Modeling Damage-Based Degradations in Stiffness and Strength in the Post-Peak Behaviour in Seismic Progressive Collapse of Reinforced Concrete Structures

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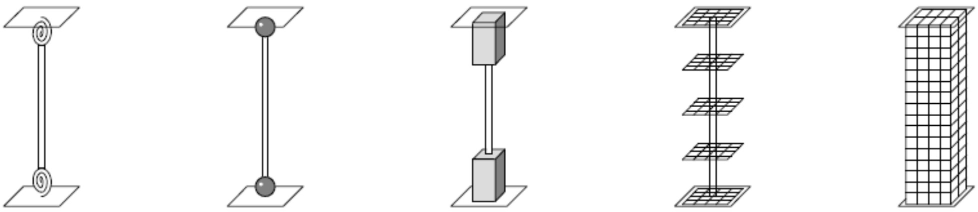
SUMMARY:
This paper presents a comprehensive inelastic damage-based degradation model for earthquake response analysis of reinforced concrete structures subjected to multi-component seismic loading. The proposed model is capable of capturing all the important effects of axial-flexure-shear interactions on the inelastic behaviour of reinforced concrete members and the degradations in stiffness and strength in the post-peak response of structural components in seismic progressive collapse of structures by considering the effects of progression and accumulation of damage in inelastic actions. The formulation of the beam-column element model is based on the stress-resultant concentrated plasticity approach and incorporates damage models that capture hysteretic damage accumulated during inelastic excursions in repeated reversed cycles. The developed model is an important prediction tool of realistic seismic progressive collapse behaviour of reinforced concrete structures.

Keywords: reinforced concrete structures, numerical models, damage-based stiffness and strength degradation

1. INTRODUCTION

Realistic prediction of seismic progressive collapse behaviour is essential in vulnerability and performance assessment of reinforced concrete structures. In order to be able to more realistically assess the seismic behaviour and performance of structural systems, accurate and computationally efficient numerical models for nonlinear seismic response analysis that capture the behaviour of members designed as ductile as well as non-ductile members are needed. Discrete finite elements, schematically shown in Figure 1.1(a – d), are commonly used to model beam-column members in large structures since they allow more insight into the seismic response of structural members as well as the entire structure compared to global models. They are computationally more efficient compared to the microscopic finite element model, shown in Figure 1.1(e). Discrete finite elements, also referred to as frame elements, can be based on concentrated plasticity and distributed plasticity formulation, shown in Figure 1.1(a – b) and in Figure (c – d), respectively. Frame elements based on the concentrated- and distributed-plasticity approach have been proposed in a number of earlier studies. The section behaviour of a reinforced concrete member is described by stress resultant plasticity-based models in some of the proposed frame element models (Takizawa and Aoyama 1976, Chen and Powell 1982, El Tawil and Deierlein 1998), and by fiber models in others (Mari and Scordelis 1984, Lai et al. 1984, Zeris and Mahin 1991, Spacone et al. 1996). The focus in earlier research is mainly on modelling ductile response of reinforced concrete members governed by flexural yielding.

However, reinforced concrete columns in existing older structures, which were typically not designed following the ductile approach, may not have the lateral strength or displacement ductility to develop flexural yielding and withstand the strength and ductility demands imposed during severe ground shaking. As observed in major earthquakes and demonstrated in numerous experimental studies on component cyclic behaviour (Ghee et al. 1989, Saatcioglu and Ozcebe 1989, Watanabe and Ichinose 1992, Wong et al. 1993, Priestley et al. 1994, Sezen 2002, Elwood and Moehle 2003), reinforced concrete columns with inadequate detailing, such as insufficient or poorly detailed transverse reinforcement and inadequate length development, may exhibit brittle shear failure or ductile shear failure in the flexural plastic hinge zone under combined axial-flexure-shear effects. It has also been shown that shear-critical reinforced concrete columns may suffer severe degradations and exhibit significant pinching behaviour as more damage is accumulated. To quantify the seismic damage to individual structural components or the global structure accumulated during inelastic excursions in repeated reversed cycles, several damage indices have been proposed in previous research, including Park and Ang (1985), Kratzig et al. (1989), Kunnath et al. (1992), Mehanny and Deierlein (2001).



(a) (b) (c) (d) (e)

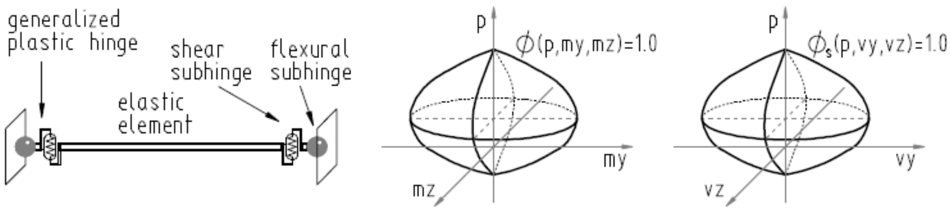
Figure 1.1. Idealized Models of Beam-Column Elements: (a) Zero-Length Nonlinear Spring Hinge Model; (b) Concentrated Generalized Plastic Hinge Model; (c) Finite Length Hinge Model; (d) Fiber Section Formulation Model; (e) Microscopic Finite Element Model (adapted from Deierlein et al. 2010)

