

Local-based damage detection of cyclically loaded bridge piers using wireless sensing units

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ABSTRACT

Concrete bridge piers are a common structural element employed in the design of bridges and elevated roadways. In order to ensure adequate behavior under earthquake-induced displacements, extensive reinforcement detailing in the form of closely spaced ties or spirals is necessary, leading to congestion problems and difficulties during concrete casting. Further, costly repairs are often necessary in bridge piers after a major earthquake which in some cases involve the total or partial shutdown of the bridge. In order to increase the damage tolerance while relaxing the transverse reinforcement requirements of bridge piers, the use of high-performance fiber reinforced cementitious composites (HPFRCC) in earthquake-resistant bridge piers is explored. HPFRCCs are a relatively new class of cementitious material for civil structures with tensile strain-hardening behavior and high damage tolerance. To monitor the behavior of this new class of material in the field, low-cost wireless monitoring technologies will be adopted to provide HPFRCC structural elements the capability to accurately monitor their performance and health. In particular, the computational core of a wireless sensing unit can be harnessed to screen HPFRCC components for damage in real-time. A seismic damage index initially proposed for flexure dominated reinforced concrete elements is modified to serve as an algorithmic tool for the rapid assessment of damage (due to flexure and shear) in HPFRCC bridge piers subjected to large shear reversals. Traditional and non-traditional sensor strategies of an HPFRCC bridge pier are proposed to optimize the correlation between the proposed damage index model and the damage observed in a circular pier test specimen. Damage index models are shown to be a sufficiently accurate rough measure of the degree of local-area damage that can then be wirelessly communicated to bridge officials.

Keyword: wireless sensors, damage detection, damage index models, high-performance fiber reinforced cementitious composites

1. INTRODUCTION

Bridges are important civil structures that are widely used within the transportation network of a country. In the United States, there are well over 575,000 bridges dispersed throughout the country. However, almost 60 percent of those bridges were constructed before 1970 when little or no consideration was given to their seismic resistance [1]. As a result, the Northridge earthquake (1994) revealed the poor seismic performance of several highway bridges constructed before 1971 with some bridge piers failing in shear [2]. Even some bridge structures built after 1971 experienced considerable structural damage that necessitated expensive post-earthquake repairs. In particular, bridge piers have been seen to be vulnerable to shear failure under the lateral displacement demands posed by earthquakes.

Concrete shear resistance mechanisms in flexural members are known to degrade under the action of inelastic displacement reversals [3-6]. This is primarily due to the formation and opening of intersecting flexural-shear cracks during load reversals, which reduce aggregate interlock and shear carried in the member compression zone. Thus, the logical design approach is to provide sufficient transverse reinforcement such that the total shear demand could be resisted through truss action. For the case of members subjected to combined bending and axial load, some "concrete" shear resistance is generally assumed [5-8], but it is known to degrade with increasing inelastic displacement demand. Closely spaced transverse reinforcement is also required to provide confinement to the concrete and prevent or delay buckling of longitudinal bars. The consequence of the need to provide sufficient shear resistance and confinement is substantial reinforcement congestion in plastic hinge regions of earthquake-resistant flexural members, let alone associated increases in labor costs and construction difficulties.

Research in the design of fiber reinforced cementitious composites (FRCC) has resulted in new materials with ultra-ductility, high damage tolerance and strain hardening behavior. Termed high-performance FRCC (HPFRCC), the material is composed of a small volume fraction of short fibers within a cement matrix [9,10]. Compared to the strain softening behavior of ordinary FRCC, HPFRCC possess pseudo strain hardening behavior, which is preferred for the design of earthquake-resistant structures. Some additional characteristics of HPFRCC include extreme tensile strain capacity (of up to 5.0%) and self-confinement. These outstanding material properties have led to HPFRCC becoming an attractive alternative to concrete for the construction of structures located in seismic regions. Structural members including columns, beams and shear walls are all structural elements vulnerable to failure under large displacement reversals, such as those experienced in seismic events. Because of the inherent brittleness of concrete, extensive transverse reinforcement detailing is required in the critical regions of these elements so that adequate shear performance can be achieved under extreme loading [8]. However, the dense amount of steel reinforcement required often leads to reinforcement congestion and concrete placement difficulties. Alternatively, HPFRCC could be used to alleviate much of the reinforcement congestion in regions of members subjected to inelastic displacement reversals. With ultra-ductility in tension and self-confining attributes, HPFRCC is a material capable of providing sufficient shear strength and confinement to allow for reductions in the amount of transverse reinforcement needed. To date, HPFRCC have been explored by several laboratory studies in structural systems whose responses are strongly dominated by shear, including shear walls, beam column connections, and coupling beams under reversed cyclic loading [11]. Based on the superior performance of HPFRCC materials in these shear critical elements, this study will explore the use of the material for the design of earthquake-resistant bridge piers.

