EVALUATION OF HISTORIC REINFORCED CONCRETE BRIDGES

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ABSTRACT: The procedures used to evaluate seven historic reinforced concrete bridges in different regions of the United States are discussed. Constructed between 1907 and 1938, three of the bridges have soil infill above arched vaults, three are open spandrel arch structures, and the seventh is a continuous span structure in the states of Nevada, Indiana, Rhode Island, Washington and Pennsylvania. A combination of visual inspection, nondestructive methods and coring followed by laboratory testing was used in all cases. Advanced NDT methods were used to maximize information obtained, while minimizing invasive investigation and disruption to bridge operations. Laboratory testing addressed such topics as the extent of deterioration and chloride concentration in the concrete, as well as identification of aggregates and cement constituents, so repair concrete could be developed to match existing. These cost-effective programs provided vital structural and material information necessary for successful rehabilitation.

INTRODUCTION

Over recent years there has been a marked increase in the demand for the evaluation of reinforced concrete bridges built in the first forty years of the 20th century. This demand is principally a result of:
- An increased interest in historic preservation
- Safety issues such as approach alignments and lane width
- A change in the use or an upgrading of the structure often accompanied by an increase in load rating.

Nearly all these structures have unknown as built conditions including both the concrete and steel reinforcement properties. Air-entrained concrete was not available during this construction era. Service and maintenance records are generally very sparse or not available, particularly from the early years of the structure's existence. As an example, information about the use of chlorides as de-icing salts is very limited.

Rehabilitation or upgrading of these structures can be less economical than demolition and replacement with new bridges. In addition the uncertainties inherent in rehabilitation result in expensive problems being discovered in the middle of the repair contract that could not be identified during the evaluation phase. Consequently, preservation of the bridge as a historical structure is often the driving force behind the owner's decision to restore. Any evaluation program must therefore address the issues of preservation of the historic design and matching the repair materials with existing concrete and finishes, as well as reaching an economical repair solution.

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It is useful when considering rehabilitation of these bridges to divide the structure into three principal units:
- Main supporting structure (columns, piers, arches and abutments)
- Bridge deck
- Ancillary features such as parapets and services.

Very often it is economically advantageous to preserve the supporting structure while reconstructing the deck and parapets entirely.

EVALUATION APPROACH

The authors participated in the evaluation of seven historic reinforced concrete bridges in different regions of the United States over the last four years. Constructed between 1907 and 1938, three have soil infill above arched vaults, three are open spandrel arch structures, and the seventh is a continuous span structure. A project approach has been developed to maximize the information obtained during the evaluation for a minimal cost outlay. This approach comprised the following stages:
- Document review (when available)
- Structural calculations
- Field investigation
- Laboratory examination and testing
- Analysis of data and report preparation.

At first sight this appears to be a classical list of operations; however in these instances emphasis was placed upon the application of advanced nondestructive testing (NDT) in the field investigation phase, together with petrographic and chemical analysis of samples in the laboratory.

Field investigations include a visual review of the structure with development of surface feature/crack maps and the selection and removal of samples, usually by coring. NDT was used initially to identify areas with potential problems, to characterize the structure's material properties and to provide a database for future maintenance programs. The principal methods used as tools in this evaluation process were:
- Impulse Response (IR)
- Impact-Echo (I-E)
- Ultrasonic Pulse Velocity (UPV)
- Impulse Radar
- Covermeter
- Corrosion testing (half-cell potential and rate of corrosion).

All these methods are fully described in ACI Report 228.2R (1998). They were integrated to varying degrees, depending on the type and configuration of the structure under evaluation.

Many engineers have limited experience, if any, with most of these NDT techniques. In particular, the authors have extended the application of the Impulse Response test to bridge evaluation as a means of qualitatively assessing the structure's general behavior and to rapidly identify localized zones of poor quality concrete. This test has also been successfully used in the evaluation of deep foundations (Davis 1995) and large plate structures such as concrete pavements (Davis, Hertlein, Lim & Michols 1996) and tall chimney stacks (Davis & Hertlein 1995).

