Canadian Conference - Pacific Conference on Earthquake Engineering 2023
Vancouver, British Columbia
June 25th – June 30th, 2023

Novel Materials and Technologies in Seismic Retrofit of Existing Reinforced Concrete Structures

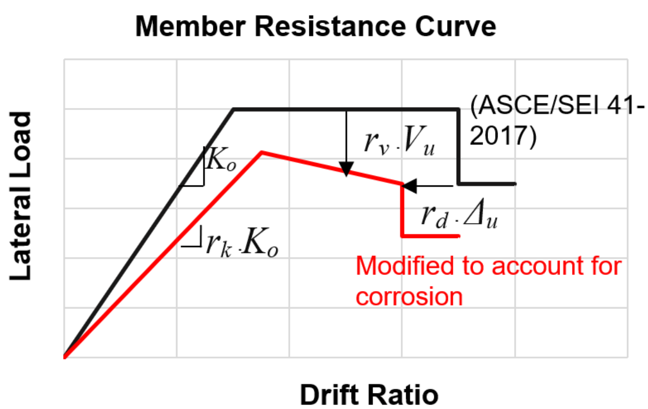
S. J. (Voula) Pantazopoulou
Department of Civil Engineering, York University, Toronto, ON M3J 1P3, Canada.

Abstract:

The presentation will review the background of the introduction of new technologies and materials in the seismic retrofit of structures. Emphasis is placed in the transition towards the new generation of tension-hardening Ultra High-Performance Cementitious Materials, (UHPC) and the prospects and opportunities that these materials provide for seismic design and seismic retrofitting solutions. These materials are used already in bridge construction and bridge rehabilitation, as well as in 3D printing technologies. Apart from very high compressive and tensile strengths (>120MPa and >6 MPa respectively), their particular, very useful characteristic is the extended tensile deformation capacity, the tension hardening characteristic property after cracking, and their exceptional durability. These qualities bypass some of the weaknesses of the existing methods and render the tension-hardening materials ideal solutions for application in structures subjected to significant seismic demands. After reviewing the state-of-the-art and experimental evidence regarding the performance of retrofitted components using novel tension hardening materials under reversed cyclic loading, the presentation is focused on the formulation of performance criteria and their integration in the framework of seismic design of structural retrofits using these technologies. The same principles are extended to cover recently developed strategies for seismic retrofitting of corrosion-damaged components. A review of the experimental evidence is relied upon for the development of performance-based criteria and their implementation in the context of seismic design and assessment procedures for seismically deficient, corrosion-affected structures.
***Keywords**: UHPC; Seismic; Retrofit; Corrosion; Confinement; Jacketing*

Introduction and Background

In the discourse for sustainability in construction a prevalent idea is that *the most sustainable structures are those that are already built* (Carl Elefante, American Institution of Architects); this was said, of course, with reference to the CO2 footprint and energy consumption. We, as Structural Earthquake Engineers, might add - "*if they would only be resilient against seismic hazards*". The concern of seismic vulnerability of the built environment is recurring every time a catastrophic earthquake reminds us that about 65% of buildings and infrastructure in most of the developed world, were built prior to the introduction of modern seismic design procedures. Seismic vulnerability is ever-increasing with the deprecating accumulation of reinforcement corrosion. Figure 1 depicts the effects of corrosion on the lateral load resistance curve of a typical column element. Factors r_k, r_v and r_d have been calibrated from collective evaluation of experimental and computational evidence, and are used to attenuate the stiffness, strength, and deformation capacity, respectively, of a structural component affected by reinforcement corrosion. Parameter x represents the percentage of mass loss in the reinforcement, assuming uniform corrosion; it should be noted that the implications of pitting corrosion could be even more dramatic, with disproportionate implications and difficult to assess[1]. When implementing the degraded properties in the case of an example building – a reinforced concrete frame with 10% mass loss in the reinforcement of the first storey, it is found that drift demand is at least doubled – this is tested under a suite of 10 different earthquake records having either near field or far field-characteristics. The increased demand is owing to the stiffness reduction represented by factor r_k; with the available deformation capacity reduced by r_d, it is evident that satisfying the design inequality for any given performance limit state is increasingly more precarious as the building continues to age. This concern is particularly important for structures with poor-quality concrete and thin covers which are typical of old construction. But they may also prevail in newer structures (e.g. bridge piers), where exposure to deicing salts is another, alternative means of condition deprecation leading practically to the same result.



