for transforming existing pay schedules of FAA specifications P501 (see table D.1) to equivalent PWL-based pay schedule based on seismic measurements. The transformation methodology, which is then based on regression models for determining the equivalent mean values and ratios of COV for determining the equivalent standard deviation, consists of the following steps:

Step 1: Develop regression models for determining the equivalent mean seismic modulus to any mean values of flexural strength and compressive strength.

Step 2: Estimate the ratios of the coefficient of variation for determining the equivalent seismic modulus standard deviation to any flexural strength standard deviation and compressive strength standard deviation.

Step 3: Simulate a large number of lots (with various combinations of mean and standard deviations for each existing quality characteristic (e.g., flexural strength).

Step 4: Compute the PWL of flexural strength and compressive strength for each simulated lot.

Step 5: Compute the equivalent mean and standard deviation of the seismic modulus obtained from PSPA using the correlations developed in Steps 1 and 2.

Step 6: Compute the PWL of the seismic modulus obtained from PSPA for each simulated lot.

Step 7: Develop regression correlations between the PWL of the seismic modulus obtained from PSPA and flexural strength or compressive strength.

Step 8: Convert the existing PWL-based pay schedule using the regression correlations developed in Step 7 for the seismic modulus obtained from PSPA.

Each step is described below.

Regression Models for Transforming Mean Values

As indicated in Equation D.1, the quality index (and thus the PWL) is a function of the mean value amongst other factors. To relate the mean strength to mean modulus, a lab-developed relationship is required. Regression models between flexural strength and seismic modulus and between compressive strength and seismic modulus were developed using the FFRC test results for different aggregate types. The data points and their models are shown in figure D.4. Generally, the models show good correlation for all aggregate types and can be used with confidence to estimate the mean modulus from the mean strength for each aggregate type.

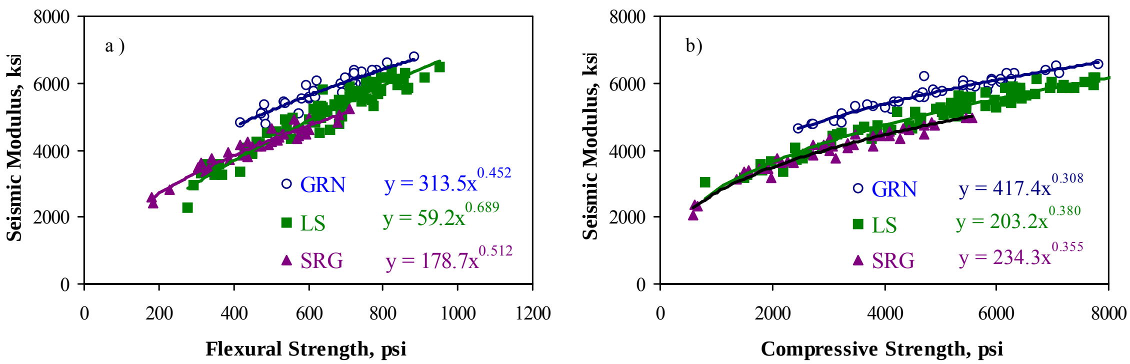

Figure D.4 – Correlations of Seismic Modulus with Flexural Strength (a) and Compressive Strength (b)

COV Ratios for Transforming Standard Deviation Values

Also required for estimating the quality index is the standard deviation of the measurements. Since the relationship between the strength (either flexural or compressive) and seismic modulus is dependent on the coarse aggregate type, a means of estimating the standard deviation for the seismic method corresponding to a given strength standard deviation should be developed. The process followed to relate these two parameters is described below. In this process, it is assumed that the ratio of the COVs from the strength and seismic modulus is constant for any given strength.

The cumulative distributions of COVs, developed from duplicate specimens from all three coarse aggregates for flexural strength, compressive strength and seismic moduli, are shown in figure D.5. The COVs at a confidence level of 90% for these parameters were obtained from the cumulative distribution curves so that they can be used in developing the COV transformation ratios. Such a high confidence level was used to incorporate some conservatism into the process. These COVs are shown in table D.2. For the FFRC, the COV is about 3.3%; whereas for the other parameters, the COVs are about 9%. It should be mentioned that for the PSPA, the results from a number of points along the pavement was used. Therefore, the COVs associated with the PSPA contain variability related to both the test method and the material variations, whereas for the other tests, the COVs correspond to primarily the variability due to testing. This will result in more conservative PWL and pay schedule. Since the database is rather small, such conservatism is warranted.

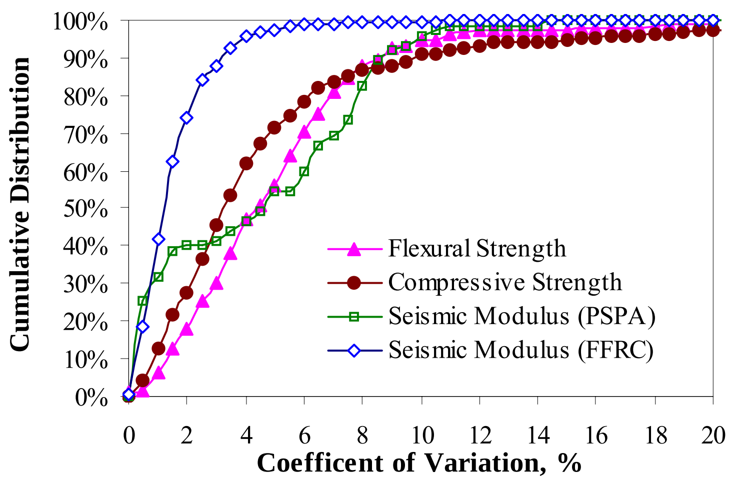

Figure D.5 - Cumulative Distribution of COV for all Testing Methods

Table D.2 – Coefficients of Variation from Different Methods at 90 Percentile Cumulative Distribution

Parameter	COV
Flexural Strength	8.6
Compressive Strength	9.7
FFRC Seismic Modulus	3.3
PSPA Seismic Modulus	8.6

With these representative COVs, the representative standard deviations for the seismic methods at any given mean strength can be obtained using:

$$s_{\text{seismic}} = \frac{\text{COV}_{\text{seissmic}}}{\text{COV}_{\text{Strength}}} * s * \left[\frac{mean_{\text{seismic}}}{mean_{\text{strength}}} \right] \tag{D.4}$$

where $S_{seismic}$ = standard deviation for seismic testing, $COV_{seismic}$ = COV of seismic testing at

90% percentile cumulative distribution (from table D.2), $COV_{strength}$ = COV of strength testing at 90% percentile cumulative distribution, s = standard deviation of strength testing, $mean_{seismic}$ = mean modulus of seismic testing from regression correlation, $mean_{strength}$ = mean of strength testing.

Transformation of Strength PWL to Seismic PWL

Under ideal conditions, it is desirable to develop the seismic-based PWL based on actual field data. However, since the database developed under this project is rather limited, this transformation was carried out through numerical simulations. This process is described below.

As specified in P-501 specifications, the design flexural strength is 600 psi and the design compressive strength is 4200 ksi. To simulate actual construction scenarios, a large number of lots with different strength and standard deviations were generated. For the flexural strength, the strength was varied from 600 psi to 800 psi and the standard deviation was varied from 20 psi to 200 psi. Software @risk was used to simulate 2000 lots for each aggregate type.

In the next step, the mean strength of the simulated lot was converted to the equivalent mean seismic modulus using the regression models shown in figure D.4. The equivalent standard deviation was also determined using Equation D.4. For each simulated lot, the traditional (strength-based) quality index and PWL were obtained using these equivalent mean and standard deviation. The strength-based and seismic-based PWLs were then compared and related.

A typical relationship between the existing flexural strength PWL and seismic PWL obtained from the process described above for the SRG aggregates is shown in figure D.6a. Only the results between PWL of 55 (lower limit of acceptance) and 100 are shown. Some scatter in the data is observed primarily because the relationship between the strength and modulus is not linear. The best fit-line through the data is also shown in the figure. The best fit line exhibits a good correlation with an R^2 value of about 0.94. Similar results but for the compressive strength are shown in figure D.6b. The existing and seismic PWLs are better correlated in this case with an R^2 value of about 0.98. These relationships can be used in the development of the pay schedule as discussed in the next section.

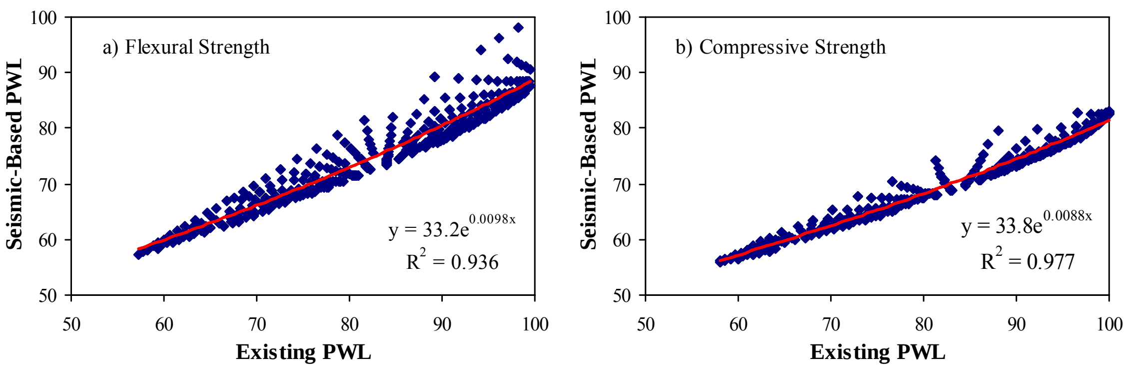

Figure D.6 - Variation in Simulated PWL from Strength Tests and Seismic Tests for SRG Aggregates

The same exercise was carried out for the other two aggregate types to investigate the impact of the variations in strength-modulus relationships on the relationships developed between the existing PWL and the seismic-based PWL. In general, the trends in the results for the limestone and granite aggregates were similar to those for the SRG aggregates.

The best fit lines relating the existing PWL to the seismic-based PWL from the three aggregates are compared in figure D.7. As anticipated, some variations between the three relationships are observed; however, the differences are not very significant. As such, the global best-fit line through all the simulated data from all three aggregates was developed and used in the development of the pay schedule.

Pay Adjustment Schedules for New Testing Methods

Pay schedules for the seismic modulus and pavement thickness (when obtained from PSPA) were then developed. These pay schedules are equivalent to the existing pay adjustment schedule (used in Item P501) for concrete strength and pavement thickness. In other words, the new pay schedules are simply a transformation of the existing pay schedule.

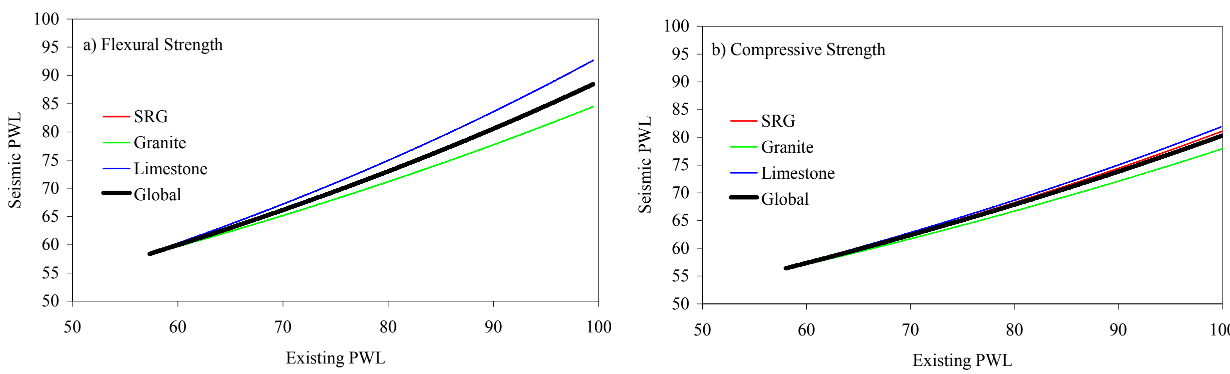

Figure D.7 - Variations in Simulated PWL from Strength Tests and Seismic Tests for All Three Types of Aggregates

For any given strength PWL, the new testing methods receive higher pay factor, especially for compressive strength. For the thickness, the traditional PWL and pay factor procedures can be implemented since no transformation is needed. Since the IE method is less precise than actual measurement of the core length, the pay factor will be less at a given PWL.

In summary, a preliminary set of PWLs and pay schedules based on the seismic methods are proposed by relating them to existing strength-based PWL and pay schedule. These results can be improved and updated as more data becomes available. Also, in the development of the presented PWLs, it is assumed that only four PSPA measurements will be carried out within a lot. However, it is envisioned that a much larger number of points can be conveniently tested. The increase in sample size will result in higher confidence in the quality assurance program of concrete that can be readily reflected in the buyer's and seller's risks.

References

American Association of State Highway and Transportation Officials (AASHTO), *Standard Recommended Practice for Acceptance Sampling Plans for Highway Construction,* AASHTO R 9-90, 1997.

Burati J. L., Weed R. M., Hughes C. S., Hill H. S., *Optimal Procedures for Quality Assurance Specifications,* FHWA-RD-02-095, 2004.

Demos, M., N.G. Gharaibeh, and M.I. Darter, *Evaluation of Potential Application of End-Result and Performance-Related Specifications, Portland Cement Concrete Pavements*, Illinois Cooperative Highway Research Report No. 425, FHWA-IL-UI-256, 1995.

Federal Aviation Administration (FAA), *Engineering Brief No. 57*, *Extended Q-Value Table for Estimating Percent of Lot Within Limits (PWL)*, Federal Aviation Administration, Washington, DC, 1999.

Federal Aviation Administration (FAA), *Item P-501, Portland Cement Concrete Pavement, Standards for Specifying Construction of Airports*. Advisory Circular (AC) 150/5370-10A. Federal Aviation Administration, Washington, DC, 1994.

Hoerner T.E. and Darter, M.I., *Guide to Developing Performance-Related Specifications for PCC Pavements*, Volume I, Practical Guide, Final Report, and Appendix A, FHWA-RD-98-155, Federal Highway Administration, Washington, DC, February 1999.

Appendix E

Guide Specification for Estimating Concrete Strength with Seismic/Maturity Methods

The combined use of the maturity and seimic method is recommended in this study. In this appendix, a process guideline specification for implementing the combined sesimic/ maturity for accpetance is included.

Trost et al. (2006)[1] developed a guide specdification for the use of the maturity method. This specification is developed in a similar format to that specification. The proposed specification here can be used for implementing combined methods or can be simplified to be used without maturity method.

[1] Trost, S., Fick, G., Hunt, J., and Pruitt, J. (2006), "Using Maturity Testing for Airfield Concrete Pavement Construction and Repair," Appendix I, Report IPRF-01-G-002-03-6, Innovative Pavement Research Foundation, Skokie, IL 60077.

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GENERAL NOTES TO ENGINEER:

Prior to using this guide specification, additional resources should be consulted so that the user thoroughly understands the concrete seismic and maturity mothods and their inherent limitations and potential sources of error.

This guide specification is provided as a guide only. The user is responsible for verifying all details, procedures, and protocals and their suitability for use on a given project.

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1. DESCRIPTION

1.1. SCOPE

This specification outlines procedures for developing mix-specific strength-seismic modulus (or combined strength-seimic modulus/maturity) relationships, determining concrete strength in the field using seismic modulus-based (or combined seimic/maturity-based) relationship data, facilitating staged opening to traffic decisions, and implementing a seismic modulus-based (or combined seimic/maturity-based) quality control program.

Performance of this specification may involve the handling of hazardous materials. This specification does not purport to address all the health and safety issues pertaining to its use. The user of this specification retains any and all responsibility for consulting and adhering to appropriate safety and health standards and practices and for identifying and adhering to pertinent regulatory issues prior to use.

1.2. REFERENCES

1.2.1. American Society for Testing and Materials (ASTM) Standards

C31	Making and Curing Concrete Test Specimens in the Field
C39	Compressive Strength of Cylindrical Concrete Specimens
C78	Flexural Strength of Concrete (Using Simple Beam with Third Point Loading)
C192	Making and Curing Concrete Test Specimens in the Laboratory
C215	Fundamental Transverse, Longitudinal, and Torsional Resonant Frequencies of Concrete Specimens
C496	Splitting Tensile Strength of Cylindrical Concrete Specimens
C1074	Estimating Concrete Strength by the Maturity Method

1.3. SIGNIFICANCE AND USE

This specification provides procedures for developing mix-specific strength-seismic modulus (or combined strength-seimic modulus/maturity) relationship curves and procedures for determining concrete strength in the field using seismic modulus-based (or combined seimic/maturity-based) relationship data. In addition, this specification provides guidelines initiating a quality control program for concrete strength using strength-seismic modulus (or combined seimic/maturity-based) relationship data.

1.4. SUBMISSION AND ACCEPTANCE

Strength-seismic modulus (or combined strength-seimic modulus/maturity) relationship data, field seismic modulus and maturity data, seismic modulus and maturity quality control reports, and supporting documentation shall be reported as specified herein and in accordance with the approved Quality Control Program.

1.5. CONTRACTOR QUALITY CONTROL PROGRAM

A comprehensive Contractor Quality Control Program shall be instituted as specified in Section 100 - Contractor Oucrliry Control Program.

1.5.1. Concrete Quality Control Plan

Whereas reliable determinations of concrete strength using seismic methods (or combined strength-seimic modulus/maturity) depend heavily upon the consistent implementation of adequate process- and quality-control procedures throughout the concrete production, placement, and curing operations, a comprehensive Concrete Quality Control Plan shall be incorporated into the Contractor's Quality Control Program.

