Research Article

Application of Impact Imaging Method on Nondestructive Detection of Void Defects in Sandwich-Structured Immersed Tunnel

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Sandwich-structured immersed tunnel is an essential component in the field of cross-sea tunnel structural engineering which has a complicated internal structure. The void defects between the contacting surface of the steel and concrete reduce the carrying capacity of the structure. There are presently no effective, rapid, and accurate methods for detecting the casting quality of steel-shell concrete. Based on the propagation law of elastic stress waves in different media and the theory of a near-source wave field, a prototype experiment on the nondestructive detection of void defects in sandwich-structured immersed tunnel was conducted. The experiment firstly determined the detection criterion for the impact imaging method through the features of the waveform data collected from 2 compartments (12 opening positions) of the full-scale model and then used the detection criterion to perform blind tests on the other 12 compartments of the full-scale model. The blind tests’ results of 72 opening positions showed that the prediction precision is 87.5% and thereby verify the applicability and accuracy of the detection criterion of the impact imaging method.

1. Introduction

Immersed tunnel structures are widely adopted in construction projects for large underwater tunnels worldwide [1]. It has two main structural forms, namely, reinforced concrete (RC) and steel-shell concrete immersed tunnel structures. Generally, RC immersed tubes are initially poured in a dry-dock prefabrication yard and then floated to the tube element immersion site. This process has high demands for the prefabrication site [2]. Steel-shell concrete immersed tubes are generally divided into single-layer and double-layer steel shells (see Figure 1). The steel shell is used as the outer shell of the immersed tube; a 6–10 mm thick steel plate is often employed [1, 3, 4]. In the 1990s, the double-layer steel-shell concrete immersed tube structure was further developed in Japan, where it was proposed that the steel structures should be constructed as closed compartments. Then, self-compacting concrete is poured inside the compartments. The tubes fabricated using this construction method are referred to as sandwich-structured steel-shell concrete immersed tubes (SSIT) [5, 6], a composite structure in which the concrete is wrapped by steel [5, 7–9]. At present, there are only two fully sandwich-structured immersed tunnels that have been completed and are in operation, i.e., the Naha and Shinwakato Tunnels, both in Japan [10].

Sandwich-structured immersed tubes utilise reserved casting holes and vent holes in the compartments for pouring the self-compacting concrete. As the pouring process cannot include vibration, the bonding surface between the steel plate and concrete is highly prone to void defects owing to uncompacted pouring. The presence of void defects reduces the binding force of the concrete on the stiffeners and welding studs, thereby affecting the overall
bearing capacity of the structure and the safe operation of the project is compromised [9, 11]. Therefore, after the pouring is completed, the casting quality of SSIT must be tested, the void defect area should be found in time, and grouting should be carried out to reinforce it before sinking. However, owing to the presence of the steel shell, various electromagnetic wave-based detection methods cannot be applied [12–14], such as the magnetic method, which has wide application in long-distance pipeline inspection [15–17]. At present, the detections in SSIT structure only rely on the elastic wave and radiation detection methods such as the ultrasonic method, impact acoustic method, the infrared method [18, 19], and the neutron method.

Yanagihara et al. conducted void detection experiments on steel-concrete composite structures using low-frequency ultrasound. They found that the dense region and defect region could be distinguished, according to the energy of the reflected echo. However, as it remains limited by the size of the receiving transducer, the ultrasonic method can only identify void defects with relatively small diameters [20]. And the transducer must be bonded to the structure, which requires considerable time, limiting its application in large structures [21–24]. The impact elastic wave method analyses the characteristics of input and output waveforms to determine the internal structure of the test area [25]. The experiments indicated that when the thickness of the steel plate is greater than 6 mm or the sizes of the defects are small, the void defects cannot be detected using this method [26, 27]. The impact acoustic method is widely studied and applied in the Japanese engineering field [28], but the method cannot distinguish between small gaps and void defects [29–31]. In actual engineering applications, the detection accuracy of the impact acoustic method is generally 60%–70%.

Hiroshi’s experimental study shows that the infrared method could identify the outlines of defects at a steel plate thickness of 8 mm and void height of 20 mm [32]. However, when the steel tube wall thickness was greater than 10 mm, this method could only estimate the location of the defect, and the defect outline was unclear [33–36]. The neutron method is highly susceptible to water. When the areas between the steel plate and concrete are immersed in water or the water on the surface of the steel plate is not sufficiently dried, the neutron method cannot be applied. Moreover, affected by the thickness of the steel plate, its detection time is extremely long [37–39].