	$v \leq 0.2$	$0.2 \leq v < 0.4$
r_k	$1\text{-}1.29 \cdot (x/100)$	$1\text{-}1.07 \cdot (x/100)$
r_v	$1\text{-}0.5 \cdot \theta_d \cdot (x/100)$	$1\text{-}0.8 \cdot \theta_d \cdot (x/100)$
r_d	$1\text{-}2.2 \cdot (x/100)$	$1\text{-}2.75 \cdot (x/100)$

θ_d is the drift ratio. The corrosion rate x is given as the number before the % (e.g. for 10% mass loss, x=10).

Figure 1: Reduction of Effective Stiffness of the Corroded Component by the stiffness attenuation factor, r_k; Nonlinear response of Plastic Hinges through the attenuation coefficients r_d & r_v. Values are calibrated against test data and numerical simulations[1] and they depend on the axial load ratio $v = P/A_g f_c'$, where P is the column axial load and A_g is the gross sectional area.

Nevertheless, the extent of investment represented by existing structures is vast – replacing it would take a huge amount of resources and would leave an even greater environmental footprint. A great range of techniques for upgrading the seismic resistance of existing construction are known and, with the advent of novel engineering materials, the number of plausible solutions is ever-expanding. These are generally classified into two general classes depending on their effects on structural seismic response:

(a) Global interventions – objective in implementing interventions that belong in this category is to increase stiffness and strength – thereby moderating the anticipated damage by achieving a reduction in the effective structural period (and therefore the lateral displacement demands) and improving the distribution of these demands in the individual structural components[2,3]. From the nomenclature it is evident that a global intervention may extend over several floors in a building. An example of a global intervention is, for example, the addition of conventional reinforced concrete jackets on columns of a structural system; to avoid relocation of damage in adjacent floors to the one retrofitted, which may become locations of stiffness discontinuity and therefore points of potential damage relocation, this particular type of intervention must be continued by piercing through the floor slab (Fig. 2(a)).