Recent efforts have been undertaken in the development of numerical models for capturing the nonlinear response of reinforced concrete members susceptible to nonductile and ductile shear failure. Concentrated plasticity models considering shear effects based on single-component hinge approach have been proposed by Pincheira et al. (1999), Lee and Elnashai (2002), Elwood and Moehle (2003), Sezen and Chowdhury (2009), Xu and Zhang (2011). In most single-component hinge models, the influence of the axial force-moment interaction on the post-peak behaviour of reinforced concrete members is not considered, resulting in important limitations since the effects of variable axial load on the flexural and shear capacity have been shown to have significant impact on the degradations of stiffness and strength. In addition, single-component hinge models are mainly suitable for modelling the planar behaviour of reinforced concrete components since interaction effects due to bidirectional loading are ignored. To overcome some of the limitations in single-component hinge models, recent studies have employed yield-surface and evolution models approach to account for the force interaction in the case of multiaxial loading (Ricles et al. 1998, Abou-Elfath et al. 1998, ElMandooh Galal and Ghobarah 2003, Kaul 2004). Review of concentrated plasticity beam-column models incorporating degradation models for capturing the effects of shear failure and post-shear failure behaviour of reinforced concrete components is presented in Reshotkina and Lau (2011). Distributed plasticity models incorporating flexural and shear behaviours based on the fiber approach, in which the axial load-moment interaction is readily taken into account, have been proposed by Petrangeli et al. (1999), Shirai et al. (2001), Marini and Spacone (2006), Mazars et al. (2006), Martinelli (2008), Ceresa et al. (2009), Mullapudi and Ayoub (2010), among others. Distributed plasticity fiber models are computationally demanding and require complex constitutive models for concrete that capture the post-peak degradation behaviour, which are not readily available. On the other hand, stress-resultant concentrated plasticity models are computationally more efficient than the distributed plasticity fiber models, and also more practical for modelling complex phenomena like post-shear failure cyclic response using hysteretic models that require fewer parameters to capture degradation behaviours under cyclic loading and can be calibrated based on force-deformation relations available from previous experimental research on cyclic behaviour of reinforced concrete components. However, the current frame elements incorporating stiffness and strength degradation models are mostly suitable for analysis of 2D plane behaviour of reinforced concrete components or do not capture the full axial-flexure- shear interaction effects and the effects of accumulated damage on the 3D behaviour.

As it can be observed from the review presented herein, there is a need for further developments of comprehensive and computationally efficient models that capture accurately the three-dimensional behaviour of reinforced concrete structural components in seismic response analysis of structures. This is particularly important for design of structures with high earthquake resilience and in devising effective retrofit and repair schemes for old deficient and earthquake damaged structures. This research is focused on the development of a comprehensive model for seismic response analysis of reinforced concrete structures subjected to multi-component seismic loading that captures the behaviour of members in new structures designed as ductile as well as non-ductile members in existing older deficient structures. The proposed formulation is based on the stress-resultant concentrated plasticity approach and considers the full axial-flexure-shear interaction effects on the inelastic behaviour as well as biaxial interaction effects on the degradation of stiffness and strength by incorporating damage models capturing hysteretic damage in multi-directions. The proposed model described herein is an important tool for accurate capturing of the degradation behaviours of reinforced concrete components due to the effects of accumulated damage and the damage characteristics of global structures from initiation and progression of failure until ultimate collapse during major earthquakes. The proposed analysis model is thus suitable for assessing structures against the full range of performance objectives, including collapse prevention.

2. BEAM-COLUMN ELEMENT FORMULATION

The formulation consists of a beam-column element with zero-length generalized plastic hinges concentrated at the ends of the element, as shown in Figure 2.1(a). The element between the hinges is elastic. Each hinge has three deformations, axial deformation and rotations about the local element axes. The contributions from shear deformations in the hinge are considered in the total deformations. Each hinge is modelled as two subhinges in series, one for the flexural behaviour and one for the shear behaviour. Yield surface is defined to model the interaction between axial force and biaxial bending moments, as shown in Figure 2.1(b). Shear failure is captured by shear failure surface defined in terms of axial force and shear forces, as shown in Figure 2.1(c). Post-yield inelastic hardening behaviour and post-shear failure softening behaviour are captured by different evolution models defined for the yield surface and the shear failure surface, respectively, as described in details in the following section.



(a) (b) (c)

Figure 2.1. Schematics of the New Beam-Column Element: (a) Concentrated Plasticity Model; (b) Yield Surface Model; (c) Shear Failure Surface Model

2.1. Inelastic Element Formulation

2.1.1. Yield surface formulation

Yielding occurs when the force state of a member reaches the yield surface and the force state remains on the yield surface during continued inelastic loading. The yield surface is assumed to be a continuous, convex, rate independent function of axial force and bending moments on a cross section of the member, and can be represented as follows:

