Nonlinear analysis of unreinforced beam-column joints

Análisis no-lineal de nudos de concreto sin refuerzo

DOI: http://doi.org/10.17981/ingecuc.16.1.2020.09

Artículo de Investigación Científica. Fecha de Recepción: 10/10/2018. Fecha de Aceptación: 24/09/2019

Daniel Ricardo Salinas Guayacundo
Fundación Universitaria Juan de Castellanos. Tunja (Colombia)
dricardosalinas@hotmail.com

Roberto Tomas Leon Saenz
Virginia Polytechnic Institute and State University (Virginia Tech). Virginia (USA)
rleon@vt.edu

Para citar este artículo:
D. Salinas Guayacundo, R. Leon Saenz, "Aplicación de la Búsqueda Armónica para el problema de formación de celdas de manufactura", INGE CUC, vol. 16, no. 1, pp. 129–140, 2020. DOI: http://doi.org/10.17981/ingecuc.16.1.2020.09

Abstract

Introduction– Reinforced Concrete Frames (RCF) constitute a significant portion of the building stock in areas with seismic hazard. Many older buildings of this type were designed and constructed with little or no consideration of lateral load effects. When not properly designed, the Beam-Column Joints (BCJ) can be the weak links in the RCF. Unreinforced BCJ are still quite prevalent in older-type construction especially in Asia and Latin America. The unreinforced BCJ are key components that have a significant impact on the structure's behavior of RCF. Regarding the analytical approaches applicable to BCJ, the approaches range from simplified to more elaborate and phenomenological-oriented. Unfortunately, most of them lack of simplicity, numerical stability and practicality to robustly evaluate the performance of unreinforced BCJ. This paper presents an analytical approach to modeling unreinforced BCJ.

Objective– The aim of this paper is to present a modified modeling approach to simulate the nonlinear behavior of unreinforced BCJ in RCF structures.

Method– In the proposed approach, the BCJ subassembly is represented by (1) a set of rigid links placed in cross-shape are used to represent the joint geometry, (2) a zero-length element with an empirical quad-backbone curve, placed at the middle point of the rigid links, to represent the joint shear behavior, and (3) columns and beams elements modeled with fiber formulation and five integration points to capture the material nonlinearity of the elements that frame into the joint. The approach was implemented in the OpenSEES platform, and this was validated with 13 test results of unreinforced BCJ documented in the literature.

Results– The proposed modelling approach can satisfactorily predict the joint shear capacity. A 2% difference and a standard deviation of about 11% were obtained when compared to 13 test results of unreinforced BCJ documented in the literature. In terms of cyclic behavior, the proposed modelling approach shown to adequately capture the initial stiffness, strength degradation, reloading stiffness, pre-capping, and post-capping capacity.

Conclusions– The method proposed presents satisfactory agreement with the test results analyzed. Taking into account the minor modifications applied to the proposed method and the uncertainties associated with the materials, test measurements, test setup, and the tolerances, the proposed method can satisfactorily predict the unreinforced BCJ shear capacity in RCF structures. It is assumed that the procedures presented here will contribute in the incorporation of the unreinforced BCJ flexibility when modeling older-type RCF construction in a pragmatic manner.

Keywords– Nonlinear analysis; older-type construction; reinforced concrete frames; unreinforced beam-column joints

Resumen

Introducción– El Sistema estructural Pórticos de Concreto Reforzado (PCR) constituye una parte significativa del inventario de edificaciones en zonas sísmicamente activas en el mundo. Muchas de las edificaciones construidas antes de la década de los 80's fueron diseñadas y construidas con poca, o ninguna consideración de cargas sísmicas. Cuando el nudo de concreto reforzado no se ha diseñado competentemente puede convertirse en el eslabón débil del sistemas de PCR. La presencia de nudos sin refuerzo, aun es común en países emergentes localizados en Asia y América Latina. Los nudos tienen un impacto significativo en el comportamiento de PCR. Las metodologías relacionadas con el análisis de nudos de concreto pueden catalogarse como aproximadas, o muy complejas, o de enfoque fenomenológico. Desafortunadamente la mayoría de ellas carece de la simplicidad, estabilidad, y practicidad requerida para evaluar el comportamiento de los nudos en PCR. Este artículo presenta una alternativa analítica aplicable a este tipo de elementos estructurales.

Objetivo– El propósito del presente artículo es presentar un método analítico modificado aplicable al análisis no lineal de nudos no reforzados en estructuras de PCR.

Metodología– En el modelo analítico, el nudo es representado a través de: (1) elementos rígidos en cruz para idealizar la geometría del nudo, (2) un resorte rotacional con una curva empírica de comportamiento tetralineal localizado en la mitad de los elementos rígidos para representar el comportamiento en cortante del nudo, y (3) las vigas y columnas que llegan al nudos, son modeladas con análisis seccional basado en fibras, con 5 puntos de integración; con la finalidad de incorporar el comportamiento no-lineal de los elementos que llegan al nudo. El modelo propuesto fue implementado en la plataforma OpenSEES y al mismo tiempo se validó con el resultado de 13 ensayos de laboratorio encontrados en la literatura de nudos carentes de acero de refuerzo.

Resultados– El modelo propuesto puede capturar adecuadamente la capacidad a cortante del nudo. Al comparar los resultados analíticos con 13 resultados de nudos de concreto encontrados en la literatura, se encontró una diferencia en la capacidad del 2% con una desviación estándar del 11%. En relación al comportamiento del nudo ante carga cíclica se observó que se captura en forma adecuada: la rigidez inicial, resistencia, degradación de la resistencia, rigidez de recarga y capacidad antes y después del pico de resistencia.

Conclusiones– El método propuesto presenta una adecuada correlación con los resultados de laboratorio estudiados. La metodología propuesta competentemente captura la capacidad del nudo a cortante, a pesar de las modificaciones incorporadas, sin mencionar las incertidumbres asociadas a los materiales, resultados de laboratorio, y tolerancias. Se espera que el procedimiento presentado en el presente documento contribuya, de una forma práctica, en la incorporación de la flexibilidad del nudo en PCR diseñados primariamente para cargas gravitacionales.

Palabras clave– Análisis no-lineal; pórticos de gravedad; pórticos de concreto reforzado; nudos de concreto no-reforzados

I. Introduccion

Reinforced Concrete Frames (RCF) constitute a significant portion of the building stock in areas with seismic hazard. Many older buildings of this type were designed and constructed with little or no consideration of lateral load effects. The vulnerability of such structures, hereinafter termed non-ductile frames, may have serious implications for the resilience of built communities. When a RCF is subjected to seismic forces, Beam Column Joints (BCJ) play a major role in transferring internal forces among adjacent columns and beam elements. When not properly designed, BCJ can be the weak links in the structural system.

The vulnerability of unreinforced BCJ is understood today. Unconfined beam–column joints are still quite prevalent in older-type construction especially in Asia and Latin America. The unreinforced BCJ are key components that have a significant impact on the structure's behavior of RCF. Based on the fact that a gross percentage of the non-ductile RCF were constructed without following any standard and detailing rules, those structures are more likely to present this vulnerability. The following sections discuss in more detail the literature review associated to unreinforced BCJ and present a practical-oriented approach to model this vulnerability in non-ductile frames.

II. Unreinforced beam-column joints

For many years, the importance of Beam Column Joints (BCJ) was overlooked. This was true because there was little evidence of major damage or collapse that could be attributed to the failure of BCJ. Poor detailing of the columns and beams held the attention of the researchers in post-earthquake damage surveys. When subjected to large ground motions, many RCF buildings have shown soft story mechanisms from lack of rotational capacity associated with improper detailing of plastic hinges in both columns and beams. In a RCF, older-type columns are characterized by having widely spaced transverse reinforcement which induces large inelastic action at the column ends. This type of columns typically fail in shear with a subsequent loss of lateral and/or axial-load-carrying capacity. After learning from these previous experiences, requirements to produce much better curvature ductility in the beam and column elements were developed and implemented into codes [3, Ch. 19]. At this stage, BCJ became the weak links in the structural system, and their importance was recognized [2].

