USING STRUCTURAL HEALTH MONITORING DATA TO ESTIMATE MATERIAL PROPERTIES OF MASONRY STRUCTURES

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ABSTRACT

Brick masonry arches have been used for centuries in countless historic masonry structures and bridges. These structures have been in-service for many years and most have passed their designed service life but are showing major signs of deterioration. In order to create effective and practical methods of rehabilitation, many research programs are being conducted to better understand these structures. One of the most important aspects of the research is to accurately determine the material properties of the structures under study. Current methods of constructing test prisms of material with properties similar to the original materials in the structure do not offer accurate values due to the fact that the prisms are constructed using new materials. While the new materials have properties comparable to the original material, they are still dissimilar because they have not deteriorated to the level of the original material.

A recent study conducted jointly by researchers at the University of Manitoba (U of M) and the University of Illinois at Chicago (UIC), investigated methods for more accurately computing material properties by using data gained from real-time Structural Health Monitoring (SHM). Researchers at the UIC developed and implemented a SHM system on the Brooklyn Bridge in New York and fabricated several scale models of the arches to develop a strategy to determine the safety criteria for the structure. Researchers from U of M used one of the scale models to develop a computational method of using strain gauge data from the SHM system to estimate the modulus of elasticity of the material.

KEYWORDS: effective modulus of elasticity, shear modulus, brick arch, arching action structural health monitoring

INTRODUCTION

Many public and privately owned historic structures are being rehabilitated and/or retrofitted as part of a repair program or a specific upgrading program. The damages being repaired are caused by either structural issues or, in some cases, occupancy changes. A majority of these structures are constructed using reinforced or unreinforced masonry which are almost always load bearing. These load bearing structures are the focus of this study.

In order to formulate the best course of action for rehabilitation design and to determine the most suitable material to use with the existing materials in the structure, it is essential to know the property of the materials; not when it was constructed but at the time of rehabilitation. However, it is extremely difficult to determine the in situ material properties for many reasons. Some of these reasons are listed below:

- It is inherently impossible to take samples from the existing structures for testing,
- The material properties determined from testing newer materials will be considerably different from the existing materials used in the existing structures,
- The most common method of determining material properties is to use similar materials that may even be from the same era. However, the results may still vary considerably from the in situ properties because the materials used for testing have, almost certainly, not experienced the same type and level of deterioration.

It is for these fundamental reasons that the material properties determined and used for formulating rehabilitation strategies are different from the material properties in the existing structure. As a result, many of the rehabilitation strategies developed by designers may not be the suitable option.

Meanwhile, Structural Health Monitoring (SHM) has been advancing considerably, and many historic structures have been instrumented prior to rehabilitation to determine and monitor some of the aspects of the structures such as displacement and strain. To resolve the discrepancy between the material properties of testing similar materials verses the material properties of in-situ material, a question was formulated: “Would it be possible to determine the material properties by collecting real-time data from the structures that are instrumented for SHM?”

The opportunity to answer this question arose when researchers of the UIC instrumented the masonry arches of the Brooklyn Bridge for SHM [1]. The researchers at UIC developed a new methodology for analyzing masonry arches using a combination of rigid block analysis and finite element modeling. They recognized that there is a need for simplified methods that could be used for rapid analysis of masonry arches using generic finite element programs. Such methods provide the opportunity for interpretation of real-time data from structural health monitoring systems. As part of their study several scaled models of the masonry arches were constructed and tested. One of these scaled models was tested but not to complete failure and it could be instrumented and further studied. The researchers at the UIC offered the researchers at the U of M the opportunity to conduct additional testing on the specimen. Figure 1 illustrates the testing specimen.

Figure 1: Brick Arch Specimen – Scale Model of One of the Arches in Brooklyn Bridge

METHODOLOGY

The methodology for this experiment was very simple as this is a preliminary study to examine the hypothesis; could real-time data collected by structural health monitoring be used to determine the material properties of the structure.
In this experiment, lateral displacement was applied at one support of the arch while the other support was fixed. Various displacements were applied as a cyclical load and the stress at various locations was recorded. The recorded data was then compared to numerical analysis.