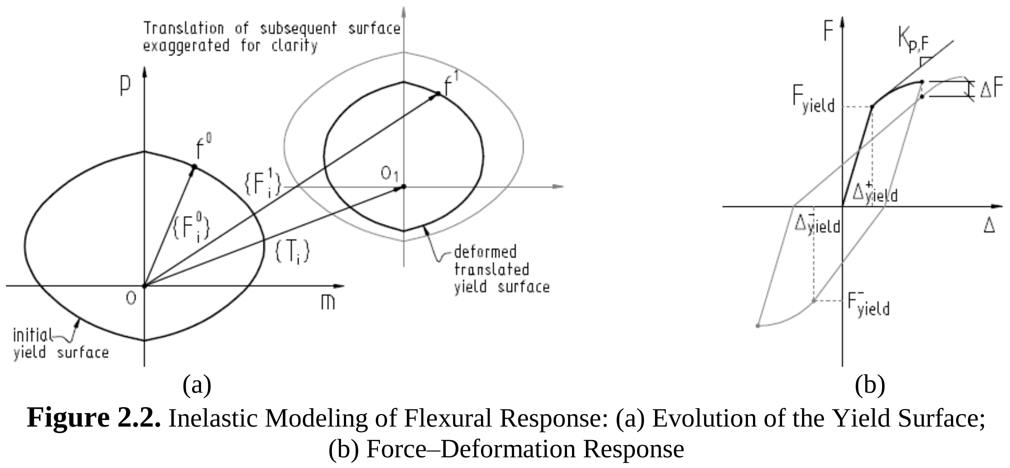
$$\Phi\left(p, m_y, m_z\right) = 1 \tag{2.1}$$

where p, m_y and m_z are normalized axial force and bending moments about y and z axis, respectively. For reinforced concrete sections, the axial force and moments can be normalized by the axial load and the moments at the balanced failure point, where the concrete starts to crush in compression as the outermost steel reinforcement bars begin to yield in tension. The equations describing the force interaction surface can be based on yield surface functions proposed in previous research, such as Tseng and Penzien (1973), El-Tawil (1996), among others.

2.1.2. Yield surface evolution models

An evolution rule determines how the yield surface evolves in the force space to reflect the change in structural behaviour upon undergoing inelastic deformations of the member, i.e. changes its position, shape and size, which results in hardening/softening in the force-deformation response. In the proposed formulation, a combination of kinematic hardening rule and non-uniform contraction rule is considered. The kinematic rule results in translation of the yield surface, while the contraction rule results in non-uniform shrinking of the yield surface, as shown in Figure 2.2(a). The evolution model developed by Kaul (2004) for yield surfaces in terms of axial force and bending moment for modelling in-plane behaviour is modified and extended in this research to yield surfaces defined in terms of axial force and biaxial bending moments to model 3D behavior by considering the effects of accumulated damage in inelastic excursions in one direction on the behaviour in other out-of-plane directions.

The kinematic rule in the proposed evolution model is used to model the hardening response in flexure within the same cycle of inelastic excursion. The magnitude of hardening is governed by the plastic stiffness, $[K_p]$, a 3x3 diagonal matrix which contains terms for the axial force, $(K_{p,P})$, and for the bending moments, (K_{p,M_y}) and (K_{p,M_z}). In the proposed formulation, the plastic stiffness terms are gradually reduced from high initial values to zero as the member accumulates damage in large inelastic excursions and repeated load reversals. This results in the gradual reduction of the amount of hardening in the force-deformation response, as shown in Figure 2.2(b), and thus allows capturing the post-yield behaviour of reinforced concrete members affected by concrete crushing and bar buckling more realistically. The direction of surface translation is specified by the evolution direction. Summary of the main kinematic rules for surface translation can be found in El-Tawil (1996), Chen and Han (2007). In the proposed model, a normal, centroidal or constant-P direction of evolution can be used. In the case of normal direction of evolution, motion of the yield surface is in direction parallel to the normal vector of the current force state; in centroidal evolution, the surface motion is directed along a unit vector connecting the surface centre to the current force state; and in constant-P evolution, the surface translates in a plane parallel to the plane defined by the moment axes. The non-uniform contraction rule is formulated to model the strength degradation under repeated cyclic load reversals. At the end of each half cycle, i.e. at zero force or when the force switches direction, the yield surface is contracted using damage indices based on plastic deformations and hysteretic energy, such as those proposed by Park and Ang (1985), Kratzig et al. (1989) and Kunnath et al. (1992). As shown in Figure 2.2(b), this results in loss of strength, ΔF, in each subsequent loading cycle even when the same level of inelastic displacement is maintained.



(a) (b)

Figure 2.2. Inelastic Modeling of Flexural Response: (a) Evolution of the Yield Surface; (b) Force–Deformation Response

The formulation of the proposed evolution model is based on maintaining two states – the initial state of the yield surface and the current translated deformed state of the subsequent surface, as shown in Figure 2.2(a). Updating the state of evolution is achieved by using mapping between the state of the initial yield surface, represented by superscript 0, and the current state, represented by superscript 1. The surfaces shown in Figure 2.2(a) are drawn in $(p - m)$ plane for the purpose of clarity, where m is the normalized resultant moment, however, the formulations are valid for 3D case. A translation vector, $\{T\}$, of dimension 3x1, accounts for the motion of the yield surface in force space. A contraction factor matrix, $[DI]$, of dimension 3x3, represents the contraction of the yield surface along each force axis. The matrix $[DI]$ is equal to the identity matrix for the initial state of the yield surface or in case of kinematic hardening only. At any trial step (i) a force state on the subsequent yield surface, $\{F_i^1\}$, can be mapped to a corresponding force state on the initial yield surface, $\{F_i^0\}$, so that iterations can be performed using the original yield surface functions. The force vector $\{F_i^1\}$ can be expressed as follows

$$\{F_i^1\} = [DI_i]\{F_i^0\} + \{T_i\} \tag{2.2}$$

The contraction factor matrix at step (i), $[DI_i]$, is a diagonal matrix and its terms for axial load and bending moments, $(DI_{p,i})$, $(DI_{m_y,i})$, $(DI_{m_z,i})$, respectively, are based on damage indices. The advantage of using the mapping technique is that the force vector corresponding to the current state is updated depending on the translation and contraction of the subsequent yield surface, while the equations and parameters used for deriving the yield surface function remain unchanged during the evolution. Thus, the extensibility of the formulations is facilitated for defining and using different yield surface functions in the inelastic modelling.