In recent years, structural sensing technology has been going through dramatic change. As part of those changes, wireless sensors have emerged as novel substitutes to current wire-based structural monitoring systems. Through an eradication of wires, wireless monitoring systems are low-cost and easy to install as compared to tethered systems. Generally, wire-based monitoring systems can cost approximately \$5,000 per sensor channel with total system costs exceeding more than \$60,000 when 12 force balance accelerometer (FBA) channels are installed [12]. The use of wireless monitoring technology was first illustrated by Straser and Kiremidjian [13] who effectively illustrated that expensive wires are no longer needed between sensors and a data repository. Lynch *et al.* [14] have extended their work by integrating miniaturized computing technologies with sensors to provide decentralized computational ability for real-time interrogation of structural response measurements. Currently, low-cost wireless sensors have been installed on the Alamosa Canyon Bridge in New Mexico to monitor the acceleration response of the bridge during a validation study of the wireless structural monitoring system performance [15]. More recently, 14 wireless sensing units proposed by Wang *et al.* [16] have been installed on the Geumdang Bridge in South Korea to determine bridge natural frequencies and operational deflection shapes [17]. Wireless sensors with mobile computing technologies integrated can be used to locally interrogate structural response measurements for signs of structural damage. The majority of the damage detection methods embedded in the computing cores of wireless sensors have examined global vibration characteristics of a dynamic system to identify the existence of damage [18]. However, as costs continue to decline, an increasing number of wireless sensors will be installed in a single structure. This development is important because wireless monitoring systems with high nodal densities can be leveraged to screen structures for damage at both global (e.g. modal properties) and local length scales. To monitor locally, sensors can track the behavior of individual structural elements (e.g. beams, columns, joints). In this study, the wireless sensing units proposed by Wang *et al.* [16] are utilized to monitor the performance and health of a single HPFRCC bridge pier. In particular, a computationally simplistic damage index model embedded in the wireless sensing unit core will provide a rough measure of the amount of damage that is incurred by the pier under cyclic loading.

To address the construction problems typically created by the need for large amounts of closely spaced transverse reinforcement, as well as the potential large repair costs after earthquakes, this study seeks to devise an intelligent pier combining materials with superior shear strength and damage tolerance (HPFRCC) and embedded wireless sensing that can quantify the health of the pier in real-time. A partial scale circular bridge pier specimen constructed of HPFRCC without earthquake-resistant reinforcement detailing is laterally loaded with a quasi-static reversed cyclic displacement pattern in the laboratory. Wireless sensing units embedded with damage detection algorithms (damage index models) are employed for structural response acquisition and local data interrogation so as to achieve real-time structural health monitoring and a rough damage report of the pier.

2. DESIGN OF AN HPFRCC BRIDGE PIER

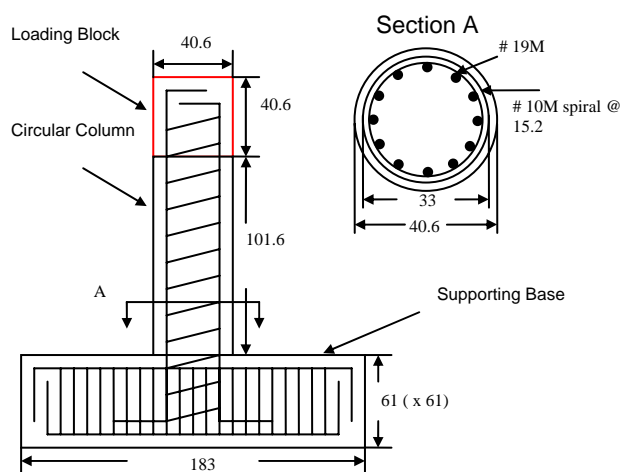
Cantilever concrete bridge piers are vulnerable during seismic loading with the pier base being the most critical region since this is where plastic hinges will be formed under the action of earthquake-induced displacements. Damage in plastic hinge regions may include large flexural and diagonal cracks, concrete crushing and spalling, and reinforcement buckling. All of these conditions could lead to premature structural performance degradations when bridges are undergoing displacement reversals. Within current codes [8], shear strength attributed to the concrete is typically neglected and a vast amount of closely spaced transverse steel reinforcement is required to ensure sufficient shear strength of the member, to provide confinement to the concrete, and to prevent longitudinal reinforcement from buckling, especially in potential plastic hinge regions. Unfortunately, this large amount of transverse reinforcement can lead to unavoidable reinforcement congestion and concrete placing problems. Unlike buildings which have a large number of plastic hinges within a single structure, bridge structures are typically less structurally redundant and have fewer plastic hinges than buildings; as a result, plastic hinges play a more critical role in the structural behavior. The strain hardening property of HPFRCC under tension renders the material strong in shear. As such, it can be used to provide bridge piers and flexural members in general with sufficient shear strength so as to reduce current shear steel reinforcement detailing mandated by building codes [7,8]. In addition to its strength in shear, HPFRCC is capable of providing support to the longitudinal bars, as well as an increased displacement capacity due to its excellent ductility both in tension and compression. In HPFRCC materials, concentrated macro-cracks only occur under large displacements when the strength of the fiber-matrix interface is exceeded (this is referred to as damage localization). Although no seismic design guidelines have yet been developed for the reasonable use of HPFRCC in plastic hinge regions of reinforced concrete structural members, studies regarding the use of fiber reinforced concrete within the plastic hinge regions of reinforced concrete flexural members have been conducted [11].

