Detection of Internal Defects in Concrete Members Using Global Vibration Characteristics

by H. Sezer Atamturktur, Christopher R. Gilligan, and Kelly A. Salyards

INTRODUCTION

Rock-pocket and honeycomb defects impair overall stiffness, accelerate aging, reduce service life, and cause structural problems in hardened concrete members. Traditional methods for detecting such deficient volumes involve visual observations or localized nondestructive methods, which are labor-intensive, time-consuming, highly sensitive to test conditions, and require knowledge of and accessibility to defect locations. The authors propose a vibration response-based nondestructive technique that combines experimental and numerical methodologies for use in identifying the location and severity of internal defects of concrete members. The experimental component entails collecting mode shape curvatures from laboratory beam specimens with size-controlled rock pocket and honeycomb defects, and the numerical component entails simulating beam vibration response through a finite element (FE) model parameterized with three defect-identifying variables indicating location (x, coordinate along the beam length) and severity of damage (α, stiffness reduction and β, mass reduction). Defects are detected by comparing the FE model predictions to experimental measurements and inferring the low number of defect-identifying variables. This method is particularly well-suited for rapid and cost-effective quality assurance for precast concrete members and for inspecting concrete members with simple geometric forms.

Keywords: concrete defect detection; experimental modal analysis; nondestructive testing and evaluation; statistical inference; uncertainty quantification.

Rock-pocket and honeycomb defects impair overall stiffness, accelerate aging, reduce service life, and cause structural problems in hardened concrete members. Traditional methods for detecting such deficient volumes involve visual observations or localized nondestructive methods, which are labor-intensive, time-consuming, highly sensitive to test conditions, and require knowledge of and accessibility to defect locations. The authors propose a vibration response-based nondestructive technique that combines experimental and numerical methodologies for use in identifying the location and severity of internal defects of concrete members. The experimental component entails collecting mode shape curvatures from laboratory beam specimens with size-controlled rock pocket and honeycomb defects, and the numerical component entails simulating beam vibration response through a finite element (FE) model parameterized with three defect-identifying variables indicating location (x, coordinate along the beam length) and severity of damage (α, stiffness reduction and β, mass reduction). Defects are detected by comparing the FE model predictions to experimental measurements and inferring the low number of defect-identifying variables. This method is particularly well-suited for rapid and cost-effective quality assurance for precast concrete members and for inspecting concrete members with simple geometric forms.

INTRODUCTION

Rock pockets and honeycombs are local, internal defects that may form in concrete members due to mortar paste failing to fill the space around the coarse aggregates. Rock pockets (clusters of loose aggregates not bound by mortar paste) and honeycomb defects (voids within the concrete) are typically the result of poor concrete mixing, leakage of wet concrete from the form, or insufficient consolidation. Consequently, the presence of such internal defects can degrade the performance of a concrete member with the severity of this degradation being proportional to the size of the defect. Depending on their size and location, the degrading effects of these internal defects may be evident immediately or may manifest years after the structural member has been in service. Such internal defects, if extensive or if located near the anchorage, can bring about compression failure (that is, crushing) in prestressed precast beams during tendon stressing (Eiji et al. 2006). Moreover, the presence of internal defects accelerates the degrading effects of aging caused by operational and environmental determinants, such as impairment of water tightness, corrosion of reinforcement bars, and crack development caused by freezing-and-thawing cycles.

