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Various non-destructive tests for infrastructures in JAPAN

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Proper maintenance of concrete structures requires close inspection and accurate understanding of the current condition of the structures. Visual observation is the most common means of inspection. However, some suitable non-destructive inspection will be needed in addition to visual inspection, especially for evaluating important factors which are deeply related to safety or durability of structures. This paper describes various non-destructive testing techniques recently put in use in the field of road or railway bridges and provides inspection methods using these techniques applicable to concrete quality evaluation, detection of cracks, voids, spalling or other defects in concrete, and evaluation of sheath grouting which is specifically important for prestressed concrete structures.

Key words: Concrete, Inspection, Visual observation, Durability, Non-destructive testing.

Introduction

Proper maintenance of concrete structures requires close inspection and accurate understanding of the current condition of the structures. Visual observation is the most common means of inspection at present but can provide only limited information which is sometimes insufficient for making diagnosis. Combined use of visual inspection with some suitable non-destructive inspection will be needed, especially for evaluating important factors which are deeply related to safety or durability of structures.

Although concrete structures can be divided into reinforced concrete (hereinafter referred to as “RC”) structures and prestressed concrete (hereinafter referred to as “PC”) structures, factors to be evaluated by non-destructive examination are basically the same. Major basic factors common to RC and PC structures are as follows: (1) concrete quality evaluation; (2) detection of cracks, voids, spalling or other defects in concrete; and (3) steel location and corrosion evaluation. Important factors inherent to PC structures are as follows: (4) sheath grouting evaluation; and (5) evaluation of breaking of steel tendons.

This paper describes various non-destructive testing techniques recently put in use in the field of road or railway bridges and provides information about available types, principles, research status and actual application cases of non-destructive inspection methods applicable to above mentioned common factors (1) and (2) and factor (4) inherent to PC structures.

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Damage extent Determination for voids and Rock Pockets

Outline

One of the well-known non-destructive testing techniques used for evaluation of internal quality of concrete is the elastic wave method using ultrasonic waves. Previous reports have shown that propagation velocity of ultrasonic waves penetrating the concrete has some extent of correlation with compressive strength or elastic modulus of concrete. Researches are under way to establish a method for determining the damage extent in concrete structures by tomographic analysis and visualization of internal defect locations, using measured ultrasonic wave propagation velocity.

Tomographic analysis method

The tomography-based analysis method consists mainly of two analysis techniques. One is a forward or direct analysis which simulates a propagation path of an elastic wave from a transmission point to a reception point and determines the travel time by calculation. The other is an inversion analysis which corrects the elastic wave propagation velocity model to obtain the optimal velocity by making the difference between the calculated and measured propagation times (remainder) minimum.

In the current study, the authors used back projection technique for forward analysis and simultaneous iterative reconstruction technique for inversion analysis. Details of each technique are described below.

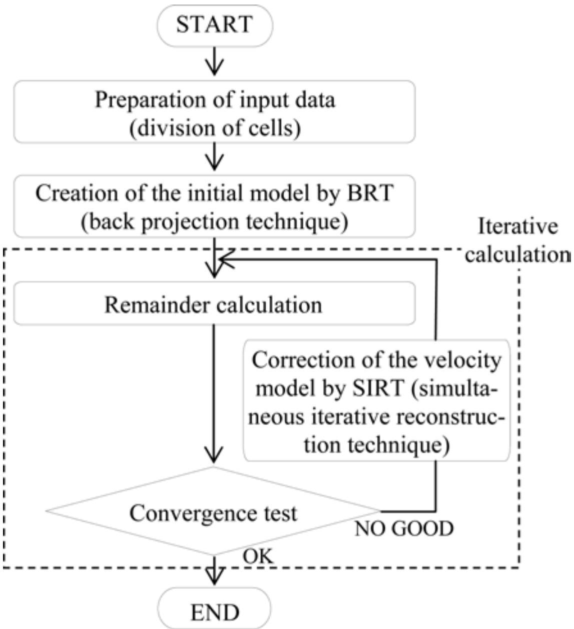


Fig. 1. Ultrasonic tomographic analysis flow.