The element tangent stiffness matrix, $[K_t]$, is obtained by modifying the element elastic stiffness matrix, $[K_e]$, using plastic reduction matrix, representing the change in stiffness resulting from inelastic behaviour of the element. It can be shown that for the hardening model the plastic reduction matrix, $[K_r]$, can be expressed by

$$[K_r] = [K_e][G]\left[[G]^T[K_e + K_p][G]\right]^{-1}[G]^T[K_e] \tag{2.3}$$

where $[G]$ is the yield surface gradient at the location of the force state.
The incremental force can be calculated by

$$\{dF\} = [K_t]\{d\Delta\} = \left[[K_e] - [K_r]\right]\{d\Delta\} \tag{2.4}$$

2.1.3. Shear failure surface formulation

Shear failure is detected when the shear force state of a member reaches the shear failure surface. Upon increasing the inelastic deformations, the shear force state remains on the subsequent post-shear failure surface. Similarly to the yield surface, the shear failure surface is assumed to be a continuous, convex function of axial force and shear forces on a cross section of the member, and can be defined as follows

$$\Phi_s(p, v_y, v_z) = 1 \tag{2.5}$$

where p, v_y and v_z are normalized axial force and shear force capacity about y and z axis, respectively. For reinforced concrete sections, the axial force and shear force can be normalized by the axial load and the shear force capacity at the balanced failure point. The shape of the shear failure surface can be described using functions similar to the yield surface functions mentioned above. For symmetric reinforced concrete sections, ElMandooh Galal and Ghobarah (2003) proposed a simplified ellipsoidal shear strength-axial strength interaction diagram based on experimental test results by Vecchio and Collins (1986).

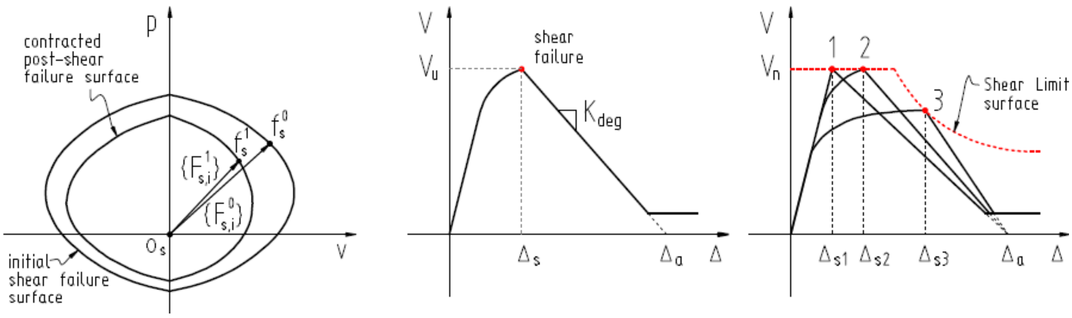
2.1.4. Shear failure surface evolution models

The post-shear failure response to further loading, i.e. increasing inelastic deformations, is characterized by softening or in-cycle strength degradation. To capture the softening behaviour of reinforced concrete members, an evolution model for the shear failure surface is proposed based on a non-uniform contraction rule. In this evolution model, the shear failure surface can shrink differently in each direction, e.g. positive and negative directions along the axial force axis and the shear force axes, which results in change in both size and shape of the surface, as shown in Figure 2.3(a). By using the non-uniform contraction rule in the proposed model, biaxial interaction effects on the evolution of the post-shear failure surface are accounted for by considering the damage accumulated in each direction. For example, damage due to excessive inelastic deformations and repeated cycles of loading in direction of y axis will affect the capacities of the member in direction of z axis, and vice versa. For this purpose, separate damage indices are calculated for the positive and negative directions of y and z axes, and interaction rules are defined, as discussed later.

The magnitude of evolution in the proposed model is determined as a function of the degradation stiffness, $[K_{deg}]$. As reported by Elwood and Moehle (2003), once the column fails in shear, a linear degradation stiffness can be used based on experimental observations that at the time of axial failure, the shear capacity of the column degrades to approximately zero (Nakamura and Yoshimura 2002). Thus, the slope of degradation, K_{deg}, is given by

$$K_{deg} = V_u/(\Delta_a - \Delta_s) \quad (2.6)$$

where V_u is the shear force in the member at shear failure, Δ_s is the corresponding drift at shear failure, and Δ_a is the drift at axial failure, as shown in Figure 2.3(b). The drifts Δ_s and Δ_a can be determined using the empirical drift capacity models proposed by Elwood and Moehle (2003).