Conventional techniques for detection of internal defects involve drilling samples—a process that is labor-intensive, time-consuming, and semi-destructive. As a result of the pervasive need for tools to detect internal defects in concrete members, a number of noninvasive and nondestructive defect detection techniques have been proposed over the past two decades (ACI Committee 228 1998). Such techniques, which are typically localized, rely primarily on the mechanical principle of stress wave or acoustic wave propagation (refer to, for example, Mori et al. [2002]). Examples of such processes include the acoustic impact method (refer to ASTM D4580 [2007] and Warnemuende and Wu [2004]), the ultrasonic wave propagation method (Na et al. 2002; Acciani et al. 2008) and the impact-echo method (Sansalone and Carino 1986). These localized nondestructive techniques: 1) require a priori knowledge about the vicinity of the defect; 2) require establishing access to the region with the defect (Sohn et al. 2004); and 3) exhibit high dependency on testing conditions, such as the surface condition of the tested specimen (Hellier 2001). In the absence of such information regarding the vicinity of the defect or access to the region with the defect, these aforementioned local nondestructive methods become prohibitively labor-intensive and time-consuming or inapplicable altogether. Aside from the location of the defect, another important step for elucidating the internal quality of a concrete member involves determining both the type and severity of the internal defect. Ultimately, the degrading effects of a defect must be determined to evaluate the significance and appropriate nature of repair schemes—either mandated or elective. Therefore, to be of practical use, a defect-detection method must also be able to quantify the defect severity.

In practical applications, neither the presence of internal defects nor their location or severity is known a priori. Focusing on the global properties of the structure (instead of local properties) may, however, avoid the necessity for such a priori knowledge. This study presents a combined experimental and numerical defect detection approach that relies solely on the global response of the concrete member. Through this proposed approach, the authors intend to detect the presence, location, and severity of internal deficient volumes in concrete members by integrating vibration testing with finite element (FE) simulation.

Currently, the majority of the quality assurance methods required for PCI Plant Certification focus on quantifying the strength and dimensional conformity of a concrete member. To check for the defects potentially formed during the casting process, such as rock pockets or honeycombing, only visual inspection of the surface is required (PCI MNL.116-99 [1999]). Although visual inspection may be appropriate for
defects that are contiguous to the surface, such inspection is ineffective in the identification of defects that are within the interior recesses of the beam. The proposed method can supply a rapid and cost-effective inspection technique to verify the compliance of concrete members with the design requirements and, thus, venues for quality assessment of precast concrete members at casting facilities.

**RESEARCH SIGNIFICANCE**

This research has led to the development of a nondestructive defect detection method for structural concrete members. This method shows that the global vibration response measurements obtained from nondestructive tests, coupled with a sound numerical model, can be used not only to detect but also to locate and determine the severity of a defect within a structural member. Therefore, this method can mitigate the reliance on visual inspection or semi-invasive methods that involve drilling samples. Thus, the proposed method has potential for use as a quality-control tool for fabricated concrete members with simple and repetitive geometries similar to those that are common in the precast concrete industry.

**BACKGROUND**

Theoretically, in the context of FE analysis, if a sufficient number of response characteristics of a structure are identified experimentally (assuming that the experiments are identically repeatable), the parameters necessary for reconstructing the elemental matrices of the structure can be retrieved. The sufficiency requirement for experimental data is satisfied if the number of equations that can be written is equal to the number of unknown properties of each individual FE. With this approach, a finite number of FEAs associated with the defected region can be identified by monitoring the regions within which the property values, such as mass and/or stiffness values, are reduced. Such an approach entails reconstructing the entirety of elemental matrices that define the mass and stiffness distribution of the structure.

For simple structures, the approach of reconstructing the discrete elemental matrices has been shown to be demonstrably successful. Refer to, for instance, Aoki et al.’s (2005) study on a brick chimney, in which correction factors are assigned to both mass and stiffness of each FE. These correction factors are sought through an optimization algorithm where the disagreement between the model predictions and the experimental results are minimized and the FEs with reduced stiffness are identified as damaged regions. Although the results presented by Aoki et al. (2005) are quite promising, for complex problems with large numbers of FEs, this reconstruction approach inevitably renders defect detection an underdetermined problem (or poses prohibitively high demands on experimental resources).