In summary, as a novel structure, SSIT structure is rarely used in actual engineering at present. There is a lack of systematic research on the nondestructive testing of the SSIT structure, and most of the existing approaches can qualitatively determine the presence of defects but cannot well-distinguish the types and shapes of defects or quantitatively assess the void heights of defects (see Table 1). Therefore, based on a prototype experimental model of SSIT structure, the impact imaging method was firstly applied to the detection of steel-shell concrete void defects in this paper. The prototype experiments considered the complex characteristics of the SSIT structure, and the frequency, amplitude, and time-frequency characteristics of the collected waveform signals were analysed. The experiment results show that the normalised impact response intensity index can accurately evaluate the void severity of the defect, which overcomes the limitations of traditional detection methods. Simultaneously, this method can achieve rapid data collection and analysis and high detection efficiency and is minimally affected by environmental noise.

2. Testing Methods and Equipment

2.1. Introduction of the Impact Imaging Method. The theoretical basis of the impact imaging method is the elastic wave theory. When impacting the surface of a test object, an elastic wavefield will be excited inside the test object, including surface waves, P-waves, S-waves, and P-SH converted waves, as shown in Figure 2. The elastic wave will reflect and transmit when it encounters the interface of two different media, which will alter the original propagation direction and change its composition [40, 41]. The interface of the different media becomes the medium of the wave transmission and conversion, and its properties will affect the wavefield distribution of the entire structure [42]. When there is a void between the concrete and steel plate, the distribution characteristics of the elastic wavefield on the surface of the steel plate will change, including energy attenuation, abnormal wave characteristics, and abnormal spectral characteristics. The internal structure of the test object can be inferred via an inverse mapping analysis of the wavefield distribution on the surface of the test object [43, 44].

2.2. Testing Equipment for the Impact Imaging Method. The testing equipment for the impact imaging method consists of a small steel hammer vibration exciter, geophone, coupler, digital seismograph, laptop, mobile power supply, and related connecting cables as shown in Figure 3. The impact-elastic wave is manually excited by the hammer, and the excitation frequency is generally tens of Hz to several
kHz. The geophone is a medium-and-high-frequency vertical geophone with a sensitivity of 0.28 ± 5% (V/cm/s). The seismograph is a Geode high-resolution seismograph (US). The main parameters of the seismograph are listed in Table 2.

### 3. Prototype Experiment

#### 3.1. Introduction of the In Situ Test on the Full-Scale Model

This study adopted a full-scale steel-shell concrete immersed tunnel model (hereinafter referred to as the full-scale model) to perform a void defect detection experiment using the impact imaging method. The full-scale model is 9.6 m long, 55.4 m wide, and 10.6 m high. It is welded using 14 mm/44 mm steel plates, and the entire steel shell weighs approximately 740 t. The amount of concrete poured is approximately 1500 m³ as shown in Figure 4. The transverse and longitudinal partitions divide the full-scale model into 105 compartments, including 51 floor compartments, 12 wall compartments, and 42 roof compartments. Figure 5 shows the compartment structure, which has a size of 3 m × 3.5 m × 1.5 m. The compartment roof is arranged with a structure consisting of T-stiffeners and welding studs. The compartment floor is arranged with longitudinal and transverse stiffeners. The pouring holes are arranged in the middle of the compartment, and the vent holes are arranged into 10 holes per single standard compartment. The feed tube and vent tube are connected in two sections; the lower section is equipped with a steel tube (50 mm high), and the upper section is a plug-in-type tube. The experimental
material parameters and mixing ratios of the self-compacting concrete are given in Tables 3 and 4, respectively.

No void defects were preset during the pouring of the compartment in the full-scale model. In the experiment, the full-scale model was fabricated, transported, and poured according to the construction conditions, processes, and methods for steel-shell concrete immersed tube production in an actual project. In situations in which void defects may occur during the production and construction of the immersed tubes were simulated. After the pouring of the compartment was completed and the initial setting period had passed, the void defect detection experiment was conducted. The experiment firstly determined the detection criterion for the impact imaging method through the features of the waveform data collected from some compartments of the full-scale model and then used the detection criterion to perform blind tests on the other compartments of the full-scale model and thereby verify the applicability and accuracy of the detection criterion of the impact imaging method.

3.2. Experimental Process. In the void detection experiment, the data collection is carried out in units of the compartment, and the measuring lines were arranged in the direction of the T-stiffener as shown in Figures 6(a)–6(c). Generally,
the spacing between each measuring line on the compartment was 10 cm. However, the voids were more likely to occur near the T-stiffener, where the measuring lines were dense and the spacing was 5 cm. The measuring points were arranged at a spacing of 10 cm in each measuring line, forming a grid of measuring points. The measuring points formed the equidistant grid of 10 cm × 10 cm and 5 cm × 10 cm in the non-T-stiffener regions and T-stiffener regions, respectively.

