

INNOVATIONS

IN NON-DESTRUCTIVE TESTING OF CONCRETE



EDITORS
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Innovations in Nondestructive Testing of Concrete

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Stephen Pessiki
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PREFACE

ACI Committee 228, Nondestructive Testing of Concrete, was organized in 1982 to promote the use of nondestructive testing techniques for concrete construction and to disseminate up-to-date information on the application of these techniques. The committee remains active in the development and reporting of information on nondestructive techniques for the evaluation of concrete. As part of its work, ACI Committee 228 has been active in the organization of technical sessions that present new developments and field experiences in the area of nondestructive testing of concrete. This volume contains many of the papers presented at the two most recent technical sessions organized by the committee. The first session was held during the ACI Spring Convention in 1993 in Vancouver, British Columbia, Canada. The second session was held during the American Concrete Institute Fall Convention in 1995 in Montreal, Quebec, Canada.

A total of 16 papers are included in this special publication. Three of the papers treat the use of radar methods for the evaluation of concrete structures, including the application of dual frequency radar to bridge decks. Six of the papers in this publication involve the application of stress wave methods to concrete structures. Of these six papers, four are related to the use of the impact-echo method for either thickness determination or flaw evaluation. Both laboratory and field studies are included. The remaining two stress wave method papers treat the use of spectral analysis of surface waves to evaluate the damage and repair of concrete bridge members, and the use of acoustic emissions to identify internal cracks in concrete structures. Two papers in this publication discuss advances in imaging and visualization techniques to the interpretation of nondestructive test results. The remaining five papers in this publication treat a range of topics including the development of an in situ chloride migration test, the use of a pulloff test to evaluate bond between old and new concrete, the evaluation of bond strength by contact electrical resistivity measurement, and case studies of the use of nondestructive methods for the evaluation of a variety of concrete structures in the field, including foundations, dams, and bridge piers.

All of the papers presented in this special publication were peer-reviewed according to ACI procedures. On behalf of ACI Committee 228, the editors thank the authors for their careful efforts in preparing these papers, and also thank the many reviewers whose thoughtful reviews contributed to the quality of the papers contained herein.

Stephen Pessiki
Larry Olson
Editors

CONTENTS

PREFACE iii

INTERPRETATION OF RADAR TEST RESULTS
by Stephen G. Millard, John H. Bungey, Marcus R. Shaw, Cledwyn Thomas, and Brian A. Austin 1

NONDESTRUCTIVE TESTING OF BRIDGE DECKS USING DUAL FREQUENCY RADAR
by Anthony B. SlatonBarker and John W. Wallace 25

NEW IMPULSE RADAR STRATEGIES FOR BRIDGE DECK ASSESSMENT
by R. Reel, T. Tharmabala, D. Wood, T. Chung, and C. R. Carter. 59

KINEMATIC IDENTIFICATION OF INTERNAL CRACKS BY SIGMA INVERSION TECHNIQUE
OF ACOUSTIC EMISSION
by M. Ohtsu, M. Shigeishi, T. Okamoto, and S. Yuyama 95

NONDESTRUCTIVE CHARACTERIZATION OF DAMAGED AND REPAIRED AREAS OF A
CONCRETE BEAM USING THE SASW METHOD
(1995 ACI-James Instruments Student Award Paper)
by Michael E. Kalinski 111

A PROCEDURE FOR DETERMINING P-WAVE SPEED IN CONCRETE FOR USE IN IMPACT-
ECHO TESTING USING A RAYLEIGH WAVE SPEED MEASUREMENT TECHNIQUE
by Jiunn-Ming Lin and Mary Sansalone 137

A PROCEDURE FOR DETERMINING CONCRETE PAVEMENT THICKNESS USING P-WAVE
SPEED MEASUREMENTS AND THE IMPACT-ECHO METHOD
by Mary Sansalone, Jiunn-Ming Lin, and William B. Streett 167

LOCATING DELAMINATIONS AND OTHER DEFECTS IN CONCRETE SILO WALLS USING THE IMPACT-ECHO PROCEDURE
by N. A. Cumming and O. S. Ooi 185

DETECTING CRACKS IN THE BEAMS AND COLUMNS OF A POST-TENSIONED PARKING GARAGE USING THE IMPACT-ECHO METHOD
by Randall Poston and Mary Sansalone 199

APPLICATION OF ACOUSTICAL IMAGING TO THE EVALUATION OF CONCRETE IN OPERATING STRUCTURES
by Jamal Rhazi, Yahya Kharrat, Gerard Ballivy, and Michel Rivest 221

THREE-DIMENSIONAL VISUALIZATION OF CONCRETE STRUCTURES
by M. D. A. Thomas, D. Wiese, and H. Caratin 233

FEASIBILITY STUDY OF THE DEVELOPMENT OF AN IN SITU CHLORIDE MIGRATION TEST
by P. A. Muhammed Basheer, Peter R. V. Gilleece, Robin Andrews, and Adrian E. Long 245

APPLICATION OF PULLOFF TEST TO ASSESS THE DURABILITY OF BOND BETWEEN NEW AND OLD CONCRETE SUBJECTED TO DEICER SALTS
by Shigen (Eric) Li, Gregory C. Frantz, and Jack E. Stephens 267

NONDESTRUCTIVE BOND STRENGTH TESTING BY CONTACT ELECTRICAL RESISTIVITY MEASUREMENT
by Xuli Fu and D. D. L. Chung 295

EVALUATION OF DEEP FOUNDATIONS BENEATH BUILDINGS DAMAGED DURING THE 1994 NORTHRIDGE EARTHQUAKE
by Allen G. Davis 319

EVALUATION OF THE INTEGRITY OF SOME LARGE CONCRETE STRUCTURES USING NDT
by Allen G. Davis and Bernard H. Hertlein 333

SI (Metric) TABLES 357

INDEX 359

SP 168-1

Interpretation of Radar Test Results

by Stephen G. Millard, John H. Bungey,
Marcus R. Shaw, Cledwyn Thomas,
and Brian A. Austin

Synopsis: Experimental work has been undertaken in the laboratory and on site to assist understanding and interpretation of the results of radar testing of structural concrete. This has included the development and use of a large-scale emulsion simulation tank in which a very large range of reinforcing steel and void configurations have been examined with field testing apparatus for a range of simulated concrete properties. A library of characteristic responses as well as limits of size and spacing upon successful resolution have been obtained. A large diameter co-axial transmission line has also been designed, fabricated and used to determine the fundamental electrical properties of a range of concretes and moisture conditions at frequencies from 1MHz up to 1GHz.

Results have been compared with those for tests on larger concrete specimens with field testing apparatus and confirm the dominant influence of moisture compared with other aspects of the concrete composition. Frequency effects are quantified and related to characteristics of field antennas, and potential errors of using 'typical' values of concrete properties in interpretation and numerical modelling are identified. The test results from transmission line studies are compared with experimental and theoretical results from other research workers.

Keywords: concretes; conductivity; microwaves; nondestructive tests

2 Millard et al.

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INTRODUCTION

Subsurface radar has become well established as a non-destructive technique for inspection of structural concrete in a wide range of applications. The principles were outlined by Cantor (1) in 1984 and subsequent developments have more recently been reviewed by Clemena (2) as well as Bungey and Millard (3). Despite significant developments in commercially available equipment, the most successful applications currently involve interpretation procedures which are largely comparative in nature and based on pattern recognition. These include location of reinforcing steel and ducts, estimation of element thickness, identification of constructional details, location of voids and cracking as well as location of regions of saturated or salt contaminated concrete.

Specialised skills are required to interpret successfully the output data from radar surveys and numerical modelling techniques including ray-tracing, finite element and finite difference techniques have been employed to simulate the responses from particular configurations (4). Validation of such models requires comparison with results from known circumstances. The scope for the use of site

results for this purpose is inevitably limited, thus laboratory studies are required and the Authors have developed a large scale simulation tank using an oil-water emulsion to permit a large number of known configurations to be systematically studied without the need for casting, curing and conditioning an impossibly large number of concrete specimens (5). Numerical models, as well as quantitative interpretation procedures, also rely on a knowledge of the fundamental electrical properties of particular concretes at the test frequencies used. It is known that moisture is a key influencing factor and Halabe et al (6) have described theoretical models to predict variations of these properties from a knowledge of the constituents and their proportions also taking account of temperature effects. Similar attempts have been reported by Tsui and Matthews (7) incorporating geometrical pore shape effects. Validation of these theoretical predictions of electrical properties is not easy. Halabe et al (6) have used tests on sand, gravel, water and salt mixtures whilst Al-Qadi et al (8) and the Authors have developed electrical transmission line systems for testing concrete samples.

Key results obtained from simulation tank and transmission line studies in the Civil Engineering Department at the University of Liverpool supplemented by other laboratory and site tests, are presented in this paper. These provide information about the likely limits to the capabilities of radar techniques applied to reinforced and prestressed concrete as well as the scope for improvements to interpretation procedures.

BASIC PRINCIPLES OF RADAR APPLIED TO CONCRETE

A diverging beam of electromagnetic radiation is directed into the concrete from a transmitting antenna scanned across the surface. Reflections from buried interfaces are detected by a receiving antenna which is usually located adjacent to the transmitter. The propagation velocity of the signal is determined predominantly by the relative permittivity of the concrete whilst the attenuation and hence effective penetration depth is related to the electrical conductivity. The strengths of reflections are governed by the contrast in relative permittivity and conductivity at an interface, whilst some energy will pass through the interface by refraction and may be reflected from deeper features.

The pattern developed during a scan will be influenced by the beam divergence angle, the shape and orientation of the reflective surface, and the scan speed. Interpretation may also involve a study of the polarity changes which may occur in reflected signals. Conductive materials such as steel reinforcing bars will provide a very strong response and are thus relatively easy to identify but no signal will pass through to greater depths.

4 Millard et al.

Beam spread and penetration capabilities are also influenced by the antenna characteristics as well as the concrete properties, with lower frequencies generally providing greater penetration but lower resolution than higher frequencies. This has been discussed in detail by Padaratz and Forde (9), and most radar testing of concrete lies in the 500MHz to 2.4GHz frequency range. Hand held 900MHz or 1GHz apparatus is most popular for structural investigations with effective penetration typically up to 300 mm in saturated concrete.

Fundamental theory is highly complex and beyond the scope of this paper although key features have been summarised by Halabe et al (6). Other aspects of the principles, apparatus, interpretation and signal processing procedures have been reviewed elsewhere (1, 2, 3, 10).

SIMULATION TANK STUDIES

A 1250mm x 1100mm x 450mm deep all-timber tank was fabricated to simulate a concrete slab with dimensions likely to provide a central zone free of edge effects. This was located with an 770mm air gap above the laboratory floor to minimise interference reflections from the floor. The tank was filled to an appropriate depth with an oil-water emulsion. Reinforcing bars were supported within the emulsion on dowelled wooden boards fitted within the longer sides of the tank, and adjustable runners set to the top surface level of the fluid facilitated surface contact scanning as illustrated in figure 1 where a 1GHz antenna is being used. Voids formed by expanded polystyrene or plastic boxes were similarly supported by small diameter timber dowelling.