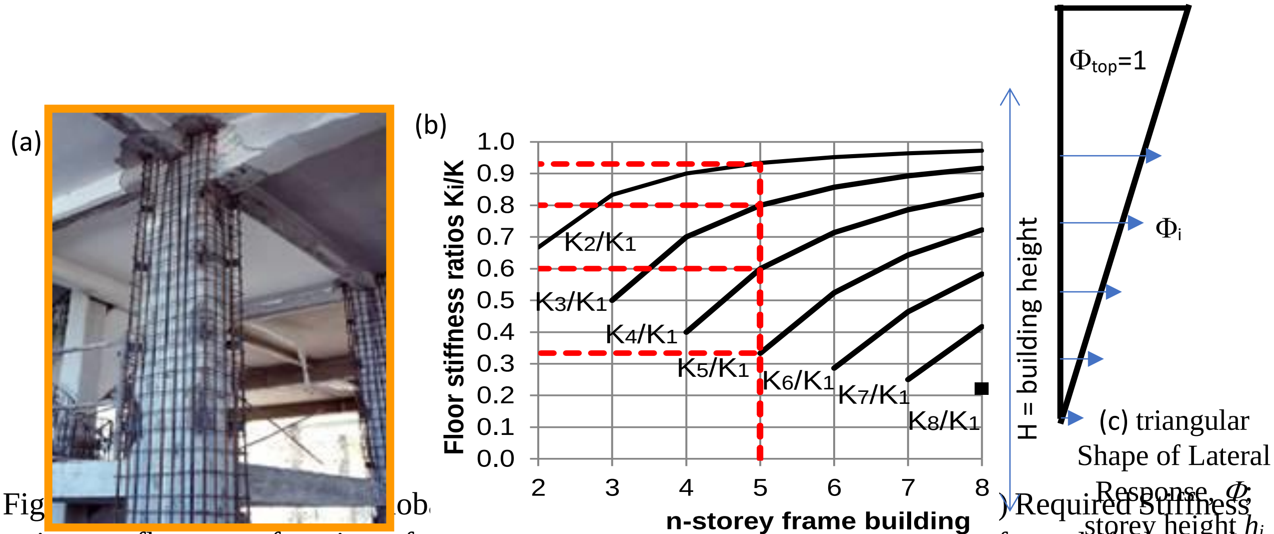


Fig... ob...) Required Stiffness ratios per floor as a function of the stiffness of the first floor to achieve uniform drift demands throughout the height (c). Graph plots requirements for frame buildings with 2- up to 8-storeys. Red dashed lines give the required values for a 5 storey building.

(b) Local interventions – objective in implementing those is to mitigate premature failures so as to enable the development of flexural yielding (i.e., development of the strength imparted by longitudinal reinforcement yielding); and to also enhance the deformation capacity of components so as to preclude premature failures that could occur in the compression zone of critical regions, either by crushing / delamination of concrete cover and/or by buckling of compression longitudinal reinforcement[4]. A local intervention is not meant to influence the demand side of the design inequality – but only the supply side, and specifically to enhance the member capacity in terms of deformation and ductility. It occurs in a critical region of a structural member – e.g. over the plastic hinge length and as a rule, it cannot contain addition of any form of longitudinal reinforcement – since this would add stiffness and strength and would therefore affect the structural demands.

In general, retrofitting of an existing structure does not necessarily require both types of intervention; it is up to the engineer to determine what type of intervention is needed during an initial stage of assessment. Nowadays, assessment has become a complex computational exercise conducted in advanced modeling software. However, it is often observed that the essence of the assessment exercise is subordinate to, or lost, in the complexity of modeling. Unfortunately, the complexity of modeling does not necessarily secure that the actual performance of the building is captured in a future seismic event. This is pilled under "epistemic uncertainty" in the field of reliability[5], but it is the duty of the engineering community to enhance the reliability of modeling by properly accounting for all the mechanisms of failure in the model that might eventually control the hierarchy of failure. For example, it is pointless to worry about considering complex fiber models in analyzing the moment-curvature behavior of a component if the model is blind to other forms of failure that may intercept the optimal flexural response of a laterally swaying column, and which can occur along the column line thereby limit the input force to the flexural mechanism. Such failure modes are, column web shear failure, bar anchorage failure, lap-splice failure, joint failure, or punching failure of the slab above[6].

Diagnosing the Need for Global Interventions in the Retrofit

Determining whether a global intervention is required should be the first step in the process of rehabilitation. There are a few diagnostic tools to that end, as listed below:

(a) The fundamental period of the structure: based on a calibration study that has been conducted as a background to current codes, new structures that comply with the provisions have a period that is close to the estimate $0.075H^{3/4}$, (or $0.05H^{0.9}$) where H is the height of the building above ground[7,8]. These values may be used as a benchmark for assessment of existing structures. The period may be estimated computationally after considering a 50% reduction in member stiffness from the elastic value, or measured in the field with pertinent instruments when the building is excited by ambient vibration; if the latter approach is used, the value ought to be increased after multiplication by $\sqrt{2} \approx 1.5$, to account for crack opening under the overturning moments that develop during seismic vibration[9]. A period value that is notably larger than the benchmark may be used to signal the need for a global intervention with the objective to add stiffness and effectively reduce the structural period – this would, in turn, reduce the displacement demand since the latter increases at a quadratic rate with the fundamental period according with: ($S_d(T) = S_a(T) \cdot (T/2\pi)^2$, [3].