(a) (b) (c)

Figure 2.3. Inelastic Modeling of Shear Response: (a) Evolution of the Shear Failure Surface; (b) Force–Deformation Response; (c) Shear Failure Modes

Similarly to the yield surface evolution model, the updating of the state of evolution of the shear failure surface is achieved by using mapping between the state of the initial surface, represented by superscript 0, and the current state, represented by superscript 1, as shown in Figure 2.3(a). Since translation is not used in this evolution model, mapping is governed only by a contraction factor matrix, $[DI_s]$, which is also of dimension 3x3 and represents the contraction of the shear failure surface along each force axis based on damage indices. The matrix $[DI_s]$ is equal to the identity matrix for the initial state of the shear failure surface. At any trial step (i), the force state on the subsequent post-shear failure surface, $\{F_{s,i}^1\}$, can be mapped to a corresponding force state on the initial yield surface, $\{F_{s,i}^0\}$, by using the following relationship:

$$\{F_{s,i}^1\} = [DI_{s,i}]\{F_{s,i}^0\} \quad (2.7)$$

The iterations can be performed using the equations and parameters of the original shear failure surface function, and thus the proposed evolution model allows extensibility of the formulations in defining and using different shear failure surface functions.

The effects of shear deformations are included in the element stiffness formulation by using the flexibility approach. The element flexibility equations are first derived, and then transformed into element stiffness equations by inversion and supplemental matrix operations using a procedure similar to that given in McGuire et al. (2000). Using this approach allows modelling the post-shear failure softening behaviour by modifying the shear components in the stiffness to achieve degradation.

Shear-critical behaviour of reinforced concrete members, including behaviour governed by brittle shear failure and behaviour governed by flexure-shear failure, is captured by the proposed model using force-based and displacement-ductility based criteria. In Figure 2.3(c), the force-deformation response of columns exhibiting behaviour dominated by brittle shear failure and limited ductile behaviour with shear failure are shown as case 1 and case 2, respectively. In the proposed model, shear failure in these cases is detected when the shear force state reaches the initial shear failure surface, i.e. shear demand exceeds the initial shear capacity. In the case where the column exhibits moderate ductile behaviour with shear failure resulting from degradation of the initial shear strength due to inelastic flexure-shear interaction, shown as case 3 in Figure 2.3(c), a displacement-ductility based criterion is used in the proposed model to detect shear failure as a function of the member drift. To capture the degradation of shear strength in the flexural plastic hinge zone, the initial shear failure surface is contracted under pronounced flexural displacement ductility demand. The shrinking of the shear failure surface under such conditions is controlled by monitoring the element's flexural displacement ductility demand imposed about each axis, and updating the shear failure surface as governed by a shear limit surface. The shear limit surface function can be defined based on the equation proposed by Kaul (2004), which is derived using the empirical drift capacity models for reinforced concrete columns prone to flexure-shear failure proposed by Elwood and Moehle (2003). In this research, shear limit functions are defined for y and z axes to govern the shear capacity in the corresponding direction. Since damage accumulated in inelastic excursions in one direction affects the shear strength and behaviour in other out-of-plane directions, biaxial interaction effects on the shear limit surfaces are considered in modelling 3D behaviour of reinforced concrete members based on multi-component damage indices, as discussed in the next section.

2.2. Formulation of Damage-Based Cyclic Degradation Models

The approach used in the cyclic behaviour modelling in this research is based on the incorporation of damage models in the beam-column element formulation to track the evolution of damage and consider its effects on the gradual deterioration in stiffness and loss of strength. The proposed model is thus capable of capturing the degradation behaviour of structural components beyond the peak response in a more realistic way. In the proposed formulation, cumulative and combined damage models based on deformations as well as hysteretic energy are used, which are suitable for representing damage in reinforced concrete members under cyclic loading. In this research, the cyclic models proposed by Kaul (2004) for modelling 2D plane hysteretic behaviour of reinforced concrete members are modified to include interaction with damage models and extended to capture full 3D behaviour by considering the biaxial interaction effects on stiffness and strength degradation.

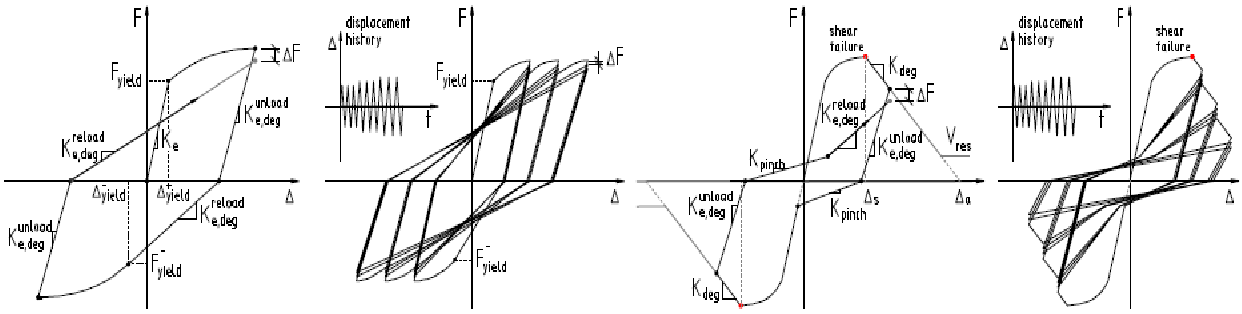
2.2.1. Cyclic stiffness model

The cyclic stiffness model is formulated separately from the generalized plastic hinge model, presented in section 2.1, to control the degrading stiffness upon unloading after yielding and subsequent reloading in the element hysteretic response dominated by flexure. The cyclic stiffness model requires force, deformation and damage index from the element response in order to determine degradation factors to vary the element stiffness components. The governing force quantity is the element shear normalized by the shear force corresponding to first yield. The governing deformation is the maximum natural rotation of the two ends of the element normalized by the rotation at first yield.