RECENT NDT STRESS WAVE METHODS

The Impulse Response test method is a stress wave test used extensively in the evaluation of machined metallic components in the aircraft industry. Its application to concrete structures in Civil Engineering is less well known, and the method has received far less publicity than the recently developed Impact-Echo test (Sansalone & Streett, 1997). The IR method is a direct descendant of the Forced Vibration method for evaluating the integrity of concrete drilled shafts, developed in France in the 1960's (Davis & Dunn, 1974). The basic theory of dynamic mobility developed at that time has not changed; however, its range of applications to different structural elements has increased to incorporate:

- voiding beneath concrete highway, spillway and floor slabs (Davis & Hertlein, 1987),
- delamination of concrete around steel reinforcement in slabs, walls and large structures such as dams, chimney stacks and silos (Davis & Hertlein, 1995),
- low density concrete (honeycombing) and cracking in concrete elements (Davis & Hertlein, 1995),
- debonding of asphalt and concrete overlays to concrete substrates (Davis *et al*, 1996).

The IR method uses a low strain impact to send a stress wave through the tested element. The impactor is usually a 1-kg sledgehammer with a built-in load cell in the hammerhead. The maximum compressive stress at the impact point in concrete is directly related to the elastic properties of the hammer tip. Typical stress levels range from 5 MPa for hard rubber tips to more than 50 MPa for aluminum tips. Response to the input stress is normally measured using a velocity transducer (geophone). This receiver is preferred to accelerometers because of its stability at low frequencies and its robust performance in practice. Both the hammer and the geophone are linked to a portable field computer for data acquisition and storage.

When testing plate-like structures, the Impact-Echo method uses the reflected stress wave from the base of the concrete element or from some anomaly within that element (requiring a frequency range normally between 10 and 50 kHz). The IR test uses a compressive stress impact approximately 100 times that of the I-E test. This greater stress input means that the plate responds to the IR hammer impact in a bending mode over a very much lower frequency range (0-1 kHz for plate structures), as opposed to the reflective mode of the I-E test.

Both the time records for the hammer force and the geophone velocity response are processed in the field computer using the Fast Fourier Transform (FFT) algorithm. The resulting velocity spectrum is divided by the force spectrum to obtain a transfer function, referred to as the *Mobility* of the element under test. Two typical mobility response spectra from one of the bridge arches described in this paper are shown in Figure 1. The test graph of Mobility plotted against frequency over the 0-1kHz range contains information on the condition and the integrity of the concrete in the tested elements, obtained from the following measured parameters:
- - *Dynamic Stiffness:* The slope of the portion of the Mobility plot below 0.1 kHz defines the compliance or flexibility of the area around the test point for

normalized force input. The inverse of the compliance is the dynamic stiffness of the structural element at the test point. This can be expressed as:

Stiffness *f [concrete quality, element thickness, element support condition]*

- *Mobility and Damping:* The test element's response to the impact-generated elastic wave will be damped by the element's intrinsic rigidity (body damping). The mean mobility value over the 0.1-1 kHz range is directly related to the density and the thickness of a plate element. A reduction in plate thickness corresponds to an increase in mean mobility. As an example, when total debonding of an upper layer is present, the mean mobility reflects the thickness of the upper, debonded layer (in other words, the slab becomes more mobile). Also, any cracking or honeycombing in the concrete will reduce the damping and hence the stability of the mobility plots over the tested frequency range.

- *Peak/Mean Mobility Ratio:* When debonding or delamination is present within a structural element, or when there is loss of support beneath a concrete slab on grade, the response behavior of the uppermost layer controls the IR result. In addition to the increase in mean mobility between 0.1 and 1 kHz, the dynamic stiffness decreases greatly. The peak mobility below 0.1 kHz becomes appreciably higher than the mean mobility from 0.1-1 kHz. The ratio of this peak to mean mobility is an indicator of the presence and degree of either debonding within the element or voiding/loss of support beneath a slab on grade.