Emulsions of rape-seed oil and water with appropriate sodium chloride additions have been successfully developed to model concretes ranging from saturated to air-dry conditions (5). Values of relative permittivity were adjusted by varying the oil and water proportions, whilst signal attenuation characteristics were matched to those obtained on similar sized concrete slabs tested with field apparatus by adjustment of salt concentration of the water component. Trials were undertaken on a range of emulsion mixtures in a small perspex tank using 1GHz centre frequency field testing apparatus to develop a relationship between emulsion proportions and relative permittivity and were supplemented by results obtained using a version of the transmission line system described below. The rape-seed oil alone had a measured relative permittivity of approximately 3. By adding up to 30% water by weight, the relative permittivity could be increased to approximately 12. However, the relationship between the relative permittivity and the water content of the emulsion is not linear. There is a marked transition between the region of 8% water and 12% water where the emulsion changes from being a water-in-oil emulsion to an oil-in-water emulsion. This transition causes

a very marked change in the physical appearance of the emulsion and a sharp increase in the measured value of the relative permittivity from 4 to 6. On the basis of these results emulsions with 12.5% and 29% water contents by weight, with relative permittivities of 6.2 and 10.6 respectively, were used to simulate dry and wet concretes. Corresponding salt contents of 0.05% and 0.14% by weight of water yielded d.c. conductivities of 15×10^{-3} and $46 \times 10^{-3} \Omega^{-1} \text{ m}^{-1}$ respectively.

Results obtained with this facility were validated by comparison with specially fabricated laboratory concrete samples and by site tests on members of known construction details. This is illustrated by figure 2 which compares scan results obtained on a 1m square 200mm thick concrete slab containing a rectangular 100mm x 50mm air void and two single reinforcing bars (25mm and 8mm) set at mid depth. These are black and white versions of colour plots, but the general similarity between these two scans 2(b) and 2(c) is apparent. There is a general tendency for the simulation tank material to give a 'cleaner' response as might be expected due to its greater homogeneity than concrete containing discrete aggregate particles which are likely to cause some scattering and 'noise' on the reflected signals. Results nevertheless provide useful 'best case' reference data and are compared in figure 2(d) with predictions obtained by ray-trace numerical modelling using appropriate materials parameters.

Tests with Reinforcing Bars

Reinforcing bars were placed across the width of the tank, perpendicular to the direction of scan, which provides the optimum orientation of the polarisation plane of the antenna for bar detection when orientated as shown in figure 1. Bar sizes ranged from 6mm to 32mm diameter set at covers between 12mm and 250mm and with spacings down to 50mm centre to centre. More than 300 different configurations have been systematically studied for emulsions designed to simulate typical air-dry and saturated concretes. Detailed results (11) have shown that whilst the clear characteristic hyperbolic response pattern will be obtained by a scan across an isolated reinforcing bar which is located (for 1GHz centre frequency) more than about 100mm below the concrete surface as shown in figure 3(a), bars at more common cover depths will produce a response which merges with the surface reflection as shown in figure 3(b). A pattern obtained by Finite Element Analysis for a simulated bar at depth is also shown in figure 3(c) for comparison. Whilst the top of the bar signal is relatively clear when at depth, this is less clearly defined when close to the surface thus making depth estimation, which is based on the return signal time between the concrete surface and the bar, more difficult. Depth estimations must thus be treated with caution, and are further complicated by uncertainties about the relative permittivity and hence signal velocity. Confidence limits of up to +45% to -30% have been

6 Millard et al.

suggested (11) using assumed values of concrete relative permittivity for bars at small covers unless calibration drilling is undertaken.

As bars become more closely spaced, the reflected responses will tend to overlap as shown in figure 4 and there is a limiting spacing below which it is not possible to resolve individual bars. This limiting spacing will be influenced by cover, bar size and concrete properties although these latter effects have been found to be relatively minor. Figure 5(a) suggests effective limits for 1GHz apparatus irrespective of bar diameter or concrete condition although there is a general tendency for wet concrete (with higher attenuation) to yield greater resolution for bars which are near to the surface. The shaded area represents a zone in which individual bars may or may not be detectable, and it will be seen that at 50mm cover it may be possible to identify bars at 100mm centres whilst it may not be possible at spacings below 150mm. Closely spaced bars will also effectively screen out deeper features due to a 'Faraday Cage' effect. This critical spacing will be related to the signal wavelength ($\sim 100\text{mm}$ at 1GHz) and experimental results in figure 5(b) illustrate the effect of bar diameter and spacing for a mesh at between 25 and 50mm cover. It can be seen that for large diameter bars, the effective limiting spacing may be as high as 200mm at 1GHz test frequency, compared with the value of 100mm often quoted in the literature. This limiting spacing may be expected to increase as frequency decreases and the size of the target bar below the mesh was found to have relatively little effect upon the ability to detect it although 'weaker' features such as air voids will be more difficult to identify. The identification of deeper reinforcing bars or meshes will thus be difficult especially if bars are located directly beneath those in the surface layer.

Changing the transducer orientation by 90° to that shown in figure 1 will significantly reduce the response obtained from transverse bars and is recommended if their effect is to be minimised when seeking other, deeper, features. Bars running parallel to the scan direction have also been demonstrated to have a constant and minor affect upon the ability to detect transverse bars even when directly under the antennas unless their diameter exceeds that of the transverse bars being sought. It is nevertheless recommended that scan lines should be located midway between bars if possible.

Accuracy of estimation of the lateral positioning of bars will depend upon the consistency of scan speed, reference grids and location marker inputs but $\pm 10\text{mm}$ should be possible. The method is thus particularly valuable in this respect offering improved clarity of practical data and greater depth range compared with conventional covermeters. It can be seen from figure 2 that it is possible to differentiate comparatively between large and small diameter reinforcing bars. Detailed analysis of a wide range of results, including study of individual reflected waveforms, leads however to the conclusion that quantitative bar size

estimation is not realistically possible from simple examination of the results of a single scan.

Tests with Voids

Large voids were formed by a 100mm x 100mm x 50mm plastic box which could be air - or water - filled, whilst smaller air voids were formed by expanded polystyrene cut to cubic or spherical shapes ranging from 20mm to 100mm in size. Tests have also included reinforcing bars and voids in combination as well as both steel and plastic 100mm diameter post-tensioning ducts in grouted and ungrouted conditions.

Results have clearly demonstrated that a water-filled void provides a stronger response with increased likelihood of secondary reflections due to reverberations of the reflected signal between the void and concrete surface than a comparable air-filled void. This is to be expected due to the increased contrast in dielectric properties between the void and the surrounding concrete. Void shape similarly has a significant effect, since a sphere will inherently reflect a smaller proportion of the signal back to the concrete surface but this will commence when the antenna is at a greater distance away from the centreline of the void than for a cubic void with one face parallel to the surface. This effect, leading to a flatter response for a spherical void is illustrated in figure 6 which compares voids of similar size and depth with identical apparatus settings. The difficulties in estimating both shape and depth of the void are readily apparent. It is sometimes suggested that the smallest non-metallic feature that can be detected will have a plan dimension of about 1 wavelength. For concrete tested at 1GHz, this typically corresponds to about 80mm to 120mm. Tests have shown however that an air-void at 150mm cover could be readily detected down to 50mm in diameter but not at smaller sizes. Careful adjustment of apparatus controls permitted cubic voids as small as 20mm to just be detected (12) without the need for signal processing, but this was only possible because of the known target depth. These sizes are also significantly smaller than that suggested as the minimum detectable using a Fresnel Zone approach (9). Adjustment of gain settings has thus been shown to be crucial in the ability to locate small voids, as demonstrated by figure 7, and in reality they are likely to be missed if their size is less than 50mm. Operator skill and experience is thus crucial if features are to be reliably detected. The presence of a layer of surface zone reinforcing steel will further hinder the ability to detect small voids. As with reinforcing bars, the centre position of a void can be defined reasonably accurately but the actual dimensions are hard to estimate due to the unknown variables such as beam divergence angle, scan speed and void shape.

8 Millard et al.

Tests on prestressing ducts, as expected, confirmed that the interior condition of a metallic duct cannot be differentiated although the duct position can be clearly defined. For plastic ducts it was found that it was possible to distinguish between those containing prestressing strand and which were either fully grouted or ungrouted. This is best achieved by comparison of individual reflected waveforms, and has subsequently been confirmed by tests on similar ducts cast into concrete and by finite element modelling. The ability to detect discrete voids within grouted ducts remains to be investigated.

TRANSMISSION LINE STUDIES

Initial attempts by the Authors to assess changes in relative permittivity due to moisture changes in laboratory concrete slab specimens (10) using field testing apparatus illustrated the difficulties associated with this approach. It was thus decided that a 'transmission line' approach was the only viable way to examine accurately a range of concrete samples under controlled moisture conditions over a sensible frequency range and to yield conductivity data as well as relative permittivity.

System and Measurement Details

For the frequency range of primary interest it was concluded that the best form would be a coaxial system comprising a cylindrical concrete sample cast on a metallic inner core and surrounded by an outer conductive casing as shown in figures 8 and 9. The detailed development of this system and the relevant theory is described elsewhere by the Authors (13) whilst Al-Qadi (8) has also more recently provided details of a similar approach. Dimensional details are determined by the need to satisfy electrical parameters, and the system shown in figures 8 and 9 is 101mm in diameter which can accommodate concretes with approximately 10mm maximum aggregate size. This has a 50 Ω intrinsic impedance and is used in conjunction with a Hewlett-Packard 8753b Network Analyser. The transmission line was designed so that it could serve as a mould for the cylindrical concrete specimens which could subsequently be dismantled to permit specimens to be removed for curing and conditioning prior to, and between, measurements. The version illustrated represents a modification of the original prototype in which the sample length has been reduced to 200mm. The introduction of an air gap at each end of the specimen improves field uniformity and reduces losses in the specimen due to its shorter length.

Both ends of the cylindrical section of the transmission line are connected to a network analyser using concentric metal conical sections. A signal is input to the transmission line from one end. The ratio of the input signal to the signal reflected back from the concrete specimen can be measured. Alternatively the signal transmitted through the concrete to the far end can be measured.

Measurements can be taken in the time domain, where the velocity of propagation and the attenuation of a transient pulse is measured and used to determine the relative permittivity and electrical conductivity of the concrete specimen. Alternatively measurements can be taken in the frequency domain, where the steady state response to a continuous sinusoidal signal is measured over the range of frequencies of interest. Each technique has its own merits but measurements taken in the frequency domain can be converted into the time domain and vice versa using appropriate Fourier Transform functions. Detailed theory (13) of the transmission line system is beyond the scope of this paper.

The dielectric properties of a material are often complex with measurements involving real and imaginary components. When a material's properties are expressed as relative permittivity and electrical conductivity, the real part of the complex permittivity is seen as the relative permittivity whilst the electrical conductivity is determined from both the real and imaginary components. An alternative way of expressing the way in which a material attenuates a radar signal is to use the ratio of imaginary and real parts of the complex permittivity which is known as $\tan\delta$, where δ is called the 'loss angle'. An understanding of these relationships will assist comparisons of reports from different sources.

This system has successfully enabled measurements of both relative permittivity and conductivity in wet concretes at discrete frequencies up to 1GHz. A larger 150mm diameter version has also been commissioned for concretes using coarse aggregate size up to 20mm but whilst this has been used successfully, the increased bulk and weight diminish ease of handling. Krause et al (14) have recently demonstrated that aggregate size ranging between 8mm and 32mm has no measurable effect when using a 900MHz antenna and this has been confirmed over a wide frequency range in the present study for gravels from 10mm to 20mm, thus the use of this larger version may not be advantageous.

Test Programme

A total of fourteen 101mm diameter and six 150mm diameter specimens have been cast in the present study and tested at a range of moisture contents (typically 3 to 5 tests per specimen) over a period of several months of natural air drying in laboratory conditions. This was assisted by controlled oven drying at the later

stages, eventually to a fully oven-dry condition, with moisture contents calculated from weighings at each stage. Measurements only commenced at an age of approximately 28 days due to the high attenuation characteristics of recently cast hydrating and immature concrete (13). Interpretation of the transmission line results involves the use of computer based curve fitting processes to match established logarithmic profiles for permittivity and linear profiles for conductivity with varying frequency over the range covered. Considerable effort was required to refine these procedures to yield information successfully relating to the higher end of the frequency range.