Fig. 1(a) and Fig. 1(b) present a typical idealization for the RCF and BCJ respectively. As can be seen in Fig. 1(b), the response of the joint region will be governed by shear forces, which are transmitted by bearing, bond, and friction. Shear cracking, when not properly controlled, induces brittle failures, as shear failure is related to diagonal cracking in concrete. Today it is understood that beam column joints are key elements in the seismic performance and integrity of RC frames.

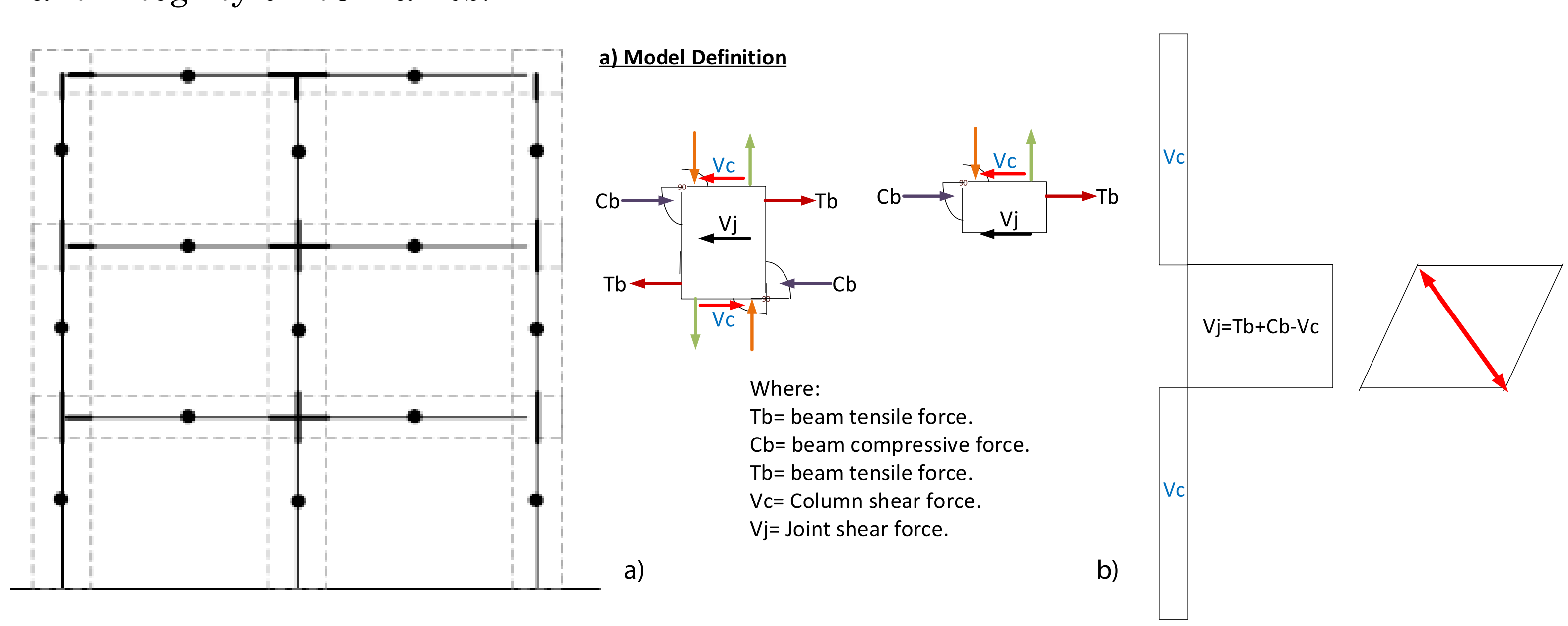


Fig. 1. BCJ idealization. a) Model idealization,
b) Internal forces at BCJ, shear demand, and deformed shape.
Source: Authors.

When a RCF experiences earthquake force, the joint region is subjected to large additive moments (i.e. large moments with the same orientation) at opposite ends of the columns and beams. As a consequence, the joint region is subjected to vertical and horizontal shears that are typically much larger than those used to size the elements that frame into the joint as shown in Fig. 1(b). The reversal in moments in the joint also means that the beam reinforcement is required to be in compression on one side and in in tension on the other side. For that reason, high bond stresses are required to sustain this steep gradient bond along the joint and bond failure may occur with corresponding degradation of moment capacity and stiffness that will lead to excessive lateral drifts.

If internal joints are assumed to be rigid in the analyses, the discrepancies from true behavior are mainly that the structure will have a shorter period of vibration. Compared with the true behaviour, the rigid assumption implies an important reduction of the lateral displacement, thus the true displacements will be underestimated. In some cases, for larger drift demands the structure will lose its lateral deformation capacity as a preamble to loss of axial load-carrying capacity.

The common denominator on the unreinforced BCJ failures is that no plastic hinges formed in the columns or beams. This is not an indication that these elements had an appropriate overcapacity and ductility, as the joint failure precluded their reaching large inelastic deformations. The failure is due to the limited capacity of the joint to transmit the forces and to keep its integrity.

A. *Literature review*

Through the years, many researchers have proposed models to represent the nonlinear behavior of BCJ. The early attempts relied on: (i) the lumped plasticity concept applied generally at the end of one elastic element, or/and (ii) the "two component model" concept [4] to idealized a steel frame as combination an elasto-plastic element representing the yielding behavior and one elastic element to represent strain hardening behavior. These types of models are referred herein as implicit models because they do not define the joint region physically, and therefore they fail to represent the exact joint kinematics. Some examples are depicted in Fig. 2. On the other hand, some of the newest approaches are based on the definition of the physical panel zone by using a macro-element and centerline analysis. The macro-element is composed of certain numbered joints, elements and springs that are assembled to represent the nonlinear behavior in the joint through the interrelation of their elements, constitutive relationships and boundary constraints. Concepts of lumped plasticity or distributed plasticity can be used. These type of models are referred here as explicit models. The difference between implicit and explicit models is in the modeling and computational effort required, and, in some cases, on the practicality of such models to be applied for everyday design. A general classification of the beam column joint models will be proposed in the following sections.

B. *Implicit models*

For about 50 years, extensive research has been done with the intent to model the hysteretic behavior of RC members under reversed cyclic loading. The first step in conducting an analysis of a BCJ, is to define the structural model that represents the nonlinear problem. The challenge is to find a set of rules that represent the hysteretic behavior of RC members under cyclic loading. Many hysteresis rules and deterioration models have been proposed to date [5]-[11]. Many models have been implemented by applying or refining some of the degradation models proposed by the above researchers. Based on the author's point of view, some of the most relevant implicit models are referenced in [12]-[17]. Some of them are depicted in Fig. 2. Readers are referred to those references for a more detailed discussion.