Like the Impulse Response test, the Impact-Echo (I-E) test uses stress waves to detect flaws within concrete structures. However, the frequency range used is considerably higher in the I-E test, since much shorter wavelengths are required to detect smaller anomalies. Surface displacements caused by reflecting stress waves can be viewed versus time as a displacement waveform. The amplitude spectrum of this waveform is computed by FFT, as for the Impulse Response. This spectrum has a periodic nature, which is a function of the depth to the reflective boundary (either the back of the element, or some anomaly such as a crack in the element under test. The depth of a concrete/air interface (internal void or external boundary) is determined by:

$$d = v_c / 2f \qquad (1)$$

d is the interface depth, v_c is the primary stress wave velocity and f is the frequency due to reflection of the P wave from the interface.

If the material beyond the reflective interface is acoustically stiffer than concrete (e.g. concrete/steel interface), then the following equation applies:

$$d = v_c / 4f \qquad (2)$$

The difference in the acoustic impedance of the two materials at an interface determines whether the presence of an interface will be detected by an I-E test. For example, a concrete/grout interface gives no reflection of the stress wave because the acoustic impedance of concrete and grout are nearly equal. In contrast, at a concrete/air interface, nearly all the energy is reflected, since the acoustic impedance of air is very much less than concrete.
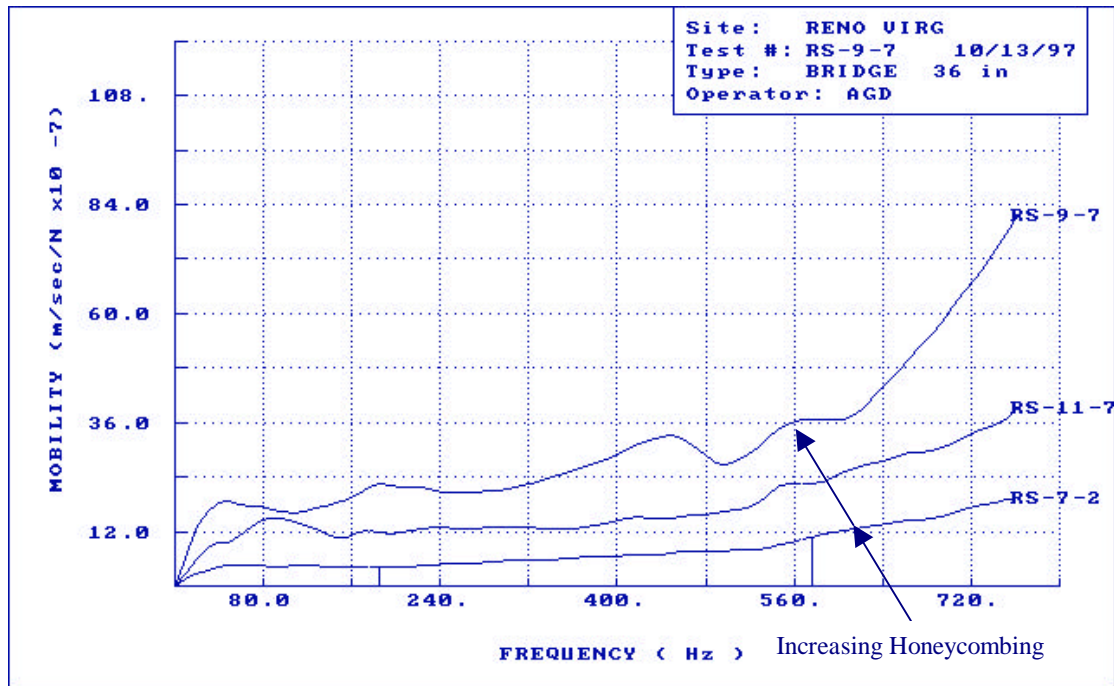
Increasing Honeycombing

Figure 1. Examples of IR Test Responses

CASE HISTORIES

The following examples are extracted from studies on the seven bridges referred to above, and are intended to emphasize the role played by NDT and specific laboratory testing in obtaining the maximum information from limited exploration budgets.