Gravel concrete samples encompassed 28 day cube strengths from approximately 17N/mm^2 to 60N/mm^2 whilst other specimens incorporated either Pulverised Fuel Ash or Ground Granulated Blast Furnace Slag as partial cement replacements over a range of proportions. Other specimens included lightweight coarse aggregates with either natural sand or lightweight fines. Typical results for a particular concrete mix are as shown in Figure 10, from which the significant effect of moisture can be seen upon both relative permittivity and conductivity.

Discussion and Analysis of Results

For comparison of different mixes, results relating to a specific 1GHz frequency are plotted in Figure 11. These demonstrate the major differences in response between lightweight and 60N/mm^2 gravel concretes for which the mean line and range of results on 3 replicate specimens are plotted. Although not plotted, results for lightweight concrete with natural sand fines lay between these two curves as might be expected, whilst mixes containing pfa or ggbfs lay close to the 60N/mm^2 gravel concrete curve for both permittivity and conductivity. Gravel concretes of much lower strength similarly lay very close to the higher strength results for conductivity but with a small reduction in relative permittivity at very low moisture levels (oven drying) as indicated. At low moisture levels it can be seen that relative permittivities all tend towards a value of about 5.0 which is that associated with the aggregate material. The use of alternative natural coarse aggregates (granite and limestone) was found to have only a relatively minor effect in relation to gravel concretes compared with the major influence of moisture, although limestone did yield a small increase in relative permittivity which is consistent with its greater water absorption characteristics.

Earlier studies of concrete properties at radar frequencies using the initial prototype transmission line (13) used frequency domain results over the entire range of 1MHz to 1GHz to produce a simulated broadband time domain pulse. From the velocity of this pulse through the transmission line, the relative permittivity could be determined. It was found that the relative permittivities

ranged from 5 for oven-dried concrete up to 11 for quite wet concrete which fitted well with the results of other studies. Results obtained with an actual broadband pulse using commercial field equipment to determine the velocity of propagation of a radar signal through concrete slabs, yielded resulting relative permittivities which fitted the same relationship but with significantly more scatter (13).

Using the modified transmission line in the present test programme, the magnitude of frequency domain reflection and transmission measurements have been matched against theoretically derived values to determine both relative permittivities and conductivities for concrete for discrete frequencies over the entire range of measurement from 1MHz to 1GHz. One result of this more sophisticated analysis technique is that when measuring more highly water saturated concrete at high frequencies, the relative permittivity is found to be significantly lower than that measured using the time domain, broadband approach. However when the same frequency dependent reflection and transmission results are used to synthesise a time domain response which is then analysed using the simpler time domain velocity approach, the permittivities are found to lie on the same curve as for the previous tests. The time domain technique unfortunately has the disadvantage that relative permittivities at specific frequencies cannot be found and furthermore no information about the conductivity of the concrete at radar frequencies is given.

The effect of band width is illustrated in figure 12 in which values of relative permittivity are compared for several frequency ranges based on a time domain re-analysis of the frequency domain transmission line results. Tests on a typical commercially available 1GHz antenna yielded a bandwidth of approximately 200 MHz to 1.5 GHz in air, thus the influence of the lower frequency components upon the response due to field testing equipment at higher saturation levels is apparent. This effect is further increased by the reduction of the signal frequency caused by the nature of the medium to which the radar antenna is coupled, and frequencies of 500MHz and approximately 750MHz have been suggested as the 'effective' value in concrete resulting from the use of a 1GHz antenna (14,9) by different investigators.

Comparison with Theoretical Models

Following the completion of experimental transmission line tests to determine the dielectric properties of various concretes and moisture conditions, it is useful to compare these results with the predictions of theoretical models discussed by Tsui and Matthews (7) and by Halabe et al (6). Both of these studies examined a number of different analogies including three inclusion models, where concrete is best modelled as comprising a number of high aspect ratio inclusions of water

and air contained within the concrete solids. The homogeneous inclusion model predicted a linear increase in the relative permittivity with increasing levels of water saturation and this was found to fit most closely to the transmission line results described here.

One disparity between the homogeneous inclusion model and the observed results was the influence of concrete strength. The model predicted that a low strength concrete with a high water-cement ratio would have a high voids ratio. When saturated, these voids will contain a large amount of water with a consequential increase in the relative permittivity and the conductivity. From the experimental measurements the influence of the concrete strength was not found to be so significant and very little difference was observed between the dielectric properties of 60N/mm² and 18N/mm² concrete. The reasons for this disparity are not yet fully understood and are the subject of further investigation.

In the study by Halabe et al (6) the theoretical models were compared with measurements on an analogue specimen comprising a mixture of coarse and fine aggregates, water and air. Direct comparisons of these tests with the transmission line tests on concrete are hampered by the high voids ratios used in the analogue specimens. However both studies did find that the relative permittivity was effectively constant over the frequency range of 100MHz to 1GHz. This finding was corroborated by tests on concrete, also using a large diameter transmission line, carried out by Al-Qadi (8). Measurements on a concrete specimen after 90 days of curing found no significant variation in the relative permittivity over the same frequency range.

DISCUSSION

The simulation tank results have highlighted many of the practical difficulties which are likely to be encountered in practice when applying subsurface radar to reinforced concrete. It is important that field applications are planned with realistic expectations in the light of these findings. Whilst the use of this tank has drawbacks, including being messy, it offers the facility to evaluate systematically the likely response from a variety of configurations to assist both planning and interpretation of test programmes. This may be particularly valuable where non-standard or complex applications are being considered, and study of likely responses may be further extended by the use of numerical modelling.

The quantitative analysis of results to yield dimensions and depths of buried features relies heavily upon a knowledge of the fundamental electrical properties of the concrete. The transmission line work has clearly demonstrated that although relative permittivity does not change significantly over the 500MHz to

1GHz centre frequency range commonly used for testing structural concrete, conductivity and hence penetration capability does change. Moisture is confirmed as the dominant parameter, and concretes made with natural aggregates differ from those made with lightweight aggregates due largely to different absorption characteristics. Cement replacements and changes in mix proportions leading to different strength levels appear to have relatively minor influences although voidage characteristics are important. Due to the broadband nature of commercial antennas, results relating to one specific frequency cannot directly be used to interpret field results. The availability of such data however improves understanding and provides values for use in numerical models to assess the capabilities of radar and the likely response in given situations. They also highlight the difficulties in precise quantitative interpretation of field data without calibration since in-situ moisture conditions will generally be both unknown and variable, and the antenna frequency characteristics change when closely coupled to concrete.

CONCLUSIONS

1. A simulation tank using oil-water emulsion may successfully be used to model the radar characteristics of concrete.
2. This facility enables a very wide range of configurations of reinforcing bars, voids and other hidden features to be studied without the need for casting and curing concrete specimens.
3. The effects of bar size, spacing and depth and of void shape and size have been studied to develop and confirm approximate guidelines concerning the limitations of their identification and influence upon detection of other features.
4. The difficulties of depth estimation, and sizing of bars and voids have been demonstrated.
5. A co-axial transmission line has been developed which can successfully measure the electrical properties of concrete from 1MHz up to 1GHz frequency.
6. The dominant influence of moisture upon both the relative permittivity and conductivity of concretes has been confirmed.

7. The broadband nature of commercial antennas makes prediction of field responses difficult from laboratory results relating to one specific frequency.
8. The role of radar applied to structural concrete, whilst important, is likely in the short term to remain largely comparative due to the large number of variables influencing responses obtained under field conditions.

ACKNOWLEDGEMENTS

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Fig. 1—Emulsion simulation tank with 1 GHz transducer

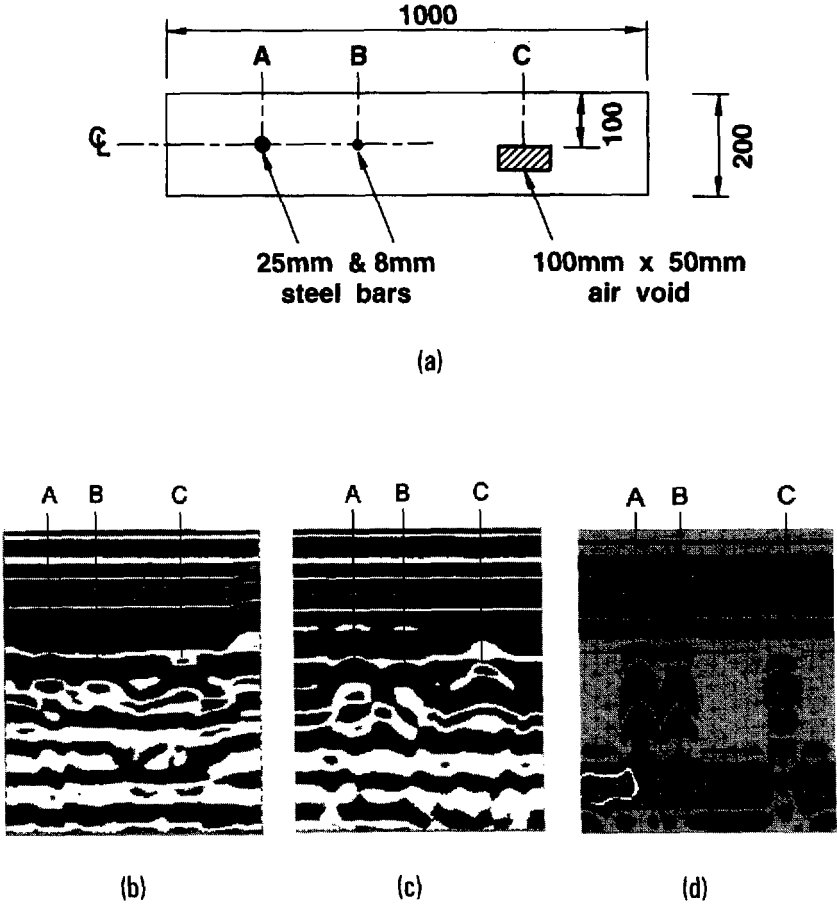


Fig. 2—Comparison of concrete slab, emulsion simulation, and numerical modeling: a) slab section; b) concrete slab; c) emulsion 'slab'; d) ray-trace model

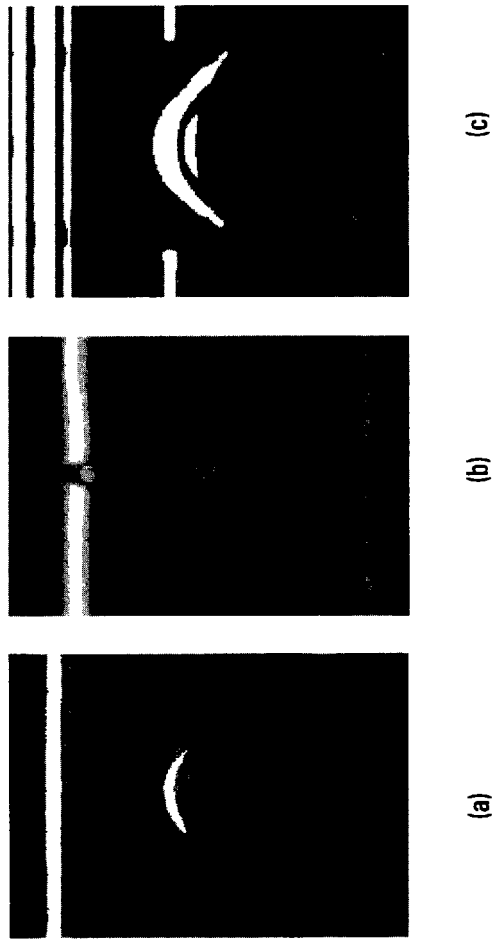


Fig. 3—Response patterns of reinforcing bars: a) single bar near surface; b) single bar at depth; c) finite element model of single bar at depth