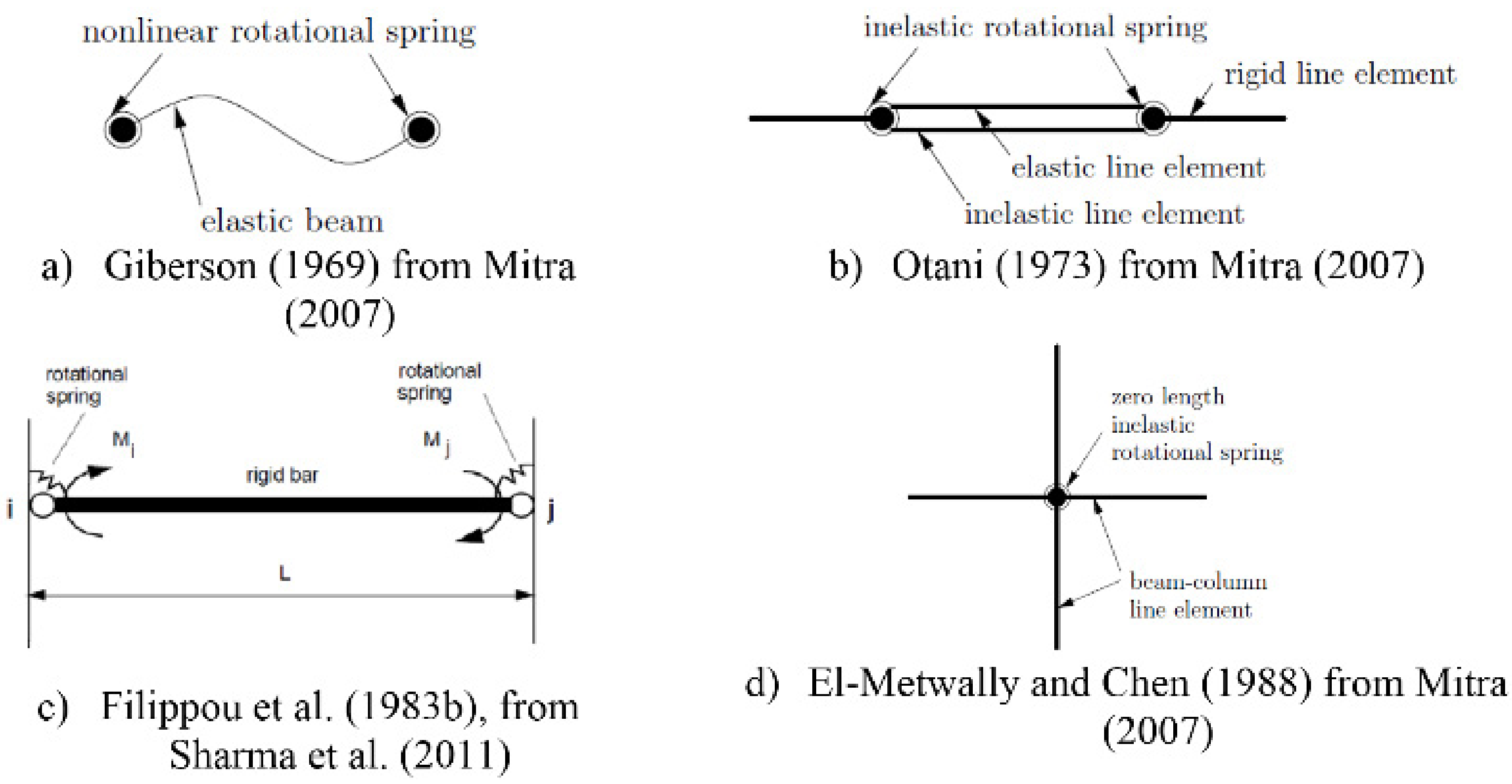


Fig. 2. Implicit BCJ idealization.
Source: [18], [19].

C. *Explicit models*

With the development of computational techniques, an important number of beam column models and super-elements (i.e. a set of elastic and inelastic elements used to mimic the kinematics of the BCJ) have been proposed during the last years. Table 1 and Fig. 4 depicts some of the most relevant explicit models.

Explicit models have been applied in powerful platforms such as OpenSEES [40]. Other authors use similar concepts and degradation rules. Some of the most relevant explicit models are: [20], [28], [31], [33], [35], [41]-[44].

For the constitutive relationship of the panel zone, [20] uses a tri-linear idealization based on a softening truss model. In model [28], in contrast, uses a plane stress element approach. The authors [43] they use axial springs connecting rigid bodies. The remaining explicit models rely on the Modified Compression Field Theory (MCFT) [23] in order to define the backbone envelope curve for the shear region. Additional features to include bond deterioration and cyclic degradation are often considered; however, the key parameter is the constitutive model used for the joint region.

Generally, despite the fact that the explicit models are more complex, mechanistically based and well-elaborated, their calibration depends almost exclusively on test results. Even though the springs are calibrated from test data, due to the difference in reliability, measurements, and overall quality and instrumentation, it is recognized that these springs do not ensure the accuracy of the analysis for either other test results found in the literature or actual BCJ in service. In fact, multi-spring models are more likely to suffer from numerical instability during frame analysis. However, for non-ductile BCJ, all the explicit models that use the MCFT for defining the shear envelope for the panel zone tend to underestimate the shear joint response [45]. In fact, the MCFT is limited to ductile joints, because once the tensile stress in the joint is greater than the cracking stress of the concrete, there is no external mechanism to balance the tensile force that is produced in the joint for the case of non-ductile joints. In other words, MCFT does not account for the dowel action provided by the longitudinal reinforcement of the columns, and once cracking is present, convergence problems will appear in the analysis.

Based on the above information, explicit models are computational demanding, and in general terms they lack the simplicity, numerical stability and practicality to robustly evaluate RCF performance under cyclic loading.

Table 1. Explicit models.

Model	Description	Joint Shear	Bond slip	Constitutive relationships	Cyclic Deterioration
[20]	Two spring joint element. One spring represented the inelastic shear response of the joint and the other represented bond-slip.	Tri-linear idealization based on a softening truss model [21].	Bilinear model based on previous analytical and experimental data [22].	MCFT [23].	[24].
[25]	Panel zone represented by a 12 node inelastic plane stress element. 10 elastic elements connected to the joint through the interposition of non-linear transitional elements.	Plane stress element.	Contact elements were introduced in between the nodes of the flexural reinforcement and the adjacent plane stress elements. The bondslip model by [26] with modifications proposed by [27].		Hysteretic relationship with no pinching effect.
[28]	Two diagonal translational springs connecting the opposite corners of the panel zone simulate the joint shear deformation. 12 translational springs located at the panel zone interface.	MCFT [23] was utilized to define the backbone envelope of the curve.	Analytical material model by [29].	[30].	[25].
[31]	Four node 12 DOFs joint element. 8 zero-length translational springs simulate the bond–slip response of beam and column longitudinal reinforcement. Joint region modeled with one zero-length rotational spring. 4 zero-length shear springs simulate the interface-shear deformations.	Three linear backbone curve using MCFT [23].	[26].	[30].	Adjusted from [32]-[33].
[33]	Four exterior nodes, constrained to a central node by multi-point constraints. One rotational spring to emulate shear response of the joint.	Three linear backbone by using modified Newton Raphson iteration curve using MCFT [23].	[26].	[30].	[32], [34].
[35]	Rigid elements located along the edges of the panel zone. Rotational springs embedded in one of the four hinges linking adjacent rigid elements.	Three linear backbone curve using MCFT [23].	The bond-slip rotational springs were calibrated using the formulation proposed by [36].	[37].	Adjusted by test results.
[38]	Four nodes, twelve degree of freedom element.	Axial springs connecting rigid bodies.	The model proposed by [36].	[37].	[36].
[39]	Beam element and a column element.	MCFT [23].	The model proposed by [36].	[18].	[36].

Source: [20], [21], [22], [23], [24], [25], [26], [27], [28], [29], [30], [31], [32], [33], [34], [35], [36], [37], [38], [39].