A single-point approach was adopted to collect the data along the measuring lines. The data collection interval was 20.83 μs, and the number of sampling points was 8,192. After an elastic wave was excited by hammering with the small steel ball, the vertical geophone was used to receive the elastic wave information on the surface of the steel shell and to record the stress wave propagation and reflection signals. Subsequently, while maintaining the distance between the excitation point and receiving point, the data acquisition system was moved to the next measuring point. Then, the above data acquisition process was repeated as shown in Figure 6(d). The impact strength should be kept as constant as possible during data collection, and it was recorded to normalise waveform data during data analysis.

4. Analysis of Experimental Results

4.1. Typical Void Phenomenon. After the data collection, to check the actual pouring situation of the compartment and establish the relationship between the response waveform characteristics and void situation in the steel-shell concrete,

| Materials | Cement | Fly ash | Slag | FA     | Coarse aggregate | Water | Plasticizer |
|-----------|--------|---------|------|--------|------------------|-------|-------------|
| Specifications | P I I42.5 | Level I | S95 | Medium | (5–10 mm) | (10–20 mm) | Tap water | Polycarboxylate |
| Production | CR Cement | Jianbi | Caofeidian | Dongjiang | Panlong stone field | — | Sobute |

Note. FA: fine aggregate; CA1: coarse aggregate (5–10 mm); CA2: coarse aggregate (10–20 mm).

| Materials | Cement | Fly ash | Slag powder | CA1 | CA2 | Silica sand | Water | Plasticizer |
|-----------|--------|---------|-------------|-----|-----|-------------|-------|-------------|
| kg        | 270    | 196     | 84          | 39  | 508 | 782         | 171   | 5.5         |

Figure 6: Schematic diagram of the measuring points layout on the compartment (unit: mm). (a) The top view of the compartment structure, (b) the grid layout of survey line, (c) the front view of compartment structure, and (d) schematic diagram of data collection.
12 parts of the compartments of the full-scale model were opened for examination. The area of the opening position was 30 cm × 30 cm (or 30 cm × 20 cm). To ensure that the opening process would not deform the steel shell or damage the concrete under the steel plate, the opening was cut via carbon arc gouging (thermal cutting). After the upper steel plate at the opening position was cut, the locations with void defects were accurately identified and the void heights were measured. A typical void phenomenon is shown in Figure 7, where the opening position in zone A is completely located on the right side of the T-stiffener. In zone A, except for the slight peeling in the lower left corner, all other locations were densely filled. The opening position of zone B spanned across the T-stiffener structure. As can be seen from Figure 7, the left side of the T-stiffener structure in zone B was densely filled with concrete, whereas the right side was the void-defect region.

\[
\overline{X} = \frac{X}{Y} = \frac{\{x_1, x_2, x_3, \ldots, x_n\}}{\text{max}\{y_1, y_2, y_3, \ldots, y_n\}} = \left\{ \frac{x_1}{y_{\text{max}}}, \frac{x_2}{y_{\text{max}}}, \ldots, \frac{x_n}{y_{\text{max}}} \right\} = \{\overline{x}_1, \overline{x}_2, \ldots, \overline{x}_n\},
\]

where \(X\) is the waveform signal collected by the geophone, \(Y\) is the impact pulse signal recorded by the impact hammer, and \(X\) is the normalised waveform signal.

The characteristics of the four waveforms are different in the following Figure 8. It is worth noting that in zone A, the measuring points \(a_1\) and \(a_2\) are both filled dense regions. However, the waveform characteristics of these two measuring points are significantly different. At \(a_1\), which is closer to the T-stiffener, the waveform amplitude is smaller, whereas at \(a_2\), which is slightly further from the T-stiffener, the vibration amplitude of the response waveform is significantly greater. The maximum amplitude at \(a_2\) is three to four times that at \(a_1\). It is inferred from this observation that the restraint effect of the T-stiffener structure may have reduced the vibration amplitude of the steel plate. Therefore, the influence of the T-stiffener structure must be considered in the defect identification analysis.