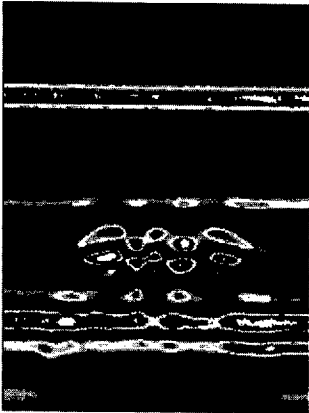


Fig. 4—Response pattern of multiple reinforcing bars (3 no. 12 mm bars at 155 mm cover and 200 mm spacing)

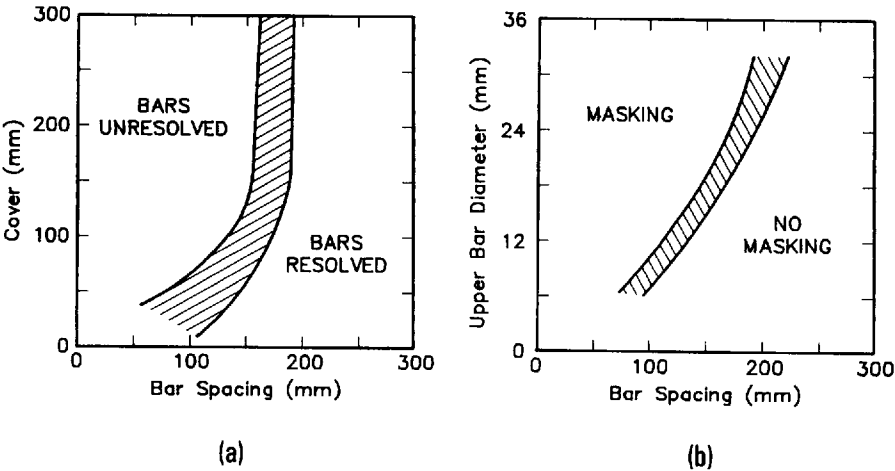
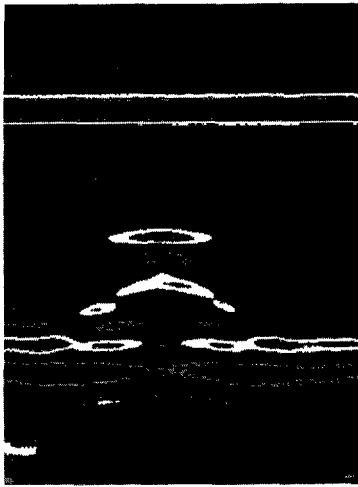
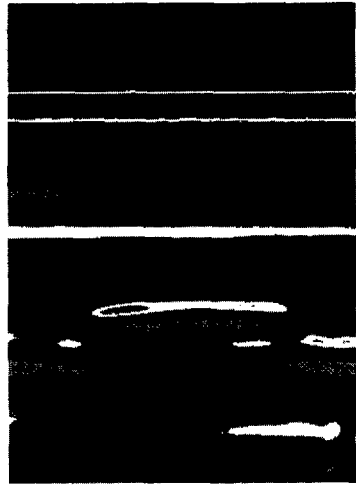


Fig. 5—Bar spacing limits (1 GHz): a) resolution of individual bar; b) masking of deeper features

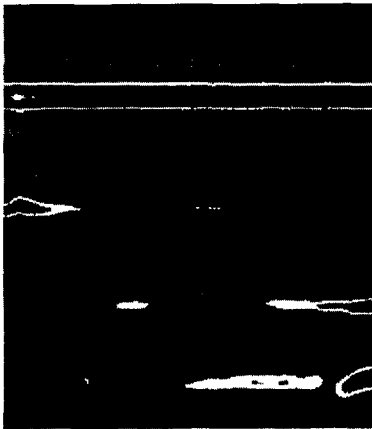


(a)

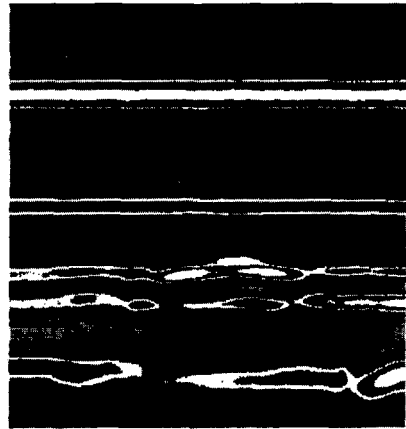


(b)

Fig. 6—Comparison of air void shapes: a) 100 mm cubic void; b) 100 mm spherical void



(a)



(b)

Fig. 7—Effects of apparatus adjustment (300 mm cubic voids at 150 mm depth): a) 'standard' settings; b) 'optimum' settings

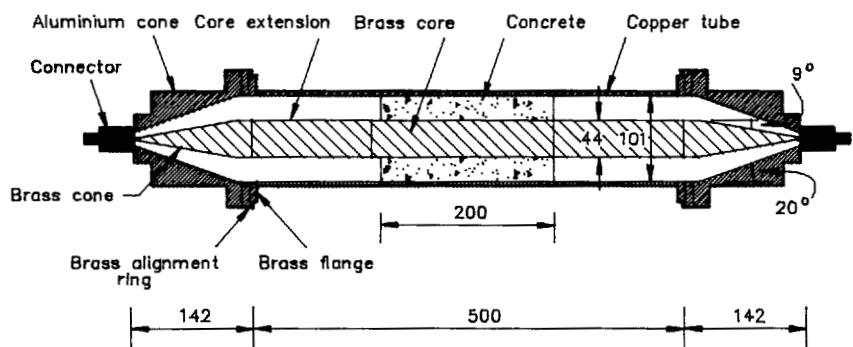


Fig. 8—100-mm-diameter transmission line details

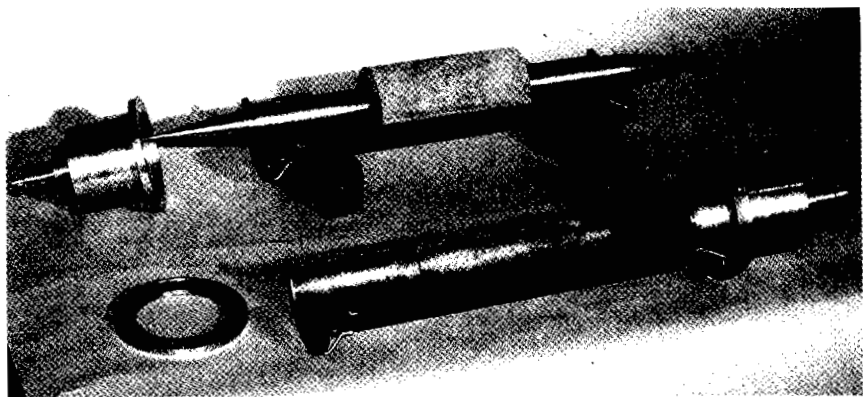


Fig. 9—Partially assembled 100-mm-diameter transmission line (photo)

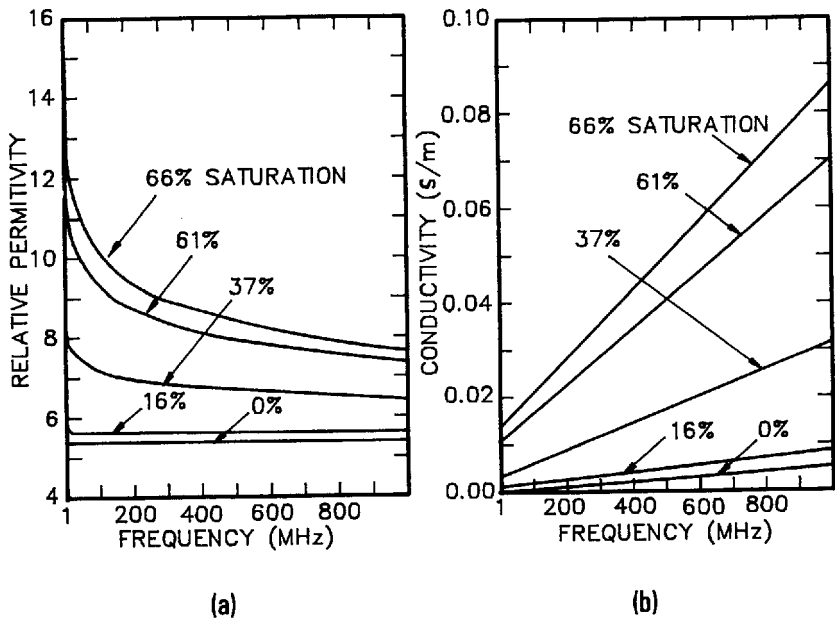
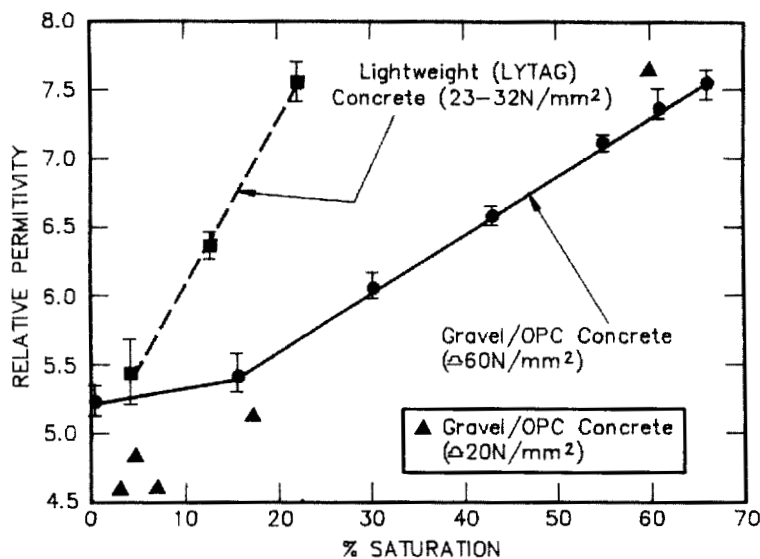
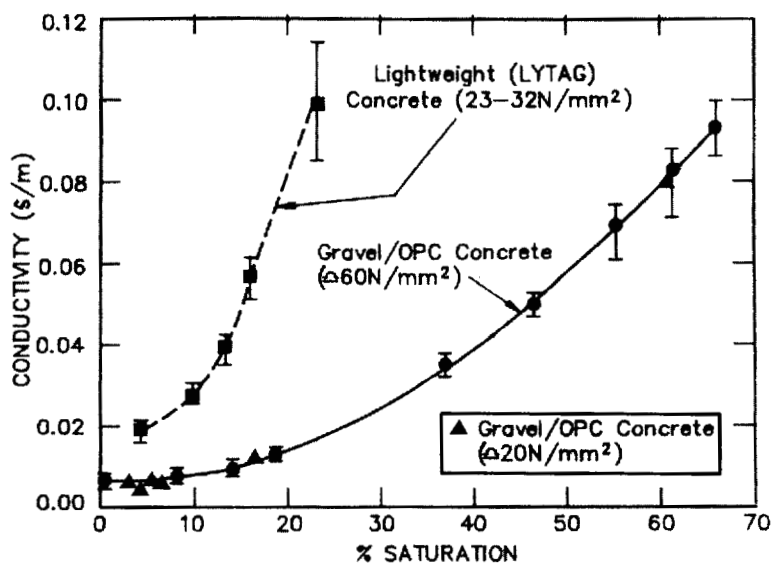


Fig. 10—Transmission line results for typical 60 N/mm² gravel concrete: a) relative permittivity; b) conductivity



(a)



(b)

Fig. 11—Transmission line results at 1 GHz for range of concretes: a) relative permittivity; b) conductivity