Back projection technique

Velocity V_j for the *j*-th cell weighted by the length of each ray traveling though the cell is obtained by Equation 1:

$$V_j = \frac{\sum(l_{ij}V_i)}{\sum l_{ij}} \tag{1}$$

where *lij*: length of the *i*-th ray traveling through the *j*-th cell.

Simultaneous iterative reconstruction technique

Errors between the measured ultrasonic wave propagation velocity and that calculated by the back projection technique for each cell are correct by using simultaneous iterative reconstruction technique. This technique obtains a correction amount of propagation time directly from the remainder and the ray length traveling through each cell. Propagation time correction amount *É¢S* for each cell is obtained by Equation 2:

$$\begin{bmatrix}\Delta S_1 \\ \dots \\ \Delta S_n\end{bmatrix} = \begin{bmatrix}\sum_{i=1}^{m}(l_{i1}OS_i)/\sum_{i=1}^{m}(l_{i1}) \\ \dots \\ \sum_{i=1}^{m}(l_{in}OS_i)/\sum_{i=1}^{m}(l_{in})\end{bmatrix} \tag{2}$$

where *OSi*: remainder of the *i*-th ray divided by the total ray length; and *m*: the number of measurement points in the cell.

Fig. 1 shows the flow chart of ultrasonic tomographic analysis used in this study.

Verification experiment using test pieces

Verification experiment for the tomographic method was carried out, using two models of rectangular test pieces shown in Fig. 2. Both models were made to the same dimensions: 500 mm wide × 500 mm long x 400 mm high. All test pieces had been exposed to natural environment for six years. The internal defect model had porous concrete parts in it to simulate rock pockets (internal defects).

Measurement points were placed at 9 points on each side at 50 mm intervals at the middle height of the test piece, starting from the 50 mm point from the edge, and measurement was taken from each point to individual points on the other three sides radially. This makes 486 measurement lines in total per test piece. The horizontal cross section of the test piece was divided into 15 elements both vertically and horizontally for analysis. Propagation velocity difference was classified and color-coded into 25 levels, with 4500 m/s as the maximum and 3500 m/s as the minimum. Fig. 3 shows the locations of the measurement points.

Analysis results

Fig. 2 shows the measurement results. The control

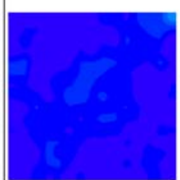
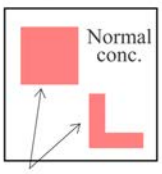
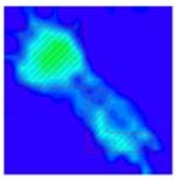
Model	Description	Schematic	Appearance	Tomography analysis result	
Control model	Dense concrete without defects	 Normal concrete			Propagation velocity is evenly distributed across the plane. 
Internal defect model	• A small defect on one side (approx. 14 x 14 cm) • A thin defect (approx. 5 cm wide)	 Normal conc. Porous concrete			The defects appear as the reduced propagation velocities. 

Fig. 2. Tomographic analysis results with the test pieces.