For the purpose of investigating the performance of HPFRCC materials in bridge pier bases under large displacement reversals, a circular bridge pier specimen with a 40.6 cm diameter is constructed of HPFRCC material and tested in laboratory under quasi-static lateral loading. The dimensions of the HPFRCC column roughly represent a 1/3 scaled specimen of a true pier. The bridge pier specimen prepared in this study is designed with non-seismic transverse steel reinforcement detailing as shown in Figure 1(a) because of the anticipated shear strength of the HPFRCC material. The longitudinal reinforcement provided in the bridge pier specimen corresponded to a reinforcement ratio of 2.6% while the volumetric ratio of spiral reinforcement is 0.56%. Note that a spiral pitch of 15.2 cm is provided, which is substantially larger than the maximum allowed in reinforced concrete members reinforced by spirals (in the order of 3 to 8 cm) [8]. The HPFRCC material is prepared by combining a 1.5% volume fraction of 3.8 cm spectra fibers with a cement mixture, as detailed in Table 1. For the rectangular supporting pedestal and the top square loading cap, a normal weight regular concrete mixture is used since these elements are not of interest in the study. Because no fibers bridge the interface between the bridge pier base and the base block, additional longitudinal reinforcement is placed at the HPFRCC-concrete interface so as to relocate the potential plastic hinge region to roughly 16 cm above the interface. This allows the full capacity of the HPFRCC material to be utilized.

The final test specimen is composed of three parts, including the concrete base which is 61 x 61 x 183 cm (width, depth and length), a 1.02 m tall HPFRCC column, and a top loading block whose dimensions are 40.6 x 40.6 x 40.6 cm, leading to a shear span-to-diameter ratio of 3.0. Construction of the whole test specimen is divided into three stages. Regular concrete is firstly poured in the supporting base. The 40.6 cm diameter HPFRCC column is then poured to a total height of 1.02 m above the base. Last, the cubic cap is poured after the HPFRCC column. Grade 420M steel bars are used for both the longitudinal and spiral reinforcement. The nominal compressive strength (f'_c) of regular concrete and HPFRCC test cylinders are measured to be 34.5 MPa and 41.8 MPa on the test day, respectively.

3. INSTRUMENTATION STRATEGY

A key element of the proposed study is the extensive use of wireless sensors to collect the response of the pier under quasi-static loading. As opposed to traditional wire-based structural monitoring systems, wireless sensing units possess many beneficial features that encourage their use for this application. First, reliable wireless communications between sensors renders the technology attractive for adoption because of the cost-benefit of not having to install extensive lengths of cables through out the structure. Second, a sophisticated computational core integrated with the wireless



(a)

(b)

Figure 1 – (a) Geometry and reinforcement detailing of the bridge pier (units shown are cm), (b) the completed HPFRCC bridge pier

Table 1 – Weight ratios of each component used in the HPFRCC cement matrix

Material	Weight Ratio
Cement (Type III)	1.00
Fly Ash	0.15
Sand (Flint)	1.00
Water	0.50
Total	2.65

sensing unit enable local execution of data interrogation algorithms by the sensor. In this study, a rapid damage detection algorithm is embedded in the core of a wireless sensing unit so that a rough measure of the amount of damage can be wirelessly reported to a bridge official in near real-time. A prototype wireless sensing unit, developed by Wang, Lynch and Law [14] is used to collect the response of the pier during lateral loading. The wireless sensing unit prototype contains a 16-bit analog-to-digital converter (ADC) for high-fidelity data acquisition for sensors such as linear variable displacement transducers (LVDTs), strain gages and accelerometers. When collecting the pier response, 128 Kbytes of on-board memory is utilized for the storage of the response data. The computational core includes a low-power 8-bit Atmel ATmega128 microcontroller where data is interrogated using embedded damage index models. To wirelessly communicate data from the test specimen to a central data repository, a 900 MHz spread spectrum radio offering reliable communications is integrated with the wireless sensing unit.

The HPFRCC bridge pier specimen is instrumented with several LVDTs at the pier base (plastic hinge region) so as to investigate its flexure and shear behavior under reversed cyclic loading. Also, a total of 20 strain gages are mounted on the surface of the longitudinal and transverse reinforcement within the HPFRCC pier to monitor strains during load testing. The displacement data collected by the LVDTs at the base will be used to calculate the shear and flexural deformation components in the pier plastic hinge. To be able to differentiate between shear and flexure, 4 LVDTs are mounted to the side of the pier base (in a plane parallel to the loading direction). As shown in Figure 2(a), 2 LVDTs are placed in an X-configuration while the remaining two are placed parallel and orthogonal to the pier longitudinal axis. An additional set of LVDTs is installed to the front and back sides of the pier to measure plastic hinge rotations, as shown in Figure 2(b). The procedure for differentiating shear and flexure from this LVDT configuration will be described in detail in a later section.