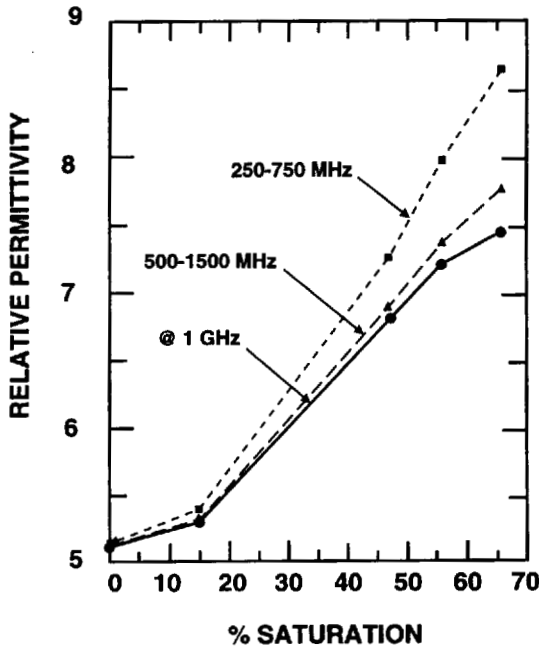


Fig. 12—Effects of frequency bandwidth upon transmission line results

SP 168-2

Nondestructive Testing of Bridge Decks Using Dual Frequency Radar

by Anthony B. SlatonBarker
and John W. Wallace

Synopsis: The paper describes a study of reinforced concrete bridge decks that addresses the need for an accurate evaluation method that is not hindered by an asphalt cover or extensive operator experience. The use of two antennas for Subsurface Interface Radar (SIR) surveys allows for more accurate evaluations by reducing false readings that may be associated with the use of a single antenna. A more accurate determination of the amount of deterioration is also achieved, because the different frequency antennas will detect different types and severities of defects. To assess the use of dual frequency radar for evaluation of bridge decks, both laboratory studies and field studies were conducted. The results of the research indicate that false readings are reduced by using dual frequency radar such that bridge deck repair jobs can be prioritized. The added information on the extent of deterioration also allows more accurate predictions of the time, effort and money necessary for bridge deck rehabilitation.

Keywords: bridge decks; field tests; nondestructive tests; reinforced concrete

26 SlatonBarker and Wallace

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INTRODUCTION

A fast, reliable and cost effective NDE method would be advantageous in areas with drastic weather changes or where road salts are used to distinguish between bridges that are unsafe (need to be replaced) and those that can be repaired/rehabilitated to extend their service life. It is also desirable to know accurately the extent of damage to a bridge deck, so that money and time are wasted by either repairing more of the deck than is necessary, or by finding out that more of the deck is damaged when rehabilitation begins.

In recent years, some states have been utilizing SIR to evaluate bridge decks. The use of SIR is inexpensive, relatively quick, reliable and has several advantages over current methods. Initial equipment outlays and training are important considerations for use of SIR; however, once these steps are taken, with SIR it only takes a few hours to survey an average two lane bridge deck. Previously, engineers have been able to obtain accurate results with SIR without removing the asphalt cover, and traffic need not be stopped during testing. SIR can also detect defects due to freeze-thaw cycles, fatigue, reinforcing bar corrosion, and salt intrusion, whereas some Non Destructive Evaluation (NDE) methods can detect only defects caused by corroding reinforcing steel. The standard bridge inspection methods used today by federal and state transportation departments are usually time consuming, labor intensive, and may not provide sufficient information about deterioration. The time and money required to investigate bridge safety with present methods can sometimes postpone the evaluation of a bridge that is unsafe, which can cost lives and money.

RELATED RESEARCH

The Transportation Research Board [1], [3], [4], [6], [8], the Virginia Highway and Transportation Research Council [2], ASCE (Urban Transportation Division) [5], and the Ontario Ministry of Transportation (to name a few) [7], have all performed research on the testing of bridges using

SIR. All of the researchers were able to accurately detect delaminations, but scaling and debonding were more difficult to detect. All of the investigations were done with a single antenna, and some false readings (defect detected where none existed) were encountered in all the tests. None of the tests were able to predict how extensive the deterioration was once a defect was located. Each study concluded that SIR is a viable option for quick and reliable bridge deck evaluations, but further investigations are necessary to improve data interpretation and to increase the reliability.

The evaluations of bridges (by other researchers) summarized below are very similar except for the methods used to interpret radar signals. The interpretation methods generally fall into three categories: (1) manual interpretation of the data using the graphic display method developed by Geophysical Survey Systems Inc. (GSSI) (as was done in this report), (2) manual interpretation of data using a topographic display, where signals are printed in a slightly overlapping fashion and variations in the signals are seen as blurred sections, and (3) computer interpretation of the signals.

Graphic Display Methods:

Clemena [1] conducted a study of overlaid and non-overlaid bridge decks. It was noted that reflection profiles from different decks, that were at similar states of deterioration, were markedly different. The decks were surveyed with SIR and then the asphalt cover was removed to compare the data with results from a chain drag test and a visual inspection. Some false readings were obtained with SIR; however, over half of these were correlated with cracks in the asphalt cover. The authors noted the overall performance of the radar to be good, but improved interpretation of the radar profiles and signatures for different types of bridges are needed.

Another study, by Clemena [2], used a 900 MHz antenna to survey 21 bridge decks, 16 of which were overlaid. Three occurrences were noted that caused difficulty in identifying defects: (1) if the concrete cover over the reinforcing bars was less than 50 mm, reflections from delaminations were sometimes masked by the reflection from the bottom of the overlay, (2) unwanted reflections existed from reverberations of the signal, between interfaces inside the deck, causing interference with the data, and (3) if the size of the delamination was smaller than 900 cm², the deterioration may not be detected. Reflection profiles for some decks were found to be more difficult to interpret depending on the design of the deck (reinforcement size and spacing, thickness of deck, thickness of asphalt cover and cover over the reinforcement). The use of radar was calculated to overestimate the damage by 16% and 5% for overlaid and clear decks, respectively. This overestimate of damage exemplifies the amount of false readings that are encountered with the radar surveys and the difficulties that exist in interpreting data.

28 SlatonBarker and Wallace

Topographic Display:

Alongi et al. [3] reported research carried out in the field on bridge decks and laboratory studies on concrete slabs with different size voids. The topographic display was used to find distress, but the distress could not be categorized into specific types of defects. The study concluded that the topographic display method could easily find distressed areas, but the procedure was very tedious and time consuming. The radar findings were accurate to 90% for both damaged and undamaged areas. Approximately 10% of their data consisted of false readings.

A study conducted by Steinway et al. [4] included laboratory testing of concrete slabs (reinforced and unreinforced), and field testing of a replica of a road with voids of different shapes and depths (debonding, scaling, delamination, moisture or salt contents were not simulated). The lab testing allowed gauging of the response of different size defects to aid in the interpretation of the data from the road replica. Some false readings were obtained during the field studies that could not be explained, although it was assumed the false readings were caused by spurious reflections from objects near the antenna. When the temperature was below 70° F (21° C) the computer processor was able to predict gap sizes with great accuracy; however, at higher temperatures, difficulties existed in the computer interpretation of the data due to increased moisture content in the concrete (attenuation).

Maser, and Roddis [5] reported the deterioration predictions for nine bridge decks. The predictions were made from a combination of radar, thermography and visual inspections. The radar and thermography data interpretations were made by using theoretical models. The model for the radar reflections took into account attenuation due to reinforcement, chloride content and the reflection coefficients at the various interfaces. The theoretical models explained the main features that were present in the field data, but drastically underestimated the amount of attenuation that existed in the field. By the combined use of all three methods, deterioration estimates were estimated to be within 5% of the actual damage. More bridge deck surveys were suggested to improve the reliability of the results.

Computer Analysis of Data:

Maser, and Infrasense Inc. [6] reported the results of radar surveys of 26 asphalt covered bridge decks. The results were compared to chain drag tests and visual inspection surveys that were completed after the asphalt was removed. The predictions of deterioration were based solely on the ratio of the reflection from the air/asphalt interface to the reflection from the asphalt/concrete interface. It should be noted this method identifies scaling and debonding directly, but can only infer delamination. The agreement with actual deterioration (chain drag and visual inspection) was about 80%. It

was noted that if the asphalt cover was less than 50 mm, or if it was a clear deck, this method could not be used. Many false readings were encountered in the data, but they were partially reduced by excluding defects that were only verified by a single radar return signal.

Chung et al. [7] presented a study that included an in-depth review of the radar data from one bridge deck survey. Delaminations were found with the chain drag and a sounding hammer for comparison to the radar data. Debonding and scaling were simulated on the deck prior to the application of an asphalt cover. Three types of deterioration were searched for: delamination, scaling, and debonding. Signal variations associated with each type of defect were recorded and then computer generated maps showing the locations of the defects were produced. The computer program provided good estimates of the actual defect locations, but many false readings were obtained while locating defects.

OBJECTIVES

The main problems encountered in the previous research include: 1) the difficulty in interpreting the radar signature on the graphic display, 2) the inability to distinguish the type, extent and location of defects (concrete, reinforcing bar or asphalt) with the topographic and graphic display method, and 3) the many false readings that were encountered for non overlaid and especially for asphalt covered bridge decks. Additionally, the above methods cannot predict how much damage has occurred once a defect is located, they only specify that some minimum amount of damage is present.

To address some of these concerns, a comprehensive laboratory and field study was undertaken in which dual frequency radar was used. A 300 MHz antenna, with greater ability to penetrate into the bridge deck, was used in combination with a 1000 MHz antenna, which could provide greater resolution in the critical region at the top of the bridge deck, to assess potential benefits of using dual frequency evaluation. The primary goals of the study were to: (1) reduce the number of false reading by using two different frequency antennas (300 MHz and 1 GHz), (2) more accurately assess the amount of damage that has occurred once a defect is located by comparisons of the two antenna responses, and (3) more accurately determine defect signatures by calibration of the antennas' response with lab specimens (bridge deck replicas).

The first goal of the research program, to reduce false readings, was considered because it could be used for "cursory" SIR evaluations. These are surveys that are performed on many different decks to determine which bridges need to be repaired and the priority of those repairs. Usually about one-quarter of the bridge deck is surveyed for these evaluations [6]. The areas where both antennas show a defect are used to estimate the approximate amount of damage in a deck. Since it is unlikely false readings will occur in both antennas at the same location, false readings are greatly

30 SlatonBarker and Wallace

reduced. These cursory surveys are used to decide if repair work is needed, and are not used to decide which areas of the deck will be repaired. These surveys can also be used to prioritize repair work at different bridges.

The second goal of the research program to accurately assess the amount and type of damage, will be applied to the "full" surveys. These evaluations are done once a bridge is identified as needing repair. SIR is used to survey the entire area of the deck. The type and amount of damage is then specified with the results from both antennas so the location and amount of repair work can be accurately determined.

The experimentation was carried out with the above goals in mind; more accurate comparisons of bridge deck damage and more specific information about the type and severity of damage. A discussion of the data display method and the theoretical basis of SIR and radar waves is found below. Then the results of the laboratory and field studies are presented.

PRINCIPLES OF SUBSURFACE INTERFACE RADAR (SIR)

Radar waves are electromagnetic waves that propagate through air at the speed of light. The way electromagnetic waves behave in other materials is governed by the material's dielectric and conductivity properties.

A typical bridge deck with two layers of reinforcing steel and the reflections R_i from different interfaces can be found in Fig. 1. Some older bridge decks include a 75 mm riding surface (with welded wire mesh) on top of the deck shown. The riding surface makes the return signals weaker, but it is still possible to conduct a radar survey.