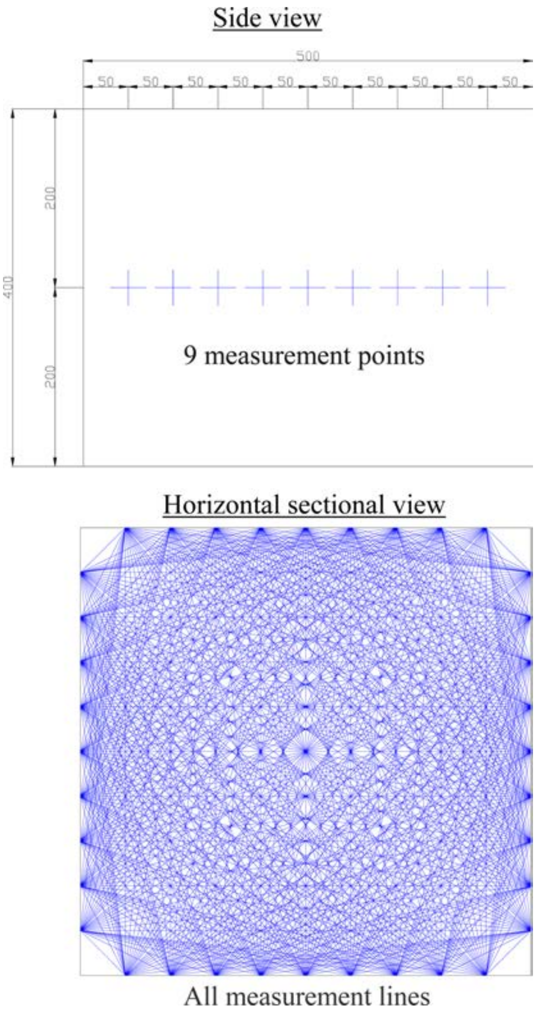


Fig. 3. Measurement point locations on the test piece.

model exhibited a generally even distribution of propagation velocity, whereas the internal defect model showed decreased propagation velocities in the rock pocket regions. These results are clearly visualized in the tomograms.

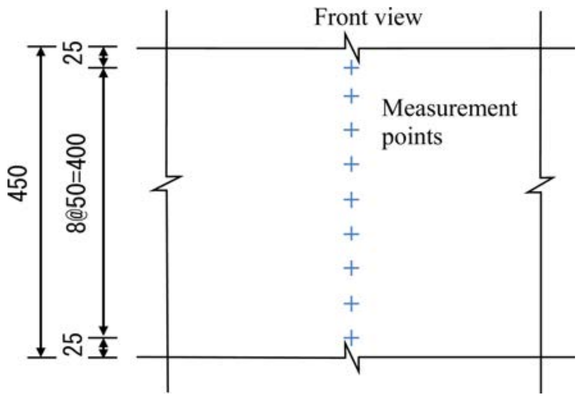


Fig. 4. Measurement point locations on the real structure.

Examination on a real structure

Poor construction joint was found between the first and second placements in a box-girder road bridge under construction. Although no other problems were found in this bridge by full hammer test, the extent of the externally visible damage had to be determined in the depth direction. Ultrasonic tomography was adopted for damage extent evaluation, with the cross section of the web divided into 8 elements horizontally and 9 elements vertically (9 measurement points at 50 mm intervals, Fig. 4). Propagation velocity difference was classified and color-coded into 25 levels, with 4500 m/s as the maximum (blue) and 4000 m/s as the minimum (red).

Fig. 5 shows the tomographic analysis results. Section 1 which is a control area visually assumed to have no defects showed an even distribution of propagation velocity of about 4500 m/s, suggesting successful bonding at the joint. Section 2 representing a defective

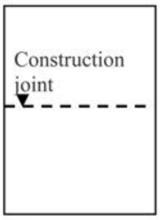
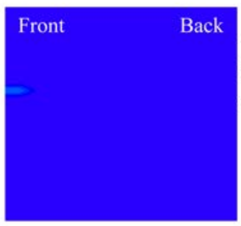
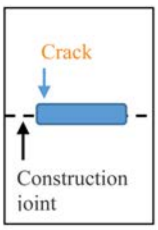
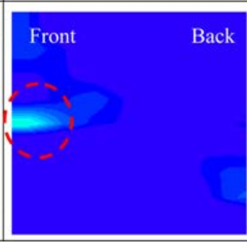
Model	Description	Schematic	Appearance	Tomography analysis result
Section 1: Control area without defects	Dense concrete without defects			 Propagation velocity was even across the plane.
Section 2: Defective area	A crack in the construction joint width: 30 mm depth: 75 mm			 Propagation velocity decreased at the defect.

Fig. 5. Tomographic analysis results on the real structure.

area exhibited a decrease in propagation velocity at the region circled with dashed line in Fig. 5. When cracks are present in the way of input ultrasonic waves, they cannot travel straight from the transmitter to the receiving sensor but needs to avoid the cracks, causing a decrease in the apparent propagation velocity. The decrease in propagation velocity suggests the presence of crack at that depth. The crack found in Section 2 in this test was considered to have a depth of about 75 mm.