1.5.2. Implementation

A comprehensive section governing the implementation of seismic modulus(or combined strength-seimic modulus/maturity) for the project shall be incorporated into the Contractor's Concrete Quality Control Plan.

1.5.2.1. Strength-Seismic Modulus/Maturity Relationships

A comprehensive section detailing the procedures for determining, reporting, and documenting strength-seismic modulus (or combined strength-seimic modulus/maturity) relationships shall be incorporated into the Contractor's Concrete Quality Control Plan. The section shall specifically detail the actions that will be taken to ensure that any testing errors are spread evenly across all measurement levels.

1.5.2.2. In-Place Flexural Strength Determination

A comprehensive section detailing the procedures for determining, reporting, and documenting in-place flexural strength using seismic modulus (or combined strength-seimic modulus/maturity) shall be incorporated into the Contractor's Concrete Quality Control Plan.

1.5.2.3. Seismic Modulus/Maturity as a Quality Control Tool/Concrete Mix Verification

A comprehensive section on the use of concrete seismic modulus/maturity as a means for quality control shall be incorporated into the Contractor's Concrete Quality Control Plan. The section shall detail the frequency and conditions under which mix verification testing will be performed. The section shall include procedures detailing the specific evaluation criteria to be applied to verification test results to determine whether or not investigative or corrective actions will be required. In addition, the section shall categorize and describe the specific investigative and corrective actions that will be taken for each of the aforementioned criteria. Investigative actions shall focus on determining the root cause of any observed deficiencies. Corrective actions shall emphasize and favor process-based changes or improvements over mere inspection-based solutions.

1.5.2.4. Strength-Seismic Modulus/Maturity Relationship Validation

A comprehensive section on the validation of strength-seismic modulus (or combined strength-seimic modulus/maturity) relationships shall be incorporated into the Contractor's Concrete Quality Control Plan. The section shall detail the frequency with which periodic validation testing will occur as well as the conditions under which non-scheduled validation testing will be required, such as whenever intentional or unintentional mix changes occur or are anticipated. The section shall detail, in the event a strength relationship is deemed invalid, the interim procedures that will govern until such time as a new strength-seismic modulus/maturity relationship can be established.

1.5.2.5. Compensating for Sources of Error

The Contractor's Concrete Quality Control Plan shall include discussion recognizing the following sources of error and identifying specific actions that will be taken to minimize their influence on the strength relationships and subsequent strength determinations.

- Batching Errors and Inconsistencies
- Changes in Raw Material Characteristics
- Testing Errors during Calibration
- Errors in Field Testing
- Human Errors when Collecting Field Data or Calculating Strength from Field Data

2. EQUIPMENT

2.1. APPARATUS

A device that complies with ASTM C215 shall be used for lab seismic tests.

A device designed for in-place concrete USW tests shall be used for field seismic tests.

A device that complies with ASTM C1074 shall be used for maturity measurements.

2.2. CALCULATIONS

2.2.1. Seismic Modulus

Seismic modulus measured in the lab with an FFRC device on a standard concrete beam or cylinder shall utilize the equation as follows:

$$E_{lab} = \rho \ (2 f_L L)^2$$

where
E_{lab} = concrete laboratory modulus (in Pa)
ρ = concrete mass density (kg/m^3)
f_L = fundamental-mode longitudinal resonant frequency (Hz)
L = length of specimen (m)

Seismic modulus measured in the field with an USW device on a concrete pavement shall utilize the equation as follows:

$$E_{field} = 2 \rho (1 + \nu) [V_R (1.13 - 0.16 \nu)]^2$$

where
E_{field} = concrete field modulus (in Pa)
ν = concrete Poisson's ratio
V_R = surface wave speed traveling in the concrete pavement (in m/s)

Laboratory and field seismic moduli, E_{lab} and E_{field}, are theoretically related as follows:

$$C_R = E_{field} / E_{lab} = (1 + \nu) (1 - 2\nu) / (1 - \nu)$$

As such, the moduli measured with the USW method should by divided by C_R.

2.2.2. Maturity

Concrete maturity systems shall utilize the Nurse-Saul maturity equation as follows:

$$M(t)=\Sigma \ \{[\max(0, (T_a-T_o)] \Delta t\}$$

where:
M(t) = time-temperature factor (TTF) (°C-hours) at age t
Δt = time interval between two consecutive measurements (in hour)
T_a = average concrete temperature during time interval, Δt
To = datum temperature (in °C)

Maturity equipment shall either be pre-programmed to 5°C as the datum temperature or shall include datum temperature as a user-selectable parameter.

Systems that allow the user to enter or select the datum temperature shall require that the user confirm the datum temperature each time prior to beginning maturity calculations and shall prominently display the datum temperature any time a maturity value is displayed or reported.

2.3. UNITS OF MEASURE

Systems for calculating concrete seismic modulus shall include units (both SI and US systems) with all displayed or printed modulus values.

Systems for calculating concrete maturity shall include units with all displayed or printed maturity values. The units of measure shall be ºC-Hours. Abbreviated unit notations, such as "CH" or "ºC-H", are acceptable.

2.4. MEASUREMENT PRECISIONS AND BIAS

Systems for determining the laboratory seismic modulus of each specimen with an FFRC device shall measure longitudinal resonant frequency to a precision and bias of plus-or-minus 10 Hz in the amplitude spectrum. Precisions of weight and dimonsion measurements for each specimen shall meet ASTM C215.

Systems for determining the field seismic modulus with an USW device shall measure surface wave velocity to a precision of plus-or-minus 10 m/sec (30 fps). At the present time, the data appropriate for determining the bias of USW measurement are not available.

Systems for determining maturity shall measure temperature to a precision and bias of plus-or-minus 1 ºC (1.8 ºF) (or better) at a resolution of 1 ºC (2 ºF) or better across an operating range of -10 ºC (14 ºF) to 85 ºC (185 ºF).

2.5. DATA SECURITY AND INTEGRITY

Systems for determining seismic modulus of each specimen with a FFRC device shall be capable of providing frequency-amplitude data in a secure, tamper-proof format.

Systems for determining seismic modulus at each location in the field with an USW device shall be capable of providing time domain data (waveforms) in a secure, tamper-proof format.

Systems for determining maturity shall be capable of providing historical maturity data in a secure, tamper-proof format.

As a minimum, the following data shall be provided with each device:

2.5.1. Serial number of the dvice or sensor

2.5.2. Calendar date and time when the sensor was activated or measurement made

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NOTE TO ENGINEER: Add this section on Data Security and Integrity if the needs of the project warrant a requirement for verifiable documentation with respect to in-place strength determinations. The Engineer shall specify the frequency with which the maturity historical data must be recorded and whether or not additional verifiable event-specific data logs will be required.

****

3. ESTABLISHING THE MIX-SPECIFIC STRENGTH-SEISMIC MODULUS/MATURITY RELATIONSHIPS FOR A CONCRETE MIX

3.1. MIX DESIGN AND MIXTURE COMPONENTS

Concrete Mix designs shall be performed and submitted for approval as specified in P-501-3.6. Concrete mixture components and admixtures shall be as specified in P-501-2. Whenever raw material sources change and/or material proportions are adjusted beyond specified batching tolerances, the strength-seismic modulus relationship shall be validated as detailed in the Contractor's Concrete Quality Control Plan.

3.2. DATA POINTS FOR ESTABLSING A STRENGTH-SEISMIC MODULUS/MATURITY RELATIONSHIP

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NOTES TO ENGINEER:

Measure seismic modulus (or seismic modulus and maturity) and flexural strength at time intervals of 1, 3, 7, 14 and 28 days, or as specified otherwise

For concrete mixtures with rapid strength development, or when strength estimates are to be made at low values of seismic modulus (or combined seismic modulus maturity index), tests should begin as soon as practicable. Subsequent tests should be scheduled to result in approximately equal increments of strength gain between test ages. At least five test ages should be used.

Three key issues need to be considered when making this determination:

1. **Collect multiple data points along the steep part of the strength-seismic modulus (or maturity index) relationship curve. The actual ages for this will depend upon how quickly the given mix design gains strength.**
2. **Collect data such that the "required" strength values (e.g. opening to light vehicle traffic through opening to loaded concrete truck traffic, and/or paving equipment) are in the middle of the data set.**
3. **NEVER EXTRAPOLATE beyond the seismic modulus (or combined seismic modulus/maturity) levels included in the strength-seismic modulus (or combined seismic modulus/maturity) relationship data. No strength determinations should be made beyond the final seismic modulus (or combined seismic modulus/maturity) level tested.**

****

Develop a minimum of five strength-seismic modulus or combined seismic modulus/maturity) relationship points at the following ages: 1, 3, 7, 14 and 28 days. Each strength-seismic modulus (or combined seismic modulus/maturity) relationship point shall be the average strength of three specimens plotted in relation to the average seismic modulus (or combined seismic modulus/maturity) of the same specimens.

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The Engineer shall specify the ages at which the strength-seismic modulus (or combined seismic modulus/maturity) relationship shall be established and the number of specimens to be tested at each age.

However, taking the averages of at least three specimens at each age for both strength and seismic modulus and maturity is desirable. This is due to the following:

- **All future determinations of in-place flexural strength will rely upon the strength-seismic modulus (or combined seismic modulus/maturity) relationship established during this calibration process.**
- **As such, testing error should be minimized during this procedure as much as possible, and**
- **Although taking the average of two specimens reduces the strength testing error by nearly 30% (compared to relying upon a single specimen), taking the average of three reduces this error by 42%. Taking the average of four reduces the error by 50%.**

****

3.3. PREPARING SPECIMENS

3.3.1. Number and Numbering of Specimens

The minimum number of test specimens shall be sixteen (16), one of which may be instrumented with one or more maturity sensors if necessary. Each specimen shall be uniquely numbered and documented concerning the batch of origin.

3.3.2. Casting Specimens

Specimens to be tested for seismic modulus/maturity/strength shall be cast in accordance with **[ASTM C31]** or **[ASTM C192]**.

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The Engineer shall specify whether the calibration specimens shall be cast in the laboratory (ASTM C192) or in the field (ASTM C31). Distinct advantages are gained when the actual production batching and mixing equipment is utilized and when the specimens are cast under field conditions.

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3.3.3. Curing Specimens

Specimens shall be cured in accordance with ASTM C 192.

3.4. TESTING

3.4.1. Seismic Modulus

Seismic modulus testing shall conform to ASTM C215

3.4.2. Maturity

Specimens to be instrumented with maturity sensors shall be identical to and cast at the same time as those to be tested for strength. As soon as practicable after casting [(i.e. within fifteen minutes or less)], the specimens shall be instrumented with maturity sensors. Maturity sensors in beam specimens shall be placed approximately 75 mm (3 in.) from each end, midway between the longitudinal sides of the beam mold and at mid-depth, [two (2)] per specimen. Maturity sensors shall be activated to begin calculating or recording within [fifteen minutes] after placement in the specimens.

3.4.3. Strength

Flexural strength testing shall conform to ASTM C78. [Compressive strength testing shall conform to ASTM C39.] [Splitting tensile strength testing shall conform to ASTM C496.] [Direct tension testing shall conform to ASTM C1583.]

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The Engineer shall specify which strength tests, if any, will be performed in addition to flexural strength. Supplemental strength tests may be used for validation or verification purposes.

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3.5. VALIDATING THE STRENGTH-SEISMIC MODULUS/MATURITY RELATIONSHIP

Each strength-seismic modulus (or combined seismic modulus/maturity) relationship shall be pcriodically validated as detailed in the Contractor's Concrete Quality Control Plan.

Whenever intentional changes are made to a concrete mix design, or when unintentional changes are suspected, the strength-seismic modulus relationship (or combined seismic modulus/maturity) shall be validated as detailed in the Contractor's Concrete Quality Control Plan.

4. ESTIMATING IN-PLACE FLEXURAL STRENGTH

4.1. MIX-SPECIFIC STRENGTH-SEISMIC MODULUS/MATURITY RELATIONSHIPS

A strength-seismic modulus (or combined seismic modulus/maturity) relationship shall be established for each mix design for which strength-from-seismic modulus (or combined seismic modulus/maturity) determinations are to be made. Strength- seismic modulus relationships shall be established in accordance with Part 3.

4.2. FIELD TESTING

4.2.1. Sesimic Modulus with USW Device

4.2.1.1. Setup of Data Acquisition Parameters and Transducer Spacing

Data acquisition parameters and transducer spacing of the USW device shall be set up as per manufacture's instructions.

4.2.1.2. Preparation of Test Surface

The test surface shall be free of staning water. Remove dirt and debris as well as excessive curing compounds (if on new construction) from the surface where the USW tests are to be conducted.

4.2.1.3. Test Location

Tests shall be conducted at specified points of each pavement extent for which seismic modulus-based (or combined seismic modulus/maturity-based) evaluation or open-to-traffic decisions will be made. For better results, test points shall not be close to cracks and pavement edges.

4.2.2. Maturity

Maturity sensors shall be placed at the beginning and end of each pavement extent for which maturity-based open-to-traffic decisions will be made. Maturity sensors shall be placed at mid-depth in the pavement.

4.3. DETERMINING IN-PLACE FLEXURAL STRENGTH USING SEISMIC MODULUS/MATURITY

Field determinations of strength using the seismic (or combined seismic modulus/maturity) method shall not be made without an approved Concrete Quality Control Plan and a valid, up-to-date strength-seismic modulus (or combined seismic modulus/maturity) relationship for each mix design for which the strength determinations are to be made.

Procedures for obtaining in-place flexural strength measurements using seismic modulus (or combined seismic modulus/maturity) shall be as detailed in the Contractor's Concrete Qutlity Control Plan.

4.4. VERIFYING THE CONCRETE MIX IN THE FIELD USING THE STRENGTH- SEISMIC MODULUS/MATURITY RELATIONSHIP

The concrete supplied to the project shall be frequently verified as detailed in the Contractor's Concrete Quality Control Plan.

Appendix F

Development and Execution of a Test Plan to Evaluate Seismic and Maturity Methods

The flexural strength (or compressive strength) and pavement thickness are the most important acceptance parameters for PCC pavement construction under the current FAA P501 specifications. Parameters that may impact the proper implementation of seismic and maturity methods were identified and studied in this project. The rationale behind selecting each parameter and they way they were assessed are discussed below.

Strength and Modulus Measurement

The parameters that impact strength were first categorized as mix or material-related, construction-related and environmental-related. These parameters are included in Table F.1. The first column in the table contains the parameters of interest. The second column summarizes the levels of variation that were considered. Most parameters were varied over a broader range than would normally be seen on an actual construction project. As such, the patterns observed may be more significant than those observed in day-to-day construction. The next three columns indicate which of the three project team members performed the testing, and whether small slabs or laboratory-prepared specimens were utilized. Any strength/modulus/maturity parameter studied using the slabs were accompanied by tests on molded specimens cured in the laboratory (for calibration purposes) and cores and beams extracted from the slabs (for validation purposes).Each item in the table is elaborated next followed by a thorough explanation.

Cement Content

The impact of the cement content on the relationships between the strength-seismic modulus, strength-maturity and strength combined seismic/maturity parameters were studied by all three partners using lab-cured specimens. The cement content was increased and decreased by 10% to simulate the worst case scenario. In this experiment, the water-cement ratio was maintained at the designed value. Due to importance of this parameter, ERDC also constructed a small-slab to ensure that the laboratory results are applicable to the field results.

Water-Cement Ratio

The impact of this parameter on the gain in strength and modulus is well known. The water-cement ratio was increased and decreased by 10% as compared with the design value. This parameter was studied through small slabs poured at UTEP. In addition, ERDC and UIC assessed its impact through laboratory cured specimens.

Air Entraining Admixtures

Air entraining admixtures have a major impact on the durability of the concrete, especially in cold regions. Since UIC was concentrating on the cold weather concrete, this parameter was studied by UIC by testing laboratory-cured specimens that contained either no, low or high levels of air-entraining admixtures.

Aggregates

The following four aggregate-related parameters were studied: type of aggregate, percent total aggregates, coarse aggregate factor and fineness modulus.

<u>*Type of Aggregate*</u> The type of coarse aggregate has the most impact on the strength-seismic modulus relationship and was a major focus of the laboratory testing. UTEP and ERDC studied this parameter through small-slab and UIC conducted a laboratory study. Siliceous river gravel,

Table F.1 - Summary of Strength-Related Activities

a) Material-Related Parameters

Parameter		This Study	UTEP		ERDC		UIC
			Slab	Specimen	Slab	Specimen	Specimen Only
Cement content		• As designed • 10% higher • 10% lower		✓	✓	✓	✓
Water-cement ratio		• As designed • 10% higher • 10% lower	✓	✓	✓	✓	✓
Air content		• No air-entrainer • Low air-entrainer • High air-entrainer					✓
Aggregates	Type of Aggregates*	• Siliceous river gravel • Limestone • Granite	✓	✓	✓	✓	✓
	% total aggregates	• As designed • 10% higher • 10% lower					✓
	Coarse Aggregate Factor	• As designed • 10% higher • 10% lower		✓	✓	✓	✓
	Fineness Modulus	• As designed • 5% Passing Sieve #50 • 25% passing Sieve #50					✓

b) Construction-Related Parameters

Parameter	This Study	UTEP		ERDC		UIC
		Slab	Specimen	Slab	Specimen	Specimen Only
Curing	• No curing compound • Curing compound • Blanket	✓		✓		
Compaction	• Appropriate compaction • Overcompaction	✓				
Grooving	• Broom finish • Standard FAA grooving			✓		
Thickness	• 6 inches • 12 inches • 18 inches	✓				

c) Environmental-Related Parameters

Parameter	This Study	UTEP		ERDC		UIC
		Slab	Specimen	Slab	Specimen	Specimen Only
Ambient Temperature	• Cold (50^o F) • Warm (70^o F) • Hot (90^o F)	✓				
Ambient Humidity	• Low (30 to 40%) • High (90 to 95%)	✓				

*** Each institution used a different coarse aggregate**

limestone, and granite were tested as the coarse aggregate in mixes prepared by UTEP, UIC, and ERDC, respectively.