The response waveforms of measuring points \((a_2, b_1,\) and \(b_2)\) on the same measuring line are analysed. It can be seen that the waveform amplitude in the dense region is relatively small. In Figure 8 \((a_2)\), the maximum amplitude is only 12–17 mV, and the vibration duration is relatively short. After 0.02 s, the amplitude value is less than 1 mV, indicating that the energy has basically attenuated. However, in the void defect region, there is a large amplitude, and the maximum amplitude of the point \(b_1\) is 70 mV, which is 4 times that of the dense region, and in the point \(b_2\), the maximum amplitude can reach 100–120 mV, approximately 7 times that of the dense region. Moreover, the vibration duration is also longer, and the amplitude energy only attenuates to a small value after 0.052 s as shown in Figure 8 \((b_2)\). Simultaneously, the result shows that compared with points \(a_2\) and \(b_1\), the point \(b_2\) with a void height of 12 mm has the largest amplitude and the longest vibration duration. It is inferred that the void height of the defect region is related to the amplitude and vibration duration of the response waveform; the higher the void height of the defect, the larger the vibration amplitude and longer the vibration duration of the response waveform will be. This phenomenon provides the possibility to correctly identify the void defect.

An FFT was performed on the waveform data of the measuring points. Figure 9 shows that the ranges of the elastic wave spectra for the dense and void defect regions are both below 2000 Hz. From dense regions \(a_1\) and \(a_2\), it can be seen that the response waveforms of the dense regions have a peak in the frequency range of 600 to 700 Hz, which only appeared once. However, the void defect regions not only had a peak at approximately 500 Hz but also showed multiple peaks in higher frequency bands. This indicates that the elastic wave encountered defects in its propagation process, causing diffraction, scattering, and reflection that generated the multiple peaks. Since the waveform data collected from the dense regions and defect regions have different characteristics, the filling compactness of the steel-shell concrete can be preliminarily determined according to the amplitude and frequency characteristics of the waveforms.

4.2. Waveform Signal Characteristic Analysis. After the data collection, the original data were normalised by the impact strength (see formula (1) for the details) and were filtered and noise-reduced in the time and frequency domains. Owing to the T-stiffener structure on the compartment roof, the waveform data collected from different locations with the same concrete filling compactness could potentially vary. To avoid the influences of the T-stiffener structure, the response waveform data of the filled/dense regions and typical defect regions of the same measuring lines were extracted for analysis. The layouts of the measuring points at typical locations in zone A and B are shown in Figure 7, and the information of the locations where measuring point data were extracted is listed in Table 5.

5. Research on the Correlation between Impact Response Intensity and Void Height

From the analysis in Section 4.2, it can be seen that the response waveforms of the void defect region and dense region are significantly different in both the time and frequency domains. In order to comprehensively consider the characteristics of amplitude and vibration duration, in this paper, an impact response intensity index \(I(t)\) was introduced as the detection criterion for the impact imaging method. The specific expression of \(I(t)\) is as follows:
\( I(t) = \int_{0}^{t} |x| \, dt \)

\[
= \int_{0}^{t} [A_1 \sin(\omega_1 t + \varphi_1) + \ldots + A_n \sin(\omega_n t + \varphi_n)] \, dt \\
\approx \left( |x_1| * dt_1 + |x_2| * dt_2 + \ldots + |x_n| * dt_n \right) \\
\approx \sum_{i=1}^{n} |F_i| * \Delta t.
\]

The impact response intensity index is the integral of the absolute value of the waveform signal over the data collection period \( t \). The original signal is composed of a set of discrete sampling points at equal intervals. Therefore, the integral of the waveform signal can be approximated as the sum of the product of the previous sampling value and sampling time. In the above equation, \( F_i \) is the amplitude of the waveform, \( \Delta t \) is the sampling interval, and \( i \) is the sampling point number within period \( t \).

5.1. The Influence of T-Stiffener Structure. To strengthen the cooperative bearing between the steel shell and concrete, 3–4 T-stiffeners (at a spacing of 70 cm) are generally placed on the compartment roof. The T-stiffener changes the internal structure of the local detection area. In this study, the measuring points \( a_1, b_1, a_2, b_2, \ldots, f \) of the dense regions are selected to study the influence of the T-stiffener structure and clarify its influence range (see Figure 10(a)). The distance between these measuring points and the T-stiffener structure is 0 cm, 5 cm, 10 cm, 15 cm, 20 cm, 25 cm, and 30 cm, respectively. Figure 10(b) shows the calculated impact response intensity \( I(t) \) of the waveform data at each measuring point. It can be seen that the farther the measuring point is from the T-stiffener structure, the greater the calculated impact response intensity. However, in different ranges, the increasing rates are different. When the measuring point is in the range of 0–15 cm, the impact response intensity increases slowly, and the increase is small. When the measuring point is in the range of 15–30 cm, the increase rate of the impact response intensity becomes faster, and the increase is more evident. The impact response intensity at a distance of 30 cm is approximately several times that at 15 cm. The results show that the restraint effect of the T-stiffener structure on the steel plate is relatively strong in the range of 0–15 cm; when the distance is beyond 15 cm, the restraint effect weakens rapidly; therefore, the calculated value of impact response intensity varies greatly within this range.