A pulse is sent to the antenna from the control unit. The emitted radar pulse can be found in Fig. 2. The vertical scale is the intensity of the signal. The left negative peak is larger than the right due to the antenna-end effect [8]. The three peaks from an interface are represented as 3 bands on the graphic display. The antenna is typically positioned about 250 mm above the surface of the deck so the 1 GHz antenna is finished transmitting before reflections return to the antenna.

Any electromagnetic wave that encounters an interface between two materials of different dielectric constants will have part of its energy reflected from the interface, and part of its energy transmitted through the interface. At the air/concrete interface, approximately 55% of the signal strength is transmitted into the deck, and 45% will be reflected back to the antenna (R_1 , Fig. 1). The antenna will receive the signals that are reflected from an interface. Any defect in a bridge deck, such as; air, water, salt water or an air gap, will cause a new dielectric interface in the concrete. This interface will cause a reflection that the radar equipment records, then the type and severity of the defect can be estimated. Only defects that are perpendicular to the direction of travel of the radar pulse (in the plane of the bridge deck)

are detected.

The larger the difference in the dielectric properties between the two materials at an interface, the larger the reflection from that interface. The size of the reflection is governed by the following equation [4,9]:

$$P_{12} = \frac{\sqrt{\epsilon_{r1}} - \sqrt{\epsilon_{r2}}}{\sqrt{\epsilon_{r1}} + \sqrt{\epsilon_{r2}}} \quad (1)$$

where ϵ_{r1} and ϵ_{r2} are the relative (to air) dielectric constants for the materials 1 and 2 at the interface under consideration, and P_{12} is the fraction of the wave that is reflected at the interface of materials 1 and 2.

Although the dielectric constant of air is well known, constants for concrete, reinforced concrete, and asphalt should be determined experimentally. The dielectric constant of concrete and asphalt vary with moisture content, aggregate type, mineral content, and air-entraining agents; therefore, no constant value for these materials can be used [9]. For the study of bridge decks, where the depth to a given interface is known (namely the concrete/air interface at the bottom of the deck), it is possible to determine the dielectric constant experimentally, using the following equation [2, 10]:

$$\epsilon_r = \left(\frac{C}{V} \right)^2 \quad (2)$$

where V is the velocity of the electromagnetic wave in the material, C is the speed of light, and ϵ_r is the ratio of the dielectric constant of the material to that of air (relative dielectric constant). C is a known quantity and the velocity can be determined from the known thickness of the bridge deck and the travel time for the wave (from the antenna to the bottom of the deck and back). Reflection strengths for signals from a given interface can be determined once ϵ_r is known.

After the wave enters the concrete, traveling at approximately 125 mm/ns, it encounters the first mat of reinforcement. The reflection from the first mat is signified by R_2 in Fig. 1. The size of the reflection from a mat of reinforcement is usually small because it does not form a complete plane (spaces exist between reinforcing bars). The next interface that the transmitted wave encounters is the second layer of reinforcement. This reflection, depicted by R_3 , can be seen in Fig. 1. The same situation that occurred at the first layer of reinforcement repeats at the second layer. The only difference is that the signal that reaches the second mat is weaker, due to the attenuation and scattering of the radar wave caused by the concrete.

The last interface that the signal encounters in the bridge deck is the bottom of the concrete deck. Again, part of the signal is reflected and part is transmitted. The transmitted portion leaves the deck and will not interfere with the data analysis. The reflected signal, R_4 from Fig. 1, returns to the antenna, but will become weaker as it passes the two layers of reinforcement and the final concrete/air interface at the top of the deck. The portion of the signal that is reflected from any interface on the way back to the antenna is

32 SlatonBarker and Wallace

generally not strong enough to be detected by the antenna.

The reflection shown in Fig. 1 as r , is a reflection of a reflection, which is referred to as ringing or reverberation. When the R_1 signal enters the antenna to be recorded, part of it will be reflected back toward the bridge deck, because the antenna and air form a dielectric interface. This signal will then return to the concrete/air interface of the top of the deck and a portion of the signal is reflected back to the antenna. These random reflections can cause difficulties in interpretation, but usually they are small enough so that they do not interfere with interpretation of the signals.

DEFECTS FOR RC BRIDGE DECKS

In reinforced concrete decks with no asphalt cover, corrosion of reinforcement causes two types of defects: delamination at the first layer of steel and delamination or spalling at the second mat of steel. When delamination around the first layer of reinforcement develops, the gap will become filled with air, water or salt water. This gap has a much larger surface area perpendicular to the incoming radar wave than the reinforcing bar; therefore, even though the reflection coefficient for a concrete/air interface is less than that for a concrete/steel interface, the reflection from the delamination is stronger. A crack usually develops like the one in Fig. 3, and the radar wave "sees" a crack as two interfaces: a concrete/gap interface first, then as a gap/concrete interface as the signal reaches the bottom of the gap. For small gaps the reflections from the top and bottom will overlap and tend to reduce the amplitude of the reflection.

Two additional types of defects occur in bridge decks with an asphalt cover, they are; debonding, which is the lack of bond between the asphalt and the concrete, and scaling, which is debonding with concrete deterioration at the asphalt/concrete interface. Debonding allows air or water to become trapped in the gap. Debonding increases the deterioration rate of the deck due to traffic, freeze thaw cycles and other loads. This gap causes a larger reflection than that of a sound asphalt/concrete interface due to the larger difference in dielectric constants, Equation (1).

After a basic understanding of the theoretical application of radar was achieved, the experimental studies were conducted. The theoretical response of the radar waves to different interfaces, encountered in the lab and the field, were considered to assist in the interpretation of the data.

EXPERIMENTAL STUDIES

Laboratory calibration was conducted using three bridge deck replicas with plan dimensions of 54 in. x 43 in. (1.37 m x 1.1 m). Defects such as debonding, scaling, delamination, moisture levels, and salt content were all

simulated and tested in the lab. The specific signatures were then used to aid in the interpretation of data for both the cursory and full surveys. Two different frequency antennas were used because each frequency detects different types and depths of defects. After the laboratory studies were completed, the findings were used to interpret the SIR surveys of four different bridges; two with an asphalt cover and two without. They were surveyed with two different frequency antennas (300 MHz and 1 GHz). The data from both antennas was compared to equipotential tests performed by the New York State Department of Transportation (NYS DOT), visual inspections, cores taken from the bridge decks, and the calibration tests done in the lab.

The lab results showed that different frequency antennas respond to different types and severities of defects. For defects occurring in the top of the bridge deck, the 300 MHz antenna detected only more advanced stages of deterioration. The 1 GHz antenna detects any defects larger than about one square foot (0.093 m^2) that occur in the top portion of the deck, but it is difficult to ascertain the amount of damage that has occurred once a defect is found. By locating the areas where both antennas show a defect, the areas in the top of the deck where more advanced stages of deterioration exist can be determined.

False readings were extracted from comparisons between methods and more accurate SIR deterioration predictions were made. Important aspects of the laboratory and field studies are outlined in the following sections. Additional information on the study is available in reference [11].

Data Display Method

The equipment for this study consisted of components built by GSSI and a 12 volt DC power source, Fig. 4. The main piece of equipment is the control module and mainframe (model 4800). The control unit transmits power to the electronics and runs the transducer (antenna and transceiver). The control unit sends a signal to the transducer to be emitted by the antenna (transmitting phase), and then receives the reflection signals from the antenna (receiving phase). The return signal, either processed or unprocessed, is sent from the 4800 to the tape recorder, an oscilloscope and the video display unit (VDU-38) or a graphic recorder.

GSSI developed a unique way to display the SIR return signal so very minute changes can be observed. The signal in Fig. 5a is a typical signal received by the antenna from a good bridge deck. The areas where there are three consecutive large peaks signify reflections from interfaces. The second set of large peaks (middle of the signal) is from the bottom of the deck; any peaks after that are either reverberations of the signal from inside the deck or interfaces below the deck (such as the surface of the water under the deck or an abutment). If additional interfaces are encountered, more peaks will be visible in the signal. The line with different shades in Fig. 5b is the same

34 SlatonBarker and Wallace

signal as in Fig. 5a after it has been transformed by the VDU or graphic recorder. For both figures the vertical scale is time. The horizontal scale in Fig. 5a is amplitude. Figure 5b represents a single vertical line, but has been expanded for clarity. The signal in Fig. 5a is converted to Fig. 5b by rotation of the signal 90 degrees about its vertical axis and then color or gray coding the peaks and zero crossings. For a simple black-and-white output, any large peaks are displayed as black and any zero crossings are displayed as white (or peaks in white and zero crossings as white). Other portions of the signal are printed shades of gray in proportion to their strength. This display method allows hundreds of signals to be printed on one screen, because each signal is represented by a single vertical line.

Laboratory Experiments

The research that was conducted in the lab was performed on three slabs. Slab A was not reinforced, whereas slabs B and C were similar to actual bridge decks that were tested in the field. The plain concrete slab was built so the affect of reinforcement on the radar data could be determined. The other slabs, B and C, were built similarly to the two types of decks explained below (thick and thin). All of the slabs were then tested under varying conditions and with simulated defects.

Slab A: Slab A contained no reinforcement, and was approximately 7 in. (180 mm) thick. No defects were present in the slab and it acted as a control specimen, Fig. 6. The only variations that were considered with the control slab were to test the slab with the surface saturated with water, and to simulate deteriorated concrete on the underside.

Thick Deck, Slab B: This category includes bridges, 30 to 60 years old. They typically consist of a structural layer of reinforced concrete approximately 8 in. (203 mm) thick with a reinforced concrete riding surface, 3 in. (76 mm) thick. The top layer of concrete (riding surface), is a less porous concrete mix than the structural layer, to help keep water and salt from penetrating into the deck.

Thin Decks, Slab C: The second type of deck is used in newer bridges, generally less than 30 years old. These decks are typically 8 in. (203 mm) thick with two layers of reinforcing bar. There is no riding surface placed over the single concrete layer; it is used as is until repair is needed. Both deck designs are usually covered with 0.75 to 3 in. (19 to 76 mm) of asphalt when the riding surface becomes rough, potholed or pitted.

Detailed Description of Laboratory Specimens

Slab B was very similar to the South Colton and Colton bridges across the Raquette river in St. Lawrence County, NY. The slab contained

longitudinal and transverse reinforcing steel of approximately the same size (US #5; 16 mm diameter) and spacing (4.33 in.; 110 mm), as was in the actual bridge decks. The slab was built with two voids that were used to simulate defects. The first void was a hollow cylinder near the bottom of the deck, 2.5 in. (64 mm) in diameter, see Fig. 7. This cylinder could be filled with different materials (water, salt water, gravel and a metal pipe) to evaluate the sensitivity of the two antennas. When the pipe hole was filled with gravel/wet gravel it simulates concrete deterioration around a single reinforcing bar. The slab was also tested with the cylinder open (air filled). A concrete cylinder was cast that could fill the hole except for a small air gap, which simulates the start of a delamination.

The slab also contained a 30 in. x 2 in. (762 mm x 51 mm) gap in the riding surface, see Fig. 7. The gap was also at the same depth as the first layer of reinforcing bar in thin decks. The gap ran the full transverse length of the slab. Twelve 10 in. x 0.75 in. x 22 in. (254 mm x 19 mm x 560 mm) rectangular concrete sheets were constructed to fill the gap and replicate multiple types and sizes of deterioration. The sheets had rough surfaces, so when they were placed together small gaps were created, similar to a delamination. Placing the sheets underneath slabs A and B simulated either deterioration of the bottom of a deck or spalling, depending on the orientation.

Slab C was similar to the Philadelphia and Chippewa Creek bridges in Jefferson County, NY. Both of these decks were similar to the Thin decks explained above. A hollow cylinder was cast into this slab as well, but it was only 2.5 in. (64 mm) in diameter, Fig. 8. The cylinder was used in the same way as it was in slab B; filled with different materials and the variations of the radar signals were recorded. The lack of the welded wire mesh and the riding surface was also studied by comparing results between slabs B and C.