Percent Total Aggregate UIC varied the amount of coarse aggregate to evaluate the impact on the strength seismic relationships. The percent coarse aggregate was increased and decreased by 20% from the baseline for this experiment.

Coarse Aggregate Factor A survey conducted near Fort Worth indicates that for the typical coarse aggregate factor used in the area, a variability of about 10% should be anticipated. The impact of this variability on estimating strength parameters was studied by UTEP and UIC by testing laboratory specimens and ERDC by testing on small slabs. The coarse aggregate fraction was increased and decreased by 10% from the corresponding design values.

Fineness Modulus The survey from the Fort Worth area also indicated that the fineness modulus may vary by as much as 15%. The fineness modulus can be changed in a number of ways. For this research, two levels, 5% and 25%, of percent materials passing Sieve No. 50 were used to simulate the range of expected variation in fineness modulus. The impact of this parameter on strength was evaluated by UIC by testing laboratory specimens.

Curing Method

The method of curing significantly impacts the final quality of concrete and can only be studied by testing slabs. UTEP evaluated the impact of curing compounds and blankets on the strength gain with age on concrete slabs. As a worst case scenario, one slab was included with no provision for curing. ERDC also evaluated the effect of mat curing versus a curing compound on one slab.

Consolidation

Proper consolidation of the in-place PCC is essential for attaining desirable performance. UTEP studied the effects of consolidation by constructing and testing slabs with appropriate consolidation and over-vibration.

Grooving

Even though grooving may not affect the strength of concrete, it may impact the in situ seismic measurements. Geomedia (2001) has carried out an extensive study of this matter, and the project team relied heavily on the results of that study. A slab was constructed with typical airfield grooving and tested by ERDC to verify the results from the Geomedia study.

Pavement Thickness

The motivation behind studying the impact of pavement thickness on strength is concerned with non-uniform curing and thus strength gain of the pavement. To study this impact, two additional small slabs with nominal thicknesses of 6 in. and 18 in. were constructed and tested at UTEP.

Ambient Temperature

Since the construction of PCC slabs are controlled by the schedule, the developed relationships should not be sensitive to the deviation of ambient temperature. UTEP constructed three slabs from the control mix and cured them at three different temperatures (nominally 50^{o}F, 70^{o}F and 90^{o}F). These slabs were periodically tested for in-situ gain in strength and seismic modulus.

Ambient Humidity

Similar to the ambient temperature, the impact of ambient humidity was also studied. Two slabs were constructed and cured under significantly different levels of the relative humidity (wet curing and dry curing) to ensure that the ambient humidity would not impact the robustness of the relationships developed. Both slabs were covered with curing compound to minimize the loss of moisture.

Thickness Measurement

The determination of thickness with seismic methods was studied with the same objectives as the strength parameters. The three major parameters studied were:

1. The texture (groove pattern) of the slab.
2. The type of material underlying the pavement.
1. The most suitable time for utilizing these methods.

Grooving

An extensive study on the impact of the texture (grooving pattern) of the concrete on the impact-echo method has been carried out by Geomedia (2001). As such, the scope of this work was mostly toward the validation of that study. To address this issue, two similar slabs were poured, one with broom finish and another with standard runway grooving. Tests were carried out on both slabs to illustrate the impact of the grooves on the impact-echo tests.

Underlying Layer

The boundary between two layers with similar mechanical impedance [2] should not be distinguishable from the impact-echo tests. Mechanical impedance of a fresh concrete, which is the product of the propagation velocity and the density, increases with time. Three 12-in. thick slabs were constructed simultaneously on (1) compacted subgrade, (2) a hot-mix asphalt base, and (3) a cement-stabilized base. These slabs were tested at different ages of concrete. The results from tests on the three slabs were used to determine at what age (if any) the contrasts in the mechanical impedance are adequate for accurate thickness estimation.

Timing of Thickness Measurements

As a concrete pavement cures, its modulus, propagation velocity and mechanical impedance increase. Since the mechanical impedance of fresh concrete vastly varies with time, the earliest age at which there is enough contrast to accurately estimate pavement thickness with the impact-echo method must be determined. This parameter was studied on the three slabs poured for studying the impact of the underlying layer.

References

Geomedia Research and Development (2001), "Determining Capacity of Military Pavements with a Portable Sonic-Ultrasonic Stress-Wave Testing Device," SBIR Phase I Report, Submitted to the Department of Defense, Washington, DC.

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[2] Mechanical impedance is the product of the propagation velocity and the density

Appendix G

Analysis of Results for Strength Estimation

Test Results from Laboratory-Cured Specimens

To develop solid relationships between strength parameters and maturity or seismic modulus, the accuracy and precision of individual tests are critical. The results from tests on laboratory-cured specimens for all mixes with granite (GRN), limestone (LS) and siliceous river gravel (SRG) coarse aggregates are summarized in tables G.1 through G.3.

As reflected in table G.3, UTEP used two standard mixes for the SRG aggregates, namely Standard 1 and Standard 2. Standard 1 corresponds to the mixes delivered by a concrete truck (large mixer), whereas Standard 2 was batched in the laboratory using a small mixer. All experiments involving small slabs used Standard-1 mixes. The strengths and moduli from Standard 1 are consistently higher than those from Standard 2.

The global average measurement errors in terms of coefficient of variation (COV) are 4.9%, 4.5% and 1.5% for flexural strength, compressive strength and seismic (FFRC) modulus, respectively.

Accuracy and Precision of Laboratory-Developed Relationships

The relationships or models between strength and maturity or seismic modulus and between seismic modulus and maturity were developed from the data provided in tables G.1 through G.3. The quality of these relationships is judged by the corresponding R^2 values and the standard errors of estimate (SEE). The results are included in tables G.4 through G.10 for all individual mixes, with the general form of model shown at the bottom of each table. For each model, four data points were available as shown in tables G.1 through G.3.

In general, all models show reasonably high R^2 and small SEE. For those models of relatively low R^2 and large SEE, as stated above, the most part of the error source is related to strength testing.

Impact of Mix-Related Parameters on Lab-Developed Relationships

The data available from all mixes for each aggregate type are presented in figures G.1 through G.7. The solid line in each figure corresponds to the relationship developed for the standard mix design with the given type of coarse aggregates. The dashed lines on each figure correspond to 10% error bands for reference.

The variations in flexural strength, compressive strength and seismic modulus with TTF are shown in figures G.1 through G.3. In general, as the TTF increases, the two strength parameters and seismic modulus increase. However, some scatter is evident in the results.

For the mixes with SRG (figures G.1a and G.2a), the highest strengths at a given TTF correspond to the mix with the lower-than-design water-cement ratio, and the lowest correspond to the less-than-design cement content.

For the mixes with LS (figures G.1b and G.2b), most strength parameters fall above those for the standard mix at a given TTF. For a number of mixes, the strength parameters fall outside the 10% error band, indicating that for temperature-based quality management, a rigid process control is needed during construction to ensure that the laboratory-developed relationships can be reliably used for field testing.

Table G.1 – Summary of Test Results from Lab-Cured Specimens with Granite Coarse Aggregates

Mix	Age	Maturity TTF	Cylinders				Beams			
			FFRC Modulus		Compressive Strength		FFRC Modulus		Flexural Strength	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)
Standard	1	1174	4888	0.7	3103	4.9	5118	0.6	478	6.1
	3	3443	5245	0.9	4130	3.5	5557	0.4	603	10.0
	7	6897	5606	1.6	4722	5.7	5925	0.4	595	10.9
	14	12757	5762	0.9	5018	15.1	6220	0.8	719	5.2
	28	23916	5932	1.8	5849	7.5	6271	0.3	723	7.4
Low WCR	1	1201	5264	1.6	4013	1.4	5437	0.8	535	5.8
	3	3489	5915	1.0	5626	2.4	6042	0.7	743	4.0
	7	6849	6297	1.5	6326	4.6	6381	1.1	784	6.6
	14	12631	6503	0.1	7050	11.8	6593	0.9	810	4.0
	28	24174	6581	0.2	7816	4.3	6774	1.4	885	4.3
High WCR	1	1163	4653	2.3	2469	5.3	4846	0.8	418	23.0
	3	3310	5310	1.3	3805	4.3	5524	0.7	583	3.0
	7	6614	5736	0.7	4686	4.1	5847	0.9	715	2.7
	14	12324	6062	1.0	5417	2.0	6050	1.1	699	1.4
	28	23632	6132	2.5	5917	5.6	6173	1.3	719	1.4
More Cement	1	1200	5322	2.1	3479	3.5	5353	1.5	481	6.3
	3	3538	5819	2.8	4861	2.8	5574	3.2	617	1.0
	7	6994	6089	2.4	5926	2.7	6096	0.4	613	6.9
	14	13748	6200	2.2	4714	14.7	6357	0.3	742	2.7
	28	23640	6306	2.7	7124	0.3	6394	1.2	722	5.3
Less Cement	1	1273	4793	0.7	2696	4.4	5006	0.5	482	0.7
	3	3508	5459	1.3	4176	1.9	5579	0.7	617	4.0
	7	6688	5628	1.4	4579	3.9	5829	0.6	626	5.2
	14	12396	5931	0.7	5540	1.0	5956	0.1	759	8.5
	28	23908	6037	0.8	6111	2.6	6103	0.3	740	7.3
High CAF	1	1254	4835	2.2	3235	9.5	5100		575	
	3	3454	5472	2.0	4256	17.8	5699	1.2	691	1.6
	7	6701	5735	1.0	5410	4.1	5980	1.7	728	1.1
	14	12352	6055	1.4	6075	6.8	6265	0.7	775	8.2
	28	22976	6280	1.7	6181	11.2	6397	1.4	770	11.0
Low CAF	1	1153	4763	1.3	2741	3.2	4764	2.6	484	5.7
	3	3291	5345	1.7	3702	1.9	5424	1.4	540	7.1
	7	6592	5739	0.7	4920	2.5	5767	0.1	616	5.1
	14	11858	5783	3.2	5826	0.9	5909	0.7	717	1.2
	28	22140	6108	0.4	6062	8.6	6060	0.2	719	6.2
FAA Standard	1	1154	5079	0.5	3250	5.0	5114	0.7	473	7.8
	3	3223	5567	0.8	4642	1.8	5745	1.0	614	2.4
	7	6243	5930	0.6	4657	8.3	5977	0.8	687	4.5
	14	11536	6142	0.6	6268	1.4	6185	0.9	698	8.5
	28	23362	6407	0.1	6870	3.1	6313	0.6	722	6.7

Table G.2 – Summary of Test Results from Lab-Cured Specimens with Limestone Coarse Aggregates

Mix	Age	Maturity TTF	Cylinders				Beams			
			FFRC Modulus		Compressive Strength		FFRC Modulus		Flexural Strength	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)
Standard	1	1009	3682	2.3	1982	3.5	3580	3.2	331	6.5
	3	2611	4734	1.8	3893	3.2	4575	2.6	513	4.0
	7	5738	5215	1.2	4918	2.1	5140	2.5	624	2.6
	14	11094	5481	0.9	5356	2.2	5519	1.3	682	3.4
	28	21594	5851	1.2	6142	2.8	5668	2.0	731	7.4
Low WCR	1	1179	4682	2.0	3631	19.4	4942	0.7	557	1.5
	3	2770	5581	2.1	5420	16.1	5520	5.9	754	7.2
	7	5805	5943	1.7	7203	6.2	6109	0.4	831	5.1
	14	10967	6031	5.2	7580	9.9	6304	1.0	858	5.9
	28	20345	6414	3.1	8789	6.0	6477	0.5	949	5.4
High WCR	1	1119	4051	0.7	2883	3.5	3460	9.4	353	18.3
	3	2675	4750	1.4	4438	12.4	4196	7.0	470	21.0
	7	5780	5221		5453	5.8	5280	0.7	637	4.9
	14	10984	5495	1.3	5958	3.6	5143	4.3	647	15.3
	28	21367	5727	1.5	6386	2.8	5338	2.2	677	1.5
More Cement	1	1024	3731	3.0	2434	7.6	4050	1.1	428	2.8
	3	2398	4567	0.4	3822	3.3	4802	1.4	542	6.9
	7	4968	5134	1.0	5019	2.3	5307	1.0	632	4.0
	14	9519	5381	0.4	5465	1.4	5556	1.0	748	6.4
	28	18279	5623	0.4	6399	8.8	5791	0.3	783	6.0
Less Cement	1	910	3362		2192		3362	2.1	417	4.0
	3	2164	4502	1.3	3236	5.9	4894	3.1	536	2.2
	7	4864	5135	1.4	4550	1.0	5324	2.2	626	5.6
	14	9414	5398	0.4	5190	2.6	5450	1.2	688	6.3
	28	19135	5627	2.0	5328	5.1	6070	0.9	756	3.6
High CAF	1	679	3164	3.7	1510	2.6	2959	1.5	292	6.6
	3	2291	4624	3.0	4115	2.9	4513	1.0	629	3.6
	7	5356	5195	0.7	5171	7.3	5187	1.6	649	6.4
	14	10708	5590	1.5	6387	1.9	5409	2.9	700	4.9
	28	21840	5621	1.4	6523	6.6	5767	2.4	793	5.6
Low CAF	1	939	4063		2399		3280	3.9	368	0.2
	3	2584	5087		5246		4625	4.9	660	7.1
	7	6008	5396	5.8	6261		5512	3.3	770	5.6
	14	11753	5871	1.0	7398	1.3	5806	3.1	862	2.2
	28	23332	6165	1.2	7736	4.9	6159	2.2	912	2.7
Less Water	1	1060	4444	5.9	3082	2.0	4533	1.4	491	1.6
	3	2548	5147	1.1	4216	3.8	5216	1.2	614	2.2
	7	5541	5578	1.3	5527	2.4	5739	1.5	717	3.9
	14	10814	5752	4.9	6264	2.1	6034	0.4	773	3.4
	28	20604	6104	1.8	6713	8.4	6184	2.0	810	4.0

Table G.2 – Summary of Test Results from Lab-Cured Specimens with Limestone Coarse Aggregates (Con't)

Mix	Age	Maturity TTF	Cylinders				Beams			
			FFRC Modulus		Compressive Strength		FFRC Modulus		Flexural Strength	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)
More Water	1	1117	3835	2.0	2498	4.9	4236	3.1	463	2.7
	3	2746	4660	0.3	4054	4.5	4832	0.4	619	7.9
	7	6091	5288	0.5	5370	1.4	5374	0.3	669	18.0
	14	11838	5545	1.8	6458	1.7	5811	4.0	640	3.4
	28	22010	5744	1.1	6431	2.8	5901	1.2	784	5.6
High FM	1	630	3018	2.2	798	3.5	2268	0.4	275	1.0
	3	2152	4588	3.8	3526	6.5	4545	1.6	589	5.4
	7	5195	5385	1.0	5339	0.8	5350	1.4	696	5.2
	14	10520	5665	0.1	6293	3.5	5753	1.1	736	4.4
	28	21112	5843	1.4	6421	3.6	5935	1.4	838	5.3
Low FM	1	666	3259	1.1	1556	0.4	3288	3.3	314	7.0
	3	2073	4452	0.5	3163	3.2	4512	1.1	533	3.8
	7	4849	5274	1.6	5001	3.0	5166	0.5	648	2.2
	14	9561	5702	1.1	5808	4.6	5570	0.5	709	5.0
	28	19799	5905	1.3	6288	7.3	5672	1.8	746	5.0
High PTA	1	682	3560	1.1	1788	3.5	3576	3.7	363	4.7
	3	2002	4973	1.6	4659	4.5	4900	1.8	602	3.1
	7	4586	5430	1.6	5435	6.8	5581	2.7	714	0.6
	14	9340	5833	0.1	6343	2.6	5767	1.3	788	3.3
	28	18927	6028	1.2	6980	2.2	6145	1.1	795	4.6
Low PTA	1	920	3738	1.3	2542	3.1	3909	0.9	450	0.8
	3	2410	4657	0.3	4501	0.9	4800	0.8	681	4.1
	7	5250	5299	1.0	5811	6.1	5436	1.5	742	3.4
	14	10034	5622	0.8	6367	3.5	5697	0.2	785	0.8
	28	19787	5864	0.5	6950	2.7	5873	1.3	868	4.8
High AEA	1	989	3396	1.1	1810	0.6	3262	3.1	348	3.7
	3	2634	4336	1.8	3786	3.9	4335	3.5	555	3.7
	7	5707	5026	1.6	5319	3.6	4953	3.9	635	10.6
	14	10603	5277	0.4	6000	2.1	5193	3.6	688	4.1
	28	19752	5454	1.1	6149	2.4	5338	3.0	774	1.6
Low AEA	1	916	3889	2.1	2623	2.9	3952	3.9	449	3.5
	3	2408	4813	1.5	4632	1.3	4996	2.8	691	2.1
	7	5344	5528	2.4	5354	30.9	5641	3.7	736	3.3
	14	10401	5859	0.8	7171	1.8	5883	1.2	757	4.6
	28	19438	5859	2.0	7314	0.7	6124	0.3	854	3.7
No AEA	1	875	4166	0.9	3046	5.4	4534	2.3	543	6.5
	3	2446	5039	2.1	5069	2.7	5352	0.8	699	1.8
	7	5433	5655	0.1	6696	3.9	5904	1.6	712	2.5
	14	10667	5949	1.2	7727	3.0	6148	0.4	808	5.3
	28	21085	6171	0.5	7766	3.2	6383	0.3	823	3.0