It should be noted that arches are designed for, and for most parts during their service life are under, uniform loading. However, the original study [1] proves that the cracking pattern in the existing structure was due to lateral forces which are essentially introducing flexural stress into the arch. It is for this reason the lateral load was applied on to the arch for this experiment. The only axial loads present in this experiment were the self-weight of the arch and the nominal load (1 kN) applied on the arch in Phase 2 of the experiment. Therefore, it should be further noted this loading condition is not a realistic load condition for arches since they are designed to sustain major axial compression forces due to uniform vertical loading.

TEST SETUP

The arch was fabricated on a steel railing system (Figure 1); the arch could move laterally at one support by means of an actuator and was fixed at the other support.
The laboratory experiment consisted of two phases. In Phase 1 the actuator applied lateral load on one support of the arch, based on a prescribed lateral displacement, while no vertical loads were applied on the arch. In Phase 2 the same lateral displacement was applied with nominal vertical load on the arch and the results of the two phases were compared.
Figure 2 shows the arch, the force due to the actuator and the location of the strain gauges. The strain gauges were marked as SG1, SG2 and SG3. One LVDT was installed at the centre of the arch to monitor the displacement at the centre-span. The displacement at the moveable end of the support was monitored and recorded by the actuator.

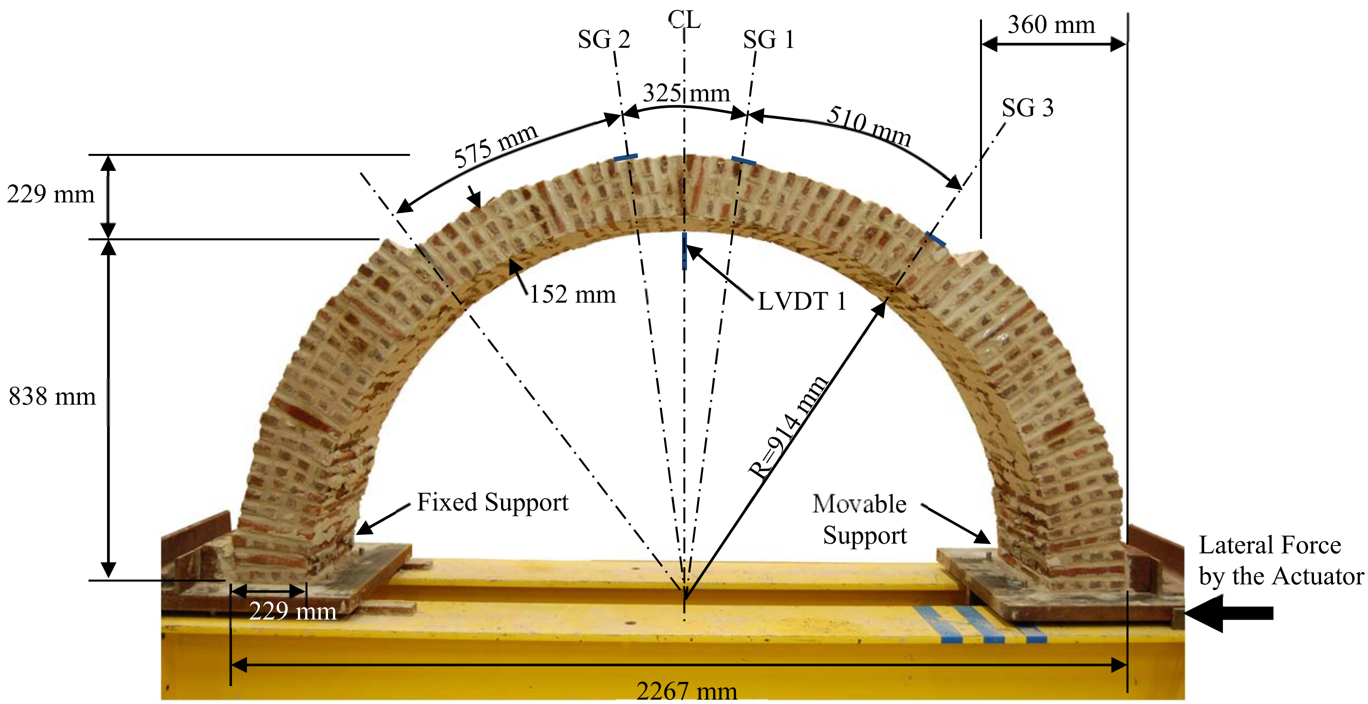


Figure 2: Brick Arch Specimen – Phase 1 Test Setup

In Phase 2, the tes t set up was the sam e except for the additional vertical load applied close the centre of the arch. Fig ure 3 shows the location of the loads. The strain gauges an d the LVDT locations are not shown in Figure 3 for clarity.