A bilinear peak-oriented model is considered in the proposed cyclic stiffness model. The unloading stiffness, $\left[K_{e,deg}^{unload}\right]$, is different from the reloading stiffness, $\left[K_{e,deg}^{reload}\right]$, but they are assumed to remain constant within one loading cycle, as shown in Figure 2.4(a). Degradation in stiffness is achieved by applying degradation factors to the flexural $\left(K_f = EI\right)$ and shear $(K_s = GA_s/L)$ stiffness components so that $K_{deg} = K\left(\alpha_y EI_y,\ \beta_y K_s;\ \alpha_z EI_z,\ \beta_z K_s\right)$ and the degradation factors are as follows:

$$
\begin{aligned}
\alpha_y &= w_{\alpha y1} DI_{y+} + w_{\alpha y2} DI_{y-} + w_{\alpha y3} DI_{z+} + w_{\alpha y4} DI_{z-} \\
\alpha_z &= w_{\alpha z1} DI_{y+} + w_{\alpha z2} DI_{y-} + w_{\alpha z3} DI_{z+} + w_{\alpha z4} DI_{z-} \\
\beta_y &= w_{\beta y1} DI_{y+} + w_{\beta y2} DI_{y-} + w_{\beta y3} DI_{z+} + w_{\beta y4} DI_{z-} \\
\beta_z &= w_{\beta z1} DI_{y+} + w_{\beta z2} DI_{y-} + w_{\beta z3} DI_{z+} + w_{\beta z4} DI_{z-}
\end{aligned}
\tag{2.8}
$$

where $w_{\alpha y1-4}$, $w_{\alpha z1-4}$, $w_{\beta y1-4}$, $w_{\beta z1-4}$ are weighting factors; DI_{y+}, DI_{y-}, DI_{z+} and DI_{z-} are damage indices for positive and negative excursions in y and z axes, respectively. The damage indices are calculated by damage models that interact with the element during inelastic excursions and keep track of the accumulated damage during the loading history. The unloading stiffness is degraded using damage indices based on flexural deformations and dissipated hysteretic energy, such as those proposed by Park and Ang (1985) and Kunnath et al. (1992). The reloading stiffness is determined based on force and deformation quantities from previous cycles to obtain peak-oriented response. The peak-oriented response is then updated to be consistent with the contracted yield surface, which allows the model to also capture the cyclic strength deterioration. Degraded elastic stiffness for unloading and reloading as well as the yield surface are updated for each loading cycle considering the accumulated damage, and thus capturing the degradation characteristics of the cyclic response in flexure, as illustrated in Figure 2.4(b).



(a) (b) (c) (d)

Figure 2.4. Degradation Models: (a) Cyclic Stiffness Model; (b) Cyclic Response in Flexure; (c) Cyclic Pinching Model; (d) Cyclic Response in Shear

2.2.2. Cyclic pinching model

After failing in shear and softening, the cyclic pinching model controls the degrading stiffness upon unloading and during subsequent reloading, which is characterized by pinching. Similarly to the cyclic stiffness model described in the previous section, the cyclic pinching model is formulated separately from the generalized plastic hinge model and uses force, deformation and damage index quantities from the element response to provide degradation factors for the stiffness components. In the proposed cyclic pinching model, shown in Figure 2.4(c), the unloading stiffness, $[K_{e,deg}^{unload}]$, is degraded using damage indices based on shear deformations and dissipated hysteretic energy. The degradation factors used to vary the flexural and shear stiffness components are calculated using Eqn. 2.8. The pinching stiffness, $[K_{pinch}]$, is determined based on force and deformation quantities from previous cycles, and the pinching is assumed to continue until the imposed deformation equals the peak deformation after shear failure. Upon reaching the target pinching deformation, the stiffness is changed to the degraded reloading stiffness, $[K_{e,deg}^{reload}]$, to obtain peak-oriented response. Cyclic strength degradation is also considered in the pinching model by modifying the peak-oriented response to be consistent with the contracted post-shear failure surface. Similarly to the cyclic stiffness model, degraded elastic stiffness for unloading, pinching and reloading as well as the post-shear failure surface are updated for each loading cycle considering the accumulated damage, and thus capturing the degradation characteristics of the post-peak cyclic response in shear, as illustrated in Figure 2.4(d).

The state determination in the formulation of the proposed cyclic models is based on a consistent event-to-event strategy. The concept of primary and follower half cycles is applied in the developed event-based strategy to facilitate the separation of positive and negative excursions. The concept of primary and follower half cycles has previously been used in Kratzig et al. (1989) in the development of cumulative damage index based on dissipated energy.

The proposed beam-column element is to be incorporated into the OpenSees software framework (McKenna 1997). The formulation of the new models presented herein is developed considering flexibility and extensibility of the implementations based on object-oriented design concepts.