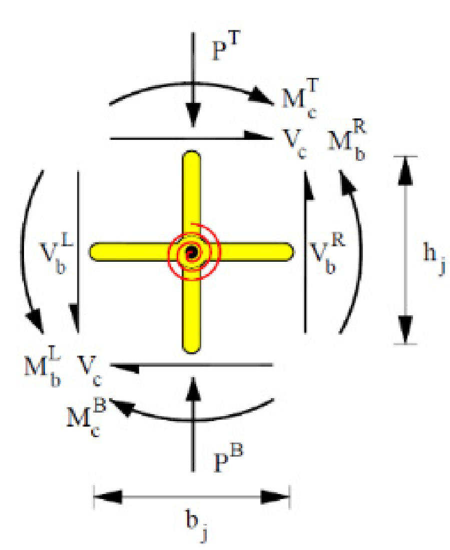


Fig. 3. BCJ model idealization.
Source: [46].

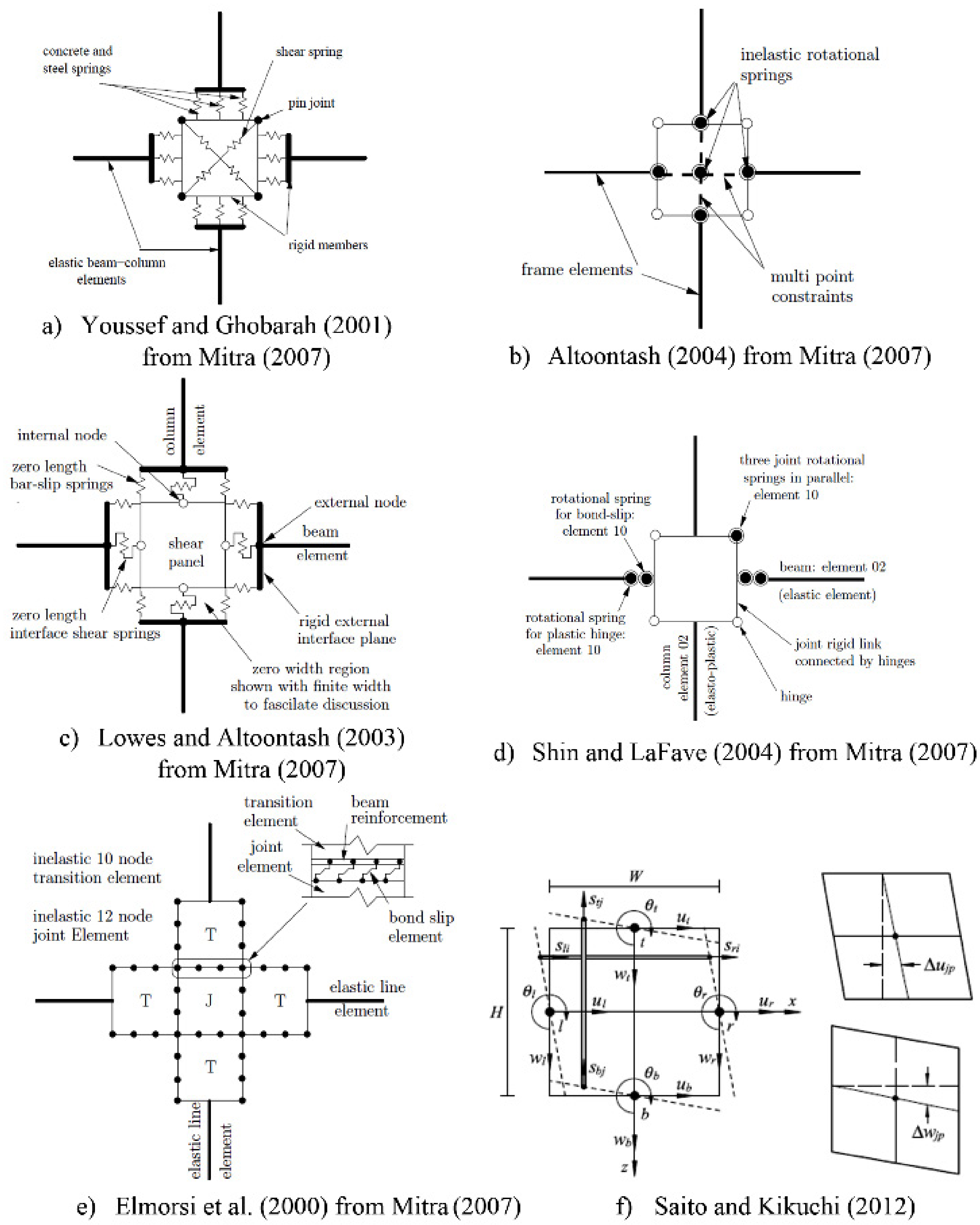


Fig. 4. Examples of explicit BCJ models.
Source: [18], [43].

III. Proposed approach to modeling unreinforced beam-column joints

The simplest approach to performing nonlinear analysis of BCJ is by using a nonlinear spring at the intersection of the beam-column line elements with the inclusion of rigid offsets to define a physical size of the joint. Fig. 3 depicts this beam column joint representation. Even though it is required more refinement in order to represent the true geometry and complex kinematic behavior of the BCJ, the procedures presented as follow will allow to model the unreinforced BCJ flexibility in older-type RCF construction.

This approach uses the concentrated plasticity concept in conjunction with a set of elements including: (a) rigid links to represent the joint geometry (b) in the middle point of the rigid links, a nonlinear rotational spring is created by using a zero length element (i.e. the rotational spring represents a physical kinematics of the BCJ by means of the moment rotation curve) and (c) nonlinear beam column elements to represent the BCJ subassembly. This approach is easily implemented in the OpenSEES platform and due to its overall simplicity, lack of numerical problems and perceived accuracy, has been used by many researchers in the past few years [1], [47]-[49].

The suggested formulation for the scissors model is a force formulation, which implies that a definition of the plastic hinge length is not required. Distributed plasticity elements are modeled with fiber elements and five integration points are used in columns and beams to capture the material nonlinearity of the elements that frame into the joint. The advantage of the fiber formulation is to facilitate the specification of unconfined and confined concrete to account for the effects of confinement and ductility. The joint region is represented by rigid link elements. The constitutive relationship for the joint is assigned to a zero length element located in the center of the joint. The quad-linear backbone curve and the semi-empirical joint shear capacity proposed by [50] is implemented in this study. Once these variables are defined, a centerline analysis is applied. The main features of this methodology, as applied herein, are summarized on Fig. 5. The joint capacity is calculated with equation (1) and equation (2) from [1] where is calculated accordance with [2], and f, c, b_j, h_b are the compressive strength resistance of concrete, joint width, column height, and beam height, respectively.

$$Vn = \gamma\sqrt{f'c}\, b_j h_c \frac{\cos(\theta)}{\cos\left(\frac{\pi}{4}\right)} \tag{1}$$

$$\theta = tan^{-1}\left(\frac{h_b}{h_c}\right) \tag{2}$$

The first step is to define the RCF geometry (i.e Beam-colum elements, BCJ, rigid links, and rotational springs) using centerline analysis, inflections points (i.e. points of zero moment at the elements which frame into the joint) are assumed to be located at midpoint of the beam and column elements, see Fig. 5(a). In Fig. 5(b), the joint shear strength is selected in accordance of [2]. Finally the strength modification factor is calculated, thus the joint shear capacity, backbone curve and the moment rotation behavior of the nonlinear spring can be determined. Fig. 5(c) to Fig. 5(f) depicts the above procedure.

The moment-rotation backbone relationship depicted in Fig. 5(f) is calculated using the geometry and equilibrium equations applied to the isolated subassemblies. Similar procedures can be found in [46], [51]-[53]. Once the moment-rotation relationships are calculated, they can be implemented with the Pinching4 [33] model in OpenSEES.