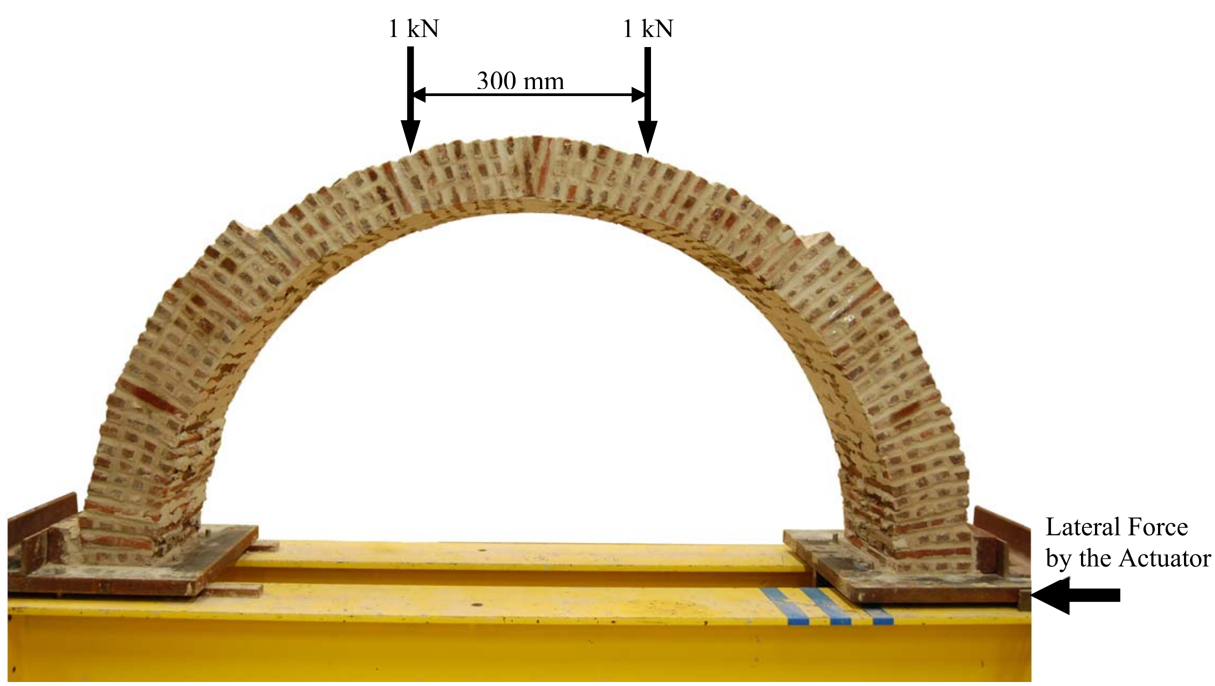


Figure 3: Brick Arch Specimen – Phase 2 Test Setup

Figure 4 shows the glass fibre strain gauges used for these experiments. The strain gauges used here were fabricated by Micron Optics, Inc. and are the same type of gauges used in the field studies.

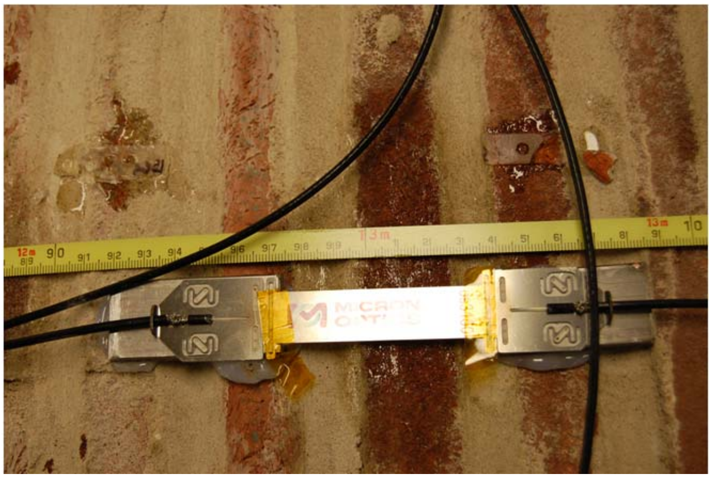


Figure 4: Glass Fibre Strain Gauges Fabricated by Micron Optics, Inc.