Table G.3 – Summary of Test Results from Lab-Cured Specimens with Siliceous River Gravel Coarse Aggregates

Mix	Age	Maturity TTF	Cylinders				Beams			
			FFRC Modulus		Compressive Strength		FFRC Modulus		Flexural Strength	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)
Standard (1)	1	858	3463	1.8	1928	4.6	3739	0.2	339	7.1
	3	2346	4115	1.8	3090	6.4	4254	0.6	439	5.0
	7	5238	4484	1.5	3929	2.7	4627	0.3	503	7.3
	14	10282	4630	1.6	4324	4.5	4783	1.5	570	4.1
	28	20053	4827	2.2	4833	3.9	4970	1.2	602	5.3
Low WCR	1	890	3836	0.5	2738	3.8	4158	2.0	418	5.9
	3	2362	4422	0.6	3946	1.1	4633	0.8	573	6.8
	7	5198	4759	0.5	4888	0.6	4918	0.0	563	3.1
	14	10483	4979	1.0	5469	2.7	5063	0.6	683	6.7
	28	20429	4983	3.6	5577	0.9	5247	0.5	710	9.0
High WCR	1	861	3176	0.4	1973	3.1	3339	3.5	325	8.7
	3	2341	3735	4.7	3122	3.3	3798	1.3	438	2.4
	7	5094	4111	1.9	3788	1.2	4126	0.8	475	1.5
	14	10328	4356	0.6	4270	3.8	4382	1.3	520	2.7
	28	20135	4485	0.8	4719	1.2	4464	10.6	533	3.3
Standard (2)	1	877	3136	2.4	1357	1.0	3470	0.6	305	2.3
	3	2275	3666	3.0	2267	1.4	3848	1.7	420	3.4
	7	5157	4177	1.0	2925	2.7	4170	2.0	470	1.5
	14	10413	4217	1.0	3338	5.2	4393	0.1	518	8.5
	28	20922	4404	1.6	3789	1.8	4411	0.3	575	1.3
More Cement	1	706	3430	2.0	1759	2.3	3608	1.6	313	7.9
	3	1794	4099	1.1	2947	6.2	4093	1.1	465	4.6
	7	3970	4454	0.5	3476	10.9	4458	0.2	530	9.3
	14	7593	4555	0.0	4091	0.3	4600	0.2	588	1.8
	28	14162	4718	1.0	4712	0.8	4832	0.1	678	1.0
Less Cement	1	664	2365	3.4	626	4.2	2602	3.3	180	3.9
	3	1874	3342	3.2	1499	7.6	3438	0.9	335	1.1
	7	4301	3734	2.2	2087	1.2	3922	0.1	385	0.9
	14	8692	4152	1.0	2728	5.8	4092	0.4	450	7.9
	28	16683	4356	1.3	3029	9.6	4285	2.1	517	8.6
High CAF	1	617	2316	0.8	681	24.1	2828	0.7	230	3.1
	3	1853	3598	0.4	2176	3.6	3731	0.6	388	2.7
	7	4335	4142	3.4	3447	2.2	4255	1.0	503	2.1
	14	8249	4425	2.7	4103	0.7	4488	1.4	593	0.6
	28	16233	4586	1.4	4449	3.4	4603	0.6	600	2.4
Low CAF	1	615	2056	3.5	601	1.8	2431	1.2	185	3.8
	3	1753	3380	0.9	1570	2.4	3453	1.6	335	0.0
	7	4022	3607	0.8	2371	2.8	3858	2.1	423	0.8
	14	8084	3992	3.3	3016	2.4	4174	0.1	493	3.6
	28	15778	4080	3.6	3344	2.4	4353	0.7	570	7.4

Table G.4 – Summary of Laboratory-Developed Relationships between Flexural Strength and Maturity Parameter

Coarse Aggregate	Mix Designation	Fit Parameters*		R^2	SEE ** (psi)
		α_1	β_1		
SRG	Standard 1 (Large Mixer)	84.8	-226	0.99	9
	Low WCR	90.3	-175	0.92	30
	High WCR	65.3	-92.8	0.94	19
	Standard 2 (Small Mixer)	81.6	-232	0.98	12
	More Cement	115	-426	0.98	16
	Less Cement	99.7	-448	0.98	17
	High CAF	119.6	-519	0.96	27
	Low CAF	116.4	-550	0.99	10
	Global	98.3	-350	0.75	64
Limestone	Standard	130	-535	0.96	28
	Low WCR	127	-303	0.94	33
	High WCR	115	-429	0.91	38
	More Cement	129	-459	0.99	16
	Less Cement	110	-321	0.99	10
	High CAF	132	-497	0.88	60
	Low CAF	166	-710	0.94	48
	Less Water	109	-251	0.97	19
	More Water	90.2	-141	0.84	41
	High FM	152	-650	0.94	48
	Low FM	128	-480	0.95	36
	High PTA	132	-444	0.91	47
	Low PTA	127	-368	0.93	38
	High AEA	136	-555	0.97	26
	Low AEA	119	-306	0.89	45
	No AEA	86.6	-17.6	0.93	27
	Global	**125**	**-414**	**0.78**	**75**
Granite	Standard	83.0	-100	0.91	27
	Low WCR	109	-199	0.92	33
	High WCR	102	-259	0.84	46
	More Cement	84.9	-105	0.91	28
	Less Cement	94.8	-185	0.91	29
	High CAF	69.1	106	0.91	22
	Low CAF	88.8	-154	0.94	23
	FFA Standard	82.3	-73.5	0.89	30
	Global	**90.0**	**-125**	**0.74**	**55**

*** Flex Strength = α_1 Log (TTF)$^{\beta 1}$** **SEE = Standard Error of Estimate**

Table G.5 – Summary of Laboratory-Developed Relationships between Compressive Strength and Maturity Parameter

Coarse Aggregate	Mix Designation	Fit Parameters*		R^2	SEE ** (psi)
		α_1	β_1		
SRG	Standard 1 (Large Mixer)	883	-3843	0.99	115
	Low WCR	943	-3457	0.95	233
	High WCR	862	-3701	0.98	121
	Standard 2 (Small Mixer)	760	-3688	0.99	87
	More Cement	951	-4373	0.99	101
	Less Cement	760	-4266	0.99	67
	High CAF	1195	-6839	0.98	209
	Low CAF	871	-4933	0.99	91
	Global	**943**	**-4720**	**0.67**	**742**
Limestone	Standard	1312	-6751	0.96	273
	Low WCR	1778	-8730	0.98	279
	High WCR	1182	-5107	0.96	259
	More Cement	1349	-6754	0.98	170
	Less Cement	1099	-5127	0.95	271
	High CAF	1484	-7761	0.95	399
	Low CAF	1649	-8322	0.94	469
	Less Water	1270	-5675	0.98	173
	More Water	1404	-7120	0.95	326
	High FM	1673	-9541	0.95	484
	Low FM	1471	-7897	0.97	282
	High PTA	1507	-7483	0.94	442
	Low PTA	1436	-6911	0.96	307
	High AEA	1511	-8252	0.94	389
	Low AEA	1586	-8003	0.96	333
	No AEA	1571	-7266	0.95	417
	Global	**1474**	**-7453**	**0.84**	**721**
Granite	Standard	871	-3032	0.99	102
	Low WCR	1249	-4727	0.99	95
	High WCR	1168	-5687	0.99	103
	More Cement	2860	-5980	0.99	184
	Less Cement	1157	-5476	0.99	134
	High CAF	1096	-4514	0.96	232
	Low CAF	1216	-5888	0.97	212
	FFA Standard	1221	-5277	0.99	94
	Global	**1099**	**-4614**	**0.80**	**611**

*** Comp. Strength = α_2 Log (TTF)$^{\beta 2}$** **SEE = Standard Error of Estimate**

Table G.6 – Summary of Laboratory-Developed Relationships between Seismic Modulus and Maturity Parameter

Coarse Aggregate	Mix Designation	Fit Parameters*		R^2	SEE ** (psi)
		α_1	β_1		
SRG	Standard 1 (Large Mixer)	390	1094	0.96	82
	Low WCR	358	1669	0.95	90
	High WCR	395	663	0.97	75
	Standard 2 (Small Mixer)	356	980	0.95	95
	More Cement	410	938	0.96	90
	Less Cement	563	-1012	0.96	134
	High CAF	621	-1199	0.93	188
	Low CAF	592	-1298	0.91	207
	Global	**479**	**70.1**	**0.67**	**377**
Limestone	Standard	686	-915	0.96	157
	Low WCR	557	1019	0.95	125
	High WCR	613	-384	0.92	185
	More Cement	626	-290	0.96	135
	Less Cement	766	-1520	0.91	255
	High CAF	757	-1570	0.93	258
	Low CAF	764	-1351	0.96	167
	Less Water	552	767	0.97	102
	More Water	616	-175	0.97	114
	High FM	923	-2899	0.91	351
	Low FM	761	-1462	0.95	202
	High PTA	739	-965	0.93	234
	Low PTA	675	-620	0.96	153
	High AEA	699	-1300	0.94	181
	Low AEA	690	-589	0.94	188
	No AEA	611	348	0.97	127
	Global	**701**	**-847**	**0.82**	**355**
Granite	Standard	357	2374	0.99	43
	Low WCR	453	2164	0.95	110
	High WCR	514	1116	0.96	105
	More Cement	330	3066	0.96	74
	Less Cement	424	1871	0.95	94
	High CAF	495	1363	0.99	51
	Low CAF	443	1711	0.97	81
	FFA Standard	445	1970	0.99	39
	Global	**412**	**2118**	**0.77**	**229**

*** Modulus = α_3 Log (TTF)$^{\beta 3}$** **SEE = Standard Error of Estimate**

Table G.7 – Summary of Laboratory-Developed Relationships between Flexural Strength and Seismic Modulus

Coarse Aggregate	Mix Designation	Fit Parameters*		R^2	SEE ** (psi)
		α_1	β_1		
SRG	Standard 1 (Large Mixer)	2.02E-05	2.022	0.99	10
	Low WCR	4.08E-06	2.215	0.92	31
	High WCR	5.05E-04	1.652	0.97	12
	Standard 2 (Small Mixer)	1.44E-06	2.355	0.96	20
	More Cement	2.67E-07	2.552	0.99	14
	Less Cement	2.33E-05	2.018	0.99	18
	High CAF	2.95E-05	1.995	1.00	11
	Low CAF	7.90E-05	1.878	0.99	16
	Global	**8.70E-05**	**1.859**	**0.95**	**34**
Limestone	Standard	2.98E-04	1.702	1.00	8
	Low WCR	1.51E-04	1.782	0.95	30
	High WCR	2.16E-03	1.473	0.99	12
	More Cement	2.81E-04	1.711	0.98	22
	Less Cement	1.24E-01	0.997	0.94	31
	High CAF	2.71E-03	1.454	0.97	37
	Low CAF	3.48E-03	1.432	0.99	24
	Less Water	6.67E-04	1.605	1.00	2
	More Water	1.44E-02	1.249	0.81	49
	High FM	5.39E-02	1.105	0.99	25
	Low FM	9.56E-04	1.570	1.00	9
	High PTA	1.62E-03	1.507	0.99	23
	Low PTA	1.91E-03	1.500	0.96	29
	High AEA	1.58E-03	1.521	0.99	23
	Low AEA	5.78E-03	1.363	0.96	32
	No AEA	2.64E-02	1.181	0.95	24
	Global	**6.79E-03**	**1.342**	**0.93**	**45**
Granite	Standard	4.12E-05	1.907	0.91	27
	Low WCR	4.42E-06	2.167	0.95	26
	High WCR	9.83E-07	2.343	0.96	29
	More Cement	2.44E-05	1.964	0.83	37
	Less Cement	2.32E-06	2.247	0.92	32
	High CAF	6.44E-03	1.337	0.98	12
	Low CAF	2.47E-04	1.706	0.89	32
	FFA Standard	1.19E-05	2.051	0.98	16
	Global	**4.01E-05**	**1.914**	**0.86**	**40**

*** Flex Strength = α_4 (Modulus)$^{\beta 4}$** **SEE = Standard Error of Estimate**

Table G.8 – Summary of Laboratory-Developed Relationships between Compressive Strength and Seismic Modulus

Coarse Aggregate	Mix Designation	Fit Parameters*		R^2	SEE ** (psi)
		α_1	β_1		
SRG	Standard 1 (Large Mixer)	2.94E-07	2.773	1.00	14
	Low WCR	5.75E-07	2.700	1.00	40
	High WCR	4.86E-06	2.461	0.99	75
	Standard 2 (Small Mixer)	9.50E-08	2.906	0.99	130
	More Cement	5.73E-08	2.965	0.98	172
	Less Cement	9.96E-07	2.607	1.00	30
	High CAF	3.22E-07	2.770	1.00	65
	Low CAF	4.26E-06	2.453	0.97	214
	Global	**5.57E-07**	**2.699**	**0.96**	**308**
Limestone	Standard	3.25E-06	2.466	1.00	128
	Low WCR	9.67E-08	2.878	0.98	244
	High WCR	1.37E-05	2.310	0.99	135
	More Cement	1.45E-05	2.302	1.00	109
	Less Cement	1.02E-03	1.791	0.98	190
	High CAF	2.20E-06	2.526	1.00	97
	Low CAF	9.18E-08	2.894	0.97	442
	Less Water	1.09E-06	2.589	0.98	205
	More Water	5.30E-06	2.421	0.99	189
	High FM	5.57E-09	3.211	0.99	271
	Low FM	7.14E-06	2.373	1.00	81
	High PTA	1.32E-06	2.575	0.99	180
	Low PTA	2.98E-05	2.223	0.99	155
	High AEA	1.03E-06	2.622	0.99	190
	Low AEA	8.86E-06	2.361	0.97	385
	No AEA	3.54E-06	2.470	0.99	230
	Global	**2.38E-06**	**2.510**	**0.95**	**397**
Granite	Standard	2.22E-08	3.024	0.98	152
	Low WCR	1.76E-07	2.783	0.98	219
	High WCR	1.58E-08	3.053	1.00	104
	More Cement	1.718E-05	2.286	0.87	454
	Less Cement	4.44E-10	3.472	1.00	87
	High CAF	5.37E-07	2.654	0.97	239
	Low CAF	7.80E-10	3.409	0.95	326
	FFA Standard	3.63E-09	3.228	0.99	130
	Global	**1.79E-08**	**3.043**	**0.94**	**362**

*** Comp. Strength = α_5 (Modulus)$^{\beta 5}$** **SEE = Standard Error of Estimate**

Table G.9 – Summary of Laboratory-Developed Relationships between Flexural Strength, Seismic Modulus and Maturity Parameter

Coarse Aggregate	Mix Designation	Fit Parameters*				R^2	SEE** (psi)
		α_6	β_6	γ_6	δ_6		
SRG	Standard 1 (Large Mixer)	8.256	0.398	59.2	-253	0.91	23
	Low WCR	3.813	0.627	55.5	-656	0.92	29
	High WCR	4.563	0.684	-19.2	-714	0.99	9
	Standard 2 (Small Mixer)	7.149	0.508	62.4	-555	0.98	11
	More Cement	3.518	0.670	54.6	-893	0.99	13
	Less Cement	5.273	0.547	62.4	-609	0.99	12
	High CAF	0.398	0.889	40.9	-506	0.99	14
	Low CAF	2.614	0.553	93.7	-611	1.00	3
	Global	**0.709**	**0.842**	**24.4**	**-517**	**0.95**	**70**
Limestone	Standard	5.491	0.629	26.6	-793	1.00	5
	Low WCR	7.521	0.563	48.4	-723	0.98	18
	High WCR	5.464	0.646	36.2	-1005	0.96	27
	More Cement	0.846	0.688	104.0	-551	0.99	15
	Less Cement	3.329	0.513	87.8	-395	1.00	4
	High CAF	8.430	0.595	12.4	-765	0.94	37
	Low CAF	8.221	0.602	13.2	-790	0.99	17
	Less Water	2.215	0.727	21.6	-672	1.00	2
	More Water	-2.089	0.717	168.7	150	0.86	39
	High FM	8.398	0.531	57.9	-602	0.99	17
	Low FM	10.004	0.567	21.8	-811	1.00	3
	High PTA	8.542	0.598	4.5	-807	0.99	18
	Low PTA	7.485	0.605	24.0	-814	0.97	26
	High AEA	6.142	0.572	67.5	-736	0.99	15
	Low AEA	6.592	0.629	-5.9	-708	0.95	29
	No AEA	4.926	0.606	27.8	-445	0.95	23
	Global	**0.624**	**0.819**	**30.5**	**-412**	**0.91**	**47**
Granite	Standard	1.817	0.683	50.4	-492	0.92	26
	Low WCR	5.225	0.721	-39.6	-1744	0.97	22
	High WCR	3.567	0.773	-71.1	-1600	0.97	20
	More Cement	-1.494	0.600	96.2	73	0.91	28
	Less Cement	3.218	0.614	68.8	-610	0.92	29
	High CAF	-2.851	0.596	112.0	131	0.94	22
	Low CAF	3.889	0.740	-68.0	-1093	1.00	1
	FFA Standard	5.146	0.706	-31.5	-1447	0.99	10
	Global	**2.228**	**0.746**	**12.2**	**-885**	**0.87**	**39**

***Flex. Strength = $\alpha_6(\text{Modulus})^{\beta_6} + \gamma_6 \text{ Log (TTF)} + \delta_6$** ****SEE = Standard Error of Estimate**

Table G.10 – Summary of Laboratory-Developed Relationships between Compressive Strength and Combined Seismic Modulus and Maturity Parameters

Coarse Aggregate	Mix Designation	Fit Parameters*				R^2	SEE** (psi)
		α_7	β_7	γ_7	δ_7		
SRG	Standard 1 (Large Mixer)	1.397	0.833	630.2	-3008	1.00	49
	Low WCR	5.821	0.878	253.5	-7183	1.00	50
	High WCR	17.89	0.722	310.1	-6158	1.00	39
	Standard 2 (Small Mixer)	10.46	0.704	502.9	-5052	1.00	29
	More Cement	9.187	0.689	752.2	-5647	0.99	80
	Less Cement	0.854	0.906	538.8	-3867	1.00	46
	High CAF	1.398	0.932	679.4	-5637	0.99	121
	Low CAF	0.304	0.928	771.6	-4732	0.99	77
	Global	**8.256**	**0.398**	**59.2**	**-253**	**0.90**	**638**
Limestone	Standard	5.739	0.871	166.0	-6504	1.00	39
	Low WCR	4.710	0.867	1046.2	-11002	0.99	180
	High WCR	8.687	0.869	-228.1	-7402	1.00	8
	More Cement	3.630	0.858	734.3	-6895	1.00	92
	Less Cement	0.865	1.000	465.1	-3979	0.98	185
	High CAF	16.13	0.815	-251	-7505	0.99	107
	Low CAF	19.22	0.813	-372.7	-11585	0.99	154
	Less Water	3.681	0.925	244.8	-6859	0.98	192
	More Water	13.79	0.703	862.3	-7978	0.99	153
	High FM	1.455	1.012	355.8	-6388	0.99	161
	Low FM	1.418	0.985	484.4	-5804	0.99	158
	High PTA	21.74	0.733	296.7	-8877	1.00	78
	Low PTA	16.70	0.779	32.3	-7827	1.00	36
	High AEA	5.536	0.905	-73.9	-6412	1.00	77
	Low AEA	2.813	0.863	1074.0	-8191	0.97	294
	No AEA	6.289	0.916	-229.1	-8527	0.99	164
	Global	**1.172**	**1.051**	**205.2**	**-5798**	**0.94**	**420**
Granite	Standard	-4.678	0.739	1002.9	-1458	0.99	100
	Low WCR	27.155	0.609	1002.4	-8046	1.00	69
	High WCR	4.648	0.838	675.1	-7785	1.00	4
	More Cement	7.536	0.849	404.6	-10328	0.69	683
	Less Cement	4.216	0.861	693.7	-8507	0.99	86
	High CAF	-4.323	0.760	1401.5	-4474	0.97	211
	Low CAF	11.050	0.790	388.2	-8566	0.96	218
	FFA Standard	9.419	0.804	600.6	-9928	1.00	74
	Global	**1.817**	**0.683**	**50.4**	**-492**	**0.92**	**363**

***Comp. Strength = $\alpha_7(\text{Modulus})^{\beta7} + \gamma_7 \text{ Log (TTF)} + \delta_7$** ****SEE = Standard Error of Estimate**

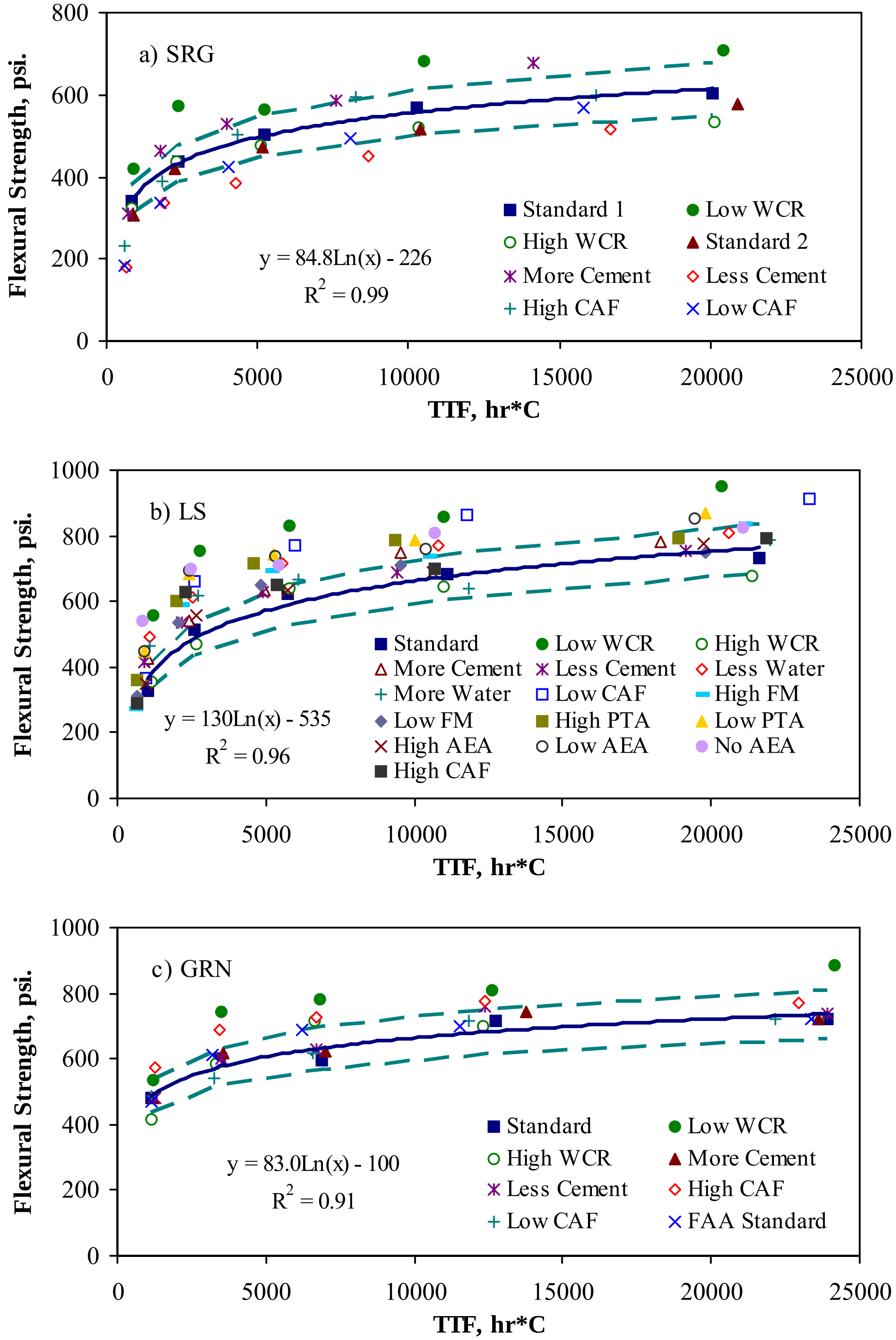


Figure G.1 - Impact of Mix-Related Parameters on Flexural Strength with Maturity

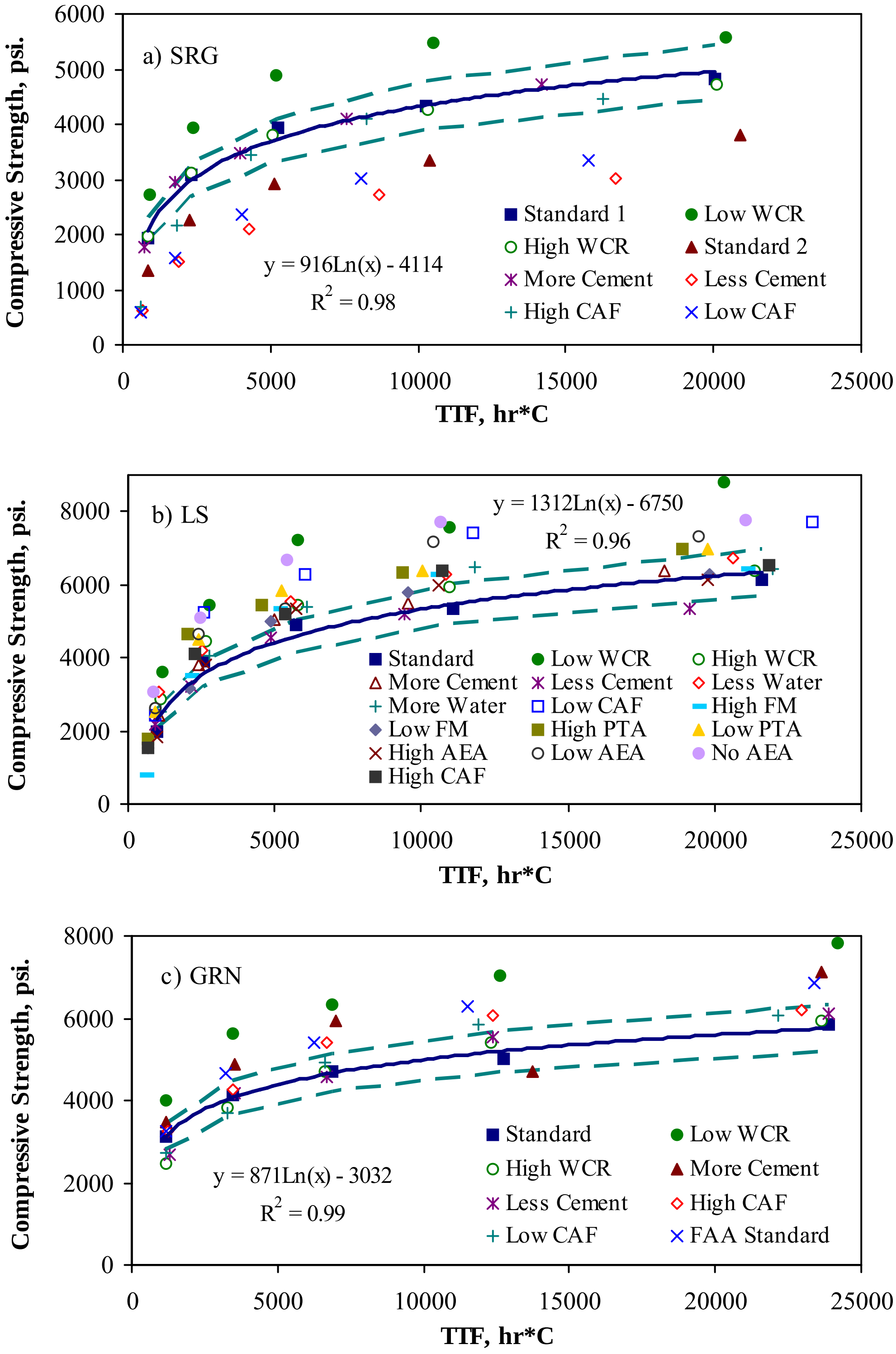


Figure G.2 - Impact of Mix-Related Parameters on Compressive Strength with Maturity

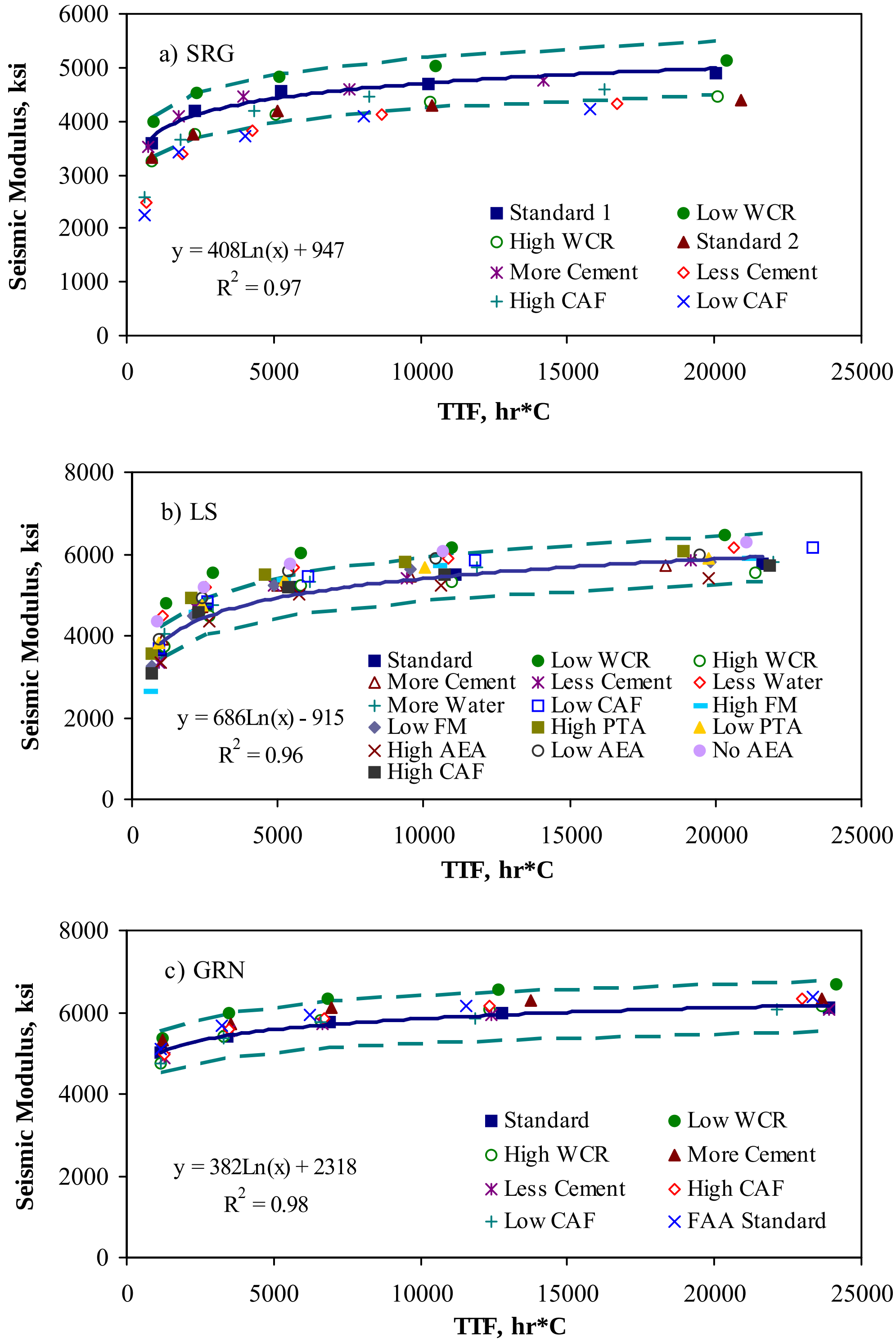


Figure G.3 - Impact of Mix-Related Parameters on Seismic Modulus with Maturity

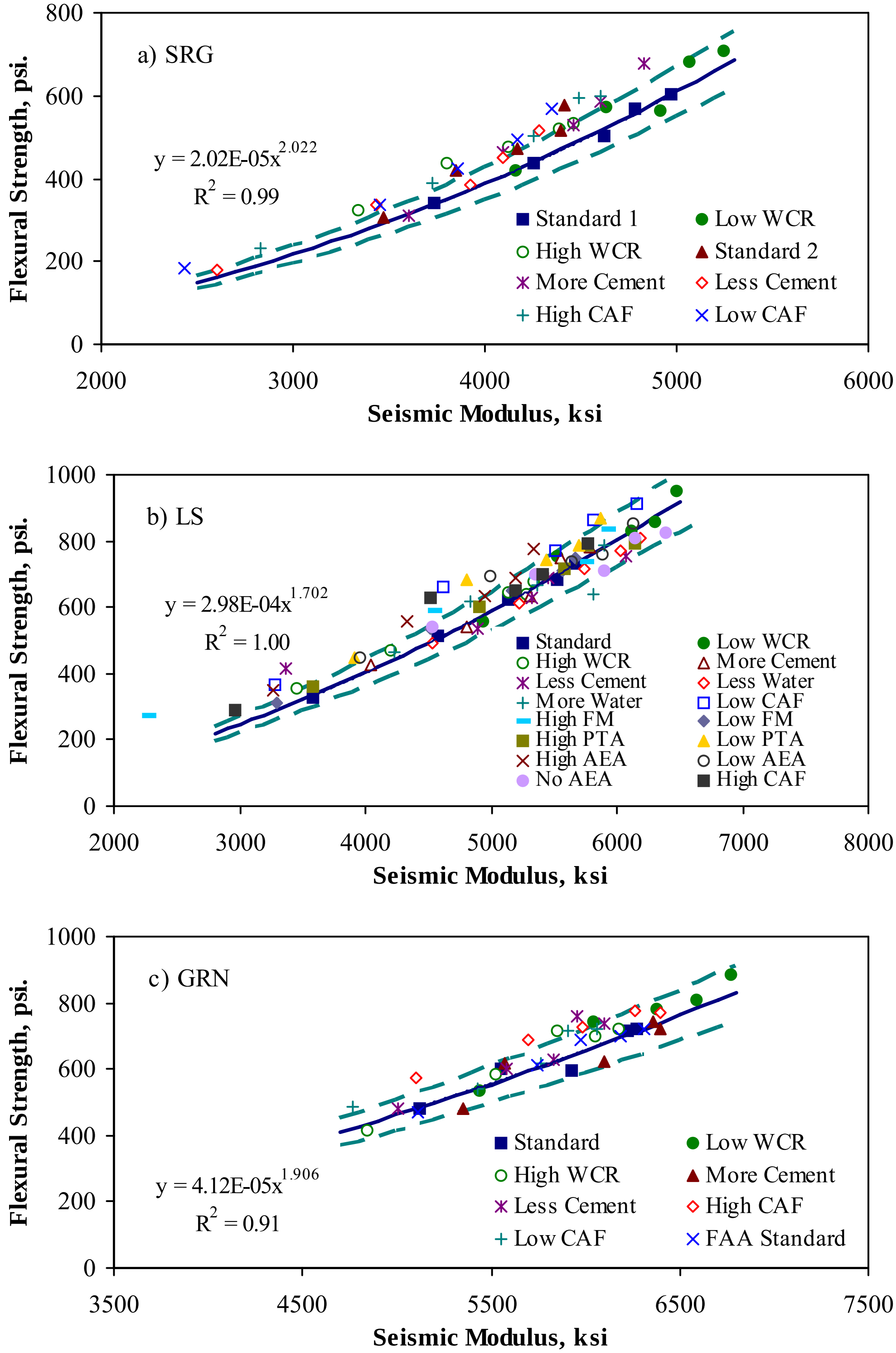


Figure G.4 - Impact of Mix-Related Parameters on Flexural Strength with Seismic Modulus

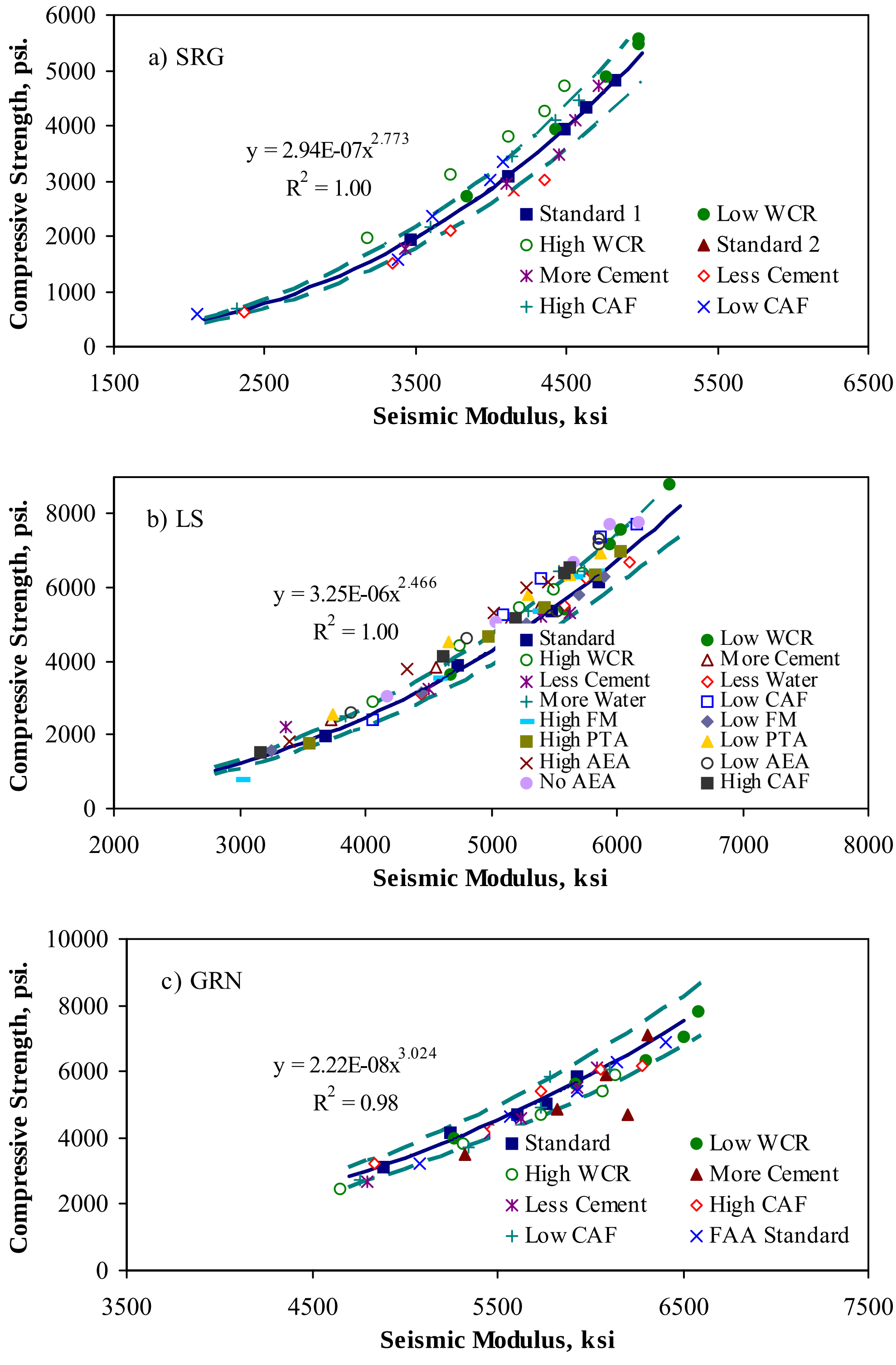


Figure G.5 - Impact of Mix-Related Parameters on Compressive Strength with Seismic Modulus

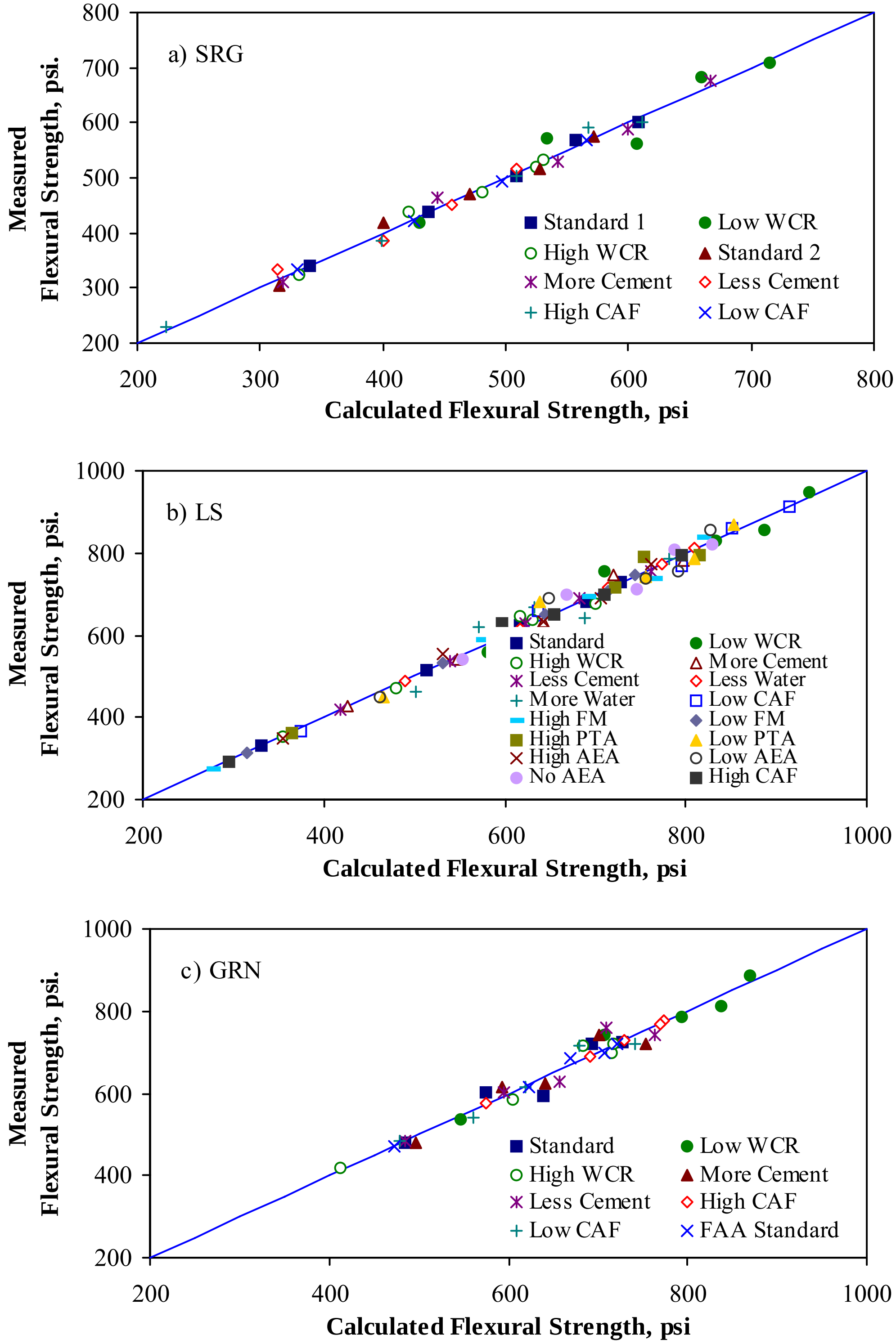


Figure G.6 - Impact of Mix-Related Parameters on Flexural Strength with Seismic Modulus and Maturity

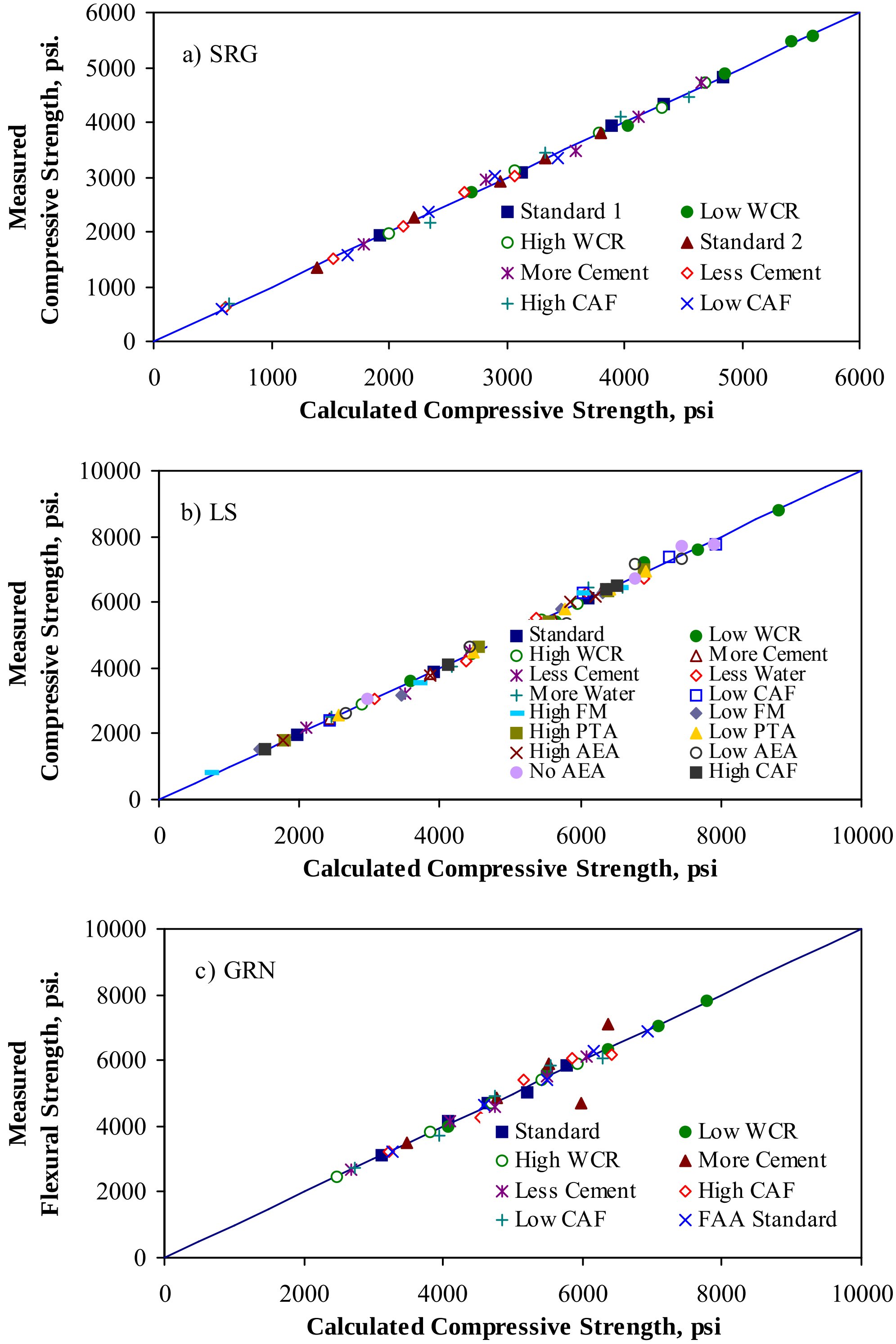


Figure G.7 - Impact of Mix-Related Parameters on Compressive Strength with Seismic Modulus and Maturity

For the mixes with GRN (figures G.1c and G.2c), some results are outside the error band, again suggesting that for maturity-based quality control, a reasonably rigid process control should be considered.

The relationships between seismic modulus and maturity are shown figure G.4. For mixes with all three types of coarse aggregates, most data points fall within the 10% error band. This indicates that as compared with the effect of errors in strength measurement on the laboratory-developed relationships, the effect of mix-related parameters is quite minor. Since the errors in both maturity and seismic (FFRC) modulus measurements are small, the accuracy and precision of strength tests play a critical role in developing any reliable relationships between strength parameters and maturity or seismic modulus.

Similarly, the variations in flexural strength and compressive strength with seismic modulus are presented in figures G.4 and G.5. These relationships are better defined independent of the variations in mix. Most strength parameters at a given seismic modulus fall within the 10% error band. Better relationships are anticipated for compressive strength as compared to flexural strength because of higher precision associated with the compressive strength test. From these figures, the seismic-based relationships are shown to be robust, require less stringent process control, and are perhaps adequate for quality acceptance.

The variations in flexural and compressive strengths with the combination of seismic modulus and TTF are shown in figures G.6 and G.7. Since it would be difficult to demonstrate the relationships in a three-dimensional format, the predicted values from the given relationships are compared to the measured values. The models provide similar results.

Test Results from Small Slabs

The results from tests on small slabs and the cores and beams retrieved from the slabs for all mixes with GRN and SRG coarse aggregates are summarized in tables G.11 and G.12. These results contain the effects of either construction-related or environmental-related or both. For SRG coarse aggregates, Standard-1 mix was used for all small slabs.

The average measurement errors in terms of COV are 6.9%, 4.8% and 1.7% for flexural strength, compressive strength and seismic (FFRC) modulus, respectively. These errors are higher than those for laboratory-cured specimens. For direct modulus measurements with the PSPA on small slabs, the average COV is about 6%.

Impact of Grooving on In-Situ Seismic Modulus Measurements

Grooved concrete slabs constitute a large proportion of rigid pavement construction, especially for runways. The major question to be answered was the reliability of the interpretations of stress waves for grooved concrete pavements. If reliable interpretations can be made, the secondary issue is the sensitivity and accuracy of in-situ measurement with respect to the presence, size and relative orientation of grooves to the placement of a measurement apparatus.

Even though grooving may not affect the strength of concrete, it may impact the in situ seismic measurements. The project team relied heavily on the results from Geomedia (2001) conducted to study of this matter. In that study, tests were conducted in five sections of a large concrete slab (30 ft by 40 ft) just before and after grooving with the PSPA placed parallel to the grooves (termed the parallel position) and with the perpendicular to the grooves (termed the transverse position). The grooving patterns investigated are summarized in table G.13.

Table G.11 – Summary of Test Results from Cores, Sawed Beams and Small Slabs with Granite Coarse Aggregates

Mix	Age	TTF	Cores				Sawed Beams				Slab	
			FRRC Modulus		Compressive Strength		FRRC Modulus		Flexural Strength		PSPA Modulus	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)
Standard	1	1174									4319	
	3	3443	5418	1.4	4107	3.7	5716	0.6	693	3.8	4504	
	7	6897	5365	1.3	4391	1.9	5772	3.3	645	8.5	5772	
	14	12757	5740	2.1	4388	8.8	5835	1.9	632	2.7	5754	
	28	23916	5885	1.2	5189	5.4	5902	0.2	692	1.7	5743	
Low WCR	1	1201									5355	5.6
	3	3489	5896	1.8	5615	1.9	6245	1.0	691	7.1	6832	1.4
	7	6849	6004	0.9	6011	3.6	6337	0.2	703	10.9	5899	4.0
	14	12631	6100	1.7	6717	3.7	6376	0.4	684	3.7	6149	5.9
	28	24174	6196	0.4	6855	1.3	6381	1.8	777	9.8	6432	7.6
High WCR	1	1163									4764	3.6
	3	3310	5323	1.4	3075	0.7	5570	1.3	570	4.2	5821	0.4
	7	6614	5520	0.4	3304	20.1	5679	1.7	623	3.8	5590	5.6
	14	12324	5630	1.5	4055	4.0	5814	1.9	707	2.8	6106	5.7
	28	23632	5757	1.1	4428	2.7	5747	2.1	735	21.8	5546	0.7
More Cement	1	1200									4946	2.6
	3	3538	5855	1.3	5041	8.4	5979	0.6	592	7.1	5319	5.9
	7	6994	5855	1.4	5426	1.7	6033	1.5	602	2.9	5667	0.3
	14	13748	5783	1.3	4926	6.3	6118	1.6	627	3.6	6174	0.7
	28	23640	5814	2.7	6218	1.6	6171	1.8	673	6.6	6334	0.8
Less Cement	1	1273									5304	
	3	3508	5033	2.2	3247	12.0	5647	4.5	547	1.7	4989	
	7	6688	5238	2.1	3700	1.8	5493	1.0	612	7.5	5326	
	14	12396	5366	3.4	4153	9.5	5713	0.4	633	8.6	5159	
	28	23908	5512	1.9	4612	6.4	5689	1.4	707	9.8	5296	
High CAF	1	1254									4981	5.5
	3	3454	5691	2.7	3922	7.8	5844	0.7	635	9.8	5242	7.1
	7	6701	5793	1.0	4512	4.5	6014	1.0	684	5.5	5723	6.4
	14	12352	6074	1.4	5491	2.5	5995	0.4	708	1.8	6359	1.3
	28	22976	6195	1.2	6218	4.3	6181	0.6	742	3.0	6486	6.6
Low CAF	1	1153									4568	1.4
	3	3291	5840	2.2	3376	2.2	5455	3.6	590	10.4	5345	1.1
	7	6592	5394	1.2	3824	4.0	5577	0.3	603	2.2	6092	2.1
	14	11858	5711	1.8	4483	0.5	5686	0.6	608	3.2	5500	4.7
	28	22140	5710	0.6	4890	3.9	5717	1.5	625	18.0	6549	
Blanketed (Standard)	1	1115									5448	1.5
	3	3100	5405	1.1	3730	1.1	5781	1.0	619	3.1	6014	1.2
	7	6318	5549	1.3	3970	15.0	5879	1.2	627	7.9	6538	9.1
	14	11512	5913	1.5	4790	7.0	5923	1.7	645	4.2	6345	12.1
	28		5944	2.1	4797	4.9	6067	5.2	648	8.3	6592	

Table G.11 – Summary of Test Results from Cores, Sawed Beams and Small Slabs with Granite Coarse Aggregates (Con't)

Mix	Age	TTF	Cores				Sawed Beams				Slab	
			FRRC Modulus		Compressive Strength		FRRC Modulus		Flexural Strength		PSPA Modulus	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)
Untreated (Standard)	1	965									4505	3.7
	3	2685										
	7	6118	5516	1.5	4538	2.4	5938	8.1	536	3.5	5837	6.0
	14	11977	5820	2.4	4504	2.2	5951	0.6	580	3.4	5875	10.4
	28	23124	5910	3.2	5075	7.6	6063	3.6	663	1.9	6565	1.8
FAA Standard	1	1154									5639	2.2
	3	3223	5344	2.7	3560	0.8	5678	0.9	627	8.4	5473	9.4
	7	6243	5593	1.9	3790	4.8	5788	0.8	627	3.2	5171	8.3
	14	11536	5745	2.7	4519	7.4	5813	2.9	585	4.9	5139	9.1
	28	23362	5935	1.3	5260	7.7	6011	2.7	640	12.2	5551	10.7

Table G.12 – Summary of Test Results from Cores, Sawed Beams and Small Slabs with Siliceous River Gravel Coarse Aggregates

Mix	Age	TTF	Cores				Sawed Beams				Slab	
			FRRC Modulus		Compressive Strength		FRRC Modulus		Flexural Strength		PSPA Modulus	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)
Standard	1	1472									3660	10.2
	3	3706	3846	0.6	2903	8.9	4078	0.5	465	16.5	3906	5.7
	7	7621	3998	0.1	3230	5.3	4111	1.2	496	2.7	4090	4.8
	14	14909									4044	9.8
	28	29649	4263	0.9	3711	3.9	4062	2.1	452	0.7	3709	4.9
Low WCR	1	1431									4015	10.1
	3	3591	4431	1.3	3542	0.8	4584	1.4	565	7.9	4683	6.4
	7	7550	4391	1.6	3844	2.6	4762	1.5	620	11.9	4696	6.1
	14	14937									4573	9.3
	28	29798	4644	1.8	4815	5.8	4679	0.4	588	5.6	4519	7.3
High WCR	1	1443									3407	6.8
	3	3636	3481	0.8	2417	10.0	3733	2.2	466	3.8	3532	5.8
	7	7522	3667	0.8	2684	2.4	3869	1.0	504	4.8	3563	8.9
	14	14761									3574	6.4
	28	29391	3888	0.1	3190	0.0	3807	0.7	444	1.4	3621	7.9
18 in. Thick	1	1523									3309	6.7
	3	4084	3736	1.6	3130	5.6	4031	0.6	466	0.0	3576	8.1
	7	8622	3942	1.0	3533	2.9	4187	2.7	484	4.3	3545	6.5
	14	15568									4017	4.4
	28	29835	4042	1.1	4075	2.6	4222	0.5	534	1.1	3675	5.6
6 in. Thick	1	1355									3558	11.2
	3	3504					3991	0.0	516	21.1	3852	8.3
	7	8184					4185	0.7	446	5.7	3918	8.3
	14	15419									3785	11.3
	28	30353					4190	0.7	449	7.9	3956	5.3
Over Compacted	1	1326									3183	7.7
	3	3631	4075	1.2	2295	3.7	4241	1.4	464	1.7	3326	7.8
	7	7841	4224	3.1	2690	2.3	4347	1.2	448	6.5	3131	7.8
	14	14246									3704	7.3
	28	27841	4472	0.1	3122	1.7	4494	2.3	476	13.0	3778	9.6
Untreated	1	1029									3508	8.5
	3	2776	3996	0.1	1911	9.5	4283	2.9	445	14.7	4239	7.9
	7	5825	4251	2.9	2678	0.9	4431	0.7	424	9.0	4256	10.3
	14	11098									4390	8.9
	28	19987	4482	1.5	3454	5.1	4537	4.7	442	4.3	4488	7.7
Blanketed	1	981									3692	6.4
	3	2489	4131	0.9	2087	4.9	4345	2.9	465	17.8	4294	4.8
	7	5383	4437	1.9	3316	4.1	4552	0.7	470	9.0	4435	7.2
	14	10590									4642	6.8
	28	19586	4613	5.8	4062	3.9	4738	1.0	485	4.0	4769	7.3

Table G.12 – Summary of Test Results from Cores, Sawed Beams and Small Slabs with Siliceous River Gravel Coarse Aggregates (Con't)

Mix	Age	TTF	Cores				Sawed Beams				Slab	
			FRRC Modulus		Compressive Strength		FRRC Modulus		Flexural Strength		PSPA Modulus	
	day	hr*C	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)	Mean (psi)	C.V. (%)	Mean (ksi)	C.V. (%)
Low Humidity	1	989									3769	11.3
	3	2631	4455	2.4	2843	7.3	4420	7.0	502	10.3	3940	11.5
	7	5537	4533	0.7	3577	3.0	4622	2.4	435	21.5	4359	10.4
	14	10484									4757	4.8
	28	19900	4780	2.8	3903	12.5	4687	3.5	465	17.2	4924	8.4
High Humidity	1	1026									4218	10.4
	3	2818	4405	1.9	3373	4.1	4525	0.1	460	1.5	4430	4.9
	7	5829	4509	0.2	3881	1.3	4765	1.0	532	0.3	4528	4.9
	14	10947									4986	8.1
	28	20261	4737	2.5	4067	18.5	5092	0.5	659	1.6	5169	9.2
Hot Curing	1	1096									3897	6.2
	3	3087	4546	1.9	3300	3.3	4462	4.9	453	4.8	4156	6.8
	7	6665	4614	0.8	3744	4.2	4551	1.9	518	11.7	4222	3.9
	14	12710									4440	7.6
	28	24922	4777	3.6	3690	0.4	4605	3.7	430	2.6	4633	7.5
Cold Curing	1	965									3775	6.6
	3	2477	4402	0.7	3094	2.7	4408	3.4	407	0.0	4197	10.6
	7	5054	4518	2.6	3600	0.6	4609	7.1	419	5.4	4484	9.0
	14	9383									5076	1.0
	28	17776	4811	1.4	3854	2.8	4756	3.4	487	17.4	4902	8.1

Table G.13 - Grooving Patterns Used in Geomedia Study

Section	Grooving Pattern, inch		
	Spacing	Width	Depth
1 (Standard)	1.5	0.25	0.25
2 (Deeper Grooves)	1.5	0.25	0.50
3 (Smaller Groove Spacing)	1.0	0.25	0.25
4 (Larger Groove Spacing)	2.0	0.25	0.25
5 (Narrower Groove)	1.5	0.125	0.25

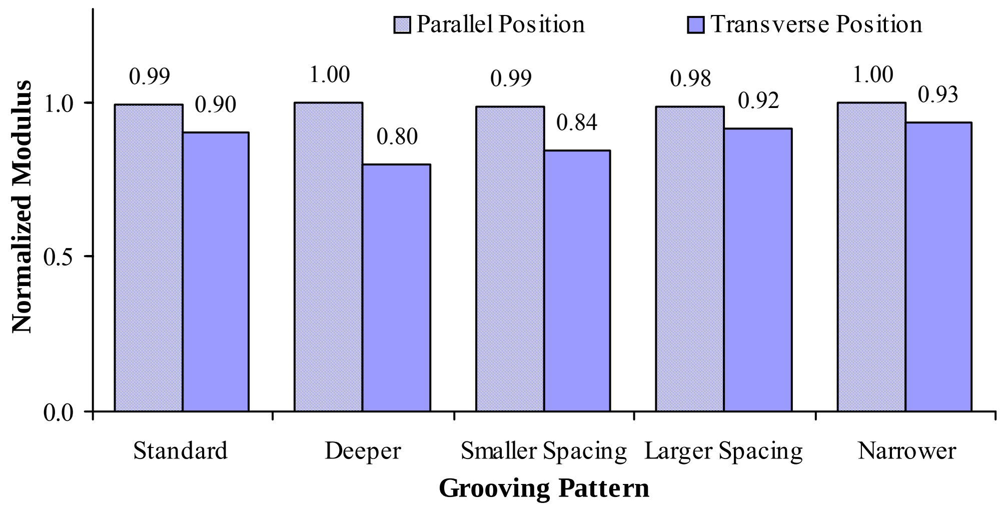
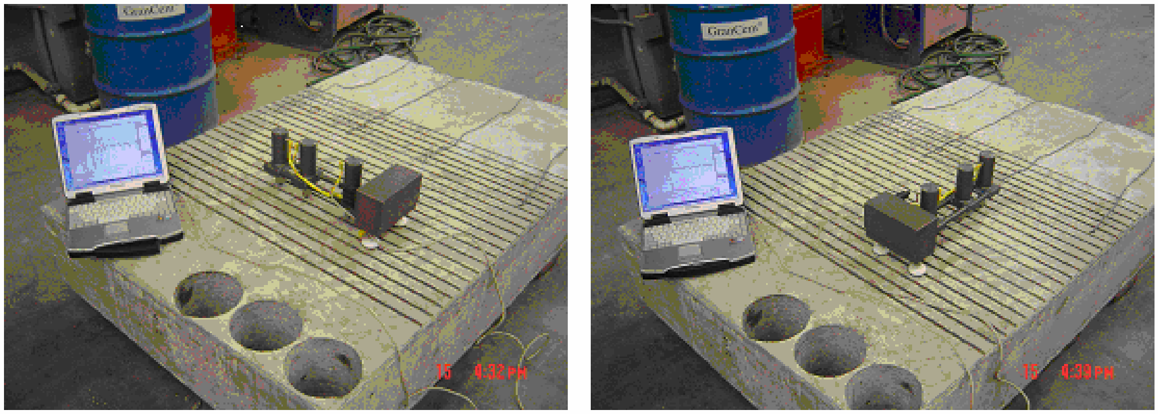


Figure G.8 - Effects of Grooving Pattern on Measured Seismic Moduli

The ratio of moduli measured before and after grooving for each section (termed normalized modulus) is shown in figure G.8. The results demonstrate a dependence on placement position of the source-sensor array relative to the grooves. Moduli measured with parallel position were quite comparable to those measured before grooving. On the other hand, moduli measured with the transverse position show about 6% to 20% decrease depending on the grooving pattern.

Under this project, ERDC provided an independent validation of these results using a small slab. The standard grooving as specified in FAA Specification P501 is used. As shown in figure G.9, the grooved slab was tested in parallel and transverse directions with a PSPA. The results from this study are shown in figure G.10. Measurements in the two directions yield similar results, with a difference of less than 5%. Even though the grooves did not substantially impact the measurements, it still seems a good practice to place the PSPA parallel to the grooves.



a) Parallel Placement   b) Transverse Placement

Figure G.9 - Two Positions of PSPA Sensor Unit Placement

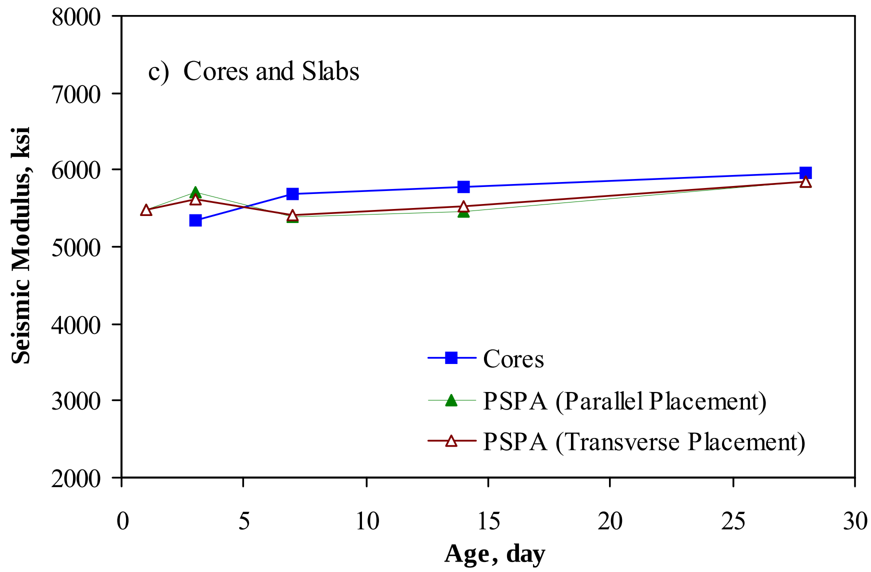


Figure G.10 - Impact of PSPA Placement on Modulus Measurement

References

Geomedia Research and Development (2001), "Determining Capacity of Military Pavements with a Portable Sonic-Ultrasonic Stress-Wave Testing Device," SBIR Phase I Report, Submitted to the Department of Defense, Washington, DC.

Appendix H

Analysis of Results for Pavement Thickness Estimation

The fundamentals of the impact-echo (IE) method as applied to the thickness measurement of a concrete pavement are rather simple. However, the factors that affect the result of the measurement are somewhat complicated. The accuracy and precision of the method performed with a PSPA as well as the effects of some construction-related factors (such as the stiffness of base material, distance from pavement edge, and surface grooving) were studied as part of this project.

Accuracy

The accuracy or uncertainty in thickness estimate with the impact-echo method depends on a pooled error of the return frequency measurement and the P-wave velocity determination.

The frequency resolution of the measurement device is one of the sources of uncertainty in return frequency measurement. A Fourier analysis is carried out to measure the return frequency. To carry out this process, the signals have to be digitized, that is the analog signal has to be approximate by discrete points. The rate at which the digitization is carried out contributes to the uncertainty in the measurements. The theoretical uncertainty as a function of digitization rate and thickness of the pavement is shown in figure H.1. As the thickness increases, the theoretical uncertainty in measured thickness increases. This occurs because as the thickness increases, the return frequency decreases. For example, typical return frequencies for a 10-inch thick pavement are between 7000 Hz to 8000 Hz and for a 20 in. pavement between 3500 Hz to 4000 Hz. As such, the theoretical uncertainty in measuring thickness due to signal processing alone is twice for a 20-in.-thick pavement as compared to a 10-in.-thick pavement. For a concrete of average modulus and a frequency resolution of about 100 Hz as adopted in the PSPA, theoretically, the relative uncertainty in thickness measurement may vary from 1% to 2%.

The determination of P-wave velocity for the concrete in a pavement is another critical factor affecting the uncertainty in thickness estimate. The P-wave velocity can be determined either from the USW tests on the pavement with an assumed Poisson's ratio or from the FFRC tests on the field-cured cylinders or the cores from the pavement. The theoretical uncertainty of the measurements of the P-wave velocity from the USW method is a function of the receiver spacing and the digitization rate of the data is shown in figure H.2. For the PSPA with a sampling rate of about 400 kHz and receiver spacing of 6 in., the uncertainty is about 2.5%. For the FFRC tests, even though not shown here, the uncertainty is about 1%. If the uncertainties in the measurements of the return frequency, the P-wave velocity and the experimental errors due to quality of contact and placement of the device are pooled, the uncertainty in the thickness measurement is about 4% to 5%.

The construction process of a long slab used in this study is shown in figure H.3. Longitudinally, half of the slab was placed on a cement-treated base and the other half on a compacted soil. To minimize the cost of coring, a detailed depth survey was conducted just before the concrete mix was poured and after the completion of the slab on a 6-in. grid. As an example, the survey result from the sloping section of the long slab is shown in figure H.4. Certain uneven changes in the depth were the result of the difficulty of compaction on the cement-treated base material at the trench bottom. The thicknesses obtained from this exercise were verified with about six cores.

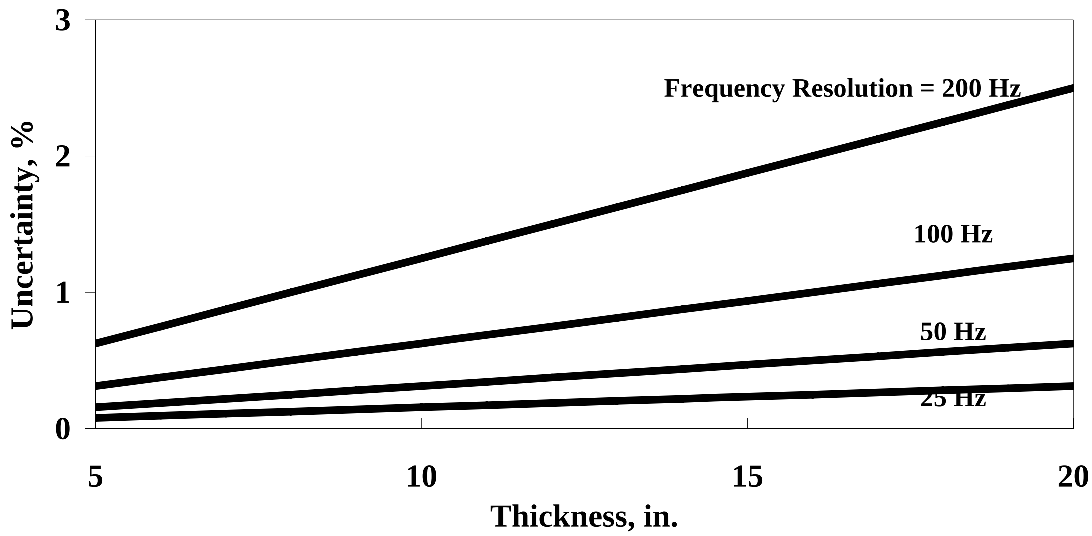


Figure H.1 - Theoretical Uncertainty in Measuring Return Frequency for Impact Echo Method

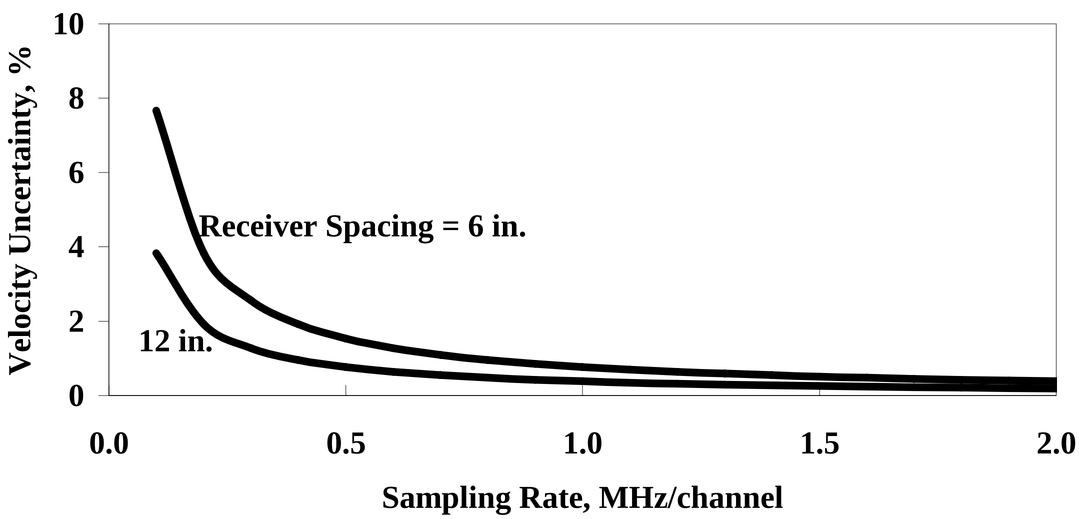


Figure H.2 - Theoretical Uncertainty in Measuring P-Wave Velocity for Impact Echo Method

a) Frame Built in a Trench for Long Slab **b) Finishing Long Slab**

Figure H.3 - Construction of the Long Slab

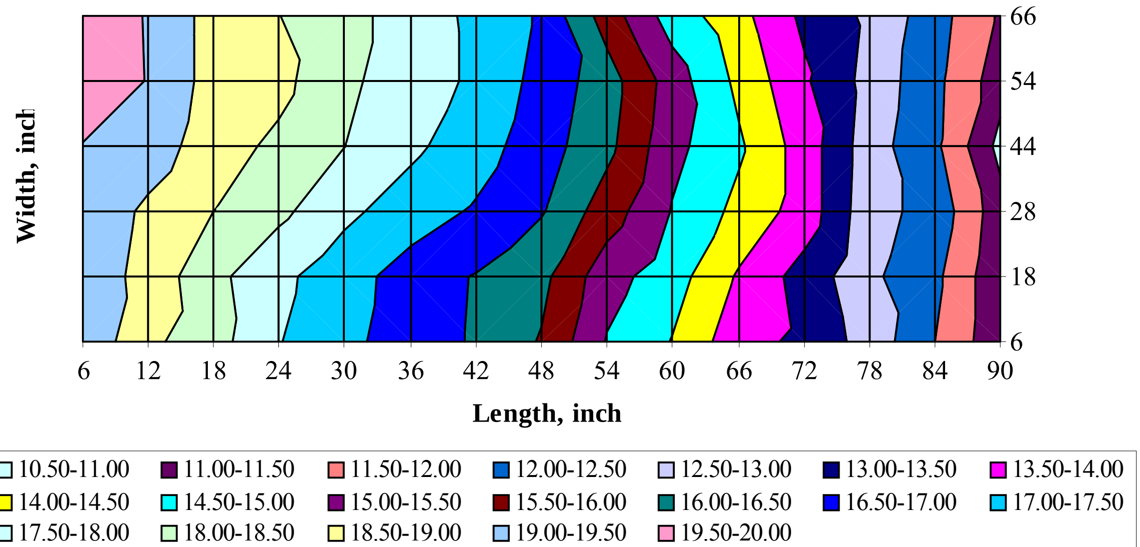


Figure H.4 - Depth Contour Map in Sloping Section for Long Slab

Thickness measurements with a PSPA were performed in the central area of each stair section and at an interval of 12 in. along the sloping section. As an example, the impact-echo spectra and thickness estimates for the slab in the sloping section are shown in figure H.5. The thicknesses are underestimated. This trend has been reported in the literature (e.g. Maser et al., 2003).

The measured and actual thicknesses along the slab are summarized in table H.1. The thicknesses estimated by the IE method differ from the actual thicknesses with a maximum of about 4% and an average of about 2.5% for all six step sections. The differences between the measured and actual thicknesses for the sloping section are as high as 8%. The bottom of the slab slopes

Table H.1 – Summary of Thickness Measurements on the Long Slab with IE Method

Section	Subsection or Location	IE Peak Frequency, kHz	Thickness (inch)		Difference	
			Actual	Measured	Value (inch)	Percentage (%)
Stair Section	1	7.8	9.9	9.6	-0.3	3.4
	2	6.0	12.8	12.5	-0.3	2.4
	3	5.3	14.8	14.2	-0.6	3.9
	4	4.9	15.8	15.3	-0.4	2.8
	5	4.2	18.0	17.8	-0.2	1.3
	6	3.9	19.5	19.4	-0.1	0.7
Sloping Section	1	4.1	19.1	18.3	-0.8	4.4
	2	4.4	18.3	17.0	-1.3	7.4
	3	4.6	17.4	16.1	-1.3	7.9
	4	4.8	16.7	15.5	-1.2	7.5
	5	5.1	15.4	14.8	-0.6	4.2
	6	5.7	14.1	13.2	-0.9	6.5
	7	6.4	12.3	11.6	-0.7	5.8
	8	7.1	10.6	10.5	-0.1	0.5

by about 10% (10 in. change in thickness over 96 in. of length). This means that for a 4 in. core, the height will vary by about 0.4 in. from one edge to the other. Since in practical application such an abrupt change in thickness is very rare, more advanced analysis of the results was not carried out.

Precision

The precision of the thickness estimate also reflects a pooled error of the return frequency measurement and the P-wave velocity determination. The precision of the IE return frequency measurements was first studied by repeating a test at one point on a slab several times without moving the PSPA sensor unit. The precision was about 1%.

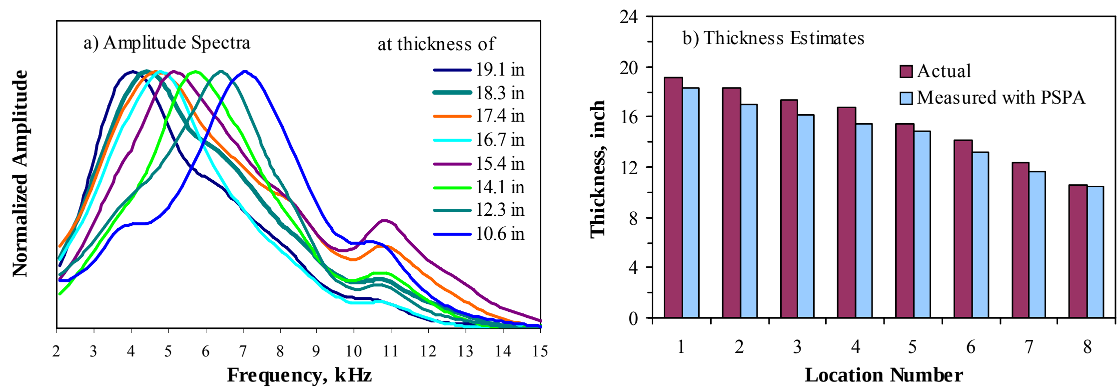


Figure H.5 - Results from Impact-Echo Tests on Sloping Section of Long Slab

The precision was about 2% to 5% when the IE tests were performed at several points on a slab. Such variation can be attributed to the superposition of P-wave and surface wave components contained in the waveform, and the coupling between the impact source and the slab surface, and the non-uniformity in the properties of concrete, especially in the early ages.

Effect of Base Stiffness

The contrast in mechanical impedance (product of density and wave velocity) between the concrete pavement and the underlying materials can impact the accurate determination of the thickness of the pavement with the IE method. Three types of base layers were used for pavement thickness study. They are compacted soil, cement-treated base (CTB), and asphalt concrete base (ACB).

The IE tests were performed on the two sides of four stair sections of the long slab. The average thickness for each section is shown in figure H.6a. The differences in the thickness estimates were within the measurement error. This means that this effect can be ignored for these two types of base materials.

Two small slabs were poured on a cement-treated base course (CTB) and an asphalt layer (ACB). As shown in figure H.6b. The measured thicknesses are typically greater than the actual ones for the slab placed on the ACB. The impedance contrast between the concrete slab and the ACB is not large enough to be distinguished with the IE measurement. As a result, the return frequency represents a response to the composite thickness of the concrete slab and the thin ACB layer. However, again the slab placed on CTB provides accurate thickness.

Effect of Slab Edges

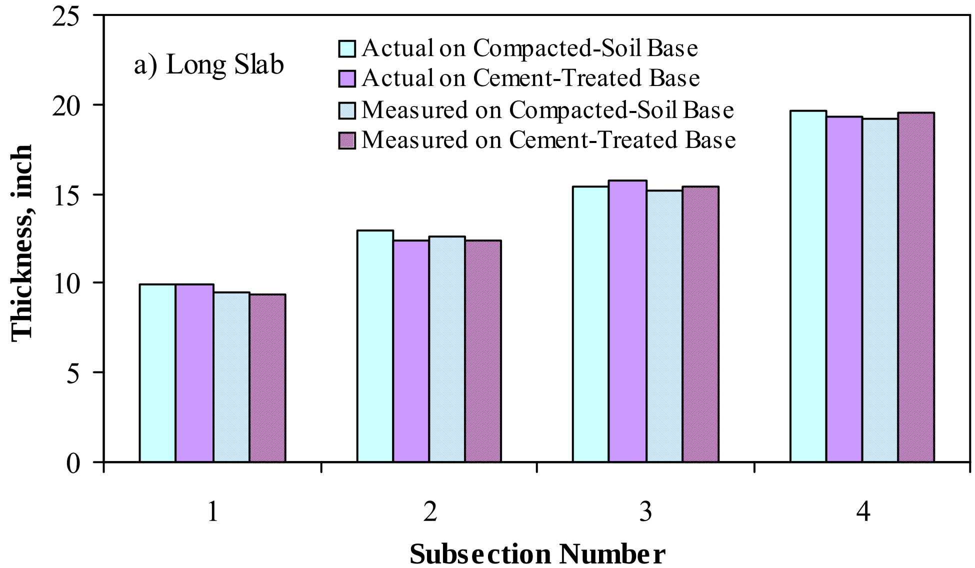
One practical item to be addressed is how close an IE test should be performed relative to the edge of a slab to get the return frequency for the slab thickness. Figure H.7 shows a typical amplitude spectrum measured at a distance of about 6 in. from one edge of a 12 in. thick slab. Two peaks are apparent: one related to the thickness return frequency and another to the reflection from the edge. When the measurements are taken at points close to the edge, the second peak may confuse the analysis algorithm. For a better result, the IE measurements should be conducted at a distance at least two times the thickness of the slab from any edge to minimize the edge effect.

Effect of Grooving

Surface grooving is a common practice for concrete airfield runway pavements. Its effect on slab thickness measurement with the impact-echo method was studied through a small slab. The thickness return frequencies measured parallel and perpendicular to the grooves (see figure G.9) were compared for each testing age. The maximum difference in average return frequency was about 2.5% and was independent on the placement orientation. Given the uncertainty in return frequency measurements, such a difference is considered insignificant.

References

Maser, K. R, T. J. Holland, R. Roberts, J. Popovics, and A. Heinz (2003), "Technology for Quality Assurance of New Pavement Thickness." Proceedings, 82nd Annual Meeting of the Transportation Research Board, Washington, DC.



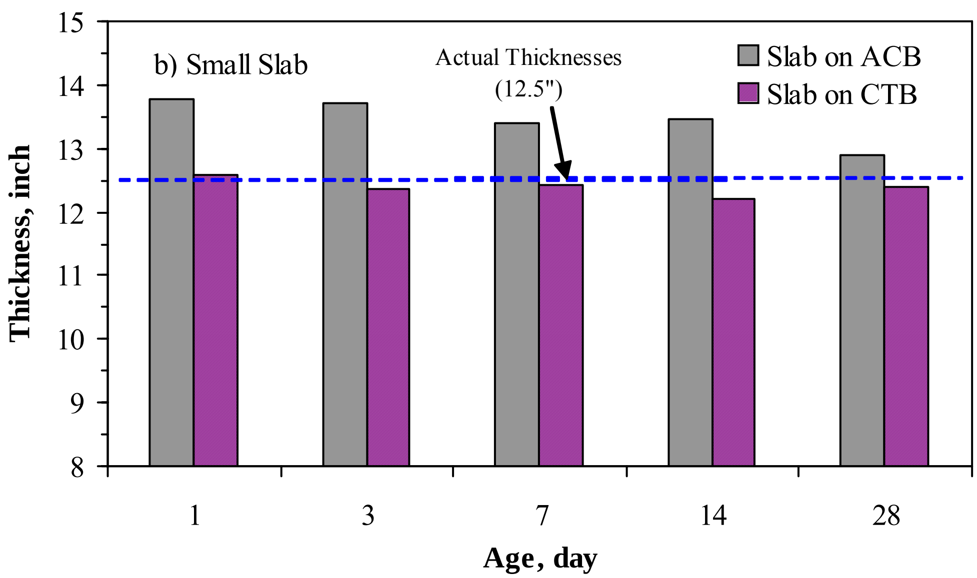


Figure H.6 - Impact of Base Material Stiffness on Measured Thickness with Impact-Echo Method

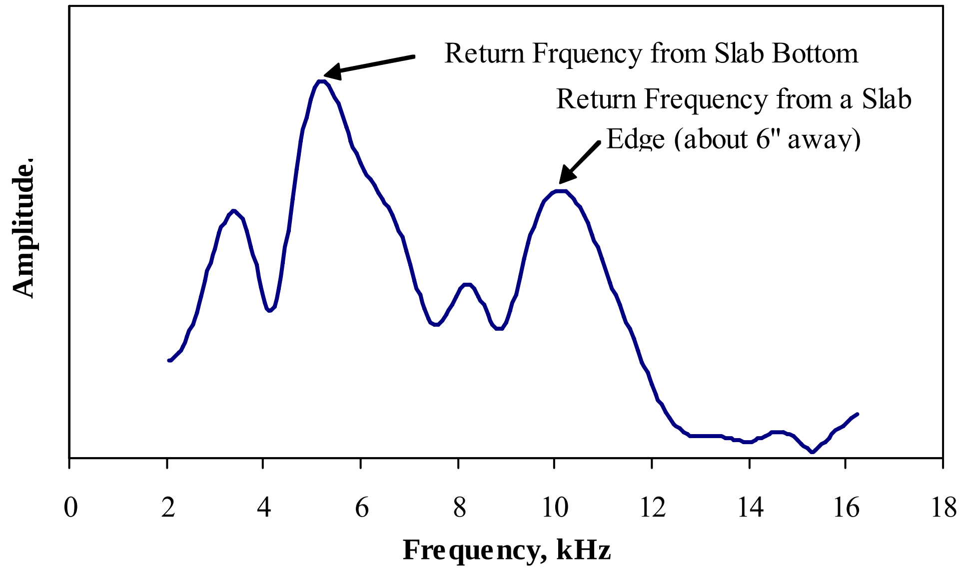


Figure H.7 - Effect of Edge Reflection on Measurements with Impact-Echo Method