(b) The axial load ratio of the columns when considering the G+0.3Q combination: Consider that the balance point occurs at an axial load ratio of 0.4 (compression positive). Therefore, for further increase of the axial load ratio, which would typically affect the exterior columns of the building as a result of the overturning action of the earthquake, the flexural capacity of the column is reduced from its peak value. To minimize this likelihood, axial load ratios resulting from gravity loads and the service load components of the Earthquake Load Pattern combination should be reduced below the 0.4 benchmark, by the addition of cross-sectional area (e.g. jackets).

(c) The most comprehensive indicator of the vulnerability of a structural system is its fundamental mode of lateral vibration – because its shape reflects the presence of irregularities in plan and in height, as well as the tendency for localization of the deformation demands: the mode shape is normalized to have a unit value at the control node, or point of reference. In the shape function, $\Phi(x)$, locations where $d\Phi(x)/dx$ is maximum are the regions where damage is anticipated to occur. (Note that several researchers use the interstorey drift ratio, $\Delta\Phi_i/h_i$ to identify the tendency for localization. This is only accurate if the building is of "shear-type" – i.e. has relatively stiff beams, whereas all the deformation occurs in the columns; in the general case, the tangential interstorey drift ought to be used to identify the tendency for "localization", i.e., the concentration of high demands in a small critical region. The tangential interstorey drift is obtained from $\Delta\Phi_i/h_i$ after subtracting the end joint rotations, which are owing to beam deformation)[3].

Lateral stiffness of a building structural system is defined by:

$$K = \sum_{i=1,N} K_i \cdot \Delta\Phi_i^2 = \left[\sum_{j=1,n_c} K_{i,j}^{c} + \sum_{k=1,n_{wc}} K_{i,k}^{w,c} + \sum_{p=1,n_{wm}} K_{i,p}^{w,m}\right] \Delta\Phi_i^2 \quad (1)$$

Where K_i is the i-th storey stiffness and $\Delta\Phi_i$ is the change in the mode shape coordinate that occurs within the floor in question. Summation occurs over $i=1,N$, where N is the total number of storeys; summations in the brackets represent the total floor stiffness; the first term represents contributions of the n_c columns at a given floor level, the second represents contributions of the $n_{w,c}$ walls, and the last term contributions of the $n_{w,m}$ masonry infills in the direction of the earthquake, at the same floor. A paradox of structural response is that the more compliant floors in a structural system control the total stiffness and therefore the period of the structure. The reason is that the term $\Delta\Phi_i$ is raised to the power of 2 in Eqn. (1) – therefore the floor with the larger drift dominates the final result[3].

Thus, the diagnostic criteria (a) and (c) can also be used to guide the retrofit is global interventions are deemed necessary: the optimal response is achieved if the tendency for localization of demand is mitigated – and this is best achieved by modifying the response shape towards a target form that aims towards an improved distribution of tangential interstorey drift so that a larger number of structural components are engaged in the response rather than a few locations of intense damage.

The procedure has been described in [3]: The designer may target to an improved (lower) value of period, as close as possible to the benchmark value for the given building height; and to an improved lateral mode of response to eliminate soft storeys. For a linear target shape – which is , the effective mass is calculated from the sum

$$M = \sum_{i=1}^{N} M_i \cdot \Phi_i^2 \quad (2)$$

Where, M_i are storey masses and Φ_i the coordinates of the target shape in each floor (for equal floor heights, $\Phi_i = i/N$. With the selected target period, T_{tar}, the effective stiffness is estimated as,

$$K = 4\pi^2 M / T_{tar}^2 \quad (3)$$

From this value, and for equal storey heights it is possible to determine the required storey stiffness for the retrofitted structure using the chart depicted in Fig. 2(b). If the available storey stiffness is lower than this reference value it can be increased using a number of alternative solutions as described in [2, 10].