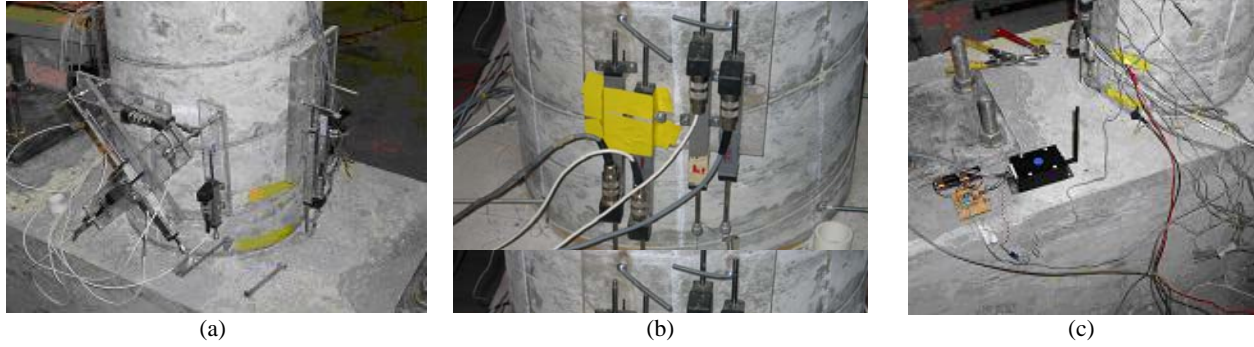


Figure 2 – (a) LVDTs instrumentation at the pier base, (b) two LVDT pairs mounted to the front and back pier faces, and (c) a wireless sensing unit mounted to the top of the concrete support pedestal

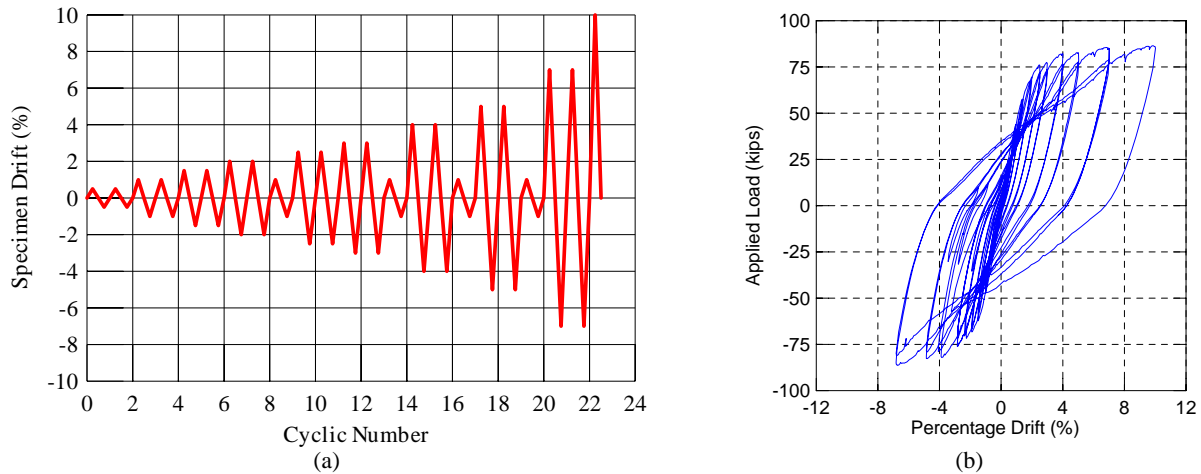


Figure 3 – (a) Applied loading, and (b) the corresponding shear force-drift response

4. CYCLIC LATERAL LOAD TEST OF THE HPFRCC BRIDGE PIER

The seismic behavior of the HPFRCC bridge pier is evaluated through a quasi-static laboratory load test. A 450kN actuator, which will provide lateral displacement reversals, is fitly attached to the cubic block on top of the bridge column. Cyclic loading is slowly applied to the specimen under displacement control; at the displacement peaks, the test is paused to allow for tracing crack patterns on the pier surface using markers. As shown in Figure 3(a), the HPFRCC bridge pier specimen was subjected to drift levels (the lateral displacement divided by the distance between the pier base and the loading point) ranging from 0.25% to 10%. The corresponding lateral force versus drift response of the HPFRCC bridge pier is shown in Figure 3(b).

As shown in Figure 3(b), the bridge pier maintains its lateral strength up to 10% drift in its positive and negative loading directions, which is an extremely large displacement for such a structural component. Damage in the bridge pier consists of a dense array of hairline flexural and diagonal cracks, which formed during the first few loading cycles (<2.0% drift). At 4.0% drift, damage localization occurs at one of the flexural cracks, leading to significant widening of that crack at larger drift demands. However, the opening of such a crack does not compromise the integrity of the specimen because flexural resistance does not rely on the tensile behavior of the HPFRCC material. Further, the HPFRCC material maintains its integrity through the test, being effective in providing confinement to the longitudinal reinforcement even after damage localization. Even though a peak shear stress demand of 2.7 MPa ($0.42\sqrt{f'_c}$) is imposed on the bridge pier specimen, only hairline diagonal cracks form throughout the test. As a result, total damage of the bridge pier specimen can be considered minor since no shear-related damage and strength decay occur. As shown in



Figure 4 – (a) Crack pattern at 4% drift at the base (b) Crack pattern at 10% drift at the base

Figures 4(a) and 4(b), there is no spalling and crushing of the HPFRCC in the plastic hinge region during the test. This is mainly due to the excellent tensile and compression properties of the HPFRCC, which also provides excellent confinement to the longitudinal reinforcement bars thereby preventing them from buckling during large displacement reversals. The results from the bridge pier test suggest that minor post-earthquake repairs would be needed in HPFRCC bridge piers after a major earthquake with no interruption of bridge service.