LOADING CONDITION

In this experiment the lateral displacement was applied at one support while the other end support remained fixed. As such the lateral displacement applied by the actuator was controlled rather than the load. Table 1 shows the displacement conditions that were applied to the arch during the experiment. The displacement conditions were exactly the same for cases 1, 2 and 3 in both phases; therefore, Table 1 illustrates the displacement condition for both phases.
In case number 4 of Phase 2, the arch was tested to failure.

Table 1: Applied Displacement (Cycle Amplitude) Condition

Case No.	Cycle Amplitue [mm]	No. of cycles
1	±2	5
2	±5	5
3	±10	5
4 (no load)	±25	5
4 (loaded)	±25	To failure

The number of cycles in the above table denotes the repetition of the displacement applied or the number of movements of the actuator. The actuator would start at 0, move in towards the centre of the arch, the negative direction for 10mm and move back past the 0 point and move outward in the positive direction for 10mm, hence, ±10mm.
The number of cycles for load case 4 was increased in phase 2 when the arch was vertically loaded. It was determined that since this was the last loading condition it would be beneficial to determine the number of cycles required to induce complete failure of the arch.

EXPERIMENTAL RESULTS

Only a summary of two experiments is shown herein due to the extensive amount of information collected. The result of two cases of the experiment has been illustrated in the following figures. Each figure shows a graph for each load case. One graph demonstrates the strain from SG1, SG2 and SG3 and the other graph shows the displacem ent at the centre-sp an as reco rded by the LVDT1.