A tangent of this reconstruction approach focuses on components within a structure (that is, a beam, column, or wall) instead of individual FEs. This alternative approach, compared to the full reconstruction of elemental matrices, can significantly reduce the number of the correction factors that must be inferred from the experimental measurements. However, if the number of components included in the identification problem is substantial, and if the experiments are scarce, this approach may still render detection an underdetermined problem. On the other hand, if the number of components is too small (that is, the components are large in size), the method cannot identify the precise location of the defect, but instead smears the effect of the defect over the entire component. This approach, which is more suitable for detecting distributed defects, is successfully demonstrated by Gentile and Saisi (2007) on a masonry tower where masonry walls with diffused cracks are identified using experimentally obtained natural frequencies.

Focusing on components within a structure (or segments within a component), let us consider a beam divided into 18 segments (Fig. 1). With this configuration, to infer the stiffness and mass properties of the defected region, individual parameters of 18 segments must be calibrated against experimental data yielding a total of 36 calibration parameters ($i = 1:18; K_i$ and $M_i$). This approach may yield nonunique calibration solutions unless the available experimental information is of sufficiently high quantity (and of course, quality). From an empirical perspective, however, such high demands on experimentation may not be feasible.

There is a need for a method that can identify a localized defect and can still be formulated as a determined problem with a feasible amount of experimental data. This manuscript presents such an approach by only seeking the parameters that characterize the defect, such as location and severity, instead of aiming at the reconstruction of the entirety of elemental matrices. In this approach, a defect is defined in terms of three parameters: its coordinate across the length of the beam ($x$) and two reduction factors (analogous to the correction factors of Aoki et al. (2005)), and $\alpha$ and $\beta$ corresponding to stiffness and mass, respectively (Fig. 2). Therefore, the structural parameters of mass and stiffness are considered uniform and equal to a baseline value throughout the structure, except at distinct locations.
indicated by the presence of a defect. Herein, the stiffness reduction ratio $\alpha$ refers to the ratio of the stiffness of the segment with the internal defect to that of a segment without the defect. The mass reduction ratio $\beta$ is similar to $\alpha$ but focuses on the mass. In the presence of a single defect, this approach reduces the model calibration to three parameters ($x$, $\alpha$, and $\beta$) only, instead of 864 variables necessary for the reconstruction of the entirety of elemental matrices or 18 variables necessary for the reconstruction-of-components approach. The proposed approach thereby significantly reduces the demands on experimental data. Moreover, if the goal is to only detect and locate the defect, only one variable needs to be defined. Note that when more than one defect is present in the member, the number of calibration parameters increase in proportion to the number of defects; that is, for two defects, six parameters become necessary. In this paper, the authors successfully demonstrate the application of this approach for detecting single rock-pocket and honeycomb defects in concrete beam-like structures.

### SCALED CONCRETE BEAMS WITH CONTROLLED DEFECTS

The case study structures encompass half-scale lightly reinforced concrete beams 15 x 22.5 cm (5.91 x 8.86 in.) in cross section and 1.83 m (72 in.) in length. A total of five beams are cast: one with no internal defects; two with small and large rock-pocket defects; and two with small and large honeycomb defects, which are described in terms of their corresponding stiffness and mass reduction ratios in Table 1. The rock-pocket defects were reproduced by placing an aggregate cluster surrounded by an impermeable net within the wood formwork before the concrete was poured, and the honeycomb defects were reproduced by placing prismatic foam of predetermined size (refer to Fig. 3). Both rock-pocket and honeycomb defects are generated with two different severities, thus yielding two levels of reduction in the moment of inertia and the area of the concrete cross section (Fig. 4). The length of the defects across the length of the beam is kept constant at 10.2 cm (4 in.). For each beam, the center of the defect is located 47 cm (18.5 in.) away from the end.