Results of Laboratory Tests

The tests were of two types, moving and stationary. The stationary tests showed the exact signal received from a certain defect. The signal was displayed on the oscilloscope and the graphic recorder and the variations in the signal due to different defects were noted.

The main purpose for the lab tests was to find the different deck signatures for each antenna and to determine how they change for given defects (the signatures can be found in Fig. 9). These signatures were then used to interpret the SIR data from actual bridge deck surveys. The ways the signatures were affected by gaps of differing sizes; pipe holes filled with water air or gravel; stringers; damaged concrete in the deck (wet/dry); and damaged concrete at the base of the deck (wet/dry) are summarized in Table (1), and discussed in the following sections; (1) 1 GHz Antenna - Stationary, (2) 300 MHz Antenna - Stationary, (3) 1 GHz Antenna - moving and (4) Defect Signatures.

36 SlatonBarker and Wallace

(1) 1 GHz Antenna - Stationary: The 1 GHz antenna was positioned above the slab such that the antenna received signals reflected from approximately a 1 ft² (0.093 m²) area of the bridge deck. The pulse width of the antenna is short (1ns), so signals from interfaces less than 2.5 in. (64 mm) apart will overlap in the graphic display. The higher frequency of the antenna also reduces the ability of the 1 GHz radar wave to penetrate concrete. The 1 GHz antenna can penetrate only about 6 in. (150 mm) of reinforced concrete. The results of the stationary tests are reported in Table 1.

The pipe hole in slab C was detected more easily than the pipe hole in slab B, mainly because of the simulated concrete damage that was present in the bottom of the riding surface in slab B. The pipe hole was discernible for slab B, but it could easily be missed. The pipe hole gave no reflection when the damaged concrete at the riding surface/structural concrete interface was saturated with water. When the damaged concrete in slab B was soaked with water (usually in the field damaged concrete is damp) reflections from below the damaged area (deep signals) were weaker compared to the dry damaged concrete. The weaker deep signals were caused by the larger loss in signal strength at the concrete/damaged concrete interface (the presence of water increases the reflection coefficient for an interface, therefore accounting for the weaker deep signals). The saturated damaged concrete also results in more attenuation of the signal.

When the pipe hole was filled with water, rather than air it gave a slightly larger return signal through the damaged concrete, but it was still difficult to discern. As expected, the added attenuation caused by damaged concrete makes interpretation of the reflections from the middle and bottom of the bridge deck difficult with the 1 GHz antenna. When there is no damaged concrete above the pipe hole, as in slab C, the opening was easily detected. The reflection signal was larger when the pipe hole was filled with water.

Damaged concrete under the slab was not simulated with slab B because the reflection from the bottom of the deck was already so weak that additional deeper reflections would have been too small to yield meaningful information. Saturated (damaged) concrete underneath slab C could not be detected with the 1 GHz antenna due to attenuation; therefore, defects such as spalling and delamination of the bottom mat of reinforcement, are difficult to detect.

Based on the lab studies the 1 GHz antenna could easily locate defects that were present in the top portion of a bridge deck. In addition, saturated defects could be detected that occurred at depths up to 130 mm. However, dry defects in the middle to bottom portion of the deck were difficult to recognize. In general the 1 GHz antenna is more reliable at detecting delaminations at the top of the slab or at the asphalt/concrete interface.

(2) 300 MHz Antenna - Stationary: The 300 MHz antenna is much larger than the 1 GHz, approximately 77 cm by 64 cm (field of view 0.5 m²). This gives the 300 MHz antenna a much larger field of view; however, smaller

defects can be more easily missed. The lower frequency allows the radar waves to more easily penetrate the concrete and identify defects deeper in the slab. Because the pulse of the 300 MHz antenna has a long duration (5 ns), any reflections from gaps are invariably overlapped by other reflections because the total time spent in the concrete deck is approximately 6 to 10 ns (depending on the thickness of the deck). This overlapping of the signals makes it difficult to identify specific reflections from interfaces, thus, variations in the overall signal pattern are searched for in the data as opposed to locating specific peaks, which was done with the 1 GHz antenna. The results of the 300 MHz lab tests can be found in Table 1.

With the 300 MHz antenna it was not possible to detect variations between sizes of gaps, but there were more zero crossings in the signal when gaps existed. The deep signals were also weaker when the signals passed through damaged concrete or gaps. In general damaged concrete resulted in weaker deep signals as compared to gaps.

The pipe hole was not visible in either slab B or C, whether it was filled with water or air. The surface area of the pipe hole perpendicular to the antenna is too small to be readily identified. When the pipe hole was filled with a metal pipe, a slight variation was seen; however, it was difficult to discern with the levels of background noise that exist in the system.

When the damaged concrete in slab C was soaked, a larger reflection appeared at approximately 4 ns in the return signal. The added size of the reflection coefficient due to the water was large enough to allow detection of the damaged concrete in the top portion of the slab; therefore, delaminations that had water at the interface could be detected. The deep signals, from the bottom of the deck, were much weaker when they had passed through the saturated damaged concrete, as opposed to the dry damaged concrete. This weakening of the signal aided in the identification of delaminations in the field data.

The damaged concrete on the bottom of the slabs affected the waveform by adding a stronger reflection from the bottom of the deck. It was not possible to differentiate between 25 to 50 mm of damaged concrete under the deck, but its presence could be recognized. When the damaged concrete under the deck was saturated, a larger reflection was received as compared to the dry concrete. The signals from the bottom of slabs B and C were increased similarly from the dry to saturated case, but the signals from the bottom of slab B were slightly weaker due to the attenuation caused by the damaged concrete in the top of that slab.

From the lab studies it was determined that the 300 MHz antenna was effective at identifying defects that occurred in the bottom portion of a bridge deck. In addition any saturated defects that occurred in the top half of the deck could be recognized. Dry defects in the top of the slab and damage at the asphalt/concrete interface could not be identified with accuracy. The 300 MHz is most suited to identifying large ($>$ than 0.25 m^2) saturated deteriorated sections in the top half of the deck or spalling and delamination

38 SlatonBarker and Wallace

at the bottom mat of reinforcement.

(3) 1 GHz Antenna - moving: Moving simulations were performed only with the 1 GHz antenna because the 300 MHz antenna could not be moved across the slabs without getting interference from the sides of the slab.

When the 1 GHz antenna was moved across slab B, with the damaged concrete in the top portion of the deck, there was considerable distortion of the signal from 2 to 3.5 ns, and some distortion occurred around 6 ns (the reflection from bottom of deck). As was seen in the stationary case, the deeper signals were weaker after having been scattered and attenuated by the damaged concrete. When the damaged concrete was saturated, the reflections from the damaged concrete were stronger, but the deeper signals were weaker as compared with the dry case.

For the simulations of gaps in the top portion of slab B, reflections from the gaps were not as disrupted as the reflections from the damaged concrete. As the antenna passed over the gaps the peaks from the bottom of the deck moved upward slightly (earlier reception time), but there was very little distortion of the signal. As the gaps became larger the reflections from the gaps became clearer in the graphic display. This made it easy to distinguish gaps from damaged concrete.

Reflections from other defects simulated (gaps of different sizes and the pipe hole filled with different materials) were similar to the static cases. As the antenna passed from sound concrete to the damaged section, the signal changed from the reflection signature for sound concrete to the reflection signature for the damaged section found in the static case. As the speed at which the antenna passed over the defect increased, the change in the signal took place in a shorter horizontal distance in the graphic display.

(4) Defect Signatures: Three main signatures were determined from the graphic output for non-overlaid bridge decks, Fig. 9. The first is the signature for the 1 GHz antenna from damaged concrete in the top of the deck (a delamination). This signature appeared as a delay in the return time of the peaks (bands) at 3 ns. In the graphic display this delay appears as a depression in the bands, Fig. 9a. The depression signature was also reported previously by Clemena [2]. The reflection signals intensity also increased in strength. The signature could also be verified by a weakening of the signal from the bottom of the deck.

The second signature is from a saturated delamination at the top mat of reinforcement for the 300 MHz antenna. This signature was identified by an addition of one or two thin bands on the graphic display around 3 ns, Fig. 9b. The second band occurred at approximately the same return time as the reflection from the bottom of the deck.

The last signature for the 300 MHz is for damaged concrete at the bottom of the deck, Fig. 9c. Because the pulse for the 300 MHz is 5 ns long, the reflection from the bottom of the deck, and the reflection from the top

map of reinforcement overlap. This overlapping made the signature for the bottom defects difficult to discern from the signature for damage in the top of the deck. The signatures for both types of damage were reported together and were differentiated later by comparison with the 1 GHz antenna (which cannot detect delamination in the bottom of the deck). The signal for spalling or delamination at the bottom mat of reinforcement was displayed as an earlier return for the signal from the bottom of the deck. This signature would also be obscured when there was a large defect in the top of the deck.

Signatures for delaminations in the top of the slab were identified for both antennas, and a signature for delamination in the bottom of the deck for the 300 MHz antenna was also obtained. The signatures obtained could be readily applied to the field data to ascertain which signals were from each specific type of defect in a bridge deck. Because the signatures for defects in the bottom of the deck and top of the deck, for the 300 MHz, were difficult to discern from each other (Fig. 9b and Fig. 9c), they were reported together. Once comparisons with the 1 GHz antenna were made, the areas that were deterioration at the bottom of the deck could be noted. More lab testing should be done that allows for more detailed descriptions of the signatures for top and bottom defects with the 300 MHz antenna. Then the defects can be recognized without comparison to the 1 GHz antenna data.

Field Studies

The field studies consisted of surveying four different bridge decks. The decks were of two types: no asphalt (removed by NYS DOT for the equipotential test) and asphalt covered decks. For the decks without an asphalt wearing surface, the SIR results were compared to results obtained by visual inspection as well as to results obtained from equipotential tests conducted by the NYS DOT. The comparisons to the equipotential tests provide a convenient reference state to report numerical results from which to assess the reliability of the SIR evaluation. It is noted that visual inspections were conducted in conjunction with the equipotential tests such that the results reported as "equipotential tests" are deemed somewhat more reliable than "blind" equipotential test results. For the decks with an asphalt cover, the SIR results were compared to visual inspections of the asphalt and the underside of the deck, as well as to results from cores taken at specific locations.

Chippewa and Philadelphia Bridges: The three span Philadelphia bridge and the one span Chippewa bridge were both similar to the think deck laboratory specimen. Both bridges were located in areas where the road was being repaired. The bridges were of approximately equal age and rehabilitation history (asphalt covered at similar times). Both bridges appeared to be in good shape from visual inspections of the underside.

Once the asphalt was removed, the Philadelphia bridge had many deteriorated sections of concrete and the top mat of reinforcement was visible

40 SlatonBarker and Wallace

in some locations. Many small cracks were also visible on the deck surface. The edge of the deck where runoff occurs had extensive deterioration, with the concrete resembling sand and gravel for the first 10 to 20 mm of depth. The joints on the deck also had large sections of deterioration. The poor shape of the deck was partially attributed to the lack of a waterproofing membrane between the concrete and the asphalt. The asphalt was removed with a milling machine and 5 to 20 mm of asphalt remained attached to the concrete surface over large areas located randomly around the deck.

After the asphalt was removed, a visual inspection of the Chippewa bridge deck indicated that it was in good shape. A few sections of deteriorated concrete existed near the side where runoff occurred, and some small cracks were also present. The bridge had a bituminous waterproofing membrane between the concrete and the asphalt cover and it appeared to protect the top portion of the concrete from water and salt penetration. The asphalt cover was removed with a back hoe, and since a membrane existed between the concrete and asphalt no sections of asphalt were adhered to the surface of the deck during the SIR survey.