Reno, Nevada

This case history describes in detail the NDT approach adopted for all seven bridges. This double spandrel arch bridge was built in 1907, and each arch is approximately 25 m (80 ft) wide by 25 m (80 ft) in length. The scanty information available suggested that the arches are approximately 1.15 m (45 in) thick at the spring line, decreasing to 0.6 m (24 in) thick at the soffit, and that they are filled with soil to form the subgrade for the flexible paving deck. A visual survey revealed that the concrete in the piers had been placed in 0.3-m (1-ft) lifts in wooden forms, and the arches had then been constructed in 3-m (10-ft) wide lifts, 1.2 m (4 ft) high from just above the water line. There was still evidence of the paper used to line the forms for easy removal, and the concrete surface presented a very smooth finish. Spalling was evident on the underside, particularly along construction joints in the bridge alignment axis and along the water line. Water seepage was evident at the time of the survey through some spalls, presumably from the soil infill above the arch below the deck. The reinforcing steel exposed in the arches exhibited deformed square bars, 25-mm (1-in) square.

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Figure 2. Snooper Vehicle Investigation

Impulse Response testing was used to locate and map changes in concrete quality such as the degree of concrete consolidation and the presence of any delamination around steel reinforcement. A basic test grid 1.5 m x 1.5 m (5 ft x 5 ft) was set up for the underside of each arch, with access assured to the test surface by snooper truck, stationed on the bridge (Figure 2). All individual test results were stored on the field computer, allowing plotting of the IR parameters on a grid representing the underside of each arch. Impact-Echo tests were performed at selected locations to measure the thickness of the concrete in the arch, or the depth to reflective interfaces such as cracks or delaminations. The combination of results from both methods influenced the selection of core locations on the underside of each arch. Finally, direct transmission stress wave tests were made on the concrete in the central pier noses to measure the typical average compression wave velocity for the concrete in the piers and arches. This was found to be approximately 3,900 m/s (12,800 ft/s).