The defect detection approach described herein relies on the successive comparisons of the test and FE analysis results; thus, the proposed approach requires a baseline numerical model that is an accurate and unbiased representation of the beam without defects. Therefore, it is of utmost importance to ensure that the boundary conditions and material properties of the numerical model closely match those of the concrete beams cast in the laboratory. For boundary conditions, free-free supports are preferred due to their ease to simulate in the numerical model and to approximate in the laboratory. However, the method discussed herein is also applicable for concrete components with hinged and fixed boundary conditions (Atamturktur 2010). The concrete batch for the scaled beam specimens was designed according to ACI 211.1-91 (ACI Committee 211 1991) exhibiting a design compressive strength of 5000 psi (34.5 MPa). Material properties, such as Young’s modulus and density, are determined experimentally from two cylinder samples by conducting compression tests to establish Young’s modulus, and by measuring the weight of the specimens to back-calculate the density.

### EXPERIMENTAL PROGRAM

The objective of the experimental campaign is to determine the modal parameters and their derivatives, such as mode shape curvatures for the beam without defects, Beam 1; and the distortions in these properties for the beams with controlled defects, Beams 2 to 5.

To reproduce a free-free support condition, the beams are suspended by bungee cords at each end of the beam attached to a steel frame, as shown in Fig. 5. While reproducing a free-free boundary condition in the laboratory, a rule of thumb is assuring a ratio of 10:1 for the first natural frequency of

### Table 1—Reductions in stiffness and mass for each beam segment

<table>
<thead>
<tr>
<th>Beam</th>
<th>Stiffness reduction ratio $\alpha$</th>
<th>Mass reduction ratio $\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 1: Control beam</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Beam 2: Small rock pocket</td>
<td>0.93</td>
<td>0.98</td>
</tr>
<tr>
<td>Beam 3: Large rock pocket</td>
<td>0.86</td>
<td>0.91</td>
</tr>
<tr>
<td>Beam 4: Small honeycomb</td>
<td>0.96</td>
<td>0.92</td>
</tr>
<tr>
<td>Beam 5: Large honeycomb</td>
<td>0.88</td>
<td>0.83</td>
</tr>
</tbody>
</table>

Fig. 3—Inclusion of (left) honeycomb defect; and (right) rock pocket defect during casting of beams.
the structure and highest frequency of the rigid body modes (also known as bounce frequency) (Wolf 1984; McConnell 2006). For a lightly damped structure, Carne et al. (2007) reported a ratio of 15, which yielded less than 1% error in the natural frequencies. For the suspended beam without defects studied herein, the highest frequency of the rigid body modes is 5 Hz and the first natural frequency of the beam is 235 Hz, yielding approximately a 50:1 ratio. Hence, the boundary conditions obtained with bungee cords during the experiments are deemed to satisfactorily represent the free-free boundary conditions.

Impact hammer tests are conducted to determine the acceleration responses in the vertical direction due to a vertical impact excitation. Measurements at 57 uniformly distributed points (Fig. 6) are collected with integrated electronic piezoelectric (IEPE) uniaxial accelerometers with a sensitivity of 50 mV/ms$^{-2}$ (approximately 500 mV/g). An impact hammer, with a sensitivity of 0.2205 mV/N (1 mV/lbf) and a maximum force capacity of 22.2 kN (5000 lbf) (Fig. 7), is used to excite the structure. In that each accelerometer weighed 4.6 g (0.2205 lb), their contribution to the weight of the system in its entirety is negligible. A total of nine excitation points are selected to extract both bending and torsional mode shapes: three at 1/6-length (excitation No. 1 through 3 in Fig. 6), three at 1/3-length (excitation No. 4 through 6 in Fig. 6), and three in the middle of the beam (excitation No. 7 through 9 in Fig. 6). The frequency resolution is set at 0.5 Hz and the time resolution is set at 122.2 microseconds. The acceleration response is measured for 2 seconds to attenuate the response of the beams, thus obviating the need for an exponential windowing function. Anti-aliasing filters are used to prevent high frequencies from contaminating the measurements. Time-history vibration responses are collected using an identical configuration for each of the five beams.

