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A review on the current trends on computational modelling of masonry-infilled reinforced concrete frames

M Shadlou¹, M M Kashani¹

¹ University of Southampton, Faculty of Engineering and Physical Sciences, Southampton, SO17 1BJ, UK

Abstract. A review of methods applicable to the study of masonry-infilled reinforced concrete frames (MIRCF), consisting both traditional and advanced solutions, and located in seismic zones is presented in this research. Firstly, this research presents a brief discussion about the main challenges on modelling the RC frames, masonry infills and interaction between them for structures located in seismic zones. Then, simplified and sophisticated approaches, which are actually used or developed for modelling both RC frames and masonry infills recently, are discussed. The main available strategies including simplified methods, and sophisticated finite element solution are considered with regard to their realism, computer efficiency, data availability and real applicability to large structures.

1. Introduction

MIRCFs have been called as most common type of buildings in earthquake prone area. Seismic behaviour of these buildings is very complex because of natures of the constituents and loading conditions. This complexity may even become worst by possible interaction between infilled masonry and surrounding frame. As a result, dynamics of the system containing infilled masonry and surrounding reinforced concrete frames is affected by the material behaviour of the constituents which are nonlinear even at small loading steps.

Masonry is made by unit and mortar creating a composite material. Largest material variations are observed in mortar joints and specifically at brick-mortar interfaces. During the last 30 years several micro-physical constitutive models have been developed for masonry joints (mortar joints and brick-mortar interfaces) and applied to sophisticated numerical solutions such as nonlinear finite elements methods ([1], [2], [3], [4], [5], [6], [7]). All models presented in the literature are two-dimensional (2D) constitutive models calibrated for bed-joