Monitoring and Repair of an Impact Damaged Prestressed Bridge

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Abstract: This paper details the monitoring and repair of an impact damaged prestressed concrete bridge. The repair was required following an impact from a low-loader carrying an excavator while passing underneath the bridge. The repair was carried out by preloading the bridge in the vicinity of the damage to relieve some prestressing. This preload was removed following the hardening and considerable strength gain of the repair material. The true behaviour of damaged prestressed concrete bridges during repair is difficult to estimate theoretically due to lack of benchmarking and inadequacy of assumed damage models. A network of strain gauges at locations of interest was thus installed during the entire period of repair. Effects of various activities were qualitatively and quantitatively observed. The interaction and rapid, model-free calibration of damaged and undamaged beams, including identification of damaged gauges were also probed. This full scale experiment is expected to be of interest and benefit to the practising engineer and the researcher alike.

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1. Introduction

A full-scale experiment on an impact damaged prestressed concrete bridge was carried out during the rehabilitation works incorporating a network of strain gauges located in and around the damaged region. The works were a part of an emergency rehabilitation following the impact of a low-loader carrying an excavator to the soffit of the bridge. Estimates of the levels of stresses present in the structure in an undamaged state were made possible through the availability of production drawings. However, uncertainties existed in estimating the stresses in a damaged condition as it is very difficult to model the redistribution of stresses following an unknown impact. The existence of a credible benchmark was absent, unfeasible or error-prone and local damage often manifests very little global change in measurable parameters. The local fracture of concrete in a beam leading to redistribution of stresses at and around the affected regions, including neighbouring beams is extremely difficult, if not impossible to deal with. The monitoring of repair is thus often dependent on the detection of sudden, unusual or unacceptable levels of change in stress at critical locations of the bridge from an unknown baseline of stress. There is a dearth in existing literature reporting full-scale tests of bridges during repair to understand their macro-level behaviour.

Challenges involving uncertainties in stress distribution, very significant constraint on time, detailed traffic management, and health and safety aspects were encountered during the rehabilitation reported in this paper. Strain data obtained from a network of strain gauges from damaged and undamaged neighbouring beams throughout the rehabilitation period was tracked during the rehabilitation work to identify sudden or unexpected changes in the response of the monitored points on the bridge. Additionally, this data provided an opportunity to observe how the full scale structure behaved during the various stages of rehabilitation. Permanent bridge monitoring systems in large scale projects have been observed before in Japan and Hong Kong (Brownjohn et al., 2005). The main challenge acknowledged lies in intelligently interpreting the data into useful information. However, it has been asserted that damage or significant change in activity can be detected by identifying anomalies or sudden changes in the recorded data (Moyo & Brownjohn, 2002). The changes in strain data in such cases are often more relevant during rehabilitation than the absolute values. Bridges are expensive infrastructure assets and consequently their monitoring has significant financial implications. Condition monitoring can be used to achieve the intended service life. Even though monitoring systems are often more common for long-span bridges it has been proven to be extremely beneficial for short-span bridges (Alampalli & Fu, 1994) as well.

Visual inspections of bridges often help identify deterioration. However, potentially serious damage can be present without an apparent sign of warning. Sometimes, the type of damage can be qualitatively assessed but a requirement of quantifying and tracking the damage is important. The performance of the structure, very often from a serviceability point of view, is required to be monitored or ascertained. Monitoring systems are beneficial in all of these cases. Detection of anomalies in the measured strain data over a relatively short period of time, whether sudden or gradual, is a special aspect in this regard since such changes are impossible to be visually estimated. As indicated by Omenzetter et al. (2004) the main challenge is in interpreting the recorded data. With continuous developments in detection techniques the use of monitoring systems is becoming more efficient and reliable.

Although a significant amount of literature exists in the field of Structural Health Monitoring (SHM) (Taha et.al (2006); Peter Carden and Fanning (2004); Farrar et.al (2001)), not many studies address full scale experiments of concrete bridges. From the minority of full-scale experiments of concrete bridges, a significant number deal with detection, monitoring or measurement techniques (Rowley et.al (2009); Lee and Shinozuka (2006); Skelton and Richardson (2006)) including some chronicling the monitoring of long time scales (Onyemelukwe et.al (2003)). Seldom, monitoring associated with rehabilitation is addressed. The lack of literature can be partially explained by the rarity of opportunity to design an experiment and the time constraints associated with emergency rehabilitations.

It is important to observe the interaction between damaged and undamaged neighbouring beams during the period of repair. It is also important to experimentally demonstrate how the sensors located at different points at and around the damaged location behave with changing activities. The identification of possible sensor failure is also a practical and important problem to look at. This paper presents a full-scale experiment on an impact damaged prestressed concrete bridge during the repair works addressing these issues. The possibilities of developing rapid and useful qualitative understanding and quantitative calibrations are discussed. This is the first full-scale experimental study on the evolution of strains in an impact damaged prestressed concrete bridge.

2. Details of Damage

A two-span continuous slab-girder bridge comprising of six precast prestressed U8 simply supported concrete beams connected by a continuity diaphragm and spanning a

National Primary road of the Republic of Ireland was damaged following an impact to its soffit from a low-loader carrying an excavator passing underneath the bridge. The beams are 27.35m in span and the bridge is not skew. The edge of the outer beam was damaged in a benign fashion although one of the tendons in the lower row snapped. This was not a major issue since a rapid assessment calculation proved that the beam was well within the safe zones under stability and serviceability conditions with the exclusion of the tendon. An internal beam was more significantly damaged where the tendons remained intact but the concrete crushed from the impact. A visual survey provided information regarding the extent of damage. This was followed up with a three -dimensional laser scan visualisation that was made available for the bridge. A hammer tapping survey at and near the location of the main damage indicated that the true damage extended beyond the visually superficial regions. This fact was reinforced by carrying out an impact echo survey. Figure 1 presents the details of the damage. The tendons were unaffected by this incident. Structural cracking in the prestressed concrete beams was absent following the damage in an unloaded state or due to the passage of vehicles, qualitatively supporting the fact that the concrete was probably within linear and compressive zone. Although it was difficult to estimate the existing stresses within the beam, calculations on extreme hypothetical situations revealed that the beams had significant windows of operation on the compressive and the tensile side from its unloaded state while remaining within the linear elastic zone. Information on Young's modulus of concrete for the beams were made available through design drawings and manufacture drawings.

3. Details on Instrumentation

Instrumentation was carried out on the bridge in the form of installation of nineteen strain gauges at five preselected Monitoring Points (MP). Figure 2 provides the schematic details of the arrangement of the multichannel strain gauges and MP locations. The monitoring points were strategically chosen so that the interaction of the damaged and the undamaged beams, including the behaviour of gauges at, near and away from the damage can be probed. Three monitoring points, at the centre and at the two ends of the damage were chosen. The centre of the two undamaged beams and the two sides of the damaged beams were chosen as the two other monitoring points. Gauges were installed at the top and at the bottom of the soffit so that the deformations at these two levels could be observed simultaneously. Three gauges at MP2, the centre of damaged location were embedded to the tendons and were zeroed at a later period than the remaining gauges. The sampling rate was kept at one minute. The data was logged in microstrain units. The rest of the gauges were zeroed at the same time and were monitored before repair works when the chief activity was thermal. The gauges remained for four and a half days after repair for the purpose of monitoring. The embedded strain gauges were Geokon Vibrating Wire Embedment Gauge Model 4200 while the rest of the gauges were Geokon Vibrating Wire Strain Gauge Model 4000. These gauges were chosen based on their high durability characteristics, range of operation, resolution and the range of operable temperature. The strain gauge mounting brackets were fixed to the surface of the prestressed beams whilst conservatively taking care not to encounter the bottom row of tendon, the depth of which was made available from production drawings of the bridge. Tolerance level and stability of readings of the gauges were ensured. The temperature was measured under the bridge in the shade and thus refers only to the average temperature of the location. A Campbell

Scientific CR10X Data Logger was mounted on the North abutment of the bridge through anchors and bolts. The data was remotely monitored and downloaded automatically every six hours of monitoring to free up the available memory. The cables from the gauges were bunched together and made secure using single piece cable cleats.

4. Method of Repair

A preload was applied to either side of the damaged region thereby releasing some of the high prestressing compressive force in the soffit of the beam. The damaged concrete around the area of impact was removed after the application of preload. The preload was removed following the application and sufficient hardening of the repair material. Consequently, the prestrain due to the preloading was released and the hardened repair material was expected to experience some compression. The attempt was to restore some of the lost prestress in the concrete.

The repair can be divided into six significant zones of activity. These are:

- The instrumentation of the gauges,
- The application of preload
- Concrete removal employing hydrodemolition
- Application of repair material, shrinkage and hardening
- Removal of load and further strength gain.

Details on instrumentation have already been discussed in the previous section. Preloading consisted of placing 20t bales of concrete blocks either side of the damaged region. These were staged in three applications to a total of 120t. The damaged concrete was then removed from the beam by hydrodemolition. This method of concrete removal was chosen for this project due to its precision and low impact on the existing strands.

The repair material chosen was a fibre reinforced spray mortar. It was designed to have a 28 day compressive strength of 70MPa and was able to take greater tensile force than standard concrete. The load was removed from the top of the bridge after the repair material had gained adequate strength and some amount of the prestress was reinstated following the removal. The strain gauges remained for a further four days to allow any further strength gain to be examined.

Figure 3 shows recordings of representative gauges with various identified time zones while Figure 4 shows the damaged section of the beam before hydrodemolition, after hydrodemolition and following the hardening of the repair material. The time zones correspond to the various periods discussed in the sub-sections of the following section. As a representative typical value, 100 microstrain represents about 3-3.4MPa based on the Young's modulus of elasticity of concrete and it is expected to be on the higher side in this case. Some strain gauges have been omitted from the plots due to irrecoverable damages and malfunctioning.

5. Monitoring Results

5.1 Thermal Period

The thermal period considered is the relatively quiescent time between the instrumentation of the gauges and the application of load. This stage is prior to any rehabilitation works where fluctuations due to the diurnal temperature cycle can be observed. These fluctuations are observed in the strain recordings prior to the repair of the bridge (Figure 5). The strain response to these significant cycles of temperature appears to be well correlated. The explanation of the exception in SG10 could not be directly explained but is suspected to be related partial damage or dysfunctionality of the gauge during installation.

5.2 Preloading

The preloading consisted of placing 20t bales of concrete blocks either side of the damaged region totalling 120t. These were staged in three main applications. Preloading causes tension in the bottom of the beams releasing prestressing force. With significant loss of section, the compressive stresses at the bottom due to prestress alone increase significantly if no redistribution of stress after impact is considered. Such a consideration may lead to a conclusion that the bottom concrete due to prestress alone, and even with dead and superimposed dead loads, may be close to crushing limit. The compressive stresses, under no redistribution of forces increase further after hydrodemolition. In reality, redistribution of stresses does take place but is very hard to model. If a complete redistribution of stresses following the impact is assumed, then there is a finite chance of the preloading creating tension at the soffit. To avoid such situations, calculations were carried out prior to the rehabilitation employing damaged and undamaged finite elements models considering the two extremes and the preload was conservatively estimated. The real situation is somewhere in between these to extreme conditions. The application of preloading essentially reduces an increase of compressive stress at the soffit. This allowed for the hydrodemolition to be carried out in a safer manner. The preload also introduces a prestrain at and around the damaged zone.

At the centre of damage, the bottom embedded gauges are expected to undergo tension and the top gauges compression. Figure 6 shows the change in strain over the loading period for this location and at a location on a neighbouring undamaged beam longitudinally at the same point as the centre of damage. SG 10 is identified as damaged although it does react albeit insensitively to events during the bridge rehabilitation. These results upon first glance might potentially be explained with the negative dip of the embedded gauges SG 11 and SG 12 (incidentally SG10 also shows a dip) representing tension in the soffit of the beam. This allows for the assumption that the strain gauges at the top represent compression shown by the positive bump. This would be a reasonable explanation considering this monitoring point alone. However, problems with this explanation arise when the other monitoring points are examined. With SG1 and SG4 representing the top section of the point on the undamaged beam, it can be seen that positive bumps are experienced which correspond to those seen in the top gauges of the centre of the damaged section. This would lead to the conclusion that compression is also being experienced in these locations at the same time. This should most definitely not be the case. It is likely that these bumps in the data represent the redistribution of stresses due to application of load affecting the already fractured concrete. This is supported by the evidence that these changes of strain occur strongly within the damaged zone of the beam. On drawing this conclusion it can be acknowledged that the loading stages themselves have not been recognised very well within the data as there are no jumps clearly evident bar one at approximately 17:15.

5.3 Hydrodemolition

Due to the nature of the activity it is expected that this period would be laden with disturbance. Looking at the centre of damage, (Figure 7) it is noted that the embedded gauges (SG 11, SG 12 and SG 13) show little reaction. This is to be expected as these gauges are attached to the tendons rather than the concrete with is suffering the bulk of the disturbance from the hydrodemolition. The lack of reaction also supports the efficiency of the hydrodemolition process as it takes the concrete away while affecting the tendons minimally. Regarding the top and soffit of the beams it would be anticipated that the gauges along the soffit would experience more disturbance as they are located closer to the region of removal. The soffit gauges do indeed show greater disturbance than those at the top of the beams. The sharp jumps or noise in readings can be explained by the nature of the disturbance and for most gauges the disturbance is momentary. The gauge SG 18 was damaged in this period and went off the typical scale of the strain gauges.

5.4 Shrinkage

At the centre of damage it is expected that the embedded gauges will be significantly affected by the force due to the shrinkage of the repair material. The top gauges should show little change in strain as there is no action occurring at this location other than the secondary action from the shrinkage repair material. Figure 8 displays the strain gauge readings in this time zone following the application of the repair material. It is apparent that the embedded gauges show an approximately linear increase in strain following compaction of the repair material while the top gauges are subject to minor forces. This is even more so for the gauges that are not located centrally in the repair area as SG 5 and SG 10 are. However, SG 15 recorded an unexpected increase in tension. The embedded gauges (SG11, SG12 and SG13) expectedly show the highest changes due to their position within the repair material.

5.5 Unloading

Unloading was in the form of three stages in order to remove the full load of 120t. These stages should be reflected as stepped changes in strain. Shown in Figure 9 are the readings of the gauges during unloading period. For the top gauges this change is relatively smaller than the bottom gauges. The unloading stages are clearly identified in the bottom gauges. These points in time are shown by arrows. The approximate level of change of strain in the soffit at the location of the centre of damage for each set of removal has been observed to be about 20 microstrains.

5.6 Further Strength Gain

The period of further strength gain of the repair material following the unloading stage covers more than four days of strain gauge readings (Figure 10). Very little evidence is present of shrinkage effects bar the first few hours. From this time on diurnal temperature effects dominate the strain changes as no other action is occurring on the bridge. It can be seen from the top and bottom strain gauges that the response to thermal cycles are generally well correlated with few exceptions. Two of these poorly responsive gauges (SG 5 and SG 10) are located as external gauges on the top of the damaged region. The embedded gauges (SG 11, SG 12 and SG 13) are within the hardened repair material and are therefore shielded from the temperature effects. Consequently, the diurnal variations are not observed.

6. Participation of Gauges

As has been discussed in previous sections, it is very difficult to estimate the absolute states of stress at various locations. Additionally, it is also noted that the amount of energy in the form of input to the structure is nearly impossible to be predicted during the works. Consequently, we cannot expect the conservation of energy to be respected within the measured network of multichannel strain gauges purely from the strain data that is logged. However, to understand and obtain an estimate of the participation of beams due to various activities, it can be assumed that energy is proportional to the square of the strain within linear, elastic zone. In the absence of bias, a measure of total energy of the discrete point system network consisting of a number of strain gauges can be reasonably defined as the sum of the squares of the measured strains. Also, the sampling points are at and around the damage and located on both damaged and undamaged beams. Consequently, the specific, average participation of each beam can be represented appropriately as a ratio of the sum of squares of the strain data from number of gauges under question in a beam to the sum of squares of strains of the entire system. As discussed, we cannot compare the absolute energy values at two different points of time this way, but the normalised participation can be compared over a timeline with the reasonable underlying assumption of no-bias in terms of dissipation of energy (or the attraction) to other forms. Since the interpretations are from a macro scale and the experiment is full-scale, small variations can be safely neglected. Figure 11 presents the participation of beams along the timeline. It is important to observe how the participation is significantly fluctuating during the quiescent temperature driven periods, and how the significant switch of participations take place during the significantly loaded high strain zones. The tendency of the participation to meet each other after the removal of the load is also highlighted.

7. Correlation of Beam Response

The interaction of the beams, as described in the previous two sections become even more useful if the relationship between the damaged and the undamaged beams can be shown to be approximately linear. This is not an unreasonable guess since the structure is expected to be within the linear zone of response. If an approximately linear relationship between the responses under a number of varied and fundamentally activities can be established, the bridge-specific relation can be calibrated with a high degree of confidence. The calibration provides a way of rapidly correlating responses measured at a later stage, assuming fundamental changes in response have not taken place and the structure is in the linear zone. Additionally, these calibrations can cross correlate the responses of the damaged and undamaged locations. This means, the calibration is essentially a relatively robust and appropriate map of what is happening on the damaged beam but which is measured on an undamaged beam. The degree of correlation will provide the bounds on the confidence of such relationships highlighting the potentials and the limitations. Figure 12 presents the correlations between the beams. Two of the correlations refer to a damaged and an undamaged beam while the remaining one correlates two undamaged beams. It is observed that a very good linear correlation is observed. The fact that this is repeated for all of the combinations does increase the degree of confidence on this conclusion qualitatively. The high correlation coefficients are also noted beside each scatter plot. Based on the discussions in this section, best fit linear relationships, along with the equations, are shown on the figure as well. These relationships form a simple, yet extremely effective and powerful bridge –specific calibration.

8. Malfunctioning Gauge

The malfunctioning gauges can be identified through their responses to the various activities in the time zones. In this paper, two simple and rapid ways of identification of damaged gauges are presented. The visual data of a gauge alone is sometimes difficult to spot as a malfunctioning one and multichannel input comparison is recommended. The underlying assumption of the detection relies on the fact that most gauges are functioning. Such a situation, barring the pathological cases, is most common. The first method considers the linear relationship between temperature and strain. Even in the presence of noise, the correlation between temperature and strain should be close to one. A significant deviation of a gauge from this high correlation coefficient as compared to the background already established by the other gauges would be spotted easily as an outlier. Figure 13 presents this thermal correlation of the various gauges. The malfunctioning gauges are easily identified. Another way of identification of malfunctioning gauges without comparing them with an external environmental factor like temperature is to compare the responses among themselves. It has already been demonstrated in the previous section that there exists a high linear correlation between the relationships of the various gauges over a timeline characterised by disparate activities. Under these circumstances the surface plot of the correlation matrix of the gauges are computed. The diagonal of this matrix is obviously equal to one and the matrix is symmetric about this diagonal. A significant deviation of the correlation between a malfunctioning and a good gauge would identify the malfunctioning gauge through a line of outliers. The comparison works the best if a number of good gauges can be identified through thermal correlation or from the correlation matrix as described and the remaining gauges are checked one by one for the possible formation of a line of outlier. Once the good gauges are identified through the correlation matrix for a test time period, data obtained from later periods can be used on all of the gauges. Figure 14 shows the identification of one such malfunctioning gauge from the surface plot of correlation matrix as compared with a number of good gauges. The simplicity and the credibility of the proposed method are illustrated this way. It is important to emphasize the importance of monitoring two quiescent periods guided chiefly by thermal activity at the start and also at the end. Such monitoring and the associated correlations as described in this section can pick out significant variations and malfunctions in gauges after heavily disruptive works like hydrodemolition are carried out.

9. Conclusions

Structural monitoring of an impact damaged prestressed concrete bridge was carried out during the emergency repair works using a network of strain gauges located at and around the damaged region. The strain data was collected during the repair works included a varied nature of activities. The effects of the activities were quantified through the change in strain. The data also allowed making some qualitative observations regarding the activities. The response of the structure to thermal loading was established. The effects of preloading were observed to be masked. The disturbance due to hydrodemolition was monitored and the effect of this activity on tendons was observed to be small. The linearity in change of strain due to the shrinkage of repair material was established. The unloading events were identified and the prestrain due to preloading was quantified. Thermal loads tended to mask the effects of further strength gain following the removal of concrete. The linear behaviour of the system, the linear relationship with temperature and the linear relationship between the responses of the damaged and the undamaged beams are established. A bridge-specific robust and simple response calibration between the damaged and the undamaged beams is formed. The importance of such calibrations is explained. Correlation based rapid methods of detection of malfunctioning gauges are presented.

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