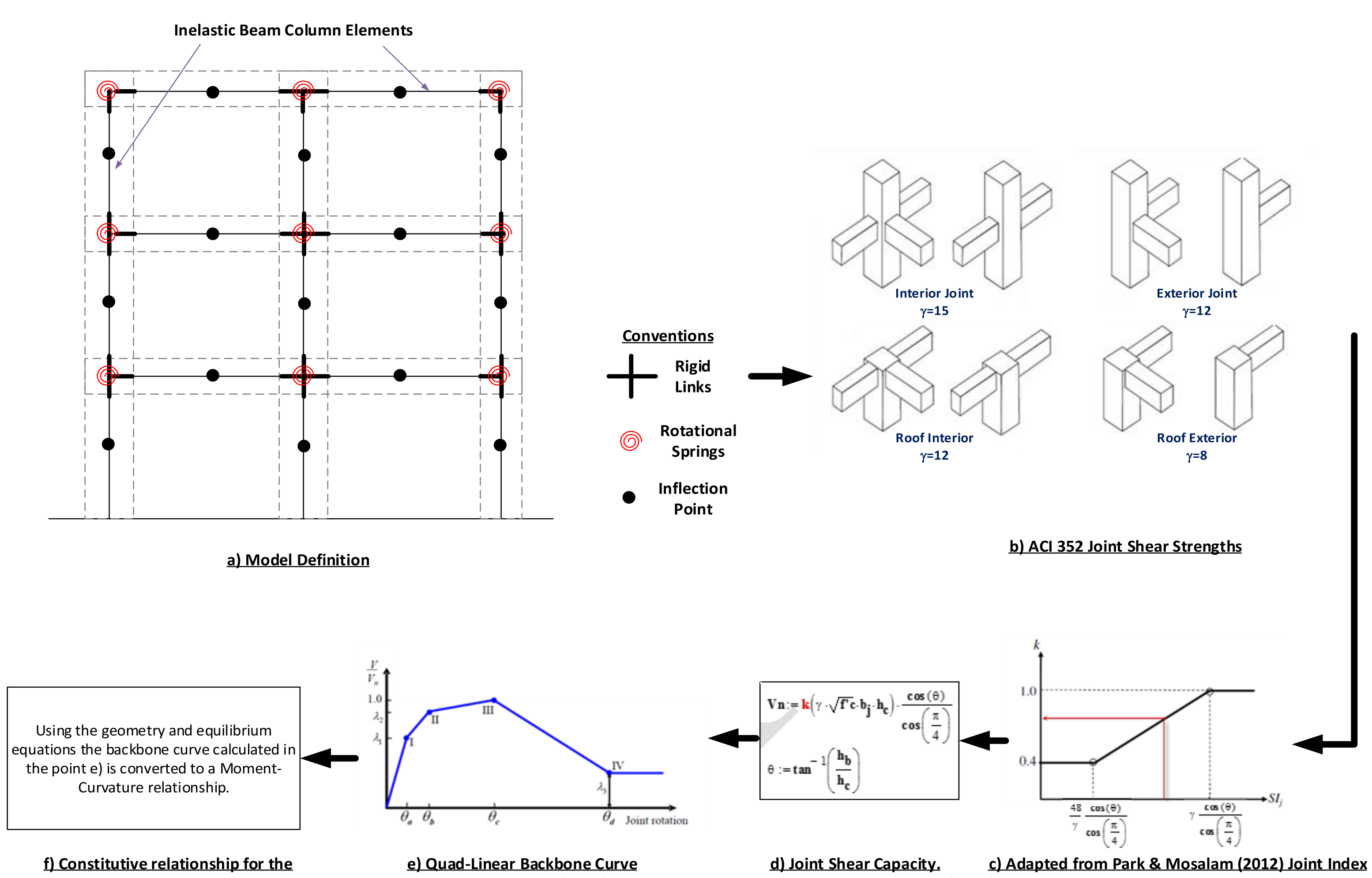


Fig. 5. Procedure outline proposed for modeling unreinforced BCJ.
Source: Author adapted [50].

For defining the backbone curve for the nonlinear spring and its constitutive relationship, the model presented here is almost identical to the model presented in [1]. In essence, the model was modified to follow the same nomenclature of [2]. The major modifications are:

1. The term Γ was omitted, instead it is proposed to use γ from [2]
2. In equation (3) of the referenced paper, the nominal shear force is expressed in terms of the exterior shear strength coefficient ($\gamma = 12$). This equation was expressed in a more general as is depicted in Fig. 5(d). This change was also applied because if equation (3) is applied directly, the nominal shear for the specimen SP1 EW would be overestimated by 20% because there is another beam framing into the joint in the perpendicular direction.

A) *Validation of the proposed approach*

The proposed model was validated with various test results of unreinforced BCJ well documented in the literature. In this study, tests by [54] and [55] are used to compare with the analytical results. In [54] an investigation about the shear strength and seismic performance of unreinforced BCJ was performed, thus 17 large-scale specimens representing exterior unreinforced BCJ under reverse cyclic loading were tested. The specimens were designed considering different parameters including: axial load ratio, type of beam anchorage, longitudinal and transverse reinforcement ratio, and beam to column depth ratio. This study focuses on the effect of the above parameters on the failure mechanism and ultimate strength of unreinforced BCJ. Additionally, [55] conducted an extensive experimental and analytical investigations to simulate the progressive collapse of older-type RC buildings construction and generate collapse fragility curves. The experimental program included 4 full-scale unreinforced corner BCJ. The specimens were tested under reverse cyclic loading and they were designed considering the joint aspect ratio and the amount of longitudinal beam reinforcement. Table 2 summarizes the comparison between the prediction equation and the test results.

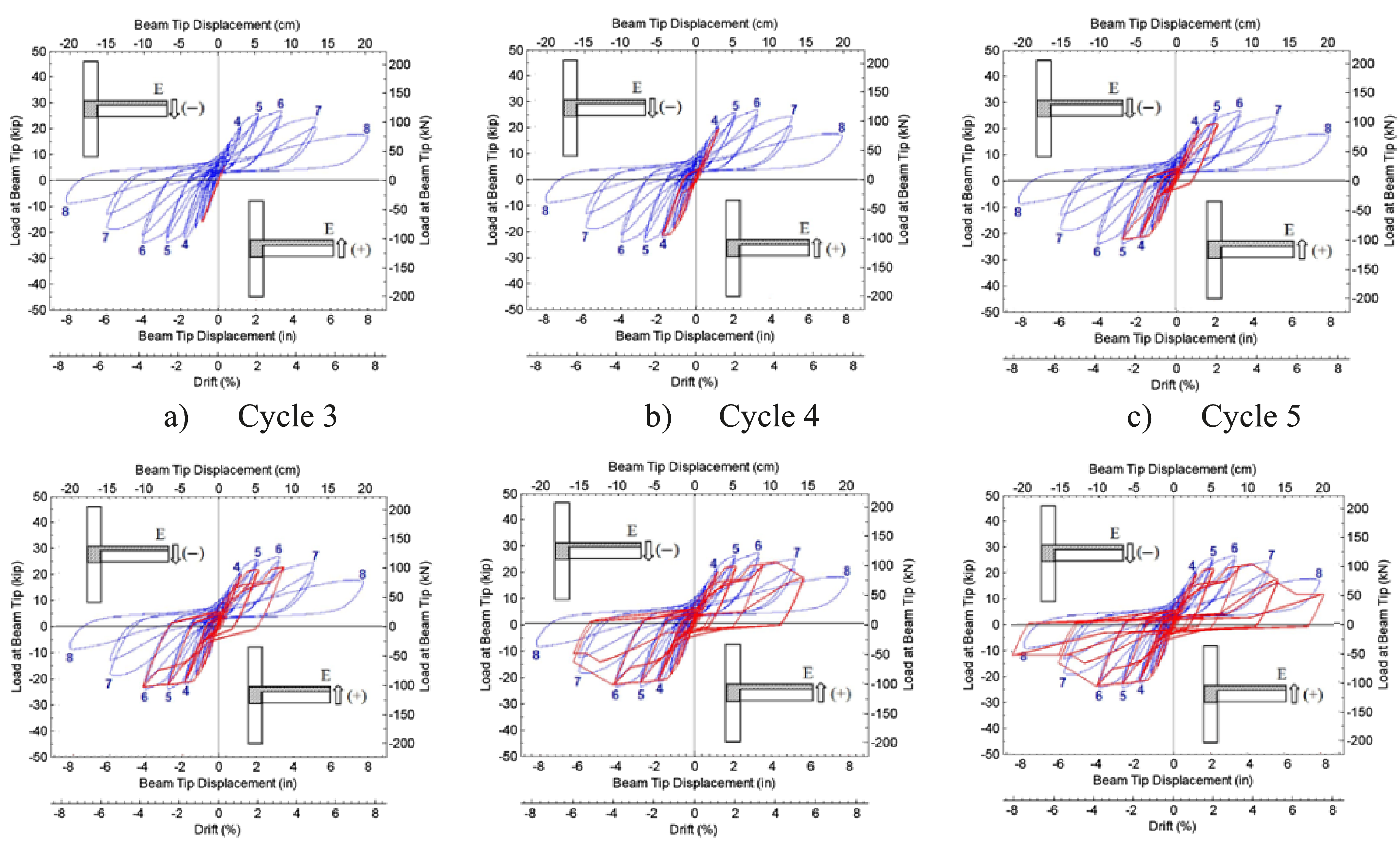
Table 2. Comparison between test results versus the proposed model.