5. SEISMIC DAMAGE INDEXES FOR HPFRCC ELEMENTS

During the design of earthquake-resistant reinforced concrete (R/C) structures, a certain amount of the structure's seismic energy is anticipated to be dissipated through inelastic deformations. During such seismic excitations, R/C structural elements should maintain their structural integrity and should be able to withstand the local accumulation of inelastic deformations without collapse. In order to assess the global structural integrity of critical structural elements and to quantify the level of structural safety, quantitative damage measurements taken of structural members are necessary. Many damage models have been proposed by the research community for tracking the progress of damage in cyclically loaded R/C structural elements [19,20]. A large number of these damage models correlate damage of a structural element to specific structural response parameters such as deformation, internal forces or dissipated hysteretic energy. These models place their assessment of structural health on a scale from 0 to 1. Such damage indexes suggest the member is in its virgin state when the index is 0 and severely damaged (to the point of failure) when the index achieves a value of 1.

Damage index models are a powerful tool for providing a rough measure of the health of a structural element immediately following a seismic event. For example, a rough picture of the degree of damage can be used by structural owners to prioritize the inspection of their structures. An accurate damage index model based solely on the energy dissipated by an R/C structural element under cyclic loading has been proposed by Kratzig *et al.* [20]. The characteristics of this damage index model are simply described as follows. By referring to the measured structural response, for example, as shown in Figure 5, positive and negative peak responses in each cycle of the total response history are employed. To provide the model with a sense of directionality, the damage index is calculated separately for the positive and negative peaks. Therefore, the damage model is divided into positive and negative indexes, D^+ and D^- , in order to account for the unsymmetrical response of a structural element. At each peak, the damage index is calculated based on the current peak value and previous peaks. As part of calculating the damage index at each peak, the set of peaks that include the current and previous peaks must be appropriately classified as "primary" or "follower" half-cycle peaks. A primary half-cycle (PHC) corresponds to a peak response that is greater than that of any of the previous peaks. However, if the structural response within a half-cycle has a peak less than the maximum structural response in any of the previous half-cycles, then the cycle is denoted as a follower half-cycle (FHC). To better understand the PHC and FHC naming convention, consider Figure 5. If the damage index is calculated at positive peak

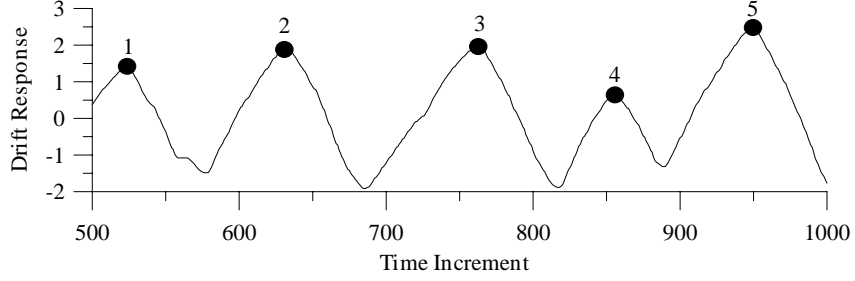


Figure 5 – A period of drift response of the HPRCC bridge pier

number 2, this peak is labeled as a PHC peak because it is larger than peak 1; peak 1 is labeled as a FHC peak. If the damage index is calculated at peak 3, it is larger than peaks 2 and 1 and is therefore labeled as a PHC peak. In contrast, peaks 1 and 2 are labeled as FHC peaks. Now, if the damage index is to be determined at peak 4, peak 4 is smaller than peak 3. As a result, peak 4 is labeled as a FHC along with peaks 2 and 1 while peak 3 retains the designation of the PHC peak. One last feature of the Kratzig damage index model is the use of the total structural capacity of monotonically loaded structural members up to failure to serve as a normalizing factor of the model. The accuracy of this damage model for R/C flexural elements is validated by finite-element simulation and experimental results reported by Kratzig *et al.* [20]. In a similar manner, damage index models describing the health of HPRCC structural members in both flexure and shear are devised in this study and are largely based on the model proposed by Kratzig *et al.* [20].

In order to differentiate between the damage caused by the shear and flexure responses during cyclic loading, separate damage index models for both the shear and flexural deformation of HPRCC bridge piers will be developed. In this study, the structural response parameter used as the input to the damage index model will be the lateral displacements of the bridge pier base that correspond to shear, U_s , and flexure, U_f , as shown in Figure 6. Figure 7(a) shows the X-configuration of LVDTs instrumented at the bridge pier base. Referring to Figure 7(b), the horizontal displacements, U_1 and U_2 , contain both shear and flexural components that should be differentiated in order to evaluate the structural behaviors attribute to shear and flexure. If only the first order shear deformation is considered, and it is assume that the lateral displacement due to flexure is equal on each side of the pier, then U_1 and U_2 can be treated as a single variable, U_t , that can be expressed as:

$$U_t = \frac{(L_1^2 - L_2^2) - a^2}{2a} \quad (1)$$

where a and b correspond to the original undeformed horizontal and vertical lengths of the instrumented pier panel, respectively. Based on the geometry shown in Figure 7(b), the associated vertical displacements, V_1 and V_2 , can be obtained once the total lateral displacement is calculated, which can be expressed as:

$$V_1 = \sqrt{(L_3 + a - U_t) * (L_3 - a + U_t)} - b \quad (2)$$

$$V_2 = \sqrt{(L_2 + U_t) * (L_2 - U_t)} - b \quad (3)$$

Flexural deformation: In analyzing the flexural deformation of plastic hinge regions of bridge piers under lateral loading, the plastic hinge rotation, θ , is first calculated and then used to estimate the corresponding flexural deformation. Generally, the hinge rotation is associated with the vertical displacements, V_1 and V_2 , and the horizontal distance between them, a , which results in:

$$\theta = \frac{V_1 - V_2}{a} \quad (4)$$

Because of the moment gradient in the bridge pier and the fact that flexural cracking concentrated more towards the lower portion of the plastic hinge region, a distance from the top of the LVDTs X-configuration to the center of rotation of 2/3 of the total vertical length, b , is assumed. Then, the flexural displacement of the pier base, U_f , can be expressed as:

$$U_f = \rho \theta b \quad (5)$$

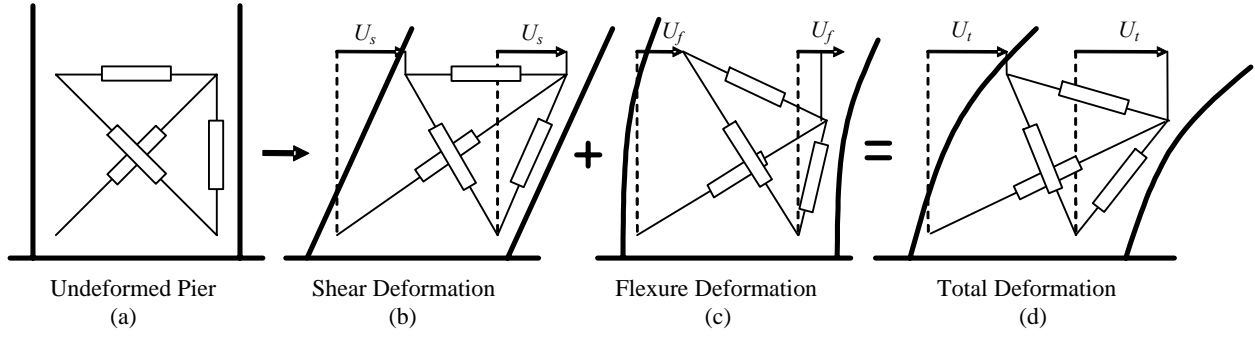


Figure 6 – Combination of shear and flexural deformations in plastic hinge region of bridge pier specimen

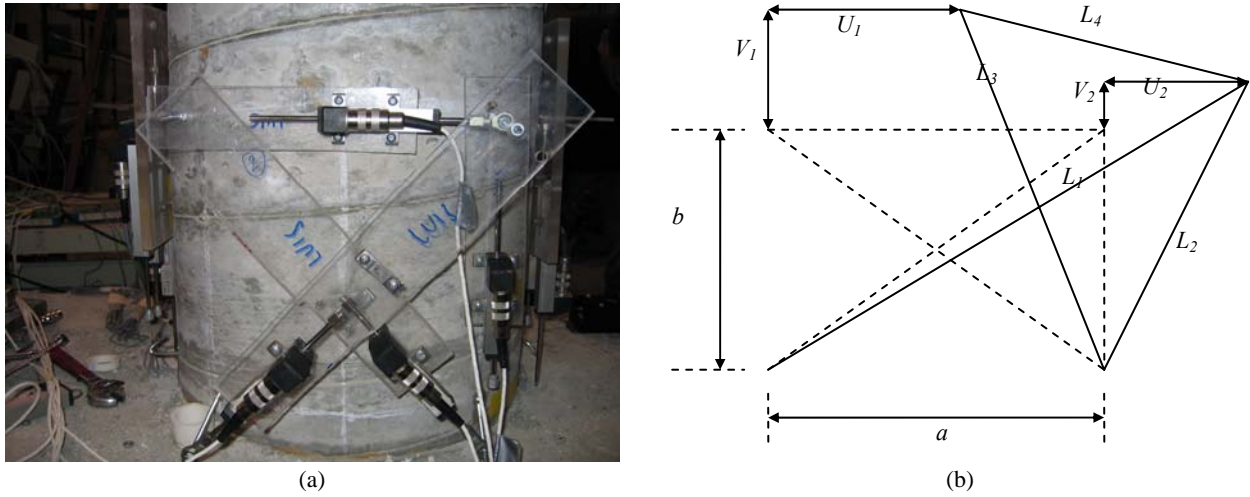


Figure 7 – (a) Instrumentation of the LVDTs at the pier base, and (b) measurements of the total deformation

where ρ is equal to $2/3$ and is consistent with the value used by others [21,22].