Both decks (Chippewa and Philadelphia) appeared to be in the same condition from the underside surveys, but once the asphalt layer was removed the Philadelphia bridge was seen to be in a more advanced state of deterioration. The difference between the two decks exemplifies the problems with visual inspections: both showed similar signs of corrosion from an underside inspection, but far more deterioration existed in the Philadelphia bridge deck.

The equipotential maps of the Philadelphia and Chippewa bridge decks can be found in Fig. 10 and 11, respectively. Only the midpoint between the tire paths and the inner tire path for the Chippewa bridge are reported. The drainage grates caused extreme interference in the graphic display for the outer tire path. The deterioration detected by each antenna is also found in these figures and is summarized in Table 2. Because the north lane of the Philadelphia bridge had only two exposed reinforcing bars, as opposed to 22 for the South lane, the comparisons of the tests are reported separately in the table.

The "Predicted Damaged (%)" in Table 2, is the percent of the bridge deck surveyed by SIR, that the specified test method reports as damaged. The "Agreement (%)" values are the percent of the damaged area specified by the second test that is verified by the first test, see Equation (3). For Table 2 and Equation (3) "A and B" refers to locations on the deck where both tests predict damage, and "A or B" refers to any area of the deck where at least one of the two tests signifies a defect.

When combining all defect predictions from both antennas and comparing that to the damaged area predicted by the equipotential test (Table 2 - (300 MHz or 1 GHz)/Equipotential), good agreement is obtained. The antennas detect 92 to 95% of the areas specified as damaged by the equipotential test and visual inspections. The findings indicate that

reasonable results can be obtained by combining results of equipotential tests with visual inspection; however, there are several advantages to using dual frequency SIR to estimate the damage present in a bridge deck. These advantages include: (1) the asphalt cover does not have to be removed to conduct a radar survey, (2) the traffic on the bridge does not have to be stopped to run the SIR test, (3) the SIR data allows the type of damage that has occurred to be specified (delamination or excessive delamination at top mat of reinforcement, or damage at the bottom layer of reinforcement), and (4) the degree of subjectivity that results from the visual inspection is eliminated. A portion of the bridge must also be closed while the asphalt is removed and again when the equipotential test is conducted. The data from the equipotential test does not detect damage caused by freeze/thaw cycles or fatigue, nor does it specify the amount of damage that has occurred. The equipotential test assumes a large area of damage from a single reading, so many areas of deterioration are not missed, but areas where no damage has occurred are included. This explains why the equipotential tests and SIR predictions are similar even though the SIR tests pick up more types of damage.

The comparisons of the data in Table 2 verified three observations: (1) some defects are more readily recognized by one of the antennas than the other, (2) that dry decks reduce the ability of the 300 MHz to detect defects in the top of the deck, and (3) that by combining the damage predictions from both antennas, over 92% of the damaged concrete is readily located (compared with equipotential tests and visual inspection). These three occurrences are explained in detail below.

(1) From Table 2, the correlation between the 300 MHz and the 1 GHz surveys can be found to average about 50% (see 300 MHz/1 GHz and 1 GHz/300 MHz comparisons). This verifies what was found in the lab; that the antennas detect some of the same defects, but will also find defects that are invisible to the other antenna. Although many of the damaged areas would be missed for a cursory survey, the defects identified by both antennas have a much lower probability of being false because it is unlikely both antennas will report a false reading at the same location. The low correlation values for the equipotential test and the individual antennas is partially due to the ability of the radar to detect defects such as delamination at the bottom mat of reinforcement, or freeze/thaw damage, which the equipotential test is unable to detect. This is verified by the correlation between the equipotential test and each individual antenna; 69/77% South lane, 71/57% North lane, and 82/75% Chippewa (see 300 MHz/Equipotential and 1 GHz/Equipotential). The antennas are detecting defects that are not included in the equipotential damage estimate.

(2) The surface of the south lane of the Philadelphia bridge had areas of exposed reinforcing bars which allowed the delaminations to dry out, making them very difficult for the 300 MHz to detect (as discovered in the lab tests, defects were more easily detected when moist or saturated as opposed to dry). The correlation for the 300 MHz and the 1 GHz for the south lane was 43/37%. The north lane of the Philadelphia bridge and Chippewa decks had relatively few exposed reinforcing bars, so delaminations would have had

42 SlatonBarker and Wallace

trapped water (making them more easily detected by the 300 MHz antenna). For the Chippewa bridge the asphalt cover was removed only a few hours before conducting the radar survey, so there was no time for the defects to dry out. The much higher correlation for the Philadelphia north lane, 77/52%, and the Chippewa deck, 52/51% between the 300 MHz and 1 GHz, exemplifies that less of the damaged concrete is missed by the 300 MHz antenna for wet decks. The 300 MHz data correlates better with the 1 GHz (which is not hindered by dry defects) when the deck has trapped moisture in it. It appears the use of two antennas could be most advantageous on bridges that have almost all of the top cover of concrete still intact and have trapped moisture in the deck. This is a promising result because bridge decks of this type appear to be in good shape from visual inspections, and the reliability of other NDE tests is low for these decks [6].

(3) The SIR's ability to recognize a large percentage of the defects was verified for the uncovered decks (compared with equipotential), but many of the correlation's between the individual antennas and the equipotential tests were quite low. The milling process that was used to remove the asphalt cover on the Philadelphia deck left many areas of asphalt bonded to the concrete surface. The areas with bonded asphalt may account for the poor correlation between the individual antennas and the equipotential test. The sections of asphalt on the north and south lanes would cause a change in the radar output because the initial air/concrete interface would be an air/asphalt interface (which has a different reflection coefficient). The presence of the added asphalt would also increase the depth to the reinforcing bar, and add an additional interface, asphalt/concrete, which would reduce the strength of the return signal. The Equipotential/(300 MHz and 1 GHz) comparison are a verification of the damage estimates for the cursory surveys. The agreement is much higher for the Chippewa bridge (90%) which had no asphalt attached to the surface, as opposed to 56% and 72% for the Philadelphia deck. A test while the asphalt cover was intact would have been advantageous, so the number of interfaces would not change during the survey.

Visual inspections, Cores and SIR comparisons, S. Colton and Colton Bridges: The decks for the Colton and South Colton bridges were tested with the asphalt cover still on the deck. Because the bridges were going to be replaced in the next two years, the NYS DOT allowed cores to be taken from the decks. Areas where only one antenna showed a defect and areas where both antennas signified a defect were cored to determine the types of defects identified.

Both bridges are approximately sixty years old and other than small patching jobs, no large scale repair of the decks has been done. The major difference between the two bridges is the span, thickness of asphalt cover, and the speed limit. The concrete designs of the two decks were identical and both decks have received multiple asphalt covers as the concrete surface deteriorated. No waterproofing membranes were placed between the asphalt and concrete or between layers of asphalt.

The asphalt cover on the South Colton deck was in good shape with only a few small (2 mm) cracks near the edges. The underside of the deck

indicated substantial deterioration with areas of 1 to 2 square meters of spalled concrete where the bottom layer of reinforcing bars were visible. The reinforcing bars were severely corroded and had many white patches of deicing chemicals surrounding them. Large cracks were also present on the underside, some up to 2 meters long and about a cm wide. Most of these cracks had stalactites of salt and deicing chemicals protruding from them, ranging from 10 to 100 mm long. About 70% of the underside had visual signs of deterioration.

The Colton deck was also in bad shape, but in different ways than the South Colton deck. The asphalt cover on the Colton deck was extensively deteriorated with large cracks present over 60% of the wearing surface, allowing water to penetrate to the concrete surface resulting in damage to the deck. The underside of the deck had a few areas where the concrete was spalled. Where visible, the reinforcing bars had undergone considerable corrosion. Cracks, iron oxide patches and deicing chemicals were present, but not to the extent present at the South Colton deck.

The comparisons between the visual inspections of the asphalt surface, underside survey, 300 MHz antenna, and 1 GHz antenna, can be found in Table 3. Figure 12 contains the radar data for one tire path on the Colton deck. The data for the South Colton deck showed damage in over 95% of the deck for each antenna, so no informative comparisons could be made.

Good agreement was obtained for the antenna results and the visual inspections of the underside and wearing surface. Only 16% of the 300 MHz defect readings were not verified by rust stains or cracks on the underside of the deck. Since a delamination can occur before visual signs of damage occur, the inability to verify the 300 MHz detection does not mean it is a false reading. Only 4% of the 1 GHz defects were not verified by damage on the underside. It appears more false readings exist for the 300 MHz antenna than the 1 GHz antenna.

The 1 GHz survey was also compared with a visual inspection of the asphalt wearing surface. For the 1 GHz antenna, 16% of the damage predictions were unable to be verified with deterioration of the asphalt cover. Extensive deterioration in the asphalt cover would allow water to penetrate to the concrete deck and cause freeze/thaw damage. Although the cracks are not proof of damage at the concrete, there is a correlation between damage in the wearing surface and 1 GHz SIR damage predictions. If all the areas not verified by top or bottom visual inspections for the 1 GHz (surface or underside) were false, the accuracy would be 92% for the 1 GHz, which is approximately the accuracy expected from an equipotential test. As was found with the non-overlaid bridge decks the correlation between the two antennas was around 50%. Again some types of defects were detected by both antennas and others were only visible with one of the antennas.

Three occurrences were determined by coring. (1) The first core type consisted of 70 mm of concrete with many damaged layers (freeze/thaw damage). The 1 GHz "sees" the damaged layers as many interfaces and the

44 SlatonBarker and Wallace

signals cancel each other out. In this situation the thick wet interface will be picked up by the 300 MHz antenna, but not by the 1 GHz antenna. The longer pulse width of the 300 MHz reduces the amount of overlap and canceling in the signal, allowing the defect to be detected. (2) In the second core type, debonding existed which was picked up by the 1 GHz antenna, but not by the 300 MHz antenna. If the structural concrete is in good shape, the interface due to debonding is too thin to be picked up by the 300 MHz antenna. (3) The third core type was for wet damaged concrete (not in layers), both antennas would identify this damage. The cursory survey, where only defects detected by both antennas are considered, would not be totally accurate for older bridge decks due to (1) and (2). More tests are needed to determine if advanced deterioration can occur for bridge decks without visual signs of damage on the underside. If the advanced deterioration does not occur in decks that show no visible signs of damage, the accuracy of the cursory survey would not be reduced by the inability of the 1 GHz antenna to pick up layered damage in the top of the deck.

Because the 300 MHz does not detect debonding or early stages of scaling, cursory comparisons will not differentiate between decks with no defects and decks with debonding. Although this is not optimal, the decks with only debonding are not a safety hazard. Bridges that have more advanced deterioration, which impacts structural integrity, would be identified.

CONCLUSIONS

Laboratory studies indicated that the use of two antennas with different frequencies increased the range of defects that can be detected. Specific signatures for delaminations at both mats of reinforcement with the 300 MHz antenna and for damage in the top of the deck for the 1 GHz antenna were obtained. These signatures were then used in analyzing the data from four bridge deck surveys. The signatures were verified in the field by coring, visual inspections, and equipotential tests.

The use of dual frequency SIR allows more accurate assessment of bridge deck deterioration. Types of deterioration other than that caused by corroding reinforcing bar can be detected. The extent of the deterioration can also be more accurately determined with SIR. This information can be obtained without removing the asphalt cover or stopping traffic on the bridge, which makes SIR more convenient and cost effective.

The drawbacks to SIR include the initial cost of the equipment and the presence of false readings that are sometimes encountered. But the false readings can be reduced appreciably for cursory surveys.

For a cursory SIR survey, any damaged area that was detected by only one antenna is assumed to be a false reading. Although some areas were excluded that are actual defects, a large percent of the false readings will be