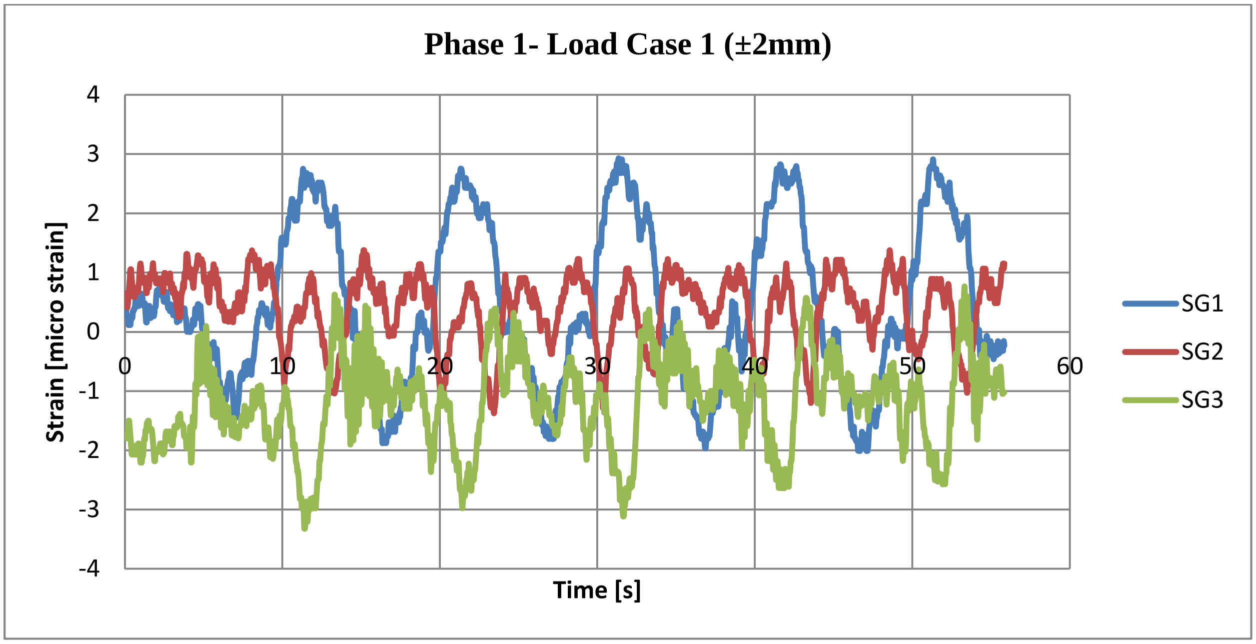


Figure 5: Strain Diagram for Phase 1 – Load Case 1

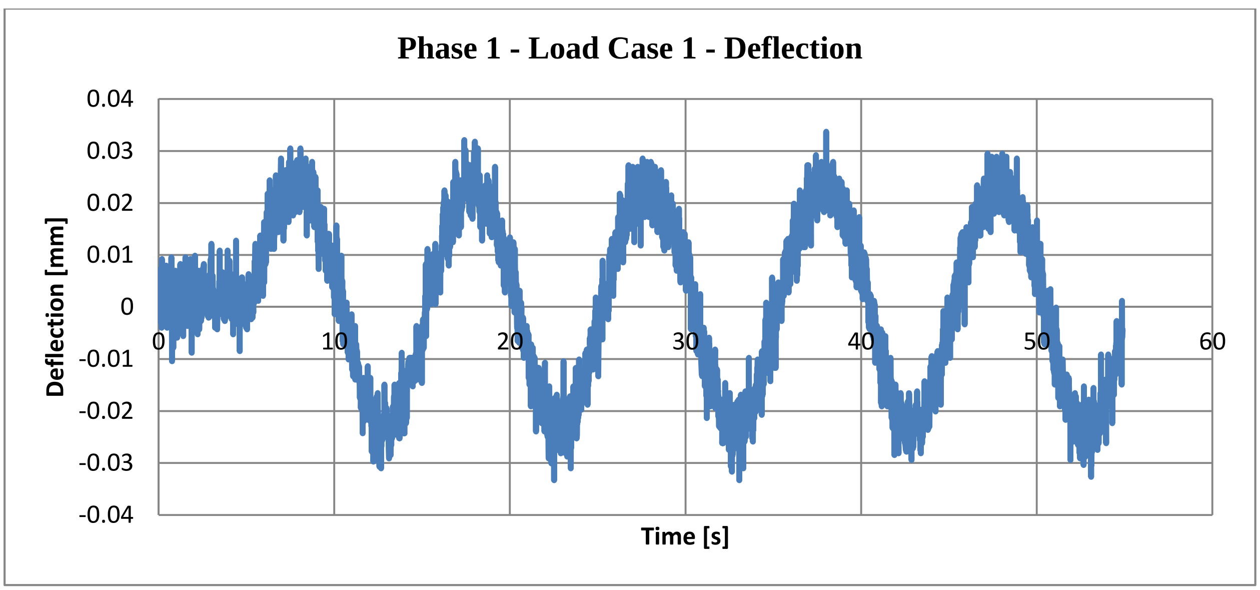


Figure 6: Centre-span Deflection for Phase 1 – Load Case 1

Figure 7 and Figure 8 show the strain and deflection for the final load case, Load Case 4, with vertical load on the arch.
The intent in the Phase 2, Load Case 4 was to cycle the displacement to complete arch failure. As illustrated in Figure 7 the experiment stopped after 42 cycles. After many cycles it was clear that the arch will not fail completely. The arch became a three-hinged arch, and developed hinges at two supports and at the centre-span and was able to sustain the load for many cycles.

