Fibre Bragg grating sensor-based damage response monitoring of an asymmetric reinforced concrete shear wall structure subjected to progressive seismic loads

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\textbf{Summary}
This research explores the seismic damage response of a frame–shear wall reinforced concrete structure exposed to progressive seismic excitations. The shear wall structure is a three-storey one-quarter-scaled reinforced concrete structure. The test was conducted by applying progressive seismic excitations using a shaking table. The shear wall structure experienced inelastic deformations under high seismic excitations that eventually caused the transformation of the structure from a state of elastic deformation to a state of highly inelastic deformation. Damage in the form of plastic hinges occurred because of induced dynamic instability in the structure. The monitored seismic responses in this research include the variation in the residual strains and their dynamic time histories. Precise and detailed measurements of varying strain responses and effective estimation of damage response within the structure have been a primary goal in this research. Successful implementation of fibre Bragg grating (FBG) strain sensor is presented and applied in this experiment because of its enhanced sensitivity towards monitoring the structural dynamic strain response. The presented research also evaluates the suitability of FBG strain sensors in dynamic testing of a frame–shear wall asymmetric structure by comparing the damage response obtained through FBG sensors and the predictions developed from the dynamic properties of the test structure, monitoring the progress in structural damage and predicting the cracks inside the structure. The monitored responses obtained through a series of tests prove that FBG sensors have the advantages of significant accuracy, small size, and good embedding abilities. It demonstrates its promising failure monitoring abilities and potentials for the crack detection. The results achieved through this research would be beneficial to validate existing numerical simulation and analytical procedures, particularly for the structures with inherent asymmetry. It has been demonstrated in this paper that (a) the structure entered into the critical state when the ground motion excitation was 0.5 g with invisible cracks, (b) a sudden change in the residual strain response can help in detecting the initiation of a crack, and (c) the damages in...
the structure were due to the formation of the plastic hinges at the flexible edge of the structure near the beam-column joints.

**KEYWORDS**
asymmetric structure, damage detection, progressive seismic loading, shake table test, structural health monitoring

# 1 | INTRODUCTION

It has been well known that old reinforced concrete (RC) buildings are more prone to collapse under earthquakes because of the fact that they were designed using outdated earthquake-resistant building guidelines. Findings based on various case studies from recent and past earthquakes demonstrate that sometimes collapses or structural damages occur even when the design of a structure is in compliance with the seismic design guidelines. The reasons for such a kind of unforeseen collapse in the structures can be assessed from various collapse aspects, for example, the uncertainties inherent to the earthquakes, featured earthquakes with higher potential of causing damage, and lack of knowledge towards the seismic-resistant design and asymmetry of structures. Numerous researchers have conducted studies on asymmetric structures.

One of the biggest hurdles in the development of understanding towards the damage process of frame–shear wall structures is the unavailability of the realistic experimental dynamic responses corresponding to the structures exposed to severe evaluation levels, that is, structures under damage states. Lately, numerous shake table experiments have been conducted to evaluate the damage states prior to collapse of the structure and to assess the numerical validation procedures used for the damage simulation of RC frame structures. For instance, Elwood and Moehle carried out shake table experiments on two single-storey RC frame structures. In this experiment, exterior ductile columns were utilised for alternate load path redistribution once the interior columns reached the damage state and all its capacity was lost. The interior columns were designed as nonductile columns, and the exterior columns were designed to have the ability to provide alternate load paths. The achieved responses were used to verify the shear and axial failure of columns obtained through nonlinear analyses. Kim et al. conducted a shaking table experiment on an RC frame wall structure containing eccentricity between the centre of stiffness and centre of strength at the first-floor level to evaluate the influence of asymmetry on structural capacity. The stiffness and strength eccentricity at the first-floor level was introduced by providing wall and column frames at the first-floor level and only wall frame at the second-floor level. Detailed definitions corresponding to the centre of stiffness and centre of strength can be found in the literature. Ghorbanirenani et al. carried out the shaking table experiment on two identical RC cantilever shear walls. These cantilever shear walls were exposed to high-frequency seismic excitations at various amplitudes, and as a result, damage response was investigated. Wu et al. evaluated the global collapse behaviour of a single-storey structure along with the investigation concerning dynamic softening of the low-ductility column of the same structure using a shake table. Wu et al. further carried out another experimental investigation on a single-storey structure damaged in axial failure mode and flexure shear failure mode. The achieved responses were utilised for the assessment and adequacy of existing simplified evaluation procedures in estimating the damage in the structures. Benavent-Climent et al. conducted a uniaxial shaking table test of a scaled RC structure and evaluated the torsional responses under seismic actions. Li et al. explored the damage process in an RC frame structure against seismic excitations by implementing the shake table experiment on a scaled frame structure. In terms of local failure, a shaking table experiment on a three-storey RC frame structure was conducted by Ghannoum and Moehle to evaluate the local failures in columns. They also utilised the achieved results to further evaluate the local failure in the columns and their potential consequences in inducing vulnerability of the structures against global damage.

The damage investigations in the presented literature involve traditional response measuring methods for damage evaluation. In the recent past, more sophisticated methods have become available for damage investigation. Among which, the use of fibre Bragg grating (FBG) sensors for structural health monitoring is gradually increasing in the existing infrastructure facilities. Several researchers have provided a systematic review on the applications of FBG sensors in the existing infrastructure facilities. In terms of applications of FBG sensors on the performance of existing infrastructure facilities, various researchers have carried out research investigations. For example, Brönnimann et al. conducted a research investigation on an under-construction cable-stayed bridge for a period of 6 months and assessed the long-term stability of FBG sensors. Kerrouche et al. investigated damage response in an RC railway bridge under inelastic deformations using FBG sensors. Kister et al. investigated the stability of pile foundations by measuring temperature and strain response using FBG sensors. Many other researchers have also evaluated the long-term monitoring
performance of FBG sensors by carrying out health monitoring assessment of existing bridges. Furthermore, in terms of new developments in the sensing technique of FBG sensor, many researchers have proposed new FBG sensing schemes for health monitoring of highway bridges. Applications of FBG sensors are also widespread to other infrastructure facilities, mainly including wind turbines and tunnels. Kim et al. investigated the deflection response of the blades of wind turbines using FBG sensors. Arsenault et al. carried out real-time monitoring of wind turbines at various loading states based on FBG sensors. Bang et al. measured bending and strain response in wind turbine towers using FBG sensors. Ye et al. carried out health monitoring investigation of a tunnel under construction using FBG sensors. Li et al. investigated the stability of tunnels by monitoring surface strain using FBG sensors. Numerous researchers have conducted research on FBG strain sensors and its applications in the field of health monitoring in various civil structures.

Considering the applications of FBG sensors in building structures, it should be noted that only few researchers have explored the use of FBG sensors to date. In this regard, one of the prominent research over the use of FBG sensors in building structures to monitor the wind response is made by Ni et al. where the use of massive deployment of FBG sensors in Canton tower, China, for temperature and strain monitoring has been demonstrated. Another important investigation was conducted by Li et al., which includes investigation on the performance of FBG sensors in an under-construction 18-storey structure. The research evaluated the temperature and strain measurements during three main stages of construction: (a) before pouring of the concrete, (b) during the pouring stage of the concrete, and (c) after pouring of the concrete.

Previous literature related to FBG-based health monitoring of RC structures does not fully guide about the damage response in building structures, and the main reason behind this is that the majority of FBG-based health monitoring studies are limited to bridges, dams, tunnels, and other infrastructure facilities. Its applications are not as widespread to buildings as its applications to other infrastructure facilities. Moreover, previous research on FBG sensors provide an insight into the structural performance with apparently no physical damage to the structure. Estimating the damage response of a physically damaged structure is yet another complicated process and highly depends upon the type sensing technique and location of the sensors. It should be noted that despite the extensive use of FBG sensors in various infrastructure facilities, previous research on FBG sensor-based damage response in RC building structures is nearly none. Therefore, there is a need to fill this research gap by detailed damage investigation in building structures using FBG sensors where a structure actually transforms from an elastic state to an inelastic state under the formation of cracks followed by progressive seismic excitations.

It should also be noted that damage investigation of RC buildings by traditional methods does not truly depict the building’s damage response. The main reason behind this is that the usual practice for damage response monitoring in a shaking table test of a structure involves the use of traditional strain-based damage measurement methods where resistance strain gauges are used on the surface of concrete for strain measurement. Such methods are useful only in monitoring the external deformation variations. Because the external deformation of the structure cannot reflect the damage degree inside of the structure, accurate measurement of strain variation and effective prediction of damage inside the structure are the major objectives of this research. Moreover, liquid adhesives such as cyanoacrylate glue, RTP-801 adhesive, or acrylate adhesive are used to attach these strain gauges onto the surface of structures. Because the strength of emulite concrete materials is smaller than the strength of these adhesives, they are not appropriate for attaching strain gauges to the external surface of the experimental models. These instant adhesives for fixed strain gauges can react with model material, and eventually, they can influence the measurement accuracy. Besides, they are also vulnerable to reduced insulation resistance under intense shaking as they are likely to get removed or displayed followed by the damage in the structure, and eventually, the accuracy of the measurement gets compromised. Some other key points that further enhance the novelty of this research are as follows:

1. As explained in the previous paragraph regarding the research on FBG-based damage response, that too on an asymmetric building undergoing inelastic deformations followed by progressive seismic excitations is nearly none. Therefore, FBG strain sensors were used in this study as these sensors have the ability of periodical variation in the index of refraction of the optical fibre core. Because of its numerous advantages over other technologies and its appropriate features that include its embedding abilities, high sensitivity, electromagnetic interference immunity, and flexibility, its applications in successfully investigating the damage process are evaluated.

2. The existing data stock on experimental damage response of asymmetric structures is not significant; therefore, data collected in this research can be used for future evaluation of damage response, verification, and development of computational models for prediction of damage behaviour in asymmetric RC structures.
2 | FBG SENSING PRINCIPLE

Careful handling and protective housing for the embedded FBG strain sensors are required when laying these sensors in concrete structures. A perfect bond between the concrete and protective housing is also required to insure faithful monitoring of the structural strain through sensors. The widely used embedded FBG sensors are the ones packaged by a capillary steel tube. In some cases, the adhesion of the steel tube with low-strength materials does not cause sufficient deformation in the steel tube. Therefore, this phenomenon causes strain transfer loss due to the decreased sensitivity of sensors. There is always a need for sensitivity enhancement of FBG strain sensors. The refractive index FBG strain sensors changes cyclically along the axial direction of the fibre. The current principle of various sensors based on FBGs can be attributed to the measurement of the centre wavelength $\lambda_B$ of the Bragg grating, that is, by measuring the drift caused by the external disturbance. The measurand parameters were obtained through Equation (1), which are related to the fibre grating length period $\Lambda$ and effective refractive index $\eta_{\text{eff}}$ of the fibre core.

$$
\lambda_B = 2\eta_{\text{eff}}\Lambda
$$

The famous principle of end-bearing piles and friction widely used in civil infrastructure-related works was utilised to increase the cohesive force between the measurand materials. The cohesive force and the surface area roughness between the measurand materials were increased by designing the cube shapes and mounting them at the supports. The mounting supports were fixed with grippers, and both sides of the FBG sensors were set with the mounting supports in order to have a better transfer of cohesive force to the sensor. A steel tube was used for packaging of the bare FBG sensors located between the grippers. Epoxy resin was used to encapsulate the fibre in FBG sensors in the grippers. The grippers on the mounting support were installed by a solder. The installed steel tube serves only as a protective housing and does not transfer strain to the sensor during the pouring process of concrete. Also, grippers and mounting supports could freely slip through the steel case. The force due to cohesion between the material and mounting support causes the slip of grippers along the steel tube. This eventually yields the deformation of bare FBG and grippers. The illustrations have been presented in Figures 1 and 2.

3 | MODEL DESIGN AND GEOMETRIC CONFIGURATION OF THE STRUCTURE

Model design is one of the important issues that need to be addressed in order to obtain successful results. Therefore, the design of the test structure was based on determining not only the similarity constants but also various other factors such as the type of production model, the material of the model, the test conditions, and a model to determine the physical similarity constants. The process of structural model testing objectively reflects the interrelationship between the relevant physical quantities involved in the work. Because of the similarity between the prototype and the model, it necessarily reflects the relationship between the similarity of prototype structure and the model. The similarity constant will thus determine the relationship between the prototype structure and model; that is, the model design needs to follow these principles.
The model test not only requires the elastic phase of the stress analysis of the data but also requires a correct reflection of the nonlinear structure of the prototype performance requirements that can then be reflected to the prototype structure, that is, the ultimate deformation capacity and the ultimate bearing capacity that are more important for the structural seismic test. In this research, keeping in view the stringent requirements of the RC structure and the actual situation available for the testing facility, similarity relationships were established. The structural properties of high-rise buildings under seismic actions are usually carried out on shaking tables using the same scale model as the prototype model. That is, the physical process contains the following physical quantities: structure size, the level of structural displacement, stress, strain, the elastic modulus of the structural material, the average density of structural materials, the weight of the structure, the vibration frequency of the structure and structural damping ratio along with the displacement amplitude of seismic excitation, and the maximum frequency of movement. With the dimensional analysis method, the similarity constants of the system were defined as the dimension matrix.

The shake table used in this research consists of dimensions 3 m × 4 m with a payload capacity of 10 tonnes located at the State Key Laboratory of Coastal and Offshore Engineering, Dalian University of Technology, China. The prototype was designed following the requirements of National Standard of P.R. China GB50011-2001-code62 for Seismic Design of Buildings following the design requirements of the Dalian region.

Table 1 refers to the similitude requirements implemented in this research, where $N$ demonstrates the ratio of prototype properties and model properties. Stress similarity factor $N_\sigma$ and length similarity factor $N_l$ are considered the key controlling parameters for defining the similitude relationships.

The stress and strain of the designed model according to the scale ratio were kept consistent with the prototype, which eventually means that the displacement of the model is consistent with the prototype. According to the scale reduction, the testing instrument required a much higher accuracy. This condition is generally more difficult to satisfy because it is difficult to maintain the damping coefficient constant because of the change in the structural properties.