Diagnosing the Need for Local Interventions in the Retrofit

The driving objective of a local retrofit is to mitigate all brittle failure modes so that the longitudinal reinforcement in the critical sections may be able to develop yielding; this is required in order for a flexural mechanism to dominate the individual member response. To diagnose the likelihood of premature brittle failures, a static model has been used[6]. The static model is used in order to relate the various failure modes that may occur along a column line using comparable measures, namely the shear force along a column that may induce failure in the following modes:

(a) Flexural failure: $V_{flex} = M_y/L_s$ – where M_y is the yield moment and L_s is the shear span of the column (i.e., half the column height)

(b) Web shear strength, V_{sh}

(c) Shear force required to develop the flexural moment at the failure of bar lap-splices or bar anchorages in the critical sections, $V_{lap/anch}$

(d) Shear force required to cause punching failure in the slabs (in flat slabs) or shear failure in the joints (in frames)

Local interventions are therefore needed only if the following equation is not satisfied (triggering the occurrence of brittle failure). Additionally, local interventions are needed to enhance the deformation capacity of concrete in compression (to mitigate cover delamination and compression reinforcement buckling) and the drift capacity of the component so as to meet the estimated ductility demands.

New Materials in Global and Local Interventions

In the context of the above framework, there are several materials and techniques available for synthesizing local interventions. These range from FRP jackets to Ductile jackets comprising Tension hardening cementitious materials such as ECC (Engineered Cementitious Composites) and UHPCs (Ultra High-Performance Concrete). FRPs comprise mostly organic fibers impregnated in high-strength epoxies, placed in a manner that mobilizes the strength of the fibers in passive confinement of the encased regions[4], [11]. Tension-hardening materials function in a similar manner – the jacket is mobilized in tension in the transverse direction, passively confining the encased structural component. Between these two extremes, several variations occur in practice, such as TRM (Textile Reinforced Mortars[12]), SRP (Steel Reinforced Polymers, and SRG (Steel Reinforced Grouts[13]) where steel is in the form of thin, high-strength strands. The principle of operation of all these options is the same, i.e., in the form of passive confinement, where the tensile strength of the material is developed in a direction that is orthogonal to the longitudinal axis of the member, mobilizing confinement in the encased component.

Of the many alternatives, the use of epoxies for the matrix of the jacket has been criticized for its performance in fire. Several laboratory studies indicate that FRP jackets are also effective as a means of delaying corrosion of the embedded reinforcement of the structural member by starving the mechanism of corrosion from oxygen because they are impenetrable after hardening[14]. Again, here there are dissenting views about the possibility of the jacket hiding underneath the occurrence of continued anaerobic corrosion since the transport of moisture cannot be prevented. The above reasons, as well as the susceptibility of local fiber rupture at corners or under pressure from bending reinforcement, have been a reason for the search of other alternatives. For example, replacement of the epoxy matrix with mortar has become a popular solution with several field applications, hailed for its greater resilience to fire, while mitigating the concerns regarding anaerobic, hidden corrosion. From the seismic perspective, both TRMs and SRMs have been shown to have similar, albeit somewhat inferior effectiveness as confining devices[12], [13].

UHPCs are the most recent addition to the collection of materials that are ideal for local interventions. Tests conducted on columns with and without previously corroded reinforcement and having endured earlier seismic damage have illustrated that thin jackets comprising UHPC materials with significant post-cracking tensile strain capacity can not only mitigate the deprecating effects of corrosion on strength and deformation capacity but may also enhance these performance indicators substantially([15], [16], [17], [18], [19]).