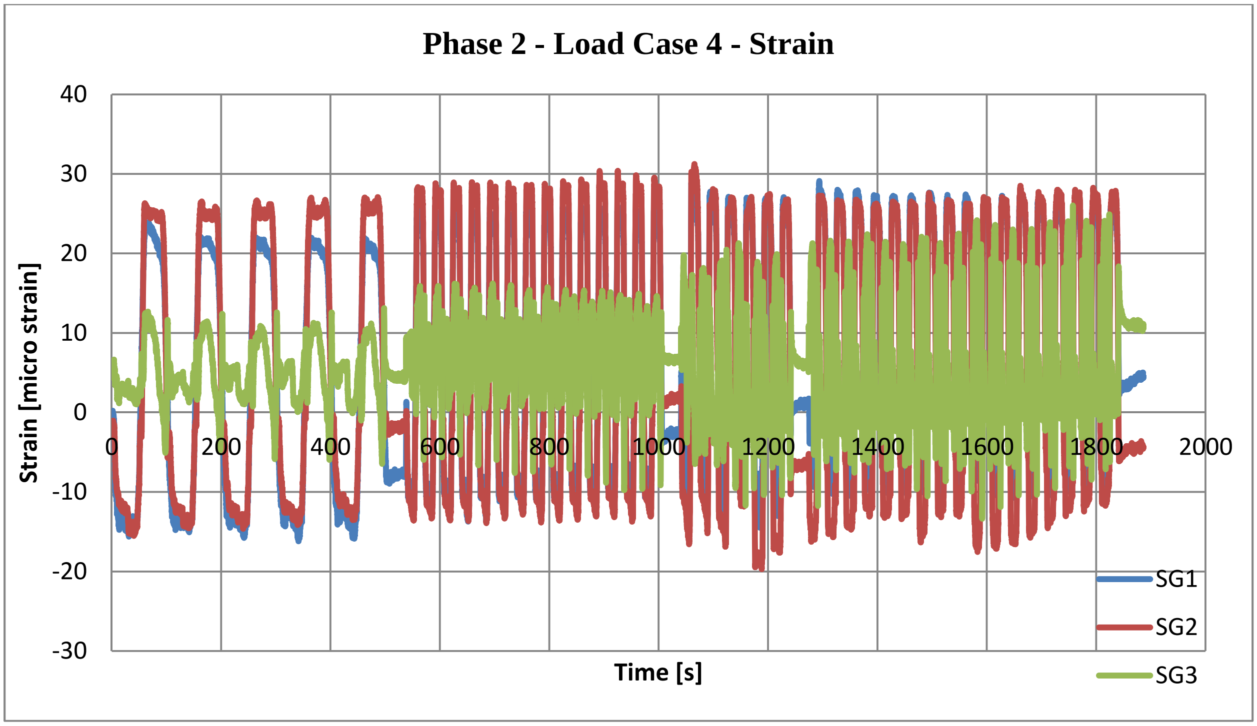


Figure 7: Strain Diagram for Phase 2 – Load Case 4

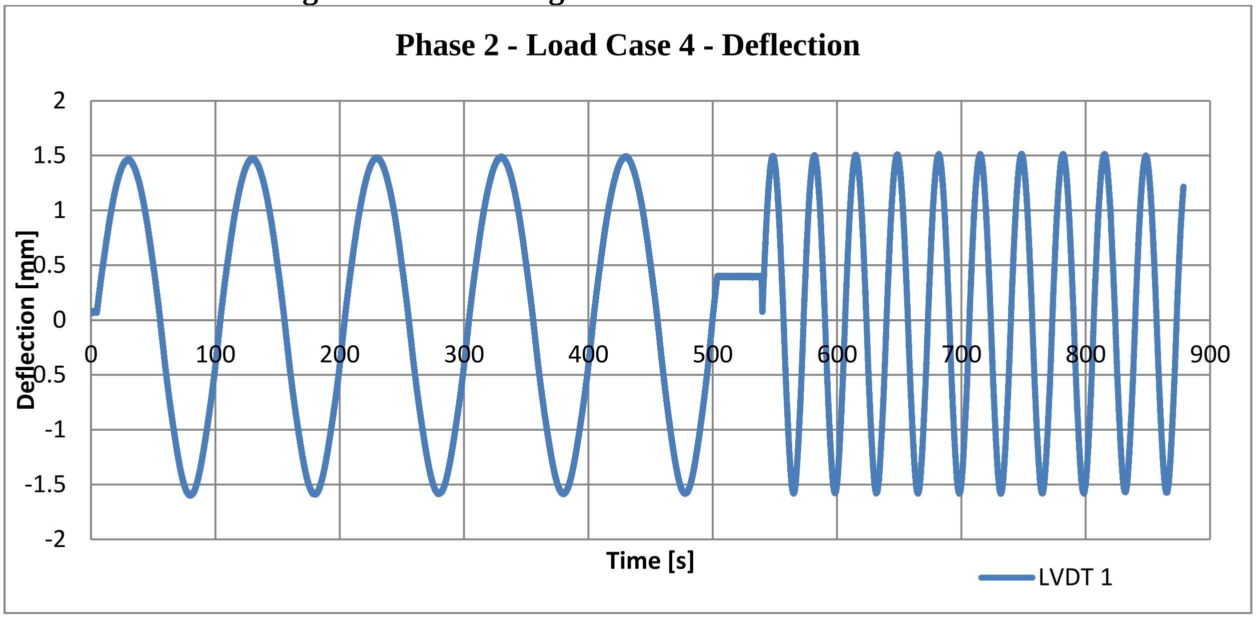


Figure 8: Centre-span Deflection for Phase 2 – Load Case 4

Figure 8 shows the deflection of the arch close to 900 seconds. This is due to the connection failure of LVDT 1. Once the centre-span hinge width increased the material (mortar) holding the seat for the LVDT was lost.