For these beams, the frequency response functions (FRFs) are obtained by calculating the ratio of FFT of the time domain vibration response over the FFT of the time domain hammer impulse (refer to Fig. 8). To reduce the degrading effects of noise due to ambient vibrations, each measurement is repeated five times for an average of five FRFs. Quality control checks, such as reciprocity and homogeneity checks as suggested by Reynolds and Pavic (2000), are completed to ensure high-quality data—that is, high coherence and a low signal-to-noise ratio.

Stability diagrams, such as the one in Fig. 9 for Beam 1, are essential mechanisms for the selection of FRF poles corresponding to the modes of the tested structure. To generate a stability diagram, the mode estimation algorithm used herein—the Rational Fraction Polynomial Method (Richardson and Formenti 1982)—is repeated iteratively with increasing model order (up to 40) and the poles are calculated at each iteration (Vanlanduit et al. 2003). A repeating of the poles with a corresponding increase in the model order implies that the pole is a global characteristic of the system and a physically relevant system mode. Conversely, poles that do not repeat at increasing iterations are mathematical poles that are often due to the presence of noise in the measurements, and as such are not physically indicative of the system modes (refer to Fig. 9 for the beam without defects; similar stabilization diagrams are obtained for the remaining four beams).

From the stabilization diagram presented in Fig. 9, a total of six modes (three flexural and three torsional modes) are identified in ranges between 235 and 1600 Hz for the beam without defects, the frequencies of which are provided in Table 2 for Beam 1. Note that the mode shape vectors of torsional modes exhibit significantly more variability...
between repeated experiments compared to the flexural modes; therefore, only flexural mode shapes, which are presented in Fig. 10, are exploited herein.

NUMERICAL METHODOLOGY

The FE model of the beam without defects—that is, the control beam or Beam 1—is constructed in ANSYS v.12. Obtained by testing two cylindrical specimens, the nominal Young's modulus is established as $2.86 \times 10^7 \text{kN/m}^2 (5.97 \times 10^8 \text{lb/ft}^2)$ and the nominal density is established as 2400 kg/m$^3$ (150 lb/ft$^3$). The model is meshed with SOLID95 elements with a total of 864 elements and 4700 nodal points (refer to Fig. 1). A subsequent mesh refinement study is completed to verify that the selected mesh size is appropriate and that the model yields converged solutions (Roache 1998). The natural frequencies estimated by the FE model are compared against those obtained experimentally (refer to Table 2). As seen, a sound numerical model is obtained as a “reference” for the beam without defects, which is an indispensable component of the proposed approach, which relies on model calibration.

Defect detection through model calibration necessitates a suitable parameterization of the defect of interest in the FE model. The FE model presented earlier in Fig. 1 contains 18 segments of 10.2 cm (4 in.), each of which exhibits an independent mass (density) and stiffness (Young’s modulus) property. Recall that internal defects have a length of 10.2 cm (4 in.) across the length of the beam. Therefore, the reduction in moment of inertia and cross-sectional area are analogous to the reduction of the stiffness and mass of the segment the defect is located, respectively (refer to Fig. 1). Of course, this statement assumes the defect to be located precisely in one of the segments shown in Fig. 1. In this study, the controlled beam specimens are constructed so that this assumption is applicable. The authors illustrate in the following section that no difficulty occurs within the detection process should the precise location of the defect not coincide with the segments.

Note that various vibration response features demonstrate variable sensitivities depending not only on the geometric form, material, and boundary condition but also on the damage type, location, and severity (Prabhu and Atamturktur 2012). Therefore, it is of paramount importance to select a sensitive response feature to increase the success of the defect detection process. Such sensitivity analysis can be completed solely based on model simulations prior to the initiation of the experimental defect detection campaign (provided that numerical model is demonstrated to agree well with experiments).

With the calibrated numerical model for the beam without defects, 162 computer runs are executed to investigate the changes in the vibration response of the beam for various scenarios of location and severity of the defects considered herein. The scenarios to be investigated are selected using a full factorial design of experiments considering the full beam length and three different levels of severity for reduction in mass and stiffness ($0.4 \leq \alpha \leq 1$ and $0.7 \leq \beta \leq 1$).