![Stainless steel tube-packaged fibre Bragg grating sensor](image)

**FIGURE 2** Stainless steel tube-packaged fibre Bragg grating sensor

| Properties               | Parameter          | Similarity equations | Dimensions | Scale factor |
|--------------------------|--------------------|----------------------|------------|-------------|
| Geometric properties     | Length ($l$)       | $N_l$                | L          | 0.25        |
|                          | Lateral displacement ($y$) | $N_y = N_l$ | L          | 0.25        |
| Material properties      | Strain ($\varepsilon$) | $N_\varepsilon = 1$  | —          | 1           |
|                          | Stress ($\sigma$)  | $N_\sigma = N_E/N_\varepsilon$ | FL$^{-1}$ | 1           |
|                          | Modulus of elasticity ($E$) | $N_E = N_\sigma$ | FL$^{-2}$ | 1           |
|                          | Density ($\rho$)   | $N_\rho = N_\rho/N_l$ | FT$^2$L$^{-4}$ | 2.165 |
|                          | Poison ratio ($\mu$) | $N_\mu = 1$ | —          | 1           |
| Dynamic properties       | Mass ($m$)         | $N_m = N_\rho N_l^3$ | FT$^2$L$^{-1}$ | 0.033 |
|                          | Period ($T$)       | $N_T = \sqrt{N_m/N_\mu}$ | T          | 0.367       |
|                          | Frequency ($f$)    | $N_f = 1/N_T$       | T$^{-1}$   | 2.719       |
|                          | Velocity ($v$)     | $N_v = N_f/N_l$     | LT$^{-1}$  | 0.67        |
|                          | Acceleration ($a$) | $N_a = N_v/N_l$     | LT$^{-2}$  | 1.848       |
|                          | Acceleration of gravity ($g$) | $N_g = 1$ | LT$^{-2}$  | 1           |
| Load                     | Surface load ($Q$) | $N_Q = N_\sigma$ | FL$^{-2}$  | 1           |

Table 1: Similarity relationships for shaking table testing where $N$ refers to similarity factor
under progressive loading. This condition can be ignored if the prototype structure damping is assumed to be small. The model was designed on a one-quarter scale; the required vibration frequency of the table was four times the frequency of the earthquake. The amplitude of the vibration table was the seismic amplitude. This is because the configuration model according to the scale ratio of the frequency itself was increased four times and the displacement was reduced and the test of the vibration table also made the corresponding changes meet the model test results similar to the prototype.

For conventional test conditions, factors that were carefully considered were materials and the premise of the construction model, the eccentric design framework of a two-way cross one-way shear wall model as shown in Figure 3, and integrally cast stratified model construction methods. The structural model was all set for the experiment after 28 days of its construction in order to have the concrete achieve its maximum strength. The reinforcement and geometry model have also been shown in Figure 4. The longitudinal reinforcing bars in beams and columns had a diameter of 6.5, and steel wires were used as shear stirrups. The concrete material was used as C20 concrete using M5 pouring mortar. All columns throughout the structure’s height and in all frames were similar (with the same reinforcement, height, and cross-sectional area), so were the beams in all structural frames, as shown in Figure 4. The beam sections were rectangular in shape with a size of 40 mm × 100 mm, and the columns were of square shape with a size of 80 mm × 80 mm.

FIGURE 3  Specimen geometry: dimensions in mm (a) elevation: Y-frame, flexible side; (b) elevation: Y-frame, stiff side; (c) floor plan; (d) 3D constructed model
EXPERIMENTAL CONFIGURATION AND PROCEDURE

The direction of excitation was considered along the Y-direction (transverse direction) only because of the limited capacity of the shake table equipment, using two waveforms: white noise and the El Centro 1940 earthquake record. Before the test, white noise was used to identify the model frequency and mode. The input small-amplitude white noise signal was used to convert the response time domain signal into Fourier transform to frequency domain signal, which can measure structural dynamic properties (including mode shape, damping ratio, and natural frequency) at each stage. The El Centro (NS 1940) earthquake record was considered for seismic testing because of the fact that it is the most widely used high-frequency earthquake around the globe; therefore, the test result will be helpful for comparison of previously available data. Seismic inputs were progressively applied during the test, that is, progressively increasing the acceleration amplitude of excitation; each level was increased with a peak ground acceleration (PGA) of 0.1 g, in order to obtain an elastic model structure at each stage until the plastic state was achieved. The progressive increase in the seismic excitation has been illustrated in Figure 5a, where a gradual increase of PGA = 0.1 g at each testing level has been shown in the form of acceleration–displacement spectra for the El Centro ground motion along with the time history and velocity spectrum (Figure 5b,c). Because this study focuses on the damage response of the structure, the input ground excitations have been presented as the acceleration–displacement spectra in Figure 5a in order to project the severity of ground motion and displacement demand behaviour of the ground motion with increasing amplitude of input excitation. Table 2 also represents the progressive increase in the ground motion at each test level.

LAYOUT AND PROTECTION OF FBG STRAIN SENSORS

The survival rate of the sensor is critical in damage detection of the structure. In addition to being careful in the deployment process, several protection measures were taken. Because the construction process of the RC structure is a typical operation, the fibre grating sensors were carefully handled during the pouring of concrete. To ensure a proper contact of the FBG sensor with reinforcement bars and to avoid the strain error caused by the strain transfer, the rebars were first...
polished with a grinding machine and then were polished with sandpaper so that the sensor could smoothly stick with the rebars, as shown in Figure 6.

In order to prevent polishing crumbs, oil, and other pollutants, cotton balls dipped in alcohol were used to clean the polished places. The fibre grating sensor along the longitudinal arrangement of rebars had issues related to the sensor buffer and moisture; therefore, at first, glue was fixed around the fibre grating, and then it was wrapped with epoxy resin and external winding gauze. The procedure corresponding to the protection process and layout of the sensor has been shown in Figures 6–9.

The transmission fibre was vulnerable to various construction impacts like concrete pouring, vibration, and impact from the inner mould. Therefore, the length of the transmission fibre was kept as minimum as possible. A hole was left in the formwork near the fibre grating sensor so that the transmission fibre could pass through it easily. The laboratory

**TABLE 2** El Centro 1940 ground motion progressive loading detail

| Test no. | Ground motion input       | PGA  |
|----------|---------------------------|------|
| 1        | IMPVALL/ELC-1940-NS       | 0.2  g|
| 3        | IMPVALL/ELC-1940-NS       | 0.3  g|
| 4        | IMPVALL/ELC-1940-NS       | 0.5  g|
| 5        | IMPVALL/ELC-1940-NS       | 0.6  g|
| 6        | IMPVALL/ELC-1940-NS       | 0.7  g|
| 7        | IMPVALL/ELC-1940-NS       | 0.8  g|
| 8        | IMPVALL/ELC-1940-NS       | 1.0  g|

**Note.** PGA: peak ground acceleration.

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environment was favourable for this to be done. Careful handling was done with the transmission fibres as they were not required to be excessively bent; otherwise, it could have led to weak optical signal or even no signal.

The monitoring data from a total of 17 sensors were available after the test (Figure 10). The sensors were equipped with a total of 20 FBG strain sensors; all of them survived, but because some of the sensors had a single head and were not able to be connected to the channel, one of them was abandoned; the other one was a single head and was destroyed during the installation. For the single-head sensor, the acquisition process was destroyed. Table 3 accompanied by Figure 11 illustrates the layout of FBG sensors in the test structure.

6 | DAMAGE IN THE STRUCTURE IN TERMS OF VARIATION IN THE DYNAMIC CHARACTERISTICS AND MAXIMUM STRAIN RESPONSE

The structural damage response was first observed through a variation in the dynamic properties of the structure as reported in Table 4. These modal characteristics were determined before each loading state. It can be seen that the first- and second-order frequencies decreased gradually, and the corresponding damping ratio increased simultaneously under progressive seismic loading. This is due to the fact that the augmentation of the damage in the structure led to the increase in the width of beams and columns; therefore, the increase in the length of the structural members caused significant reduction in the natural frequency of the structure. However, this phenomenon consequently caused a significant and rapid rise in the damping ratio of the structure. Summarising the modal characteristics variation, it can be said that the structure experienced 63% and 30% reduction in the first- and second-mode natural frequencies in the damaged state, respectively, compared with the first- and second-mode natural frequencies of the structure in the undamaged state. On the other hand, the damping ratio of the structure in the damaged state increased to about 5 times the initial damping ratio of the structure in the undamaged state.

In this experiment, the structure is a frame–shear wall asymmetric structure. When the strain was monitored, the maximum strain corresponding to the axial tensile strength was taken as \( \varepsilon_{0t} = 0.00015 - 0.0002 \), respectively, usually considered as \( \varepsilon_0 = 0.00015 (150 \mu \varepsilon_0) \). The strain response of the structure was monitored through installed sensors.

Figure 12 provides the strain history profiles of all the sensors that were installed in the structure for PGAs of 0.2 and 1.0 g. The strain history profiles have been incorporated with a zoom response at peak points. The tensile strains correspond to the positive strain in strain history profiles whereas the compressive strains correspond to negative strain in the strain history profiles.

As can be seen in Figures 13 and 14, the maximum tensile strains observed by the sensor # 1, sensor # 2, sensor # 5, sensor # 10, sensor # 11, and sensor # 12 reached the maximum limit when the maximum input ground acceleration was 0.2 g. When the test structure was exposed to maximum input ground acceleration of 0.3 g, the tensile strain
monitored by sensor # 3, sensor # 8, sensor # 9, and sensor # 13 reached the maximum limit of $\varepsilon_0$. The location of these sensors is between the frame bars of the two columns of first storey and the beam between the side column frames.

When the input ground motion was further increased to 0.5 g, the strain recorded by all the sensors reached the maximum limit of tensile strain except the two sensors installed in the shear wall named as sensor # 16 and sensor # 17, which did not reach the maximum limit of tensile strain. However, the other points were already in the limit state when a PGA of 0.5 g was applied to the test structure. It can be seen in Figures 13 and 14 that when the input ground acceleration was further augmented from 0.5 to 0.6 g, the strain in sensor # 10 and sensor # 11 became smaller due to the fact that sensor #10 and sensor #11 had a tension force. The edges of the two beams between the column frames were closer to these two sensors where the cracks were also present. The reason for the smaller strain was not due to the placement of sensors in the vicinity but due to the presence of crack near the sensor. These cracks near the sensors released some part of the sensor strain, therefore, the augmentation of input ground motion yielded smaller tensile strain due to release of the part of the strain at crack. When the PGA was further increased to 0.7 g, some very interesting phenomenon were observed. The maximum tensile strain of sensor # 4 was almost doubled whereas the

![FIGURE 9](image9.png) Transmission fibre

![FIGURE 10](image10.png) Layout of sensors: (a) grids plan, the red dash lines correspond to the region of fibre Bragg grating (FBG) sensor installation; (b) location of sensors in the structure, red dots correspond to the location of FBG sensors and the listed numbers correspond to the serial number of each sensor
Compressive strain was increased to nearly three times of compressive strain observed in the previous PGA. This shows the presence of a large crack near this region and also indicates that concrete was crushed at this point. This eventually confirms the formation of a large crack and crushing of concrete at 0.7 g. It is interesting to note that the cracks here appear to cause the original concrete to bear the tensile load developed by the rebar and eventually lead to crack in the concrete. After the concrete was crushed, the compressive strain induced in the concrete was developed and transferred from the rebar.

**FIGURE 11** Location of sensors marked with a dashed rectangle (a) First rebar of the column, (b) second rebar of the column, (c) first rebar of the beam, (d) second rebar of the beam

**TABLE 3** Sensor layout plan and details

| Sensor no. | Location of the sensor                                                                 |
|------------|----------------------------------------------------------------------------------------|
| 1          | The sensor is attached to the first rebar of the first-storey column located at the intersection of grid-1 and grid-A. The location of the sensor is the bottom of the column. |
| 2          | The sensor is attached to the second rebar of the first-storey column located at the intersection of grid-1 and grid-A. The location of the sensors is the bottom of the column. |
| 3          | The sensor is attached to the second rebar of the first-storey beam located at the intersection of grid-1 and grid-A. The location of the sensor is the 1st rebar of the transverse direction beam at the first-floor level. |
| 4          | The sensor is attached to the second rebar of the second-storey column located at the intersection of grid-1 and grid-A. The location of the sensor is the second rebar of the transverse direction beam at second-floor level. |
| 5          | The sensor is attached to the first rebar of the third-storey column located at the intersection of grid-1 and grid-A. The location of the sensor is the first rebar of the transverse direction beam at third-floor level. |
| 6          | The sensor is attached to the first rebar of the first-storey beam located at the intersection of grid-1 and grid-B. The location of the sensor is the first rebar of the transverse-direction beam. |
| 7          | The sensor is attached to the second rebar of the first-storey beam located at the intersection of grid-1 and grid-B. The location of the sensor is the first rebar of the transverse-direction beam. |
| 8          | The sensor is attached to the first rebar of the second-storey beam located at the intersection of grid-1 and grid-B. The location of the sensor is the first rebar of the transverse-direction beam. |
| 9          | The sensor is attached to the first rebar of the second-storey column located at the intersection of grid-2 and grid-B. The location of the sensor is the first rebar of the transverse-direction beam. |
| 10         | The sensor is attached to the first rebar of the longitudinal-direction beam where grid-1 intersects with grid-A at the first-floor level. |
| 11         | The sensor is attached to the first rebar of the transverse-direction beam where grid-1 intersects with grid-A at the first-floor level. |
| 12         | The sensor is attached to the first rebar of the longitudinal-direction beam where grid-1 intersects with grid-B at the first-floor level. |
| 13         | The sensor is attached to the first rebar of the transverse-direction beam where grid-1 intersects with grid-B at the first-floor level. |
| 14         | The sensor is attached to the first rebar of the second-storey column located at the intersection of grid-1 and grid-A. The location of the sensor is the first rebar of the second floor's column. |
| 15         | The sensor is attached to the first rebar of the second-storey beam located at the intersection of grid-1 and grid-A. The location of the sensor is the first rebar of the beam. |
| 16         | The sensor is attached to the shear wall longitudinal reinforcement at the first floor level at the junction of shear wall and floor. |
| 17         | The sensor is attached to the shear wall longitudinal reinforcement at the second-floor level at the junction of shear wall and floor. |
The maximum tensile strain of the sensor # 8 was also increased to double. The compressive strain did not have a significant change which indicates the fact that the concrete here was also cracked but was not crushed. When the column was pulled by the concrete, the tensile strain was developed by the rebar. Because of the compression, the concrete and the steel bars together were subjected to the compressive strain. When the peak value of ground motion was increased from 0.7 to 0.8 g, the maximum tensile strain and compressive strain of the sensor # 4 changed to one half. This shows that the structure was in the plastic state, where the plastic deformation of steel formed a plastic hinge at the joint. The compressive strain of the sensor # 15 was negligible, which confirms the fact that a plastic hinge was formed at this node as well.

7 | LOCAL RESPONSE AND STABILITY OF THE STRUCTURE AT THE FLEXIBLE EDGE UNDER PROGRESSIVE LOADING

7.1 | Local response in the elastic state

The observations made in this section from strain history profiles do not consider the initial strain caused by the casting and pouring of concrete. The explanation about the initial strain consideration is explained in Section 9. The FBG sensors labelled as “sensor # 14” and “sensor # 15” with their location being provided in Figure 20 at beam-column joint has been selected to analyse the damage characteristics and stability of the model under progressive input loading. Figure 15 illustrates the strain time history profiles in the elastic state for the input ground motions of 0.2 and 0.3 g, respectively. It can be seen that the strains of both the sensor # 14 and the sensor # 15 increased gradually with the increase in the input ground motion. The structure remained in the elastic state at PGA of 0.3 g and no visible cracks were noticed at this stage.

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**TABLE 4** Variation in the modal dynamic characteristics of the structure before each test

| Test | First natural frequency (%) | First damping ratio (%) | Second natural frequency (%) | Second damping ratio (%) |
|------|-----------------------------|-------------------------|-----------------------------|-------------------------|
| 0.2 g| 5.55                        | 1.58                    | 24.01                       | 0.67                    |
| 0.4 g| 4.73                        | 2.49                    | 21.40                       | 0.96                    |
| 0.5 g| 3.59                        | 3.50                    | 20.75                       | 1.52                    |
| 0.6 g| 3.10                        | 4.41                    | 18.95                       | 2.15                    |
| 0.7 g| 2.94                        | 5.43                    | 18.46                       | 2.77                    |
| 0.8 g| 2.61                        | 6.62                    | 17.64                       | 3.73                    |
| 1.0 g| 2.10                        | 8.26                    | 16.67                       | 4.75                    |

**FIGURE 12** Strain history profiles; recorded strain profiles corresponding to all fibre Bragg grating sensors for input ground motion of (a) 0.2 g (b) 1.0 g
7.2 | Local response in the microcracking state

The state of internal microcracking was determined from the variation in the dynamic properties of the structure as structural frequencies started to fall significantly in first mode when the input ground acceleration was 0.5 g. In Figure 16, strain profiles at the internal microcracking state has been illustrated. It can be seen that strain response in sensor # 15 shows a complete irrecoverable plastic deformation with significantly high residual strain in the tensile range. There is some amount compressive stress present as well, which can be neglected as the strain response will gradually shift into the tensile range once the seismic loading is further increased.

7.3 | Local response in the in-elastic state

As the input ground acceleration intensity increased from 0.5 to 1.0 g, the shear wall frame structure started to transform to the inelastic state. Also, it was found that tensile strain was greater than the compressive strain in most of strain-time history profiles. Finally, it was discovered that compressive strain was found to be higher than tensile strain in the plastic state for sensor named as sensor # 14 and tensile strain were found to be higher than compressive strain in the plastic state for sensor # 15 (Figure 17). In the elastic state, both sensor # 14 and sensor # 15 reported fairly equal tensile and compressive strain peaks. These observations demonstrate damage scenarios from sensor to sensor.

7.4 | Local response at the stiff edge of the structure

In comparison with the structural behaviour at the flexible edge of the structure, this section provides an insight into the structural response at the stiff edge of the structure where it can be seen that structural deformation at the stiff edge is negligible in all loading states. Figure 18 demonstrates the strain response obtained from sensor # 16 located at the first-
floor level in the shear wall. The demonstrated strain profiles prove the fact that the stiff edge of the structure has not experienced severe deformations, and thus, no apparent damage was discovered at the stiff edge of the structure. Besides, it can be seen that both compressive and tensile strains remained fairly equal in all loading states. Two sensors (sensor #16 and sensor #17) were installed in the shear wall at first- and second-floor level. However, for the reasons of compactness, results from sensor #16 have been illustrated here. For a better picture of structural response at the stiff edge, Figure 18 needs to be observed in combination with Section 6 where absolute tensile and compressive strains have been demonstrated for both sensor #16 and sensor #17.

Figure 19 demonstrates the local strain spectrum of the response monitored from sensor #15. The beam corresponding to sensor #15 has been presented in Figure 20b and 20d. It can be seen from the figure that when the ground motion input peak value is 0.2 g and 0.3 g, the strain spectrum has obvious frequency peaks. After the local vibration...
input peak value is greater than or equal to 0.5 g, a remarkable frequency shift can be observed. With an increase in the input seismic excitation, it can be seen that the strain spectrum loses obvious frequency peak. The information given by the strain spectrum indicates that the second-floor beam joint of Figure 20 has a damage problem when the ground motion input is increased from 0.5 g.

8 | INFLUENCE OF PROGRESSIVE SEISMIC EXCITATION ON RESIDUAL STRAIN

Figures 21 and 22 illustrate the typical graphs of residual strains reported by all the sensors. The neutral axis is the term in material mechanics where the strain monitored by the strain sensor is defined as a “neutral axis” under a single ground motion. At the end of the vibration, if the strain is consistent with the initial strain, that is known as unchanged neutral axis. If the strain at the end of the vibration is higher than the initial strain or below the initial strain, that is
defined as neutral axis up and neutral axis down. Also, if the neutral axis has been declining under the action of several earthquakes, it indicates the presence of cracks. The location of these cracks would be in the vicinity of the sensors because the emergence of cracks would cause the decline of the neutral axis due to the release of tensile strain acting on the sensor (Figure 20). The frequent movement of neutral axis under the action of ground motion indicates the crack near the sensor region. The tensile strain in that crack part is termed as residual strain. The concept of neutral axis was used to determine the residual strains corresponding to each of the installed sensors.

Considering the experimental results, these terms were applied in making the assessment. Also, for complex situations in which, if the neutral axis first moved up and then moved down or the case is to shift, the rear drop is very complex situation and this was analysed in combination with Figures 13 and 14. The two described situations could form when the structure changes its states from elastic to plastic forming a plastic hinge near the joint or it can be formed when the reinforced elastically deformed structure restores a portion of the plastic deformation under the action of ground motion. This analysis has been made only for the sensors installed in beams and columns and does not consider the shear walls as the installed sensors in the shear wall did not record significant change in the strain due to higher stiffness of the wall. From Figures 21 and 22, it can be said that the neutral axis moved up for sensor # 2, sensor # 3, sensor # 6, sensor # 8, sensor # 9, sensor # 12, and sensor # 13 with the first obvious upward shift when the ground motion inputs were 0.4 g, 0.4 g, 0.4 g, 0.6 g, 0.5 g, 0.5 g, and 0.5 g, respectively. The neutral axis shift with the order of neutral axis up and neutral axis down were observed for sensor #7 and sensor # 15. For sensor #7, the neutral axis moved when the input ground motion was 0.3 g, and for the same sensor, the neutral axis moved down when the input ground motion was 0.6 g, respectively. For sensor # 15, the neutral axis moved up when the input ground acceleration was 0.4 g, and for the same sensor, the neutral axis moved down when the input ground acceleration was 0.7 g, respectively. The neutral axis shift with the order of neutral axis down and neutral axis up were observed for sensor #11 and sensor # 15. For sensor #11, the decline of the neutral axis was observed when seismic excitation was 0.5 g, and for the same sensor, the upward shift was observed when seismic excitation was 1.0 g, respectively. For sensor # 14, the neutral axis drop was observed when seismic excitation was 0.4 g, and for the same sensor, the upward neutral axis shift was reported by the sensor when seismic excitation was 0.7 g, respectively.
As explained above, assuming the pretest strain of the sensor as the initial strain, the difference between post-test strain and the initial strain was termed as residual strain of the sensor. The residual strain reflects the redistribution of stress field inside the structure. Figures 21 and 22 illustrate that there was negligible residual strain in the installed sensors before the input ground motion was 0.3 g and when the structure was completely in the elastic state. The residual strains of FBG sensors began to increase remarkably when the input ground acceleration started to increase from 0.5 g. This demonstrates the fact that initial microcracks appeared inside the structure due to structure’s irreversible elasto-plastic deformation. Besides this, the residual strain in all the sensors were found to be significantly large except sensor #16 and sensor #17. It shows that shear wall region was the strongest zone of the structure and because of the high stiffness of the member, minimal relative strain changes were observed. The results also demonstrate that the sudden change in the residual strain at the flexible edge in most of the profiles in Figures 21 and 22 is either due to the initiation of crack or crack travel through the experimental model. Therefore, the damage process in the experimental model could fruitfully be investigated using FBG strain sensors as its monitoring stands well with the observations.
In addition to the movement of the neutral axis of strain, there were special variations observed, which were analysed using Figures 15–18. No significant compressive strain (negative strain) was observed in sensor # 15. In the vicinity of this sensor near the beam-column joint (Figure 20), the frame had a tensile failure with failure being particularly near the ends of beam-column joint. This tensile failure in the concrete was entirely developed by the rebars. Because the tension in the rebar caused a great plastic deformation and concrete beam was under compression, plastic deformation did not recover. In Figure 17, the strain time history profiles for sensor # 15 under the input ground acceleration (PGA) = 0.6 g, (b) cracks and damage at the beam-column joint of second floor on grid 1-B at PGA = 0.6 g, (c) cracks on the lateral side of the flexible edge at PGA = 0.6 g, (d) cracks and spalling of concrete cover at beam-column joint of second floor on grid 1-B at PGA = 0.7 g, (e) cracks and concrete spalling at beam-column joint of top roof on grid 1-B at PGA = 0.7 g, (f) damage at the beam-column joint at PGA = 0.8 g.

In addition to the movement of the neutral axis of strain, there were special variations observed, which were analysed using Figures 15–18. No significant compressive strain (negative strain) was observed in sensor # 15. In the vicinity of this sensor near the beam-column joint (Figure 20), the frame had a tensile failure with failure being particularly near the ends of beam-column joint. This tensile failure in the concrete was entirely developed by the rebars. Because the tension in the rebar caused a great plastic deformation and concrete beam was under compression, plastic deformation did not recover. In Figure 17, the strain time history profiles for sensor # 15 under the input ground acceleration (PGA) = 0.6 g, (b) cracks and damage at the beam-column joint of second floor on grid 1-B at PGA = 0.6 g, (c) cracks on the lateral side of the flexible edge at PGA = 0.6 g, (d) cracks and spalling of concrete cover at beam-column joint of second floor on grid 1-B at PGA = 0.7 g, (e) cracks and concrete spalling at beam-column joint of top roof on grid 1-B at PGA = 0.7 g, (f) damage at the beam-column joint at PGA = 0.8 g.

![Figure 20](image1.png)  
**Figure 20** Progressive damage near beam-column joint (a) crack at the beam-column joint of first floor on grid 1-A at peak ground acceleration (PGA) = 0.6 g, (b) cracks and damage at the beam-column joint of second floor on grid 1-B at PGA = 0.6 g, (c) cracks on the lateral side of the flexible edge at PGA = 0.6 g, (d) cracks and spalling of concrete cover at beam-column joint of second floor on grid 1-B at PGA = 0.7 g, (e) cracks and concrete spalling at beam-column joint of top roof on grid 1-B at PGA = 0.7 g, (f) damage at the beam-column joint at PGA = 0.8 g.

![Figure 21](image2.png)  
**Figure 21** Residual strain under progressive loading: recorded residual strain profiles corresponding to all fibre Bragg grating sensors located at the intersection of grid-A and grid-1.
 accelerations of 0.6 g, 0.7 g, 0.8 g, and 1.0 g, respectively, indicates that the decline of the neutral axis with negative strain does not have actual contribution towards the damage in the structure.

Because this experiment is based on the damaged process in the structure, when the ground motion input reached a certain peak value, the beam and column bars produced some residual strain in response to the input excitation. The FBG strain sensor provided the residual strain on the reinforcing bars by comparing the change of the wavelength before and after the change. The change in the wavelengths before and after the seismic action has been illustrated in Figures 23 and 24. Because this experiment was an indoor experiment, the influence of temperature was eliminated.

9 | INITIAL STRAIN CONSIDERATION

The wavelengths of FBG sensors have been provided to consider the initial strain (shrinkage strain). The provided strain profiles have been monitored without considering initial strain due to curing and pouring. For more reliable results and better comparisons, these wavelengths can be used in correspondence with the temperature to obtain the initial strain of the overall structure before the test.

If $T_p$ is the pouring temperature and $T_t$ is the test temperature, then change in the temperature can be written as follows:

$$\Delta T = T_t - T_p$$  \hspace{1cm} (2)

Also, if the centre wavelength of the FBG during pouring is $\lambda_p$ and centre wavelength of the FBG during the test is $\lambda_t$, then change in the wavelength $\Delta \lambda_B$ can be expressed as follows:

$$\Delta \lambda_B = \lambda_t - \lambda_p$$  \hspace{1cm} (3)

Where

$$\Delta \lambda_B = \alpha_e \varepsilon_{ini} + \alpha_T \Delta T$$  \hspace{1cm} (4)

In Equation (4), $\alpha_e$ is the strain sensitivity coefficient of the FBG sensor; $\varepsilon_{ini}$ is the initial strain, and $\alpha_T$ is the temperature sensitivity coefficient of the FBG sensor. Simplifying Equation (4), the initial strain can be obtained as follows:
Equation (5) can be used to calculate the initial strain. For sensor # 16, considering the pouring temperature as 25°C, test temperature as 5°C, centre wavelength of the FBG sensor before pouring as 1,547.837 nm, centre wavelength of the FBG sensor before the test as 1,547.362 nm, temperature sensitivity coefficient as 0.027 nm/°C and strain sensitivity coefficient as $1.2 \times 10^{-3}$ nm/µε, the initial strain turns out to be 54.5 µε.

By considering the influence of above calculated initial strain, more reliable analytical data can be demonstrated for the analysis of either yielding of steel bars or ultimate tensile strain of steel bars.

10 | DAMAGE CHARACTERISTICS AND FORMATION OF PLASTIC HINGES IN THE STRUCTURE

The prototype structure experienced progressive damage under the influence of progressive seismic excitations. The initial microcracks were determined from the dynamic properties of the structure after the test structure was exposed to a ground acceleration of 0.5 g. However, no visible cracks were noticed at this stage. The presence of microcracks were estimated from the variation in frequency of the structure. When the peak seismic acceleration reached 0.6 g, cracks started to induce in the upper frame at the column's bottom and top and at the beam's ends. This indicated that when the seismic excitation was 0.6 g, the concrete's stress exceeded the tensile strength of concrete. However, at this stage, reinforcement bars did not yield. When the input ground motion was further increased from 0.6 g to 0.7 g, the cracks started to become wider near beam-column joints. This eventually resulted in the development of plasticity in the reinforcement. The corner beam in the transverse direction started forming bigger cracks near the beam-column joints whereas the middle columns in the longitudinal direction started forming bigger cracks both at the top and bottom edges (Figure 20). For the columns at the top storey, the columns started getting cracks at the top edges after the structure was exposed to the input ground acceleration of 0.7 g and 0.8 g. Also, plastic hinges started to develop near the beam-column ends at this level. When the input seismic excitation reached a peak value of 1.0 g, cracks continued to increase, and plastic hinges developed near the column ends.
In this research, it was discovered that the development of plastic hinges in the test structure was a progressive damage action that started from structure's base and then slowly propagated towards the upward direction. Also started from the beams and then slowly and progressively propagated towards columns. The development of plastic hinges around the column and flexural damage at beam-column joints speeded up the damage process in the columns at the adjacent storeys. As a consequence, the adjacent storeys started to develop plastic hinges causing the formation of plastic hinges progressively. Therefore, it can be claimed that the initiation of structural damage started from the weak point of the structure because of lack of structural capacity following adjacent structural component's damage because of inertial forces being in the lateral direction. There were numerous factors that influenced the duration of the development process of the damage in different storeys. These factors include mass distribution, inertia forces, and input ground motion.

At times, it is easy to determine the weak locations in the structure by means of determining the internal microcracks. This analysis can be used as a tool to determine the early signs of cracking and early warning for structure that is vulnerable to damage under seismic loading.

11 | CONCLUSION

The three-storey asymmetric shear wall structure was exposed to seismic excitations with different PGA levels. The strain in the structure was monitored at different locations using FBG strain sensors. The following experimental observations have been made:

- The torsional rotation in the structure under the influence of seismic excitation affected the response of members located near the edge of the structure, which has led to the conclusion that design of the corner members on flexible sides need serious attention.
- The maximum tensile strain in some of the sensors was almost doubled whereas the compressive strain was increased to nearly three times of compressive strain observed in the previous PGA. This phenomenon indicates the presence of a large crack near the sensor region and also indicates that concrete was crushed at this point and eventually confirms the formation of a large crack and crushing of concrete.

**FIGURE 24** Wavelength shift under progressive seismic loading corresponding to all fibre Bragg grating sensors located at the intersection of grid-B-grid-1, grid-B-grid-2, and shear wall
• The residual strains of FBG sensors began to increase remarkably when the seismic excitation was increased from 0.5 g. This demonstrates the fact that initial microcracks appeared inside the structure due to structure’s irreversible elasto-plastic deformation.

• Sudden change in the trend of residual strain was caused by the initial cracks. For most of the residual strain profiles, this sudden change occurred when the excitation was increased from 0.5 g to 0.6 g, which also indicates the initiation plastic state of the structure.

• The shear-wall region was the strongest zone of the structure, and because of the high stiffness of the member, minimal relative strain changes were observed.

• The plastic hinges formation in the beams occurred at PGA of 0.8 g whereas in the columns plastic hinges were formed at PGA of 1.0 g.

• Application of FBG strain sensor in the actual structure for the evaluation of monitoring the primary and secondary components, FBG strain sensors detect the strain close to the cracks on structural components and provides the early warning signs. When the crack is present, the strain closed to the crack can be monitored to calculate the maximum crack width in the member. The warning signs for the structures can established when the maximum crack width is either close to the crack width limit or has exceeded the limit.

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REFERENCES
1. Osteraas J, Krawinkler H. The Mexico earthquake of September 19, 1985—behavior of steel buildings. Earthq Spectra. 1989;5(1):51-88.
2. Ger J-F, Cheng FY, Lu L-W. Collapse behavior of Pino Suarez building during 1985 Mexico City earthquake. Journal of Structural Engineering. 1993;119(3):852-870.
3. Minzheng Z, Yingjie J. Building damage in Dujiangyan during Wenchuan earthquake. Earthquake Engineering and Engineering Vibration. 2008;7(3):263-269.
4. Takewaki I. A comprehensive review of seismic critical excitation methods for robust design. Adv Struct Eng. 2005;8(4):349-363.
5. Li HN, Sun L, Song G. Modal combination method for earthquake-resistant design of tall structures to multidimensional excitations. Struct Design Tall Spec Build. 2004;13(4):245-263.
6. Ni S, Li S, Chang Z, Xie L. An alternative construction of normalized seismic design spectra for near-fault regions. Earthquake Engineering and Engineering Vibration. 2013;12(3):351-362.
7. Wen W-P, Zhai C-H, Li S, Chang Z, Xie L-L. Constant damage inelastic displacement ratios for the near-fault pulse-like ground motions. Eng Struct. 2014;59:599-607.
8. Raghunandan M, Liel AB. Effect of ground motion duration on earthquake-induced structural collapse. Structural Safety. 2013;41:119-133.
9. Ruiz-García J, Marín MV, Terán-Gilmore A. Effect of seismic sequences in reinforced concrete frame buildings located in soft-soil sites. Soil Dynamics and Earthquake Engineering. 2014;63:56-68.
10. Zhang C, Alam Z, Samali B. Evaluating contradictory relationship between floor rotation and torsional irregularity coefficient under varying orientations of ground motion. Earthquakes and Structures. 2016;11(6):1027-1041.
11. Anagnostopoulos S, Kyrkos M, Statopoulos K. Earthquake induced torsion in buildings: critical review and state of the art. Earthquakes and Structures. 2015;8(2):305-377.
12. Duan X, Chandler A. An optimized procedure for seismic design of torsionally unbalanced structures. *Earthquake Engineering & Structural Dynamics*. 1997;26(7):737-757.

13. Georgoussis GK. Modified seismic analysis of multistory asymmetric elastic buildings and suggestions for minimizing the rotational response. *Earthquakes and Structures*. 2014;7(1):39-55.

14. Tezcan SS, Alhan C. Parametric analysis of irregular structures under seismic loading according to the new Turkish earthquake code. *Eng Struct*. 2001;23(6):600-609.

15. Alam Z, Zhang CW, Samali B. Response uncertainty under varying orientations of ground motions. *Mechanics of Structures and Materials: Advancements and Challenges: CRC Press*. 2016;657-662.

16. Elwood KJ, Moehle JP. Dynamic shear and axial-load failure of reinforced concrete columns. *Journal of Structural Engineering*. 2008;134(7):1189-1198.

17. Elwood KJ, Moehle JP. Dynamic collapse analysis for a reinforced concrete frame sustaining shear and axial failures. *Earthquake Engineering & Structural Dynamics*. 2008;37(7):991-1012.

18. Kim Y, Kabeyesawa T, Igarashi S. Dynamic collapse test on eccentric reinforced concrete structures with and without seismic retrofit. *Eng Struct*. 2012;34:95-110.

19. Ghorbanirenani I, Tremblay R, Léger P, Leclerc M. Shake table testing of slender RC shear walls subjected to eastern North America seismic ground motions. *Journal of Structural Engineering*. 2011;138(12):1515-1529.

20. Wu C, Loh C-H, Yang Y. Shake table tests on gravity load collapse of low-ductility RC frames under near-fault earthquake excitation. *Proceedings of the Advances in Experimental Structural Engineering Nagoya, Japan*. 2005;725-732.

21. Wu C, Kuo WW, Yang YS, et al. Collapse of a nonductile concrete frame: shaking table tests. *Earthquake Engineering & Structural Dynamics*. 2009;38(2):205-224.

22. Benavent-Climent A, Morillas L, Escolano-Margarit D. Inelastic torsional seismic response of nominally symmetric reinforced concrete frame structures: shaking table tests. *Eng Struct*. 2014;80:109-117.

23. Li S, Zuo Z, Zhai C, Xu S, Xie L. Shaking table test on the collapse process of a three-story reinforced concrete frame structure. *Eng Struct*. 2016;118:156-166.

24. Ghanoun WM, Moehle JP. Shake-table tests of a concrete frame sustaining column axial failures. *ACI Structural Journal*. 2012;109(3):393.

25. Rao Y-J. Recent progress in applications of in-fibre Bragg grating sensors. *Opt Lasers Eng*. 1999;31(4):297-324.

26. Majumder M, Gangopadhyay TK, Chakraborty AK, Dasgupta K, Bhattacharya DK. Fibre Bragg gratings in structural health monitoring—present status and applications. *Sensors and Actuators a: Physical*. 2008;147(1):150-164.

27. Ye X, Su Y, Han J. Structural health monitoring of civil infrastructure using optical fiber sensing technology: a comprehensive review. *Scientific World Journal*. 2014;2014:1-11.

28. Brönnimann R, Nellen PM, Sennhauser U. Application and reliability of a fiber optical surveillance system for a stay cable bridge. *Smart Materials and Structures*. 1998;7(2):229-236.

29. Kerrouche A, Leighton J, Boyle W, et al. Strain measurement on a rail bridge loaded to failure using a fiber Bragg grating-based distributed sensor system. *IEEE Sensors Journal*. 2008;8(12):2059-2065.

30. Kerrouche A, Boyle W, Gebremichael Y, et al. Field tests of fibre Bragg grating sensors incorporated into CFRP for railway bridge strengthening condition monitoring. *Sensors and Actuators a: Physical*. 2008;148(1):68-74.

31. Kister G, Winter D, Gebremichael Y, et al. Methodology and integrity monitoring of foundation concrete piles using Bragg grating optical fibre sensors. *Eng Struct*. 2007;29(9):2048-2055.

32. Rodrigues C, Cavadas F, Félix C, Figueiras J. FBG based strain monitoring in the rehabilitation of a centenary metallic bridge. *Eng Struct*. 2012;44:281-290.

33. Barbosa C, Costa N, Ferreira L, et al. Weldable fibre Bragg grating sensors for steel bridge monitoring. *Measurement Science and Technology*. 2008;19(12):125305.

34. Kerrouche A, Boyle W, Sun T, Grattan K. Design and in-the-field performance evaluation of compact FBG sensor system for structural health monitoring applications. *Sensors and Actuators a: Physical*. 2009;151(2):107-112.

35. Chan TH, Yu L, Tam H-Y, et al. Fiber Bragg grating sensors for structural health monitoring of Tsing ma bridge: background and experimental observation. *Eng Struct*. 2006;28(5):648-659.

36. Chung W, Kim S, Kim N, et al. Deflection estimation of a full scale prestressed concrete girder using long-gauge fiber optic sensors. *Construct Build Mater*. 2008;22(3):394-401.

37. Lin YB, Pan CL, Kuo YH, Chang KC, Chern JC. Online monitoring of highway bridge construction using fiber Bragg grating sensors. *Smart Materials and Structures*. 2005;14(5):1075-1082.
39. Zhou Z, Huang M, Huang L, Ou J, Chen G. An optical fiber bragg grating sensing system for scour monitoring. Adv Struct Eng. 2011;14(1):67-78.

40. Lin Y-B, Chen J-C, Chang K-C, Chern J-C, Lai J-S. Real-time monitoring of local scour by using fiber Bragg grating sensors. Smart Materials and Structures. 2005;14(4):664-670.

41. Lin YB, Lai JS, Chang KC, Li LS. Flood scour monitoring system using fiber Bragg grating sensors. Smart Materials and Structures. 2006;15(6):1950-1959.

42. Kim S-W, Kang W-R, Jeong M-S, Lee I, Kwon I-B. Deflection estimation of a wind turbine blade using FBG sensors embedded in the blade bonding line. Smart Materials and Structures. 2013;22(12):125004.

43. Arsenault TJ, Achuthan A, Marzocca P, Grappasonni C, Coppotelli G. Development of a FBG based distributed strain sensor system for wind turbine structural health monitoring. Smart Materials and Structures. 2013;22(7):075027.

44. Bang H-J, Kim H-I, Lee K-S. Measurement of strain and bending deflection of a wind turbine tower using arrayed FBG sensors. International Journal of Precision Engineering and Manufacturing. 2012;13(12):2121-2126.

45. Ye X, Ni Y, Yin J. Safety monitoring of railway tunnel construction using FBG sensing technology. Adv Struct Eng. 2013;16(8):1401-1409.

46. Li C, Zhao Y-G, Liu H, Wan Z, Zhang C, Rong N. Monitoring second lining of tunnel with mounted fiber Bragg grating strain sensors. Automation in Construction. 2008;17(5):641-644.

47. Li C, Zhao Y-G, Liu H, et al. Strain and back cavity of tunnel engineering surveyed by FBG strain sensors and geological radar. Journal of Intelligent Material Systems and Structures. 2009;20(18):2285-2289.

48. Biswas P, Bandypadhyay S, Kesavan K, et al. Investigation on packages of fiber Bragg grating for use as embeddedable strain sensor in concrete structure. Sensors and Actuators a: Physical. 2010;157(1):77-83.

49. Chan PK, Jin W, Lau AK, Zhou L. Strain monitoring of composite-boned concrete specimen measurements by use of FMCW multiplexed fiber Bragg grating sensor array. Paper presented at: International Conference on Sensors and Control Techniques (ICSC2000). 2000.

50. Naruse H, Uchiyama Y, Kurashima T, Unno S. River levee change detection using distributed fiber optic strain sensor. IEICE Transactions on Electronics. 2000;83(3):462-467.

51. Liu J, Liu F, Kong X, Yu L. Large-scale shaking table model tests on seismically induced failure of concrete-faced rockfill dams. Soil Dynamics and Earthquake Engineering. 2016;82:11-23.

52. Schulz WL, Conte JP, Udd E. Long Gage Fiber Optic Bragg Grating Strain Sensors to Monitor Civil Structures. Blue Road Research Fairview Or. 2001.

53. Mita A, Yokoi I. Fiber Bragg grating accelerometer for buildings and civil infrastructures. Paper presented at: Proc. SPIE2001.

54. Ren L, Li H-N, Zhou J, Sun L, Li D-S. Application of tube-packaged FBG strain sensor in vibration experiment of submarine pipeline model. China Ocean Engineering. 2006;20(1):155-164.

55. Ren L, Li H-N, Zhou J, Li D-S, Sun L. Health monitoring system for offshore platform with fiber Bragg grating sensors. Optical Engineering. 2006;45(8):084401-084409.

56. Sun L, Hao H, Zhang B, Ren X, Li J. Strain transfer analysis of embedded Fiber Bragg grating strain sensor. Journal of Testing and Evaluation. 2015;44(6):2312-2320.

57. Sun L, Li C, Li J, Zhang C, Ding X. Strain transfer analysis of a clamped fiber Bragg grating sensor. Applied Sciences. 2017;7(2):188.

58. Sun L, Li H, Jin Q. FBG sensors for the measurement of the dynamic response of offshore oil platform model. Paper presented at: Health Monitoring and Smart Nondestructive Evaluation of Structural and Biological Systems IV 2005.

59. Sun L, Li H-N, Ren L, Jin Q. Dynamic response measurement of offshore platform model by FBG sensors. Sensors and Actuators a: Physical. 2007;136(2):572-579.

60. Ni Y, Xia Y, Liao W, Ko J. Technology innovation in developing the structural health monitoring system for Guangzhou new TV tower. Struct Control Health Monit. 2009;16(1):73-98.

61. Li DS, Ren L, Li H-N, Song G. Structural health monitoring of a tall building during construction with fiber Bragg grating sensors. International Journal of Distributed Sensor Networks. 2012;8(10):272190.

62. GB50011–2001. Code for Seismic Design of Buildings (GB50011–2001). Beijing: China Architecture and Building Press; 2001.

63. Kersey AD, Davis MA, Patrick HJ, et al. Fiber grating sensors. Journal of Lightwave Technology. 1997;15(8):1442-1463.

64. Li H-N, Li D-S, Song G-B. Recent applications of fiber optic sensors to health monitoring in civil engineering. Eng Struct. 2004;26(11):1647-1657.