Ref.	Specimen	f'c (ksi)	fy (ksi)	Vj Test (kip)	S*Ij*	θ	x1	x2	k	γ	Vn Calc (kip)	Vcalc / Vtest	Vn ASCE 41-06	Vcalc / Vn_{ASCE} 41-06
[54]	BS-U	4.5	75.4	76.70	11.08	0.98	3.14	9.41	1.16	12	95.06	1.24	52.24	0.68
	BS-L-LS	4.58	75.4	77.50	10.99	0.98	3.14	9.41	1.15	12	95.13	1.23	52.71	0.68
	BS-L-300	4.94	75.4	113.50	11.07	0.79	4.00	12.00	0.93	12	101.88	0.90	54.74	0.48
	BS-L-600	5.28	75.4	63.80	9.75	1.11	2.53	7.59	1.00	12	71.58	1.12	56.59	0.89
	BS-L-V2T10	4.73	75.4	89.70	10.81	0.98	3.14	9.41	1.13	12	95.27	1.06	53.56	0.60
	BS-L-V4T10	4.1	75.4	90.60	11.61	0.98	3.14	9.41	1.21	12	94.69	1.05	49.87	0.55
	JA-NN03	6.5	75.4	68.90	6.22	0.93	3.40	10.20	0.65	12	69.30	1.01	62.79	0.91
	JA-NN15	6.67	75.4	69.90	6.14	0.93	3.40	10.20	0.64	12	69.44	0.99	63.60	0.91
	JB-NN03	6.87	75.4	70.40	6.24	0.79	4.00	12.00	0.57	12	73.32	1.04	64.55	0.92
[55]	SP1	3.5	68	155.70	5.91	0.79	3.20	15.00	0.54	15	146.08	0.94	108.62	0.70
	SP2	3.53	68	228.70	10.57	0.79	3.20	15.00	0.77	15	211.31	0.92	109.08	0.48
	SP3	3.6	68	131.30	5.37	1.03	2.33	15.00	0.54	15	109.03	0.83	110.16	0.84
	SP4	3.96	68	167.60	9.20	1.03	2.33	15.00	0.73	15	152.41	0.91	115.54	0.69
Average											1.02		0.72	
Standard Deviation											0.11		0.156	

SIj = joint shear index [1], θ = from equation 2, x1 and x2 are parameters used in [1] to calculate joint resistance, and γ = coefficient based on [3].

Source: Based [1], [3], [54], [55].

For the test results analyzed, the method proposed presents satisfactory agreement with the test results analyzed. The average value obtained indicated a 2% difference and a standard deviation of about 55.05 kN (12.38 kips) for the specimens studied in this paper ranged [1017 kN (63.80 kips) – 3512 kN (228.7 kips)]. The effect of the slab reinforcement was not incorporated in the calculation of the shear index in order to be consistent with the other test results analysed by [54]. It is important to mention that taking into account the uncertainties associated with the materials, test measurements, test setup, and the tolerances, this method can satisfactorily predict the joint shear capacity. Regarding the cyclic behavior of the BCJ, Fig. 6 depicts the obtained displacement response (depicted in red) for the specimen SP1 EW [1]. Note that the Pinching4 model compares satisfactorily to the experimental results depicted in blue. For this specimen, the EW yielded first in the downward direction. Thus, in this direction the envelope, initial stiffness, strength degradation, reloading stiffness, pre-capping, and post-capping capacity match satisfactorily the test results. For the upward loading, the results indicate a minor discrepancy which can be attributed to the previous yielding in the negative direction, and is also due to the degradation produced for the loading in the NS direction as the SP1 specimen was tested as a 3D BCJ. Despite that the unloading stiffness parameters are fixed based on the recommendations made in [55], it appears that after the five cycles the unloading degradation stiffness is in some way mismatched, but other parameters are in satisfactory agreement with the test results. While recognizing some of these inherent limitations, the Pinching4 model was selected to represent the cyclic degradation of the joint shear spring.



a) Cycle 3 b) Cycle 4 c) Cycle 5

d) Cycle 6 e) Cycle 7 f) Cycle 8

Fig. 6. Obtained load-displacement response SP1 EW.
Source: Authors.

IV. Summary and conclusions

BCJ beam column joints are key elements in the seismic performance and integrity of RC frames. The response of the joint region is governed by shear, which are transmitted by bearing, bond, and friction. Shear cracking, when not properly controlled, induces brittle failures. Unreinforced BCJ failure can trigger the collapse of a RCF. The failure is presented due to the limited capacity of the joint to transmit the forces and to keep its integrity, without regard the appropriate overcapacity and ductility of the beams and columns that frame to the joint.

The proposed model is based on [1]. In essence, the model was modified to follow the same nomenclature of [2]. The proposed model was validated with various test results of unreinforced BCJ well documented in the literature. In this study, tests by [54] and [55] were used to compare with the analytical results. The validation of the model shows a good agreement when comparing with tests results. It was shown that average value obtained indicated a 2% difference and a standard deviation of about 55.05 kN (12.38 kips) for the specimens studied ranged [1017 kN (63.80 kips) – 3512 kN (228.7 kips)].

From Table 1 is clearly appreciated that the provisions [56] tend to underestimate the joint strength by about 30% with a standard deviation of 113 kN (25.4 kips) for the specimens studied in this paper ranged [1017 kN (63.80 kips) – 3512 kN (228.7 kips)]. The provisions [56] tend to have a high level of conservatism and have little success in representing the true behavior of the BCJ.

Ir was shown that the cyclic behavior of the BCJ can be satisfactorily captured by using the proposed model. The proposed model needs no special software or element for implementation. It is believed that any motivated reader can easily include the BCJ flexibility in modelling the nonlinear response of RCF.

The proposed approach is considered a practical-oriented approach easy to understand and implement with and appropriate level of accuracy. Even though it is required more refinement in order to represent the true geometry and complex kinematic behavior of the BCJ, it is assumed that the procedures presented here will contribute in the incorporation of the unreinforced BCJ flexibility when modeling older-type RCF construction.