5.2. Relationship between Void Height and Impact Response Intensity. According to the distances between the measuring line and T-stiffener, the impact response intensities of the waveform data were normalised, thereby eliminating the influence of the T-stiffener structure. After obtaining the impact response intensities of all measuring points, the waveform response intensity of the measuring point \( I(t) \) was divided by the statistical mean of the response intensities obtained from the corresponding measuring line in the dense region \( \bar{I}(t) \) to solve for the normalised impact response intensity \( \bar{I} \) as follows:
Figure 8: Time history curves of response waveforms of the measuring points. (a) $a_1$ dense region (5 cm from T-stiffener), (b) $a_2$ dense region (10 cm from T-stiffener), (c) $b_1$ void region (10 cm from T-stiffener), and (d) $b_2$ void region (10 cm from T-stiffener).

Figure 9: Continued.
The normalised impact response intensity can amplify the response characteristics of the void defect. The $I$ obtained from the opening positions on the full-scale model was compared and analysed against the measured concrete filling conditions as shown in Figure 11. At the No. 1 opening position (see Figures 11(b) and 11(c)), there is a void defect on the left side of the T-stiffener, and the right side is densely filled. Compared with the distribution of the $I$ of the measuring points in Figure 11(a), it is found that the response intensities on the left side are abnormally high, whereas the response intensities on the right side are relatively low, i.e., all below 2, which are similar to the No. 2 opening position, a typical dense area (see Figures 11(d) and 11(e)). The normalised impact response intensities at No. 2 opening position are all below 2. Moreover, it can be seen from Table 6 that the larger the void depth is, the larger the calculated value of $I$ is, which indicates that the normalised impact response intensity has a good correlation with the void height.

Based on the data obtained from the 12 opening parts of the compartments, the normalised impact response intensities $I$ and the measured void heights $h$ of the measuring points were fitted and analysed to obtain their characteristic curve as shown in Figure 12. The correlation coefficient ($R$) of the fitting formula below is 0.916, and the $R^2$ is 0.839. The correlation of the fitting results is relatively high.
Figure 11: Comparisons between the test results and the measured results after opening the compartment. (a) Normalised impact response intensity of measuring point at position No. 1, (b) measured value of the concrete filling condition after opening the compartment at position No. 1, (c) on-site photo after opening the compartment at position No. 1, (d) impact imaging test results of measuring points at position No. 2, (e) measured value of the concrete filling condition after opening the compartment at position No. 2, and (f) on-site photo after opening the compartment at position No. 2.

| No. | Impact response intensity | Calculation void height (mm) | Measured void height (mm) | Error (mm) |
|-----|--------------------------|-----------------------------|---------------------------|------------|
| 1   | 1.19                     | 0                           | 0                         | 0          |
| 2   | 1.54                     | 0                           | 0                         | 0          |
| 3   | 1.04                     | 0                           | 0                         | 0          |
| 4   | 4.02                     | 5.07                        | 5.5                       | −0.42      |
| 5   | 5.12                     | 6.63                        | 6                         | 0.63       |
| 6   | 5.57                     | 7.23                        | 7                         | 0.23       |
| 7   | 5.86                     | 7.61                        | 7                         | 0.61       |
| 8   | 6.27                     | 8.12                        | 7                         | 1.12       |
| 9   | 8.42                     | 10.57                       | 11                        | −0.43      |
| 10  | 10.73                    | 12.77                       | 16                        | −3.22      |
| 11  | 14.51                    | 15.60                       | 13                        | 2.60       |
The detection criterion for the void defects in the SSIT structure is determined according to the fitting curve as follows: when $0 < I \leq 2.3$, it is considered that the concrete is compactly poured, and there is no void defect; in contrast, when $I > 2.3$, it is considered that there are void defects. The impact response intensity of 2.3 is determined as the threshold for distinguishing void defects from the dense region. When $I > 2.3$, the void heights of the defects can be calculated from formula (3). Thus, it is possible to realise the identification of void defects in the SSIT structure, and the heights of the void defects can be quantitatively evaluated.

$$h = -25.58e^{-0.007I} + 23.61.$$  \hspace{1cm} (4)