El-Joukhadar and Pantazopoulou[20] conducted accelerated corrosion tests on steel bars embedded in UHPC and ECC covers of different thickness (cover = $1D_b$, and $2D_b$) and after pre-cracking the cover to specified crack widths ($w_{cr} = 0, 0.5mm$ and $2mm$). The two material types had the same matrix but different fiber types: in the first case steel fibers (13mm length, 0.2 mm diameter) at a volumetric ratio of 2% was used, with the material attaining an average compressive strength of 150 MPa. In the second case, 12 mm long, 0.1mm diameter PVA fibers were used at the same volumetric ratio, leading to an average compressive strength of 60 MPa. Corrosion mitigation characteristics of UHPFRC and ECC were compared to that of normal concrete and SHCC from a different study, and it was found that both UHPC and ECC offered orders of magnitude more protection than both normal concrete and SHCC. Also, the UHPC cover was significantly better in terms of slowing down the rate of corrosion in the cracked specimens as compared to ECC – the difference being attributed to the hydrophilic nature of the PVA fibers. The significant reduction in the corrosion rate effected by UHPC is attributed to its very dense matrix: its porosity is about half of what is encountered in normal concrete, and the ratio of capillary porosity to gel porosity is exactly reverse between the two material types (capillary pores account for 2/3 of the total porosity in normal concrete, but 1/3 in the case of UHPC). The absence of connected pores mitigates the transport of moisture, which is an essential requirement for sustaining anaerobic corrosion. In light of this attractive attribute, retrofitting corroded components with UHPC jacketing offer greatly improved durability of the retrofit scheme, beyond its mechanical performance advantages([21], [22], [23], [24]).

The confining pressure exerted by the jacket on the encased component may be calculated from conventional mechanics[25] (Fig. 3);

$$\sigma_{lat} = \frac{2 \cdot f_t \cdot t_j}{D_{core}} \text{ for a circular cross section, \& } \sigma_{lat} = \frac{2 \cdot \alpha_{eff} \cdot f_t \cdot t_j}{D_{core}} \text{ for a rectangular cross-section} \quad (4)$$

Where, f_t is the tensile strength of the UHPC material (in the context of the present work, this could refer to either cracking, $f_{t,cr}$ or to the peak value, $f_{t,u}$), and t_j is the jacket thickness. The effectiveness of the confinement, α_{eff} , for rectangular cross sections, has been shown to take a similar form to what is proposed for other external jackets (e.g. as in the case of FRPs[17]):

$$\alpha_{eff} = 1 - \frac{(b-2R)^2 + (h-2R)^2}{3bh(1-\rho_g)} \quad (5)$$

Whereas the confinement models used to estimate the strength and deformation capacity of the encased concrete are valid in the case of UHPC jacketing as well[25]:

$$f_{cc'} = f_{c'} + \lambda \cdot \alpha_{eff} \cdot \sigma_{lat}; \qquad \varepsilon'_{cc,o} = \varepsilon'_{co} \cdot 5 \cdot \left[\frac{f'_{cc}}{f'_c} - 0.8\right] \quad (6)$$

To assess the adequacy of the local intervention, the contribution of the confining pressure to the strength terms (a), (b), (c), and (d) discussed in the preceding section are derived from basic mechanics models using the same approach as in the case of FRP jacketing (see for example [3], [26]).

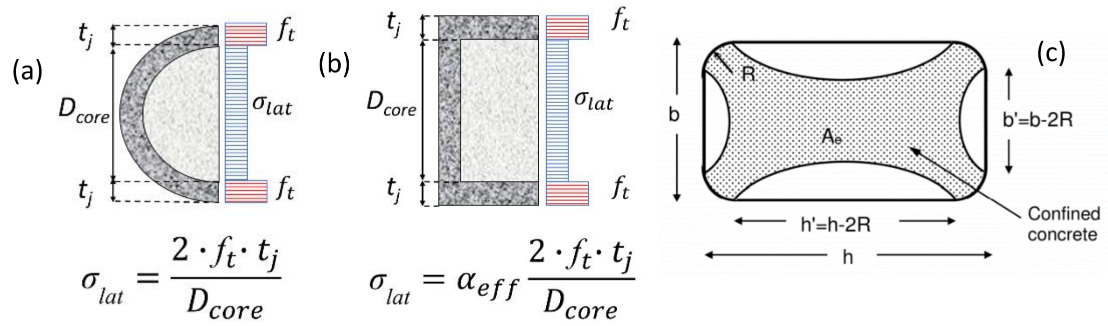


Figure 3: Definition of the confining pressure exerted by the UHPC jackets on encased concrete: (a) Circular encased cross section, (b) Rectangular encased cross section, (c) effectively confined region and definition of terms.