Cable grout fill level inspection

Outline

PC structures are highly durable when designed, constructed and maintained properly, as has already been verified in many countries. PC structures are made of dense, high-strength low water-cement ratio concrete which is applied with prestress. This makes PC structures much more resistant to penetration of external deteriorating factors than RC structures.

Despite the inherent high durability, premature deterioration occurs in some PC structures. There are various deteriorating factors, including those common to both PC and RC structures such as salt attack and alkali silica reaction. Phenomena characteristic to PC structures include damage or deterioration of tendon anchorage and steel corrosion due to poor grouting in the post-tensioning system.

When grout voids are present inside the prestressing cable sheath, rainwater, seawater or water containing deicer entering the voids may cause corrosion and even breaking of steel tendons. It has been reported that concrete spalling was caused by an impact of breaking of steel tendons which had corrosion due to poor grouting. Fig. 6 shows an example of broken longitudinal steel tendons. These defects can affect load bearing capacity of the members and lead to a danger of accident to a third party, depending on the location. Especially with those with segment girders or external post-tensioning system, there have been reports from both inside and outside Japan describing collapse of bridges caused by an abrupt drop in load bearing performance due to breaking of longitudinal steel tendons.

Accurate evaluation of cable grouting is understood as one of the most important issues for proper maintenance of PC structures, and research is under way at many institutes to develop techniques for the determination of grout fill level.

For the internal cable system where prestressing cables are placed inside the concrete, it is preferable to use non-destructive techniques for the grouting evaluation so as not to damage the PC structure. Drilling the concrete without knowing the grout fill level can damage not only the affected members but also the steel tendons. This paper describes elastic-wave based non-

Fig. 6. Broken longitudinal steel tendons.

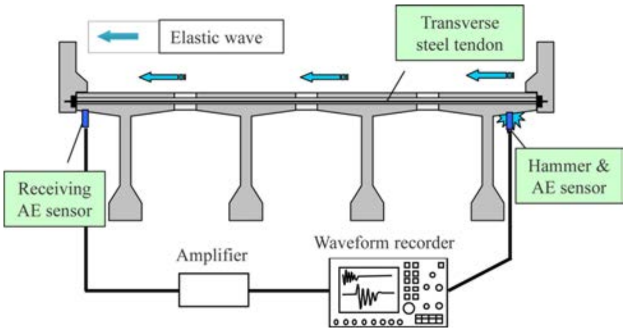


Fig. 7. Measurement set-up for impact-elastic wave method.

destructive testing techniques for concrete, focusing on impact-elastic wave method for transverse tendons in PC decks and impact-echo method for internal cables in the post-tensioning system.

Impact-elastic wave method

Outline

An impact wave is generated in a concrete member by using a steel ball, hammer or similar tool, and the elastic wave propagating through the concrete is received by AE sensors attached to the subject. Various measurement techniques are available depending on the type of elastic wave, propagation characteristics and other conditions. Use of this method for cable grouting evaluation started in the middle of the 1990s, and various improvements have been made in terms of impact-elastic wave input and measurement result evaluation.

Measurement method

The elastic wave generated on a concrete surface by hammering near an anchorage of tendon is received near the impact point as the input signal and near the anchorage on the other end as the output signal, using AE sensors. This non-destructive technique which utilizes change in propagation characteristics by grout fill level can be used for full length evaluation of the grouting of transverse prestressing cables. Fig. 7 shows the measurement set-up.

Evaluation method

Grouting evaluation by impact-elastic wave method is based on the measurement of energy attenuation and propagation velocity of the elastic waves propagating through a steel tendon.

Amplitude of output signal is small when the grouting is full, because energy of the propagating wave is attenuated by the grout confining the tendon. The reduction in the output amplitude is small when the grouting is poor, because energy attenuation is small due to the lack of confinement.

Propagation of the elastic wave is delayed when the grouting is poor as compared to full grouting. Propagation velocity increases when the grouting is poor, because the elastic wave travels mainly within the tendon. When the grouting is full, propagation velocity decreases because the elastic wave travels through composite medium of the hardened grout and the steel tendon.