5.3. Validation of the Impact Imaging Detection Criterion. The blind test was carried out on the other 12 compartments of the full-scale model to verify the accuracy of the detection criterion of the impact imaging method for void defects in the SSIT structure. In this study, a total of 72 locations were randomly selected from the 12 compartments for the opening verification. Among them, 12 compartments cover all the structural forms of the compartments used in the immersed tube structure, and 72 locations cover the locations where the compartments are most prone to void defects. The statistical analysis of the compartment-opening verification results showed that the accordance rate between the blind inspection results and the actual void situation was approximately 87.5%. Specifically, nine inspection results in the 72 opening positions were wrong, among which seven of the dense areas were identified as void defects, and 2 of the void defects were identified as dense areas. One of the verification results of the blind compartment-opening tests is shown in Figure 13. It can be seen from Figure 13(b) that the impact response intensities are all less than 2.3 in the dense region of the right part, whereas the impact response intensities of the left part are between 2.9 and 3.2, and the
calculated void heights of the left region are in the range of 3.3 to 3.8 mm, which consistent well with the measured void height 4 mm. Moreover, the data collection of the impact imaging method has a low requirement on the coupling of the detector and does not need to embed or bond the detector on the surface of the steel plate. The data collection is convenient and time-saving, which is especially suitable for the rapid detection of a large area.

6. Conclusions

The impact imaging method to detect void defects in the SSIT structure presented in this study exploits the fact that the dense region and void region can be distinguished from the waveform characteristics generated from an impact. By analyzing the wave characteristics and the influence of the T-stiffener structure, the normalised impact response intensity is proposed as the detection index parameter to quantify the void height at the defect location. This method has great potential in the voids detection and evaluation of SSITs. The main conclusions of this study are as follows.

1. The amplitude of the response waveforms in the dense region was only 12–17 mV, and the vibration duration was also short; the energy attenuation was basically completed after 0.02 s. In contrast, the maximum amplitude in the void defect region could reach 100–120 mV, i.e., approximately 7 times that of the dense region, and the vibration duration was longer. Moreover, the more serious the void defect, the larger the amplitude of the waveform and the longer the vibration duration.

2. The response waveform of the dense region was only a single dominant frequency in the frequency range of 600 to 700 Hz. However, in the void defect region, besides a peak that appeared at approximately 500 Hz, multiple peaks also appeared in higher frequency bands. The chaotic dominant frequencies indicated that the elastic wave was diffracted, scattered, and reflected during its propagation process, resulting in the multiple peaks.

3. The relationship curve between the normalised impact response intensity and void height was fitted to quantitatively evaluate the void heights of defects. A normalised impact response intensity of 2.3 was determined as the threshold for distinguishing between dense and defect regions, and the fitting curve had a correlation coefficient $R^2 = 0.839$.

4. The blind tests regarding the detection criterion of the impact imaging method showed that the accordance rate of the results was 87.5%. This method can well identify the locations of void defects and can determine the void heights of the defects. Moreover, its efficiency can meet the requirements for engineering applications.

Data Availability

The raw/processed data required to reproduce these findings cannot be shared at this time as the data also form part of an ongoing study.

Additional Points

(1) The self-compacting concrete of SSIT is prone to void defects during the casting process. (2) A novel method is applied to detect the subsurface void of SSIT. (3) A detection criterion for the impact imaging method was established. (4) The accuracy of impact imaging method is 87.5%, which is 15–20% higher than the traditional method.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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References

[1] W. C. Grantz, "Steel-shell immersed tunnels- Forty years of experience," Tunnelling and Underground Space Technology, vol. 12, no. 1, pp. 23–31, 1997.

[2] M. Lin, W. Lin, X. D. Liu, and Y. Hanada, "Development and experience of immersed tunnel in Japan," Journal of Waterway and Harbor, vol. 38, no. 1, pp. 1–7, 2017.

[3] A. Glerum, "Developments in immersed tunnelling in Holland," Tunnelling and Underground Space Technology, vol. 10, no. 4, pp. 455–462, 1995.

[4] M. Tomlinson, A. Tomlinson, and M. L. Chapman, "Shell composite construction for shallow draft immersed tube tunnels," Immersed Tunnel Techniques, 1990.

[5] H. Kimura, H. Moritaka, and I. Kojima, "Development of sandwich-structure submerged tunnel tube production method," Nippon Steel Technical Report, no. 86, pp. 86–93, 2002.

[6] A. Keiichi, H. Youichi, K. Hitoshi, and K. Kumagai, "Immersed tunnels in Japan: recent technological trends," in Proceedings of the 2002 International Symposium on Underwater Technology, IEEE, Tokyo, Japan, 19 April 2002.