The model discussed in the present paper can be further investigated and validated (i) for different types of joints, (ii) for joints with varying axial load, and (iii) for 3D BCJ simulations.

References

[1] S. Park & K. Mosalam, "Simulation of Reinforced Concrete Frames with Nonductile Beam-Column Joints," *Earthq Spectra,* vol. 29, pp. 233–257, Feb. 2013. [Online]. https://doi.org/10.1193/1.4000100

[2] ACI-ASCE Committe 352, Recommendations for design of beam-column connections in monolithic reinforced concrete structures, *ACI 352R-02*, Farmingtn Hls, USA: ACI, 1991.

[3] ACI-ASCE Committe 318, *Building Code Requirements for Structural Concrete*, *ACI-318-89*, Farmington Hills, USA: ACI, 1989.

[4] R. Clough, K. Benuska & E. Wilson, "Inelastic earthquake response of tall buildings," in *Proceedings of the Third World Conference on Earthquake Engineering, WCEE,* Auckland, NZL, 1965, pp. II-68 to II-89.

[5] T. Takeda, M. A. Sozen & N. N. Nielsen, "Reinforced concrete response to simulated earthquakes", Ohbayashi-Gumi Tech. Rsrch. TYO, JP, *Rep, no. 5*, Jan. 1970.

[6] M. Saiidi & M. Sozen, "Simple and complex models for nonlinear seismic response of reinforced concrete structures", *Struct. Rsrch. Ser,* no. 465, Aug. 1979.

[7] A. Ilki & N. Kumbasar, "Theoretical and experimental energy dissipation of reinforced concrete," in *Proceedings of the 3rd Japan-Turkey Workshop on Earthquake Engineering*, JICA, Istanbul, TR, 2000, vol. 1, pp. 167–177.

[8] H. L. Feghali, "Seismic performance of flexible concrete structures," *Dissertation Thesis*, ProQuest UMI Dissertations, Ann Arbor, MI, 1999.

[9] R. W. Clough & S. B. Johnston, *Effect of Stiffness Degradation on Earthquake Ductility Requirements.* Hoboken, USA: Wiley, 1966.

[10] L. F. Ibarra, R. A. Medina & H. Krawinkler, "Hysteretic models that incorporate strength and stiffness deterioration", *Earthq. Eng. Struct. D,* vol. 34, pp. 1489–1511, Jan. 2005. https://doi.org/10.1002/eqe.495

[11] D. G. Lignos & H. Krawinkler, "*Deterioration Modeling of Steel Components in Support of Collapse Prediction of Steel Moment Frames under Earthquake Loading,*" *J. Struct. Eng, ASCE,* vol. 137, no. 11, pp. 1291–1302, Nov. 2011. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000376

[12] M. F. Giberson, "The response of nonlinear multi-story structures subjected to earthquake excitation," *Dissertation Ph.D*, Caltech, Pasadena, CA, 1969.

[13] S. Otani, "Behavior of Multistory Reinforced Concrete Frames During Earthquakes," *Dissertation Thesis*, Croquet UMI Dissertations, Ann Arbours, MI, 1973.

[14] J. C. Anderson & W. H. Townsend, "*Models for RC Frames with Degrading Stiffness*," *J. Struct. Div,* vol. 103, no. 2, pp. 236–2376, Jan. 1977.

[15] F. C. Filippou, E. P. Popov & V. V. Bertero, "Modeling of R/C Joints under Cyclic Excitations," *J. Struct. Eng,* vol. 109, no. 11, pp. 2666–2684, 1983. https://doi.org/10.1061/(ASCE)0733-9445(1983)109:11(2666)

[16] S. E. El-Metwally & W. F. Chen, "Moment-rotation modeling of reinforced concrete beam-column connections," *ACI Struct. J,* vol. 85, no. 4, pp. 384–394, May. 1988. https://doi.org/10.14359/2656

[17] M. T. De Risi, P. Ricci & G. M. Verderame, "Modelling exterior unreinforced beam-column joints in seismic analysis of non-ductile RC frames," *Earthq. Eng. Struct. D,* vol. 46, pp. 899–923, Nov. 2016. https://doi.org/10.1002/eqe.2835

[18] N. Mitra, "An analytical study of reinforced concrete beam-column joint behavior under seismic loading," *Dissertation Thesis*, UW, WA, USA, 2007.

[19] A. Sharma, R. Eligehausen & G. R. Reddy, "A New Model to Simulate Joint Shear Behavior of Poorly Detailed Beam–Column Connections in RC Structures under Seismic Loads, Part I: Exterior Joints," *Eng. Struct,* vol. 33, no. 3, pp. 1034–1051, Mar. 2011. https://doi.org/10.1016/j.engstruct.2010.12.026

[20] A. Ghobarah & A. Biddah, "Dynamic analysis of reinforced concrete frames including joint shear deformation," *Eng. Struct,* vol. 21, no. 11, pp. 971–987, Jan. 1999. https://doi.org/10.1016/S0141-0296(98)00052-2

[21] T. Hsu, "Softened truss model theory for shear and torsion," *ACI Struct. J,* vol. 85, no. 6, pp. 624–634, May. 1988. https://doi.org/10.14359/2740

[22] T. Kaku & S. Morita, "Bond behavior of beam bars in the joint of reinforced concrete frame," in *Annual Meeting of the Architectural Institute of Japan,* AIJ, TYO, JP, Sep. 1978, pp. 1777–1778.

[23] F. J. Vecchio & M. P. Collins, "The modified compression-field theory for reinforced concrete elements subjected to shear," *ACI,* vol. 83, no. 2, pp. 219–231, Mar. 1986. https://doi.org/10.14359/10416

[24] Y. S. Chung, C. Meyer & M. Shinozuka, "Seismic damage assessment of reinforced concrete members". SUNY at Buffalo, Buffalo, NY, USA, *Tech. Rep. NCEER-87-0022*, Oct. 1987.

[25] A. Biddah & A. Ghobarah, "Modeling of shear deformation and bond slip in reinforced concrete joints", *Struct. Eng. Mech*, vol 7, no. 4, pp. 413–432, Apr. 1999. https://doi.org/10.12989/sem.1999.7.4.413

[26] R. Eligehausen, E. Popov & V. Bertero, "Local bond stress slip relationship of deformed bars under generalized excitations", PEER, CALTECH, Covina, USA, *Tech. Rep. UCB/EERC*, 1983.

[27] F. Filippou, "A simple model for reinforcing bar anchorages under cyclic excitations," *J. Struct. Eng. ASCE,* vol. 112, no. 7, pp. 1639–1659, Feb. 1986. https://doi.org/10.1061/(ASCE)0733-9445(1986)112:7(1639)

[28] M. Elmorsi, M. R. Kianoush & W. K. Tso, "Modeling bond-slip deformations in reinforced concrete beam-column joints," *Can. J Cit. Eng,* vol. 27, no. 3, pp. 490–505, Jun. 2000. https://doi.org/10.1139/l99-085

[29] E. Giuriani, G. Plizzari & C. Schumm, "Role of stirrups and residual tensile strength of cracked concrete on bond," *J. Struct. Eng. ASCE*, vol. 117, no. 1, pp. 1–18, Jan. 1969. https://doi.org/10.1061/(ASCE)0733-9445(1991)117:1(1)

[30] D. Kent & R. Park, "Flexural members with confined concrete," *J. Struct. Div, ASCE*, vol. 113, no. 10, pp. 2160–2173, Aug. 1971.