Sound concrete in plate-like structures (slabs, walls, bridge decks and arches) typically gives constant values of IR average mobility, which is indirectly proportional to the concrete element thickness. Poor consolidation and honeycombing causes a rise in average mobility usually accompanied by an increase in the mobility with increasing frequency. A reduction in the dynamic stiffness of the element is also present. Delamination of concrete along rebar planes parallel to the concrete surface is registered in the IR test by a significant increase in the average mobility, accompanied by a large reduction on dynamic stiffness. Figure 1 gives three test responses from this bridge with an increasing degree of poor concrete consolidation in the body of the concrete, which was not visible from the surface. Coring at these locations verified the zones of poorer consolidation.
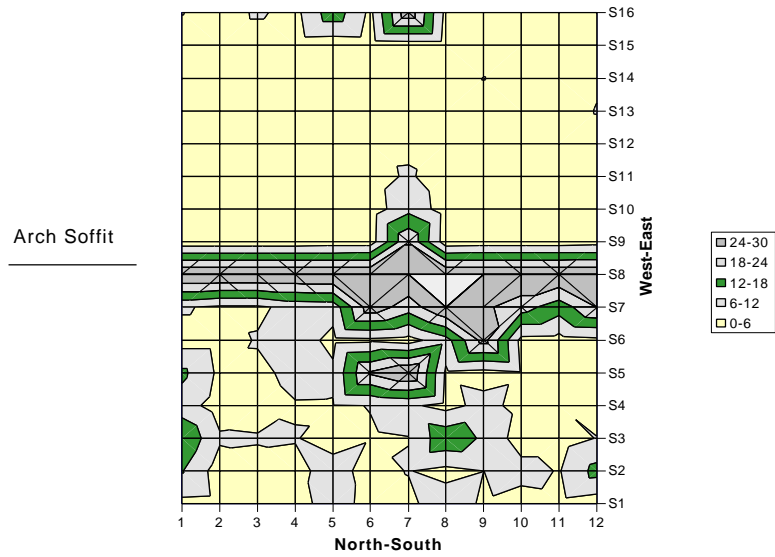
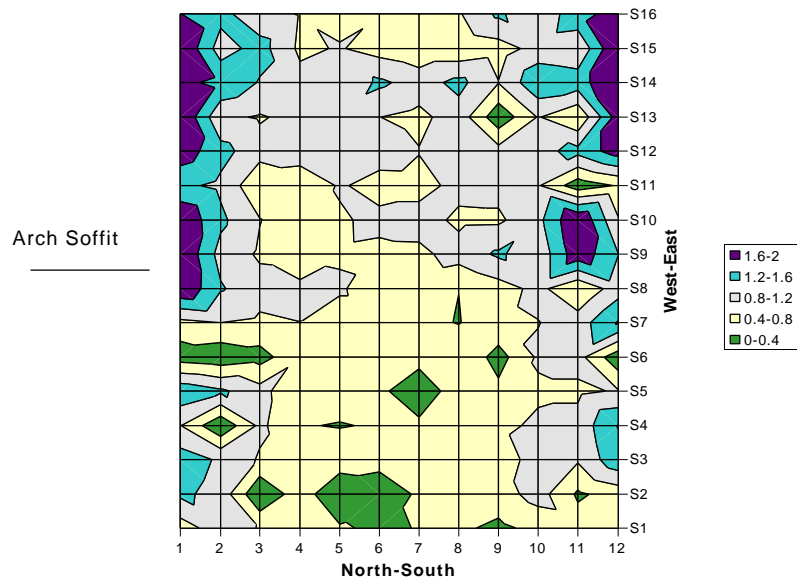**Figure 3. South Arch, Reno - Average Mobility x 1e-7 m/s/N**

Figure 4. North Arch, Reno – Stiffness (MN/mm)

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Figures 3 and 4 show examples of area plots of average mobility and dynamic stiffness for the arch undersides. For plate structures with this type of concrete, average mobility values ranged between 2 and 4.5e-7 m/s/N for a thickness between 1.15 m (45 in) and 0.6 m (24 in). Figure 3 highlights areas with mobility values greater than 6e-7 m/s/N, mainly to the west of the arch soffit. These areas of high mobility represent zones of poorly consolidated concrete, confirmed by coring. Figure 4 confirms the areas of higher stiffness towards the arch edges where the arches are reinforced by their spandrel walls.

The IR results were used to select areas for I-E testing to measure the approximate thickness of the arches at each point and to select further locations for coring. The I-E thickness at each location was determined using the average concrete compression velocity of 3,900 m/s. There was a good agreement between the I-E thickness measurements and the core lengths obtained after I-E testing. One core was selected from an area giving an I-E reflection at 0.4 m (16 in). This core revealed a crack zone at that depth.

Cores taken from zones of poorly consolidated concrete were sectioned to measure the chloride content profile through the core. Only one result showed chloride concentrations approaching the level required for active steel corrosion, at approximately 180 mm (7 in) below the top of the arch, in line with the gutters in the deck above and adjacent to steel reinforcement.

The cores were also examined by petrographic methods to search for evidence of deleterious conditions such as alkali-silica reaction (ASR), depth of carbonation and freeze-thaw damage. No ASR or freeze-thaw was evident, and carbonation appeared to be limited to the outer 50 mm (2 in) of the concrete in the arches. At the same time, the aggregate types and sizes were identified, together with the nature of the cement employed. This information was then used to design the concrete and mortar for any required repairs, in order to obtain as close a match as possible between the original and the repair finishes.