[7] S. Y. Song, J. G. Nie, G. P. Xu, J. S. Fan, L. Tang, and Y. T. Guo, "Development and application of steel-concrete-steel composite structures in immersed tunnels," China Civil Engineering Journal, vol. 52, no. 4, pp. 109–120, 2019.

[8] H. Bowerman and J. C. Chapman, "Bi-steel steel-concrete-steel sandwich construction," in Proceedings of the Composite Construction in Steel and Concrete IV Conference 2000, ASCE Library, Alberta, Canada, 28 May 2002.

[9] Y. Guo, X. Nie, M. Tao, and S. Qiu, "Bending capacity of steel-concrete-steel composite structures considering local
buckling and casting imperfection,” Journal of Structural Engineering, vol. 145, no. 10, 2019.

[10] K. Hideo, K. Ichio, and M. Hiro, “The study on deformation of immersed tunnel element during production works while aloft,” Journal of Tunnel Engineering Report, vol. 12, no. 1, pp. 117–124, 2002.

[11] J. Nie, “Application of steel–concrete composite structure in ocean engineering,” Steel Construction, vol. 35, no. 1, pp. 1–15, 2019.

[12] A. Wahab, M. M. A. Aziz, A. R. M. Sam, K. Y. A. Q. You, and K. A. Kassim, “Review on microwave nondestructive testing techniques and its applications in concrete technology,” Construction and Building Materials, vol. 209, pp. 135–146, 2019.

[13] L. Capozzoli and E. Rizzo, “Combined NDT techniques in civil engineering applications: I,” Construction and Building Materials, vol. 154, pp. 1139–1150, 2017.

[14] S. B. Im and S. Hurlebaus, “Non-destructive testing methods to identify voids in external post-tensioned tendons,” KSCE Journal of Civil Engineering, vol. 16, no. 3, pp. 388–397, 2012.

[15] B. Liu, L. He, Z. Ma, H. S. H. Zhang, and S. Perilli, “Study on internal stress damage detection in long-distance oil and gas pipelines via weak magnetic method,” ISA Transactions, vol. 89, pp. 272–280, 2019.

[16] B. Liu, L. He, H. Zhang, and S. H. S. J. Sfrara, “Quantitative study of magnetic memory signal characteristic affected by external magnetic field,” Measurement, vol. 131, pp. 730–736, 2019.

[17] B. Liu, L. He, H. Zhang, S. H. S. Sfrara, and J. Ren, “Research on stress detection technology of long-distance pipeline applying non-magnetic saturation,” IET Science, Measurement & Technology, vol. 13, no. 2, pp. 168–174, 2019.

[18] S. Amer, H. Al Zarkani, S. Sfrara, and M. Omar, “Infrared thermography approach for pipelines and cylindrical based g,” Polymers, vol. 12, no. 7, p. 1616, 2020.

[19] D. Modenini and F. F. J. Schrijver, “Heat transfer measurements in a supersonic wind tunnel through inverse temperature data reduction: application to a backward facing step,” Quantitative Infrared Thermography Journal, vol. 9, no. 2, pp. 209–230, 2012.

[20] A. Yanagihara, H. Hatanaka, and M. Tagami, “Development and application of Non-destructive inspection for steel-concrete composite structures,” Journal of Ihi Technologies, vol. 53, pp. 47–53, 2013.

[21] H. Rathod and R. Gupta, “Sub-surface simulated damage detection using Non-Destructive Testing Techniques in reinforced-concrete slabs,” Construction and Building Materials, vol. 215, no. AUG.10, pp. 754–764, 2019.

[22] D. Wei, Z. Wu, X. Zhou, and Y. Tan, “Experimental studies on void detection in concrete-filled steel tubes using ultrasound,” Construction and Building Materials, vol. 128, no. dec.15, pp. 154–162, 2016.

[23] S. Iyer, S. K. Sinha, B. R. Tittmann, and M. K. Pedrick, “Ultrasonic signal processing methods for detection of defects in concrete pipes,” Automation in Construction, vol. 22, pp. 135–148, 2012b.

[24] S. Iyer, S. K. Sinha, M. K. Pedrick, and B. R. Tittmann, “Evaluation of ultrasonic inspection and imaging systems for concrete pipes,” Automation in Construction, vol. 22, no. Mar, pp. 149–164, 2012a.

[25] M. Sansalone and N. J. Carino, Impact-echo: A Method for Flaw Detection in Concrete Using Transient Stress Waves, National Bureau of Standards (NEL), Gaithersburg, MD, 1986.

[26] K. Nishioka, K. Watanabe, D. Shigeyuki, and P. Hashimoto, “Detection of voids on the underside of steel sheet of steel-concrete composite structure using impact elastic wave method (non-destructive inspection/diagnosis),” Annual Proceedings of Concrete Engineering, vol. 302, pp. 715–720, 2008.