[31] M. Youssef & A. Ghobarah, "Modelling of RC beam-column joints and structural walls," *J. Earthq. Eng,* vol. 5, no. 1, pp. 93–111, Jan. 2001. https://doi.org/10.1080/13632460109350387

[32] Y. Park & A. Ang, "Seismic damage analysis of reinforced concrete buildings," *J. Struct. Eng*, vol. 111, no. 4, pp. 740–757, Apr. 1985. https://doi.org/10.1061/(ASCE)0733-9445(1985)111:4(740)

[33] L. N. Lowes & A. Altoontash, "Modeling Reinforced-Concrete Beam-Column Joints Subjected to Cyclic Loading," *J. Struct. Eng,* vol. 129, no. 12, pp. 1686–1697, Dic. 2003. https://doi.org/10.1061/(ASCE)0733-9445(2003)129:12(1686)

[34] M. Rahnama & H. Krawinkler, "Effects of soft soil and hysteresis model on seismic demands," John A. Blume Earthquake Engineering Center, SU, USA, *BLUME-108*, 1993.

[35] A. Altoontash, "Simulation and damage models for performance assessment of reinforced concrete beam-column joints," *Dissertation Thesis*, ProQuest UMI Dissertations, Ann Arbor, MI, Aug. 2004.

[36] S. Morita & T. Kaku, "Bond slip relationship under repeated loading," *Transactions of the AIJ*, vol. 229, pp. 15–24, Jan. 1975. https://doi.org/10.3130/aijsaxx.229.0_15

[37] R. Park, M. Priestley & W. Gill, "Ductility of square confined reinforced concrete columns," *J. Struct. Div. ASCE*, vol. 108, pp. 929–950, Jan. 1982.

[38] J. LaFave & M. Shin, "Discussion of "Modeling reinforced concrete beam-column joints subjected to cyclic loading" by L. N. Lowes & A. Altoontash," *J. Struct. Eng. ASCE*, vol. 131, no. 6, pp. 992–993, Jun. 2005. https://doi.org/10.1061/(ASCE)0733-9445(2005)131:6(992)

[39] S. Tajiri, H. Shiohara & F. Kusuhara, "A new macroelement of reinforced concrete beam column joint for elasto-plastic plane frame analysis," in *8th National Conference of Earthquake Engineering, NCEE*, SFO, USA, 2006, no. 1.

[40] F. McKenna, "OpenSees: A Framework for Earthquake Engineering Simulation," *Comp. Sci. Eng.,* vol. 13, no. 4, pp. 58–66, Jun. 2011. https://doi.org/10.1109/MCSE.2011.66

[41] P. Shin & J. LaFave, "Testing and Modeling For Cyclic Joint Shear Deformations in RC Beam-Column Connections," in *13th World Conference on Earthquake Engineering, WCEE,* Vancouver, CA, Aug. 2004, Paper 0301.

[42] F. Kusuhara, H. Shiohara & S. Tajiri, "A New Macro Element of Reinforced Concrete Beam-Column Joint for Elasto-Plastic Plane Frame Analysis," in *8th U.S. National Conference on Earthquake Engineering*, NCEE, SFO, USA, Apr. 2006, Paper 674.

[43] T. Saito & M. Kikuchi, "A New Analytical Model for Reinforced Concrete Beam-Column Joints Subjected to Cyclic Loading" in *15th World Conference on Earthquake Engineering, WCEE,* Lisboa, PT, Sep. 2012.

[44] P. Zhang, S. Hou & J. Ou, "A beam–column joint element for analysis of reinforced concrete frame structures," *Eng. Struct,* vol. 118, pp. 125–136, Jul. 2016. https://doi.org/10.1016/j.engstruct.2016.03.030

[45] L. N. Lowes, A. Altoontash & N. Mitra, "Closure to "Modeling Reinforced-Concrete Beam-Column Joints Subjected to Cyclic Loading" by Laura N. Lowes and Arash Altoontash," *J. Struct Eng,* vol. 131, no. 6, pp. 993–994, Dec. 2003.

[46] O. C. Celik, "Probabilistic Assessment of Non-Ductile Reinforced Concrete Frames Susceptible to Mid-America Ground Motions," *Dissertation Thesis*, Georgia Tech, Atlanta, GA, USA, 2007.

[47] O. C. Celik & B. R. Ellingwood, "Seismic fragilities for non-ductile reinforced concrete frames - Role of aleatoric and epistemic uncertainties," *Struct. Saf,* vol. 32, no. 1, pp. 1–12, Jan. 2010. https://doi.org/10.1016/j.strusafe.2009.04.003

[48] W. M. Hassan, "Analytical and Experimental Assessment of Seismic Vulnerability of Beam-Column Joints without Transverse Reinforcement in Concrete Buildings," *Dissertation Thesis*, UCB, Berkeley, CA, USA, 2011.

[49] J. S. Jeon, R. DesRoches, I. Brilakis & L. Lowes, "Modeling and Fragility Analysis of Non-Ductile Reinforced Concrete Buildings in Low-to-Moderate Seismic Zones," in *ASCE Structures Congress,* ASCE, Chicago, USA, 2012, vol. 2199. https://doi.org/10.1061/9780784412367.193

[50] S. Park & K. Mosalam, "Analytical Model for Predicting Shear Strength of Unreinforced Exterior Beam-Column Joints," *ACI Struct J,* vol. 109, no. 2, pp. 149–159, Mar. 2012. https://doi.org/10.14359/51683626

[51] F. Charney & R. Johnson, "The Effect of the Joint Deformations on the Drift of the Steel Frame Structures," *KKBNA, Inc., Consulting Engineers,* Oct. 1986.

[52] A. C. Birely, L. N. Lowes & D. E. Lehman, "A model for the practical nonlinear analysis of reinforced-concrete frames including joint flexibility," *Eng. Struct,* vol. 34, no. 1, pp. 455–465, Jan. 2012. https://doi.org/10.1016/j.engstruct.2011.09.003

[53] S. Park & K. Mosalam, "Parameters for shear strength prediction of exterior beam–column joints without transverse reinforcement," *Eng. Struct,* vol. 36, pp. 198–209, Mar. 2012. https://doi.org/10.1016/j.engstruct.2011.11.017

[54] H. F. Wong, "Shear strength and seismic performance of non-seismically designed reinforced concrete beam-column joints," *Dissertation Thesis*, ProQuest UMI Dissertations, Ann Arbor, MI, 2005.

[55] S. Park, "Experimental and Analytical Studies on Old Reinforced Concrete Buildings with Seismically Vulnerable Beam-Column Joints," *Dissertation Thesis*, ProQuest UMI Dissertations, Ann Arbor, MI, 2010.

[56] ASCE 41, *Seismic Rehabilitation of Existing Buildings.* Reston, USA: ASCE, 2006.

Daniel Ricardo Salinas Guayacundo received the M.Sc. degree in electrical engineering and the Ph.D. degree in solid state physics from the Technical University of Denmark, Copenhagen, Denmark, in 1974 and 1977, respectively. Since then, he has been with Danfoss Compressors GmbH, Flensburg, Germany, where he is currently the Senior Director of Technology and R&D. https://orcid.org/0000-0002-1257-5747

Roberto Tomas Leon Saenz received de David H. Burrows Professor. Obtuvo su titulo de Ph.D., Civil Engineering, University of Texas at Austin, 1983. M.S.C.E., Structural Engineering, Stanford University, 1979. B.S.C.E., Civil Engineering, University of Massachusetts, Amherst, 1978. Ostenta los siguientes professional Honors, Awards, y reconocimientos: Structural Engineering Institute (ASCE) President's Award, 2012 AISC Special Achievement Educator Award, 2011 ASCE Norman Medal, 2000 ASCE State-of-the-Art Award 1996 and 2000 Fellow, American Concrete Institute, 1999 AISC T.R. Higgins Award, 1993 ASCE E.E. Friedman Young Engineer Award, 1988.