Jackets comprising UHPC material are usually thin, ranging between 25 to 50 mm; a technique that has emerged as very effective is that of cover replacement where the UHPC material is placed to the depth of the longitudinal reinforcement after removal of the conventional concrete cover[27]. The tensile strength that they develop depends strongly on the percentage and orientation of the fibers; to exhibit tension hardening characteristics, it is estimated that the density of the fibers must be at least 50/cm^2 of material cross-section, whereas the fiber length must be adequate to develop a significant fraction of the fiber axial tensile strength through bond. (Commonly used are 13 mm long, 0.2 mm diameter brass-coated, high strength steel fibers). To be classified as UHPC([21], [24]) the compressive strength of the material, $f_{c,U}$ must exceed 120 MPa, the cracking tensile strength, $f_{t,cr}$, is at least 5 MPa, and is approximated by $0.6\sqrt{f_{c,U}}$; and the tensile strain capacity at the onset of tension localization failure should exceed 0.002. Finally, the elastic modulus is in the range of *45 GPa* or more, approximated by $4070\sqrt{f_{c,U}}$.

Calculation examples have illustrated that the significant compressive and tensile forces that develop in the jacket thickness at the compression and tension faces of a strengthened element form a couple that may enhance the flexural strength of the component by as much as 30% from its nominal value (for a rectangular cross section, the additional flexural strength provided by the jacket is estimated as: $\Delta M = 0.75 \cdot f_{t,cr} \cdot t_j \cdot (b+h-2 \cdot t_j) \cdot (h-t_j)$, where, the 0.75 coefficient is the material safety factor and h is the cross sectional height). In the available tests it has been observed that the web shear strength increase is substantial as to mitigate the likelihood of shear failure even after this flexural strength increase. The following expression is used for calculating the enhancement of shear strength contributed by the jacket([25], [27]):

$$\boldsymbol{V_j = 2t_j \cdot f_{td} \cdot (h - t_j) / \tan\theta} \qquad (7)$$

where θ is the angle of diagonal cracks with respect to the longitudinal axis, and f_{td} is the design tensile strength of the material, which is taken equal to $0.75 \cdot f_t$ (i.e., the material safety factor for UHPC in tension is taken equal to 0.75). It is noted that for $\theta=45^o$, the above reduces to the total area of the jacket in the sides of the web, multiplied by the design value of the tensile strength of the jacket material.

Most critical is the issue of bond in lap splices of reinforcement within the jacketed regions, as well as the anchorages inside footings: It is noted that bond strength, f_b, is in the order of $1.3\sqrt{f_{c,U}}$, which, for high strength materials such as UHPC lead to values exceeding 15 MPa for f_b([28], [29]). With this bond strength, it is possible to develop a Grade 60 bar over an anchorage length of $8D_b$ or less, whereas pullout slip is insignificant until advanced levels of bar strain. The implications of a very high bond strength developed by bars embedded in UHPC is that strain penetration inside the member length is limited, increasing the risk of bar fracture at the critical section. If the anchorages beyond the limit of the jacket are not confined and comprise the original concrete where bond strength is significantly lower, then the demand for strain development at the critical section of the structural component will lead to significant strain penetration inside the footing, thereby becoming the source of excessive reinforcement pullout[18].

In their work for development of a performance-based framework for the practical design of UHPC jackets, the following performance limit states for the application of the jackets in bridge piers have been proposed, to be consistent with CSA-S6 (2019)[25]:

- Minimal Damage Requirements: Compression strain in UHPC ≤ 0.004; Strain in tension reinforcement ≤ 0.008, Strain in Compression Reinforcement $\leq \varepsilon_y$.
- Repairable Damage : Strain in Tension Reinforcement ≤ 0.015
- Life Safety (Near Collapse): No compression crushing of the encased component, and strain in tension steel ≤ 0.06.