[27] S. Kashif Ur. Rehman, Z. Ibrahim, and S. A. Memon, “Nondestructive test methods for concrete bridges: a review,” Construction and Building Materials, vol. 107, no. Mar.15, pp. 58–86, 2016.

[28] D. Chen, V. Montano, L. Hsu, S. Fan, and G. Song, “Detection of subsurface voids in concrete-filled steel tubular (CFST) structure using percussion approach,” Construction and Building Materials, vol. 262, Article ID 119761, 2020.

[29] T. Akiyama, O. Kiyomiya, S. Kitazawa, and S. Shiraishi, “Full-scale experiment on tapping method inspection of unfilled part with synthetic member,” in Proceedings of the Annual Papers on Concrete Engineering, IEEE, Baltimore, MD, USA, 16 November 2002.

[30] Y. Yamashita and O. Kiyomiya, “Finite element analysis for detecting unfilled parts of synthetic members by tapping method,” in Proceedings of the 58th Annual Scientific Lecture Meeting of the Japan Society of Civil Engineers, JSCE, Tokyo, Japan, September 2003.

[31] I. Misuio, K. Kazunori, H. Kengo, H. Shin-ichi, E. Shigeru, and T. Yoshihiro, “Study on non-destructive testing method of steel plate composite concrete deck by impact acoustics,” Kawada Technical Report, vol. 27, 2008.

[32] M. Hiroshi, I. Yui, S. Takahide, M. Shigeyuki, and S. Toshiyuki, “Study on nondestructive testing for steel-concrete composite slab by infrared thermography technology,” Structural Engineering Papers, vol. 59A, 2013.

[33] D. J. Titman, “Applications of thermography in non-destructive testing of structures,” Ndt&E International, vol. 34, no. 2, pp. 149–154, 2001.

[34] C. A. Maierhofer, A. Brink, M. Röllig, and H. Wiggenhauser, “Detection of shallow voids in concrete structures with impulse thermography and radar,” Ndt&E International, vol. 36, no. 4, pp. 257–263, 2003.

[35] F. Khan and I. Bartoli, “Detection of delamination in concrete slabs combining infrared thermography and impact echo techniques: a comparative experimental study,” Structural Health Monitoring and Inspection of Advanced Materials,” Aerospace and Civil Infrastructure, vol. 9437, pp. 943701–944111, 2015.

[36] C. C. Cheng, T. M. Cheng, and C. H. Chiang, “Defect detection of concrete structures using both infrared thermography and elastic waves,” Automation in Construction, vol. 18, no. 1, pp. 87–92, 2008.

[37] H. S. Cheng, S. J. Wang, G. X. Cao, M. C. Wei, and Y. Wang, “The Non-destructive detecting of inner cavity defect of concrete under steel plate lining of the runner chamber of hydro-power plant by using neutron method,” Journal of Isotopes, vol. 16, no. 3, pp. 138–142, 2003.

[38] H. Zhang, G. Q. Liu, C. Liu, Z. W. Fan, H. B. Zhao, and H. S. Chen, “Study on application of quantitative detecting of inner cavity defect of concrete under steel plate lining of hydro-power plant,” Journal of Isotopes, vol. 30, no. 3, pp. 194–199, 2017.

[39] S. Lee, N. Kalos, and D. H. Shin, “Non-destructive testing methods in the U.S. for bridge inspection and maintenance,” KSCE Journal of Civil Engineering, vol. 18, no. 5, pp. 1322–1331, 2014.
[40] X. Liu, *Foundation of Elastic Wave Field theory*, China ocean university press, Beijing, China, 2008.
[41] X. Lv and J. Wu, *Impact Elastic Wave Theory and application*, pp. 19–43, China WaterPower Press, Beijing, China, 2016.
[42] V. Grechka, S. Theophanis, and I. Tsvankin, “Joint inversion of P- and PS-waves in orthorhombic media: theory and a physical modeling study,” *Geophysics*, vol. 64, no. 1, pp. 146–161, 1999.
[43] C. Liu, A. L. Che, and S. k. Feng, “Propagation characteristics of elastic wave in layered medium and applications of impact imaging method,” *Journal of Shanghai Jiaotong University*, vol. 18, no. 4, pp. 479–485, 2013.
[44] D. Peng, A. Che, S. Feng, and H. Wang, “An impact imaging method for defect detection and its application in geotechnical engineering,” *Rock and Soil Mechanics*, vol. 38, no. 09, pp. 2764–2772, 2017.