Measured data is evaluated based on Equation 3 of propagation velocity and Equation 4 of input-output ratio which is the ratio of maximum amplitude of the output signal to that of the input signal.

$$V = L/t_0 \tag{3}$$

where V: propagation velocity (m/s); L: distance between the measurement points (m); and t_0: travel time of the input (ms).

$$S = a_0/a_i \tag{4}$$

where S: input-output ratio ($\times 10^{-2}$); a_0: maximum amplitude of the output signal (mV); and a_i: maximum amplitude of the input signal (mV).

Output elastic wave consists of primary or longitudinal wave (P-wave) and secondary or shear wave (S-wave). Accurate input-output ratio can be obtained by using only the P-wave which reaches the output end first. It is necessary to calculate travel time of the S-wave t_a by Equation 5 and set the maximum amplitude at the output end within the range below t_a seconds.

$$t_a = L/V_s\ (\mu s) \tag{5}$$

where L: distance between the measurement points (m); and V_s: velocity of the S-wave (2500 m/s)

Figure 8 shows examples of measured waveforms. Case (a) is an example of full grouting. Amplitude of the output waveform was small, and time to rise was long. In case (b) with poor grouting, output waveform amplitude was large, and time to rise was short.

Fig. 9 shows a schematic diagram of the relationship between the input-output ratio, propagation velocity and grout fill level. The lower the grout fill level, the larger the input-output ratio is and the faster the propagation is. The higher the grout fill level, the smaller the input-output ratio is and the slower the propagation is. These tendencies allow for setting criteria for grouting evaluation in terms of input-output ratio and propagation velocity. The criteria are determined based on previous data, taking into account the type and length of the tendons to be tested and the anchorage devices. Several cables are selected from those classified to the gray area in the measurement results, and they are drilled and visually examined to determine proper criteria.

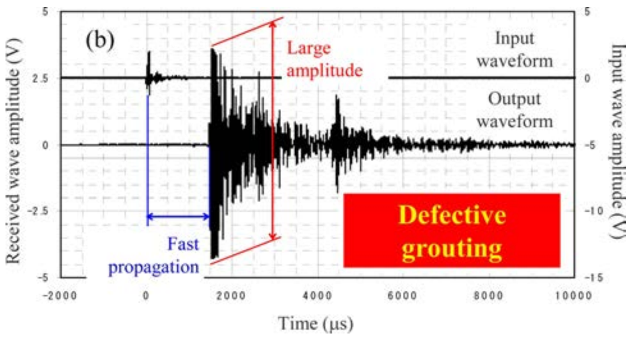


Fig. 8. Measured waveform examples by impact-elastic wave method.

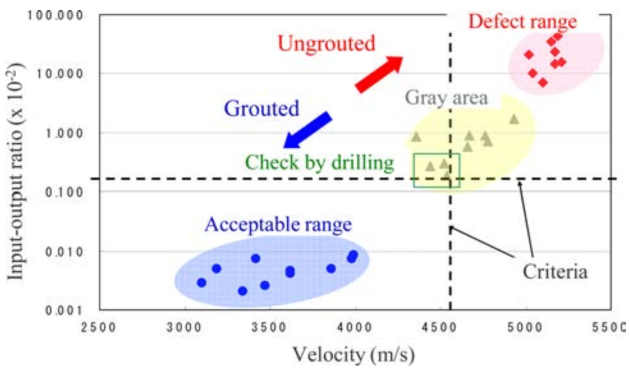


Fig. 9. Relationship between input-output ratio, propagation velocity and grout fill level.