Shear deformation: Since the total lateral displacement contains flexural and shear displacements, the later term, U_s , can be easily obtained once the flexural displacement is calculated from Eq. (5). Therefore, both flexural and shear deformations are extracted out from the measured total lateral deformation.

$$U_s = U_t - U_f \quad (6)$$

Flexural Damage Index Model: After the flexural deformation of the pier plastic hinge has been determined from the X-configuration of LVDTs, the damage index characterizing the damage due to flexure is calculated. The damage index model uses the peak flexural displacement, U_f , of the pier base in both the positive and negative directions as an input. The damage index model calculated at each positive peak is an adaptation of the damage index model proposed by Mehanny and Deierlein [23]:

$$D_f^+ = \frac{(U_{f,PHC}^+)^{\alpha} + (\sum U_{f,FHC}^+)^{\beta}}{U_{f,Monotonic}^+ + (\sum U_{f,FHC}^+)^{\beta}} \quad (7)$$

where $U_{f,PHC}^+$ is the positive peak flexural displacement of the primary half-cycle (PHC), $U_{f,FHC}^+$ is the positive peak flexural displacement of the follower half-cycles (FHC), and $U_{f,Monotonic}^+$ is the monotonic maximum flexural

displacement capacity. In this study, $U_{f,Monotonic}^+$ was not obtained at the end of the test because the maximum applied displacement (corresponding to 10% drift) was at the maximum capacity of the actuator. Therefore, the maximum flexural displacement corresponding to monotonic limit of the HPFRCC material is estimated to occur at roughly 14% drift. Once the flexural damage indexes of both positive and negative direction for the full cycle are completed, they are combined into a single flexural damage index.

$$D_f = \sqrt[\gamma]{(D_f^+)^{\gamma} + (D_f^-)^{\gamma}} \quad (8)$$

The weighting terms, α , β , and γ , allow the damage model to be tuned to empirical data taken of HPFRCC elements tested in the laboratory. Assuming that the damage induced by the PHCs and FHCs as well as the interaction between damage in both loading directions in HPFRCC members is “identical” to that in reinforced concrete members, the damage index weighting terms determined for concrete elements by Mehanny and Deierlein [23] are used in this study ($\alpha=1$, $\beta=1.5$, and $\gamma=6$).

Shear Damage Index Model: The shear damage index use the shear displacement of the pier base extracted from the total lateral deformation measured using the LVDTs. However, for an HPFRCC bridge pier, shear behavior of the pier before and after fiber pullout are much different and need to be discussed separately. Before fiber pullout, a fiber bridging mechanism allows the HPFRCC material to contribute to the total shear strength, and the shear stiffness and strength of the pier is larger than that after fiber pullout. Therefore, another term in the denominator accounting for the fiber pullout mechanism should be used in the shear damage index [24]. Fiber pullout has been experimentally observed to play a major role in the behavior of HPFRCC structural elements in shear while fiber pullout has negligible influence on flexural behavior. As a result, the shear displacement of the pier, $U_{s,FiberPullout}$, is used as a third normalizing term so as to differentiate the two different mechanisms of the shear behaviors of HPFRCC bridge piers. For example, shear damage index in the positive direction before fiber pullout can be expressed as:

$$D_s^+ = \frac{(U_{s,PHC}^+)^{\alpha} + (\Sigma U_{s,FHC}^+)^{\beta}}{U_{s,Monotonic}^+ + (U_{s,FiberPullout}^+)^{\alpha} + (\Sigma U_{s,FHC}^+)^{\beta}} \quad (9)$$

After fiber pullout, the shear displacement of the pier at fiber pullout is removed because the material behaves in a fashion similar to reinforced concrete:

$$D_s^+ = \frac{(U_{s,PHC}^+)^{\alpha} + (\Sigma U_{s,FHC}^+)^{\beta}}{U_{s,Monotonic}^+ + (\Sigma U_{s,FHC}^+)^{\beta}} \quad (10)$$

Similar to the flexural damage index model, a single damage index will be calculated at the end of each cycle once the positive and negative indexes are calculated:

$$D_s = \sqrt[\gamma]{(D_s^+)^{\gamma} + (D_s^-)^{\gamma}} \quad (11)$$

Damage index models have been previously fitted to the response parameters of HPFRCC elements dominated by shear action [24]. The weighting terms derived for these elements are adopted in this study ($\alpha=0.9$, $\beta=3$, and $\gamma=7$).

5. DAMAGE INDEX MODELS EMBEDDED IN A WIRELESS SENSOR

Figure 8(b) illustrates the firmware embedded in the wireless sensing unit to calculate the damage index models. In order to validate the ability of wireless sensing unit in monitoring structural damage evolution under cyclic loading, the structural response time-history of the X-configuration of LVDTs is stored into the wireless sensing unit memory. Based on the empirical data adjusting the index value ranging from 0 to 1, the damage index model tuning parameters α , β , and γ , are chosen to be 1, 1.5, and 6 for flexural damage index and 0.9, 3, and 7 for the shear damage index. In order to account for the different shear behaviors of HPFRCC bridge pier due to fiber pullout mechanism, $U_{s,FiberPullout}$ and

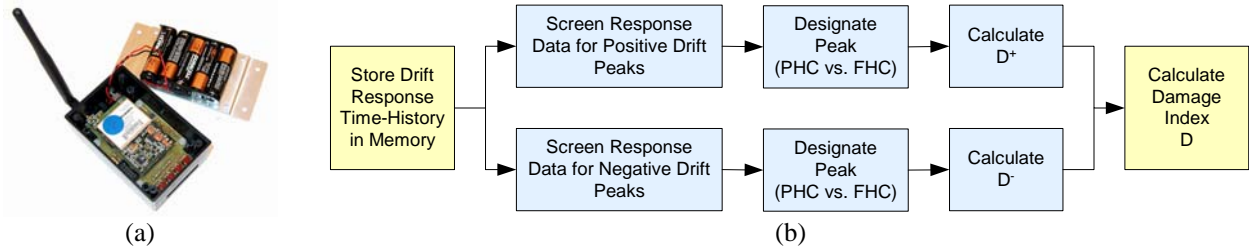


Figure 8 – (a) Prototype wireless sensing unit; (b) overview of the embedded firmware to calculate the flexural and shear damage index models