Summary and Conclusions:

The main points of the work presented here are summarized as follows:

- I. Global addition of Stiffness & strength (longitudinal reinforcement such as FRP; enlargement of member sections) at selected critical locations needed to reduce the period T and eliminate tendencies for localization of deformation demands.
- II. Local Interventions rely primarily on confinement to increase the deformation capacity of members and eliminate brittle failure modes.
- FRP repair can be durable and even delay corrosion if internal moisture is eliminated; otherwise routes for moisture convection need to be secured (wraps provided in strips + cathodic protection).
- Several combinations of FRP or steel wires with mortar (e.g. TRMs, SRMs) combine the benefits of the FRPs with better fire resiliency.
- UHPC is a very durable material with superb compressive strength and tension hardening after cracking. As cover to reinforcement, it slows down markedly the corrosion rate.
- UHPCs delay the stage of localized crack formation.
- Ideal for jacketing applications, it eliminates brittle failures, enhances deformation capacity, and increases mildly the strength and stiffness. It is a Local intervention but with moderate global effects.
- Material with superb durability and strength in compression and tension; high elastic modulus, and tensile strain capacity without loss of strength after the onset of cracking.

- Recovery of strength, stiffness, and ductility of corroded elements.
- Open design issue: Develop a framework of design guidelines to support its introduction in construction.
- Performance Limit States and safety factors have been extracted based on collective evaluation of test results from UHPC Tests and RC component UHPC jacketing performance.

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Short Bio:

Dr. Voula (S. J.) Pantazopoulou, ACI Fellow, is Professor and former Chair of Civil Engineering, Lassonde Faculty, York University. She holds an Undergraduate Degree in Civil Engineering from the National Technical University of Athens, Greece, and MSc and PhD Degrees from the University of California at Berkeley. She specializes in Reinforced Concrete Structures, Performance-based Earthquake Engineering, Structural Retrofit, and novel structural materials (such as CNTs in concrete, FRPs, UHPC, and structural biomaterials). Recent work deals with the development of sustainable geopolymer concrete, forensics of bridges damaged by Alkali-Aggregate Reactivity, Strain-hardening fiber-reinforced cementitious materials, whereas a significant part of her past activity was dedicated to Seismic Assessment and Retrofit of Reinforced Concrete Buildings and Bridges, evaluation and repair of corrosion-induced deterioration of structures, and the use of emerging materials in structural retrofit.

She has served as faculty in several Universities having obtained an international expertise through training and the various posts she has taken up during her academic career. She began her academic career at the University of Toronto, as Assistant Professor in 1988, and was promoted to Associate Professor in 1992; in 1997 she was appointed Associate Professor at Democritus University of Thrace, in Greece, where she was promoted to Full Professor in 2001; she took the position on Professor in the Department of Civil and Environmental Engineering at the University of Cyprus in 2011, where she was appointed Department Chair in 2013-2015. Since 2016 she has returned to Canada as Professor in the Lassonde School of Engineering, York University, in Toronto. She has 35 years of experience in funded research and has supervised 19 PhD students (4 ongoing) and over 50 Master of Science (Thesis) students. The research thus conducted has yielded more than 120 Journal papers with over 5500 citations to her work (she belongs within the top 1.2% of most highly cited researchers in the field). She has co-organized several International Conference Sessions on Seismic Assessment and Retrofit of RC and Masonry Structures and has co-edited a dedicated volume by Springer on Seismic Assessment of Heritage Unreinforced Masonry Constructions. She serves the profession through participation as a member in several Technical Committees by the ACI, ASCE, FIB, and CSCE, as well as a panel member of several National and International Panels reviewing proposals and academic university programs. She has also served in the Committee developing KADET on behalf of OASP in Greece and is currently member of the Canadian Standards' Association Bridge Design Committee Specializing in the use of Ultra High Performance Fiber Reinforced Concrete in Bridge Construction. She is also Member of the Board of Directors of the Canadian Society for Earthquake Engineering and the International Association of Earthquake Engineering.

Dr. Pantazopoulou has been selected as a Fellow of the Engineering Institute of Canada for 2019, is a Fellow of the American Concrete Institute, and has received a meritorious award from ASCE (Moisseiff award for "notable contributions to the science and art of Civil Engineering").