Impact-echo method

Outline

An impact is applied to a concrete surface to generate an elastic wave. P-wave component of the elastic wave is reflected at a defect inside the concrete or at the boundary surface of different materials, generating a standing wave between the concrete surface and the defect or the boundary surface (P-wave resonance). Impact-echo method utilizes this phenomenon and estimates the condition inside the concrete from the locations of frequency spectrum peaks of the waveforms measured near the input point [1]. In cable grouting evaluation, an elastic wave is input to the concrete surface where the steel tendon is located beneath, and the reflection wave is received by a sensor. The results show presence or absence of voids in the prestressing cable, thus indicating if the grouting is defective or complete. Impact-echo

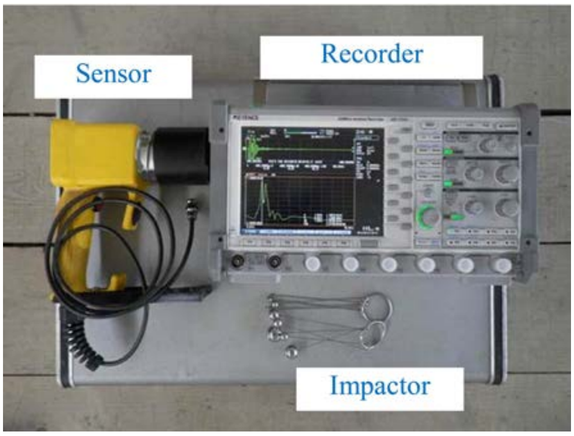


Fig. 10. Impact-echo method measurement set-up.

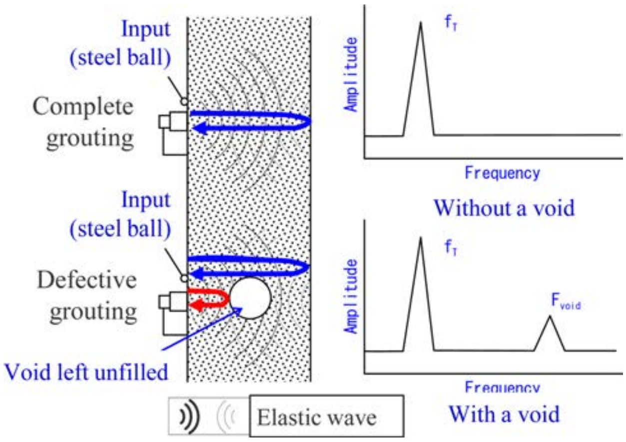


Fig. 11. Comparison of grout fill level and frequency response.

method is a technique for local evaluation of grouting in a prestressing cable and is suitable for examining main cables.

Measurement method

Measurement by impact-echo method requires steel balls (impactors), a receiving sensor, a signal amplifier and a waveform recorder (Fig. 10). Steel balls with different diameters are used as impactors to input elastic waves.

Grouting evaluation by impact-echo method starts with locating of the sheath based on design documents using electromagnetic radar method. An elastic wave is input by hitting the concrete surface directly above the sheath with a small steel ball. A conical wideband displacement sensor is installed near the input point to receive the elastic wave. The received signal is amplified, and frequency spectrum is calculated in the waveform recording system by using fast Fourier transformation (FFT).

Evaluation method

Grouting evaluation by impact-echo method is performed as described below. Fig. 11 compares sound concrete and defective concrete with internal voids (grout voids) in terms of frequency response characteristics. Concrete with full grouting exhibits a frequency spectrum peak for the thickness of the concrete member. Concrete with poor grouting exhibits another peak, f_{void}, as a reflection from the void within the sheath. These peaks can be expressed by Equation 6 as f_T for concrete with full grouting, and by Equation 7 as f_{void} for concrete with poor grouting, with V representing P-wave propagation velocity of the concrete.

$$f_T = V/2T \tag{6}$$

$$f_{void} = V/2d \tag{7}$$

where V: P-wave propagation velocity (m/s); T: member thickness (m); and d: cover depth to the sheath (m).

If the second peak appears at around the location of the sheath which has been identified by using an RC radar or similar device before measurement, it suggests presence of a void made by poor grouting.

Conclusions

More advanced inspection techniques will be necessary for rationalized maintenance of concrete structures. The importance of non-destructive inspection will be much larger in future. The techniques described in this paper are not necessarily completely established. However, there will be development in research and improvement in performance of measuring instruments which will enhance the potential of these techniques for practical applications. The authors will be pleased if this report would be a reference for or a help to the future development.

References

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