3. CONCLUSIONS

The objective of this study is the development of a comprehensive and computationally efficient beam-column finite element model for seismic response analysis of reinforced concrete structures subjected to multi-component seismic loading that captures the behaviours of members in new structures designed as ductile as well as non-ductile members in existing older deficient structures. The proposed formulation is based on stress-resultant concentrated plasticity concepts using force interaction surfaces and evolution models to capture nonlinear axial-flexure-shear interaction, flexural hardening, shear failure and post-shear failure softening of reinforced concrete members under cyclic load reversals of biaxial bending moments with variable axial load. Deterioration of shear strength in the plastic hinge zone with increasing flexural displacement ductility demand is also accounted for by using ductility-related shear limit surface. The approach used in the cyclic behaviour modelling is based on the incorporation of damage models in the beam-column element formulation to monitor the evolution of damage and capture its effects on the gradual deterioration of stiffness and strength in the post-peak response of structural components. Biaxial interaction effects on stiffness and strength degradation are also considered in the formulation based on multi-component damage models capturing hysteretic damage in multi-directions, and thus the proposed cyclic degradation models are suitable for capturing 3D behaviour of reinforced concrete members under general loading. Numerical simulation models, such as the model proposed in this research, capable of accurate capturing of damage and degradation response characteristics would lead to more resilient earthquake design of new structures as well as more efficient retrofit and rehabilitation strategies for existing older deficient structures.

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REFERENCES

Abou-Elfath, H., Ghobarah, A. and Aziz, T. S. (1998). Seismic Analysis of Non-Ductile Frames. *Sixth U.S. National Conference on Earthquake Engineering*. EERI, Oakland, California. **CD-ROM:** 12.

Ceresa, P., Petrini, L., Pinho, R. and Sousa, R. (2009). A Fibre Flexure-Shear Model for Seismic Analysis of RC Framed Stuctures. *Earthquake Engineering and Structural Dynamic*. **38**, 565-586.

Chen, P. F. S. and Powell, G. H. (1982). Generalized Plastic Hinge Concepts for 3D Beam-Column Elements. UCB/EERC-82/20, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Chen, W. F. and Han, D. J. (2007). Plasticity for Structural Engineers. J. Ross Publishing, Florida 33309.

Deierlein, G. G., Reinhorn, A. M. and Willfordm M. R. (2010) Nonlinear Structural Analysis for Seismic Design. NIST, Gaithersburg, MD.

ElMandooh Galal, K. and Ghobarah, A. (2003). Flexural and Shear Hysteretic Behaviour of Reinforced Concrete Columns with Variable Axial Load. *Engineering Structures*. **25**, 1353-1367.

Elwood, K. J. and Moehle, J. P. (2003). Shake Table Tests and Analytical Studies on the Gravity Load Collapse of Reinforced Concrete Frames. PEER Report 2003/01, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.

El-Tawil, S. (1996). Inelastic Dynamic Analysis of Mixed Steel-Concrete Space Frames. Ph.D. Dissertation, School of Civil and Environmental Engineering, Cornell University, Ithaca, NY 14583.

El-Tawil, S. and Deierlein, G. G. (1998). Stress-resultant Plasticity for Frame Structures. *ASCE Journal of Engineering Mechanics*. **124:12**, 1360-1370.

Ghee, A. B., Priestley, M. J. N. and Paulay, T. (1989). Seismic Shear Strength of Circular Reinforced Concrete Columns. *ACI Structural Journal*. **86:1**, 45-59.

Kaul, R. (2004). Object Oriented Development of Strength and Stiffness Degrading Models for Reinforced Concrete Structures. Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University, Stanford, CA.

Kratzig, W. B., Meyer, I. F. and Meskouris, K. (1989). Damage Evolution in Reinforced Concrete Members under Cyclic Loading. *Proceedings of the 5th International Conference on Structural Safety and Reliability (ICOSSAR 89),* San Francisco, CA, **Vol. II:** 795-802.

Kunnath, S. K., Reinhorn, A. M., and Lobo, R. F. (1992). IDARC version 3.0: A Program for Inelastic Damage Analysis of RC Structures. Technical Report NCEER-92-0022, National Center for Earthquake Engineering