Cumberland, Rhode Island

This bridge consists of two approach spans and five arched spans. Each arched span includes a north and south arch. The arches are cast-in-place reinforced concrete structures built between 1934 and 1936. Center-span arches measure approximately 18-m (60-ft) high and span 48 m (160 ft) across the river below. The bridge deck was considered to be too deteriorated to be saved, and was removed for reconstruction (see Figure 5).Areas of low quality concrete were reportedly detected on the center span arches during bridge rehabilitation work. Coring in several locations indicated that the low quality or damaged concrete areas were not all readily visible. Selected areas of the arch had been removed for patching, and some of the removed areas had been filled while others remained open. The NDT survey described here was intended to evaluate the location and the extent of cracking, delamination, as well as areas of freeze-thaw deterioration and poor consolidation.

The IR test was the primary test method used in this survey. After preliminary analysis of the IR data, areas of the arch were selected for I-E testing to confirm IR results as well as to locate the depth and extent of anomalous areas. The arches were marked with a 1-m (3.5-ft)

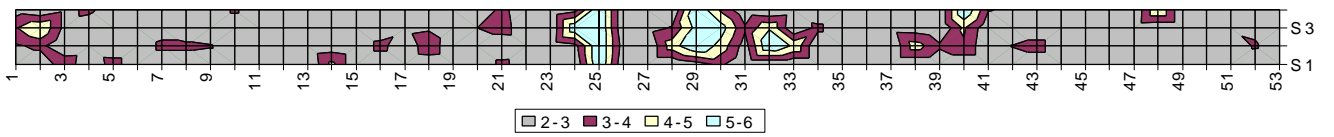**Figure 5. Rhode Island Arches**

Figure 6. IR Mobility Slope, Rhode Island South Arch

square grid for testing purposes, making 53 north-south test columns and 4 east-west test rows on the top surfaces. This grid pattern allowed complete test coverage of the arches.

The location of possible poorly consolidated or low quality concrete at this site included a review of both the average mobility and the mobility slope values for each test point. Figure 6 presents a contour plot of the mobility slope for the south arch. IR mobility slope values for sound concrete should fall below 3. Test areas showing high mobility slope were further analyzed using the I-E test method to determine the depth and extent of problem areas. In addition, areas of mobility greater than 4e-7 m/s/N were tested with the I-E equipment and used to establish an acceptable threshold value of mobility. Several areas were then selected for coring using the correlation established between the IR and I-E methods. Both arches exhibited high values of both average mobility and mobility slope in limited zones in the center section as well as in some areas close to the deck support columns and keyways, as shown in Figure 6. The consistency of average mobility and mobility slope plots indicated that the problems with the arch were restricted to very limited areas of poor consolidation. The presence of severe cracking or delamination would have shown up as discrepancies in the above mentioned plots. Delamination and or cracking would be indicated by sharp rises in the IR peak/mean mobility ratio, which were not present.

Tacoma, Washington

Certain arch columns of this open spandrel four arch bridge constructed in 1925 showed visual signs of lack of cover on reinforcement accompanied by steel corrosion, particularly on the west and south column sides. Covermeter measurements revealed that where steel corrosion was visible, the minimum concrete cover was less than 12 mm (1/2 in). Carbonation of the cover concrete had occurred over the life of the structure, and most of the corrosion can be attributed to the advancing carbonation front. The lack of cover on one side of a column also corresponded to an increase in cover above expected on the opposing side of more than 90 mm (3.5 in). This offset in the reinforcing cage was present at the time of construction, probably as a result of the tall slender cages slipping towards one side of the column forms.