Figure 9 shows the location of the hinges on the arch once the three-hinge arch was developed. It was noted that the three-hinged arched was develop prior to loading, perhaps due to shrinkage or relaxation of the support stops at each end. Therefore, the arch acted as a three-hinged arch from the onset of the very first loading. For three-hinged arch analysis refer to reference 1.

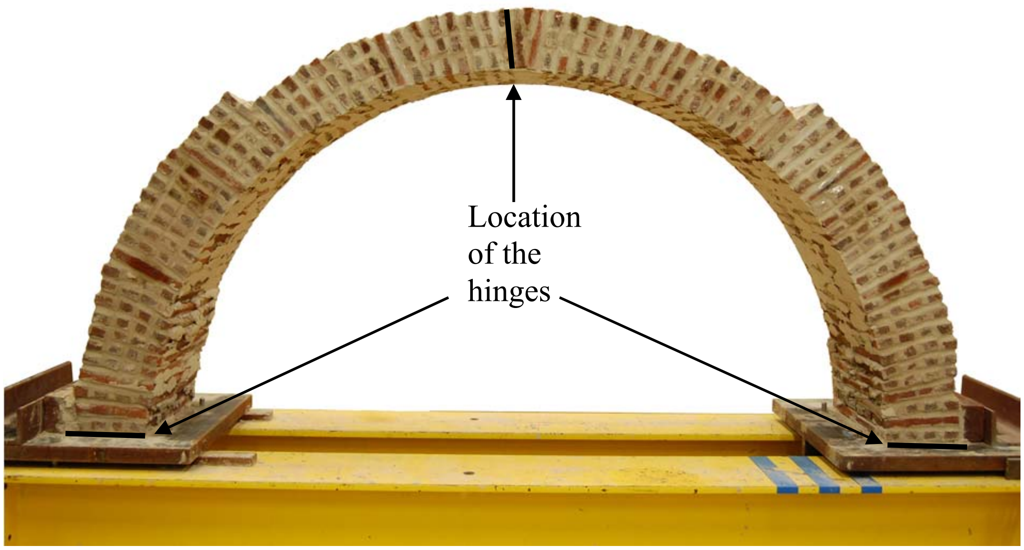


Figure 9: Three-Hinged Arch

NUMERICAL ANALYSIS

Numerical analysis was performed using Finite-Y program. This is a FORTRAN base program developed for nonlinear analysis of reinforced concrete members. This program has been modified so that it could analyse different frames and various materials such as steel-free bridge decks and fabric-formed RC beams. It was further modified to analyse masonry structures [2]. The final version of this program which was developed to analyse masonry walls was used for this analysis. For further information on this program refer to reference 2.

Material properties from the original study [1] were used for the analysis is due to material availability restraints. The masonry compressive strength, f'_m, was determined to be 12.27 MPa (1780 psi) [1]. In accordance with CSA S304.1 Modulus of Elasticity (E) and Shear Modulus (G) can be calculated as shown by equations (1) and (2):

$$E = 850\, f'_m = 850 \times 12.27 = 10429.5\ MPa \qquad (1)$$

$$G = 0.4\, E = 0.4 \times 10429.5 = 4171.8\ MPa \qquad (2)$$

In accordance with [2] the relationship between E and G can be defined as in equation (3):

$$E = r \times G \qquad (3)$$

Based on properties defined by experimental results the r can be determined as shown below:

$r = E/G = 10429.5/4171.8 = 2.5$ (4)

The analysis result presented by [1] shows very close correlation between r values of 2.5 and 3, with r=3 giving the best results for numerical analysis [2]. Using r values of 2.5 and 3 also gave results that were very similar to the experiment for this investigation. Figure 10 compares the deflection from the experiment with FEM analysis for load case one.