These 162 runs supply an excellent data set to evaluate the sensitivity of various features to not only the presence but also the location and severity of defects evaluated herein. Using this data set, the natural frequencies, mode shapes, and mode shape curvatures for the first five modes of the beam are evaluated. Natural frequencies and mode shapes are found to be insensitive to the presence of internal defects. To reiterate, the changes in the natural frequencies and mode shapes due to defects were within the same order of magnitude of the variability observed during repeated experiments. However, mode shape curvatures are observed to be more sensitive to the presence of rock-pocket and honeycomb defects. Specifically, the fifth mode shape curvature (that is, the third flexural curvature) is determined to be most sensitive in the presence of internal defects and thus the most suitable for FE model calibration. However, the observations presented herein may differ for other concrete structures and components of different geometric forms and boundary conditions, and sensitivity analysis must be repeated to find the most sensitive feature for such structures.

Fig. 6—Experimental setup including 57 measurement locations and nine excitation locations.

Fig. 7—(Left) nine uniaxial accelerometers mounted on beam; and (right) hammer operator exciting structure with impact hammer.
Mode shape curvature is the first derivative of mode shape and is inversely related to the stiffness within the proximity of its calculation. Hence, the presence of an internal defect abruptly decreases the stiffness in its vicinity and, thus, abruptly increases the mode shape curvature. From the mode shape vectors, it is thus possible to calculate the mode shape curvatures according to Eq. (1)

\[ \kappa_i = \frac{(V_{i+1} - 2V_i + V_{i-1})}{\Delta x^2} \] (1)

Herein, \( V_i \) is the normalized mode shape value at point \( i \), and \( h \) is the horizontal distance between point \( i-1 \) and point \( i+1 \). Figure 11 compares the mode shape curvatures of the reference beam (Beam 1) with that of the four beams with controlled honeycomb and rock-pocket defects (refer to Table 1).

Figure 12 presents the comparison of the experimental and simulated fifth mode shape curvature for the four beams constructed with defects (Beams 2 through 5). Herein, light gray lines represent an ensemble of mode shape curvature simulations with internal defects of varying levels of severity (0.4 ≤ \( \alpha \) ≤ 1 and 0.7 ≤ \( \beta \) ≤ 1) at various locations along the beam (1 ≤ \( x \) ≤ 18), and dark lines represent the experimentally obtained mode shape curvature for mode five for each excitation location. As is evident, the ensemble of simulated mode shape curvatures envelop the experimentally obtained mode shape curvature—that is, one FE model exists with the right configuration of defect location, type, and severity that best match the mode shape curvature collected from the beam of interest. Therefore, the objective herein is to find that model with the appropriate configuration of defect-identifying parameters.

DEFECT DETECTION RESULTS IN CONCRETE BEAMS

For the concrete members evaluated in this study, the defect-identifying variables are sought by systematically comparing the mode shape curvature predictions of the FE model to those obtained by experimental measurements. Therefore, the defect detection problem is defined as one of statistical inference, where the three defect-identifying variables are treated as parameters to be inferred. In the configuration adapted herein, the low number of calibration parameters permits the successful implementation of a variety of inference techniques. In this study, the Bayesian inference technique as described in Higdon et al. (2008) is employed.

Next, the defect detection problem is evaluated in several stages of increasing levels of information regarding the defect, where the first objective is to localize the defect. Upon localization, the purpose then becomes one of deter-
mining the reduction in stiffness and mass, respectively, as elucidated in the following sections.

**Localization of defects**

Herein, defect location is defined by unknown parameter $x$, which refers to the coordinate of the defect along the beam length, and which is inferred in a probabilistic manner using the fifth mode shape curvature. Although the correct parameters for $\alpha$ and $\beta$ are known for Beams 2 through 5 in the controlled study (refer to Table 1), these parameters are unknown to the analyst during the actual search for the defect location. Therefore, an assumption must be made regarding the likely values for these two correction factors $\alpha$ and $\beta$. To reiterate, the location of the defects must be determined in the absence of precise knowledge regarding the severity of the defect.