The 120-m (400-ft) long reinforced concrete bridge deck with a 25-mm (1-in) thick asphalt overlay appeared to be in good condition. The deck was cast integrally with transverse beams spaced at 3 m (10 ft) across the four main arch spans, and has three transverse expansion joints at approximately 9 m (30 ft), 60 m (200 ft) and 111 m (370 ft) from the south end of the deck. Two sets of steel tram rails were still present in the deck, in the center lanes in both directions. IR tests were made to evaluate the integrity of the concrete below the asphalt. The great advantage of IR testing in this case is that it is possible to test through asphalt overlays of limited thickness (up to 50 mm (2 in)) when ambient temperatures are low enough to preserve relatively high asphalt stiffness. Two test rows were selected at 1.2 m (4 ft) and 2.75 m (9 ft) from the east curb, with test intervals every 1.5-m (5 ft) along the deck. Figure 7 represents a plot of the measured average mobility along the deck from south to north.
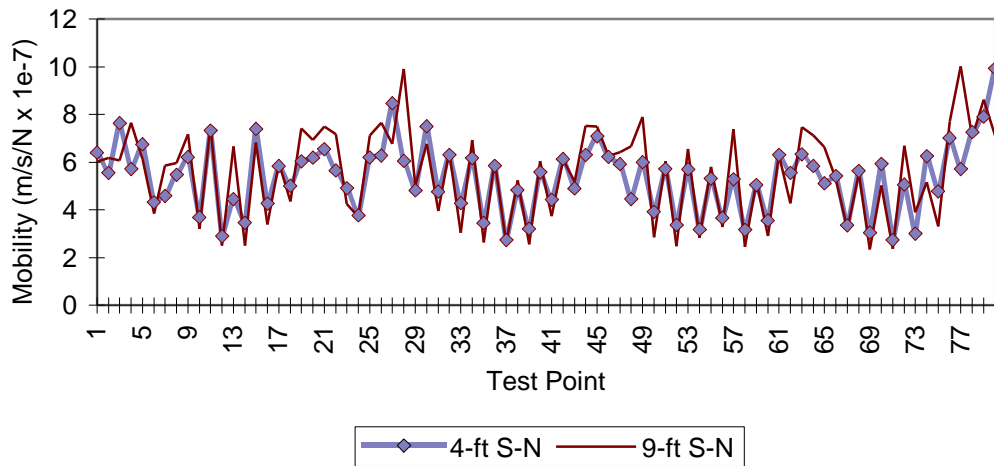
Figure 7. Tacoma Bridge Deck: IR Average Mobility

No concrete delamination was located in the deck, with all peak/mean mobility ratios below the critical value. The "saw-tooth" plot of average mobility in Figure 7 reflects the spacing of the integrally cast transverse beams at 3 m (10 ft). When the test point was above or very close to a transverse beam, the IR mobility was between 50 and 70 % that for a test point between the two beams. One test point alone gave a high mobility slope value, indicating probable deterioration in the concrete in the immediate vicinity.

Figure 8. Putnam County Bridge

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Figure 9. Coring Operation

Figure 10. View inside core hole showing poor consolidation

Putnam County Indiana

This open spandrel single-arch bridge was completed in 1929 and appeared on the surface to be in very bad condition (Figure 8). The bridge consisted of two short approaches and one 39-m (127 ft) span with a bridge width of 5.5-m (18-ft). However, IR testing along both spandrel arch beams revealed that the interior of the beams were in relatively sound condition, with good concrete consolidation and little cracking. Most of the damage had occurred because water draining through the deck beam expansion joints flowed down the arches, resulting in freeze-thaw activity in the cover concrete, followed by limited reinforcing

corrosion. Occasionally, the IR tests revealed zones with high average mobility and mobility slope. Core locations were selected as a result, and Figure 9 shows a core location in what appears to be sound concrete at the surface. Figure 10, however, reveals that the concrete in the core hole is poorly consolidated, as indicated by the IR tests. The positive conclusion about the concrete quality in the body of the arches could not have been reached based on visual and invasive testing alone.

DISCUSSION

The case histories given in this paper attempt to emphasize the advantages in incorporating nondestructive techniques in a historic concrete bridge evaluation program. In particular, the family of stress wave tests including Impulse Response and Impact-Echo give rapid coverage of relatively large concrete volumes, often with difficult access. Test data is easily stored for future analysis and reference, and is reproducible. Identification of anomalous areas for closer, invasive investigation is made on the site, thereby reducing mobilization costs. The nature of the anomalies can be determined by the nondestructive tests (cracking, delamination, poor consolidation and honeycombing), even when they are not visible at the structure surface.

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