Research, State University of New York, Buffalo, NY.
Lai, S. S., Will, G. and Otani, S. (1984). Model for Inelastic Biaxial Bending of Concrete Members. *ASCE Journal of Structural Engineering*, **110:ST11**, 2563-2584.
Lee, D. H., and Elnashai, A. S. (2002). Inelastic Seismic Analysis of RC Bridge Piers Including Flexure-Shear-Axial Interaction. *Structural Engineering and Mechanics*, **13:3**, 241-260.
Mari, A. and Scordelis, A. (1984). Nonlinear Geometric Material and Time Dependent Analysis of Three Dimensional Reinforced and Prestressted Concrete Frames. UC SESM-84/12, Department of Civil Engineering, University of California, Berkeley, CA.
Marini, A. and Spacone, E. (2006). Analysis of Reinforced Concrete Elements Including Shear Effects. *ACI Structural Journal*, **103:5**, 645-655.
Martinelli, L. (2008). Modeling Shear-Flexure Interaction in Reinforced Concrete Elements Subjected to Cyclic Lateral Loading. *ACI Structural Journal*, **105:6**, 675-684.
Mazars, J., Kotronis, P., Ragueneau, F. And Casaux, G. (2006). Using Multifibre Beams to Account for Shear and Torsion. Applications to Concrete Structural Elements. *Computer Methods in Applied Mechanics and Engineering*, **195:52**, 7264-7281.
McGuire, W., Gallagher, R. H. and Ziemian, R. D. (2000). Matrix Structural Analysis. Second Edition, John Wiley and Sons, New York.
McKenna, F. T. (1997). Object-Oriented Finite Element Programming: Frameworks for Analysis, Algorithms and Parallel Computing. Ph.D. Dissertation, Department of Civil and Environmental Engineering, University of California, Berkeley, CA.
Mehanny, S. S. F. and Deierlein, G. G. (2001). Seismic Collapse Assessment of Composite RCS Moment Frames. *ASCE Journal of Structural Engineering*, **127:9**, 1045-1053.
Mullapudi, T. R. and Ayoub, A. (2010). Modeling of the Seismic Behavior of Shear-Critical Reinforced Concrete Columns. *Engineering Structures*, **32**, 3601-3615.
Nakamura, T. and Yoshimura, M. (2002). Gravity Load Collapse of Reinforced Concrete Columns with Brittle Failure Modes. *Journal of Asian Architecture and Building Engineering*, **1:1**, 21-27.
Park, Y. J., and Ang, A. H-S. (1985). Mechanistic Seismic Damage Model for Reinforced Concrete. *ASCE Journal of Structural Engineering*, **111:ST4**, 722-739.
Petrangeli, M., Pinto, P. E. and Ciampi, V. (1999). Fibre Element for Cyclic Bending and Shear of RC Structures. I: theory. *Journal of Engineering Mechanics*, **125:9**, 994-1001.
Pincheira, J. A., Dotiwala, F. S., and D'Souza, J. T. (1999). Seismic Analysis of Older Reinforced Concrete Columns. *Earthquake Spectra*, **15:2**, 245-272.
Priestley, M. J. N., Seible, F., Xiao, Y., and Verma, R. (1994). Steel Jacket Retrofitting of Reinforced Concrete Bridge Columns for Enhanced Shear Strength – Part 2: test results and comparison with theory. *ACI Structural Journal*, **91:5**, 537-551.
Ricles, J. M., Yang, Y.-S., and Priestley, M. J. N. (1998). Modeling Nonductile R/C Columns for Seismic Analysis of Bridges. *ASCE Journal of Structural Engineering*, **124:4**, 415-425.
Reshotkina, S. S. and Lau, D. T (2011). State of the Art Review of Stiffness and Strength Degradation Models for Seismic Response Analysis of Reinforced Concrete Structures. *2nd International Engineering Mechanics and Materials Specialty Conference*, CSCE, Ottawa, Ontario, **CD-ROM:** 10.
Saatcioglu, M. and Ozcebe, G. (1989). Response of Reinforced Concrete Columns to Simulated Seismic Loading. *ACI Structural Journal*, **86:1**, 3-12.
Sezen, H. (2002). Seismic Response and Modeling of Reinforced Concrete Building Columns. Ph.D. Dissertation, Department of Civil and Environmental Engineering, University of California, Berkeley, CA.
Sezen, H. and Chowdhury, T. (2009). Hysteretic Model for Reinforced Concrete Columns Including the Effect of Shear and Axial Load Failure. *ASCE Journal of Structural Engineering*, **135:2**, 139-146.
Shirai, N., Moriizumi, K. and Terasawa, K. (2001). Cyclic Analysis of Reinforced Concrete Columns: Macro-Element Approach. *Modeling of Inelastic Behaviour of RC Structures under Seismic Load*, ASCE, Reston, Virginia, 435-453.
Spacone, E., Filippou, F. C and Taucer, F. F. (1996). Fiber Beam-Column Model for Nonlinear Analysis of R/C Frames. I: Formulation. *Earthquake Engineering and Structural Dynamics*, **25:7**, 711-725.
Takizawa, H. and Aoyama, H. (1976). Biaxial Effects in Modeling Earthquake Response of RC Structures. *Earthquake Engineering and Structural Dynamics*, **4**, 523-552.
Vecchio, F. J. and Collins, M. P. (1986). The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to Shear. *ACI Journal*, **83:2**, 219-231.
Watanabe, F., and Ichinose, T. (1992). Strength and Ductility of RC Members Subjected to Combined Bending and Shear. *Concrete Shear in Earthquake*, Elsevier Applied Science, New York, 429-438.
Wong, Y. L., Paulay, T. and Priestley, M. J. N. (1993). Response of Circular Reinforced Concrete Columns to Multi-Directional Seismic Attack. *ACI Structural Journal*, **90:2**, 180-191.
Xu, S.-Y. and Zhang, J. (2011) Hysteretic Shear-Flexure Interaction Model of Reinforced Concrete Columns for Seismic Response Assessment of Bridges. *Earthquake Engineering and Structural Dynamics*, **40:3**, 315-337.
Zeris, C. A. and Mahin, S. A. (1991). Behaviour of Reinforced Concrete Structures Subjected to Biaxial Excitation. *ASCE Journal of Structural Engineering*, **117:ST9**, 2657-2673.