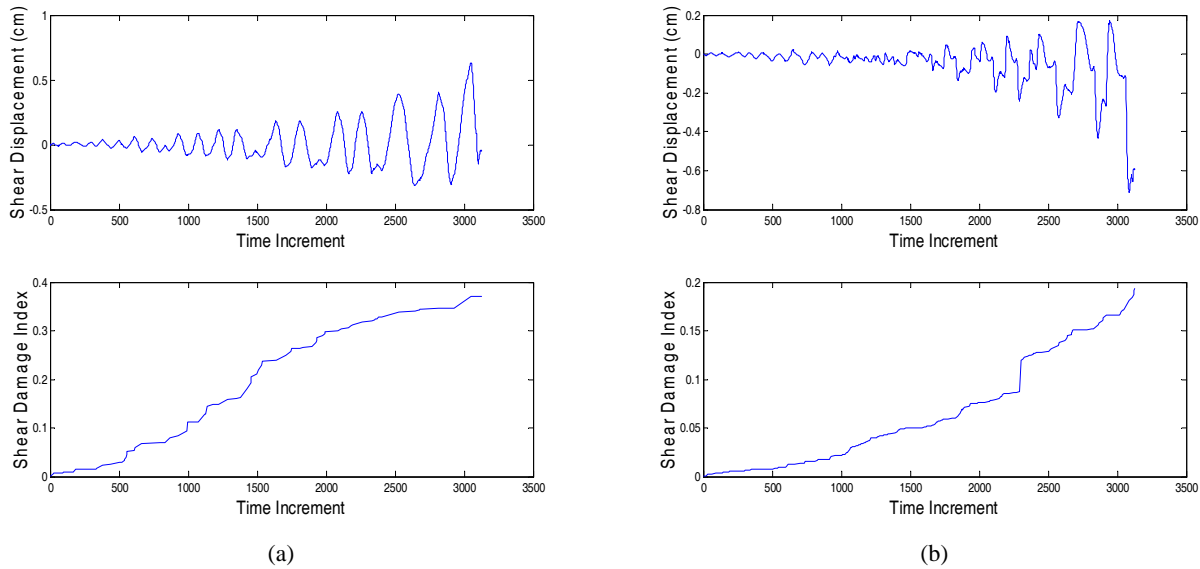


Figure 9 – HPFRCC bridge pier damage index for (a) flexural deformation and (b) shear deformation

$U_{s, Monotonic}$ are selected based on empirical data taken from other shear dominant HPFRCC elements tests. In this study, $U_{s, FiberPullout}$ and $U_{s, Monotonic}$ are chosen to be 0.28 cm and 1.12 cm corresponding to 1% and 4% shear strain, respectively.

Figure 9 shows the final damage index calculated by the wireless sensing unit for both flexure and shear. Although the HPFRCC bridge pier specimen maintains its structural integrity and does not exhibit a decay in load carrying capacity even at the end of the test (10% drift), a wide flexural crack is observed in the plastic hinge region at the pier base, which is related to moderate flexural damage, as denoted by the damage index value of 0.4. However, no major diagonal cracks are observed during the test, which implies that the shear damage of the bridge pier specimen is minor. The minor damage is consistent with the calculated damage index value of 0.2. Based on the low damage index values, the bridge pier has not experienced significant damage and still has capacity for additional load cycles and/or greater lateral displacement demands.

6. CONCLUSIONS

HPFRCC is a newly developed civil engineering material that can enhance shear strength, ductility, damage tolerance, as well as energy dissipation ability in members subjected to earthquake-induced displacements. These features are especially preferred for the construction of structures located in highly seismic zones. The HPFRCC bridge pier specimen investigated in this study has proven to have remarkable ductility and damage tolerance, maintaining its

structural integrity and load carrying capacity even under extreme large displacement reversals. As the field begins to adopt HPFRCC materials in realistic structures, an opportunity exists to monitor the performance of HPFRCC components and systems in the field. The wireless sensing unit used in this study illustrates the power of its computational core thereby facilitating real-time decentralized structural health monitoring to be performed. In order to evaluate the damage evolution of HPFRCC structural components under cyclic loading, the damage index model originally proposed by Kratzig *et al.* [20] is adopted for use with HPFRCC elements. The damage index models proposed in this study consider two deformation components in the plastic hinge region (flexural and shear). The damage index model is shown to be a suitable and an accurate indicator in determining damage situation of HPFRCC bridge piers under reversed cyclic loading. Further development of the damage index models will be carried out so as to provide the appropriate tuning parameters (α , β , and γ) of the damage indexes. A second bridge pier specimen is currently under design and construction, which will aim to improve the determination of structural damage using wireless sensors

7. ACKNOWLEDGEMENT

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