Therefore, the values for the stiffness and mass reduction factors are treated probabilistically between 0.7 and 1.0. In the absence of knowledge regarding the prior distributions of $\alpha$ and $\beta$, a uniform distribution is accepted; however, other forms of distributions may be incorporated in situations where such prior knowledge is available. The values of parameters $\alpha$ and $\beta$, being equal to 1.0, indicate the instances without defect. The value of $\alpha$ being equal to 0.7 indicates a 30% reduction in stiffness in the corresponding segment. Similarly, the value of $\beta$ equals to 0.7, indicating a 30% reduction in the cross-sectional area of the beam. A 30% reduction in stiffness and/or mass is deemed to represent a sufficiently large defect for the purposes of this study. However, in other applications, the probable severity of the defect can easily be altered by changing the ranges within which $\alpha$ and $\beta$ values are allowed to vary.

Through successive and systematic comparisons of measurements with model simulations, the posterior distributions of parameter $x$ are obtained for all four beams with defects (Beams 2 through 5). Note the results in Fig. 13 focusing on half the length of the beam. The zone between the two solid lines is the actual defect location (corresponding to the fifth segment), while the dash line is the mean value of the inferred posterior distribution—that is, likely values for parameter $x$. In this figure, it is clearly evident that the changes in mode shape curvatures from internal defects are informative and the model calibration process is able to retrieve the correct location of the defect as the fifth segment from the experimental measurements.

**Determining stiffness reduction ratio**

The segment of the beam with a defect exhibits a smaller moment of inertia due to the reduction in the cross-sectional area. Because the stiffness is proportional to the product of Young’s modulus and moment of inertia, a decrease in the moment of inertia results in a lower stiffness for the segment with the defect than that of the segment without the defect. For the scaled laboratory beams, the stiffness reduction ratio is calculated as the ratio of moment of inertia of the segment with a defect to that without the defect (refer to Table 1). During the calibration process, reduced stiffness is expressed as the reduction of Young’s modulus for that particular segment, instead of the moment of inertia. This treatment eliminates the need for the development of complex geometric forms to represent the reduction in moment of inertia while providing the same reduction in the overall stiffness of the segment.

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**Table 2—Natural frequencies obtained from Beam 1 without defects**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Experimental data</th>
<th>Simulation data</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>235</td>
<td>234.6</td>
</tr>
<tr>
<td>2</td>
<td>520</td>
<td>549.8</td>
</tr>
<tr>
<td>3</td>
<td>600</td>
<td>599.2</td>
</tr>
<tr>
<td>4</td>
<td>1044</td>
<td>1100</td>
</tr>
<tr>
<td>5</td>
<td>1070</td>
<td>1074</td>
</tr>
<tr>
<td>6</td>
<td>1600</td>
<td>1614</td>
</tr>
</tbody>
</table>

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**Fig. 10**—(Left) first three bending modes obtained experimentally; and (right) first three bending modes obtained numerically.

**Fig. 11**—Fifth mode shape curvature obtained from all five beams.
While inferring the stiffness reduction ratio $\alpha$, the prior knowledge gained in the previous section regarding the location of the defect is used and the parameter $x$ is set equal to the fifth segment. While the information regarding the location of the defect is available, the reduction in the mass due to defect still remains as an unknown and is treated as a random vari-

Fig. 12—Comparison of experimental and simulated curvature of fifth mode shape.

Fig. 13—Posterior distribution representing defect location $x$. 

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able with uniform probability between 0.7 and 1.0, while the stiffness reduction ratio \( \alpha \) is allowed to vary between 0 to 1.0.