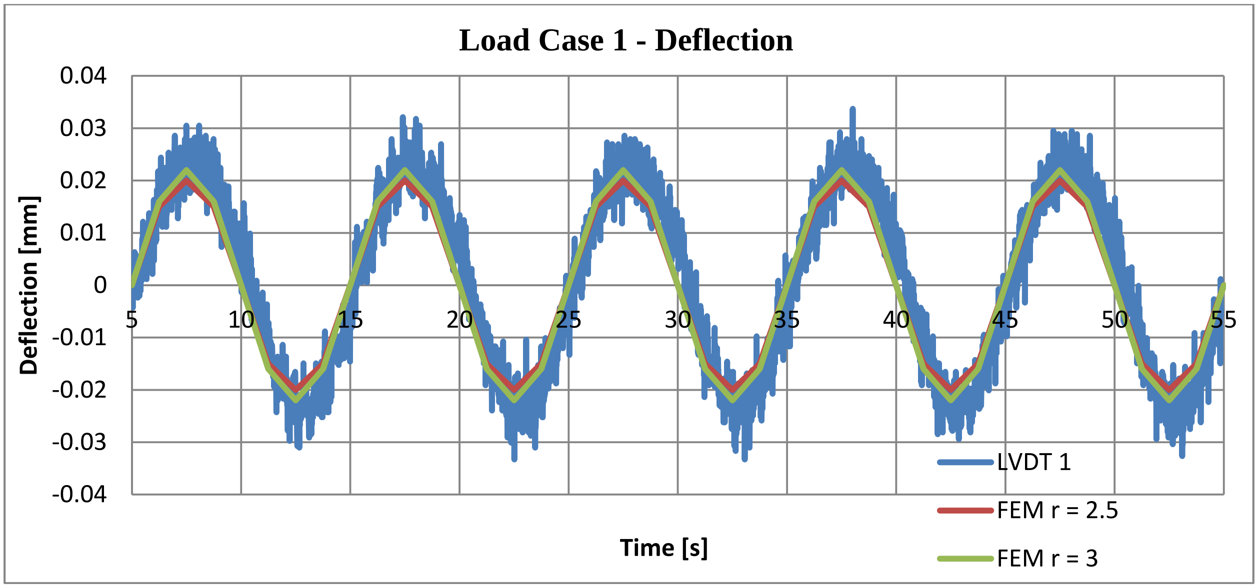
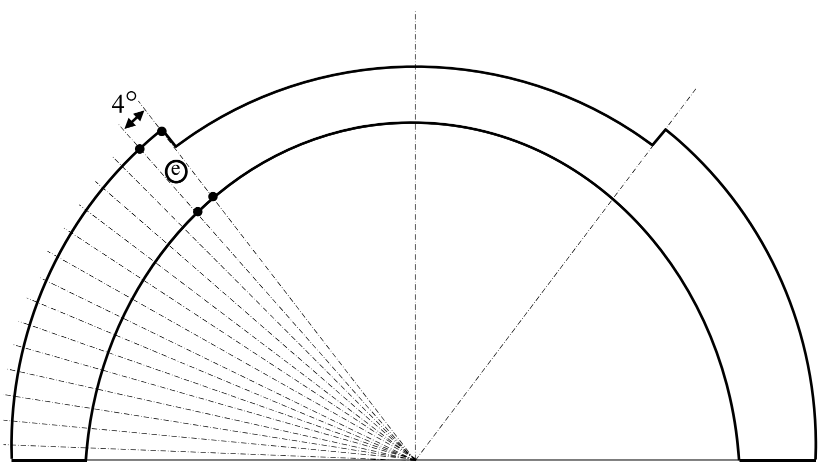


Figure 10: Deflection Comparison between Experiment and FEM Analysis

Quadrilateral elements were used to model the arch as shown in Figure 11. The width of each element was set as the thickness of the arch and the height was determined by dividing the arch in 4° increments as shown in the figure. For clarity, Figure 11 only shows a representative model.



(1)

Figure 11: Representative Model for FEM Analysis

CONCLUSION AND RECOMMENDATIONS

The primary purpose of this study is to present a possible solution to the issue of determining accurate in situ material properties for rehabilitation and remediation projects. As such, this was a preliminary study done to examine the hypothesis of the possibility of determining material properties from real-time structural health monitoring data and much more analysis of data is required. The study, however, showed close correlation between experimental r value and those suggested by Mufti and Jaeger [3]. Concluding that SHM data may be used to determine or verify in situ material properties. Although considerable additional studies are required due to the limited specimen and data collected in this study.

This research will continue by analysing strain values and comparing the experimental data with numerical date using the same r values used in deflection.

ACKNOWLEDGEMENTS

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