The obtained posterior distribution plots for defect-identifying variable \( \alpha \)—that is, the likely values for the stiffness reduction factor, are presented in Fig. 14, with dashed lines representing the known, correct values for variable \( \alpha \). The numerical comparison of the correct and inferred values for parameter \( \alpha \), given in Table 3, are within 5% for Beams 2 and 3, both of which have rock-pocket defects. For Beams 4 and 5, the results indicate that the inferred values for parameter \( \alpha \) are also in acceptable agreement (approximately 10%) with the correct values calculated for the laboratory beams with controlled honeycomb defects. Of particular interest are the likely values for the stiffness reduction factor of Beams 4 and 5, which encompass a greater range compared to that of Beams 2 and 3. Such a discrepancy can potentially be elucidated by the contaminating effects of the reduction in mass, which is more pronounced for honeycomb defects compared to rock-pocket defects.

**Determining mass reduction ratio**

Similar to the manner in which the stiffness reduction ratio is defined, the mass reduction ratio \( \beta \) is the ratio of the mass of the segment with a defect to the mass of the segment without a defect. Each beam segment is again configured to have an independent mass reduction ratio.

The posterior distributions for parameter \( \beta \) are given in Fig. 15. Again, the dashed line represents the known, correct values for the parameter. Figure 15 demonstrates that the proposed approach overestimates the reduction in mass due to the internal honeycomb defects.

**Table 3—Comparison of calibrated a values with correct \( \alpha \) values**

<table>
<thead>
<tr>
<th>Beam</th>
<th>Inferred value</th>
<th>Correct value</th>
<th>Percent difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.89</td>
<td>0.93</td>
<td>4.3%</td>
</tr>
<tr>
<td>3</td>
<td>0.88</td>
<td>0.86</td>
<td>2.3%</td>
</tr>
<tr>
<td>4</td>
<td>0.89</td>
<td>0.96</td>
<td>7.3%</td>
</tr>
<tr>
<td>5</td>
<td>0.88</td>
<td>0.88</td>
<td>0%</td>
</tr>
</tbody>
</table>

**Fig. 14—Posterior distribution representing stiffness reduction ratio \( \alpha \).**

**Fig. 15—Posterior distribution representing mass reduction ratio \( \beta \).**
CONCLUSIONS
In this paper, a methodology to detect the presence and determine the location and severity of internal defects in concrete members was demonstrated. This method is particularly suitable for the precast concrete industry as an FE model, for a structural element can be calibrated once with a full experimental analysis and the method can then be applied to identical structural elements with significantly fewer measurements required.

With this approach, the consequence of internal defects can be represented at different precision levels that vary from either a very refined FE to a substructure consisting of several FE. Because the desired precision is reflected in the parameterization of the defect, for improved precision, a greater amount of experimental information becomes necessary. In this study, because only lower-order vibration modes were used, the defects are represented with a relatively crude model. However, a more sophisticated and precise approach than described herein may require extensive experimental information and is unlikely to yield practical tools for rapid assessment of existing concrete structures.

The internal defects considered in this study are localized and are limited to a single defect per beam. In the future, the ability of the method in detecting multiple defects within a concrete member will be investigated. Furthermore, although 57 experimental data points were used in this study, the applicability and reliability of this approach with fewer experimental data remains to be verified. The approach presented herein is extendable to a two-dimensional and three-dimensional (3-D) analysis to enhance its capability as a nondestructive quality assessment tool of existing systems. Therefore, future work will entail investigating the feasibility of this method on in-service, full-scale structures through 3-D analysis.

Increasing the service life of buildings through inspection and maintenance is of paramount importance, especially in countries where the infrastructure is approaching the end of its design life. As such, detecting internal defects in concrete structures, according to the method provided herein or those outlined in IAEA (2002), can help determine whether the structure still meets the needs of its designed use and can lead to timely and effective maintenance, thus reducing not only the risk for economic loss but also the environmental impacts of new concrete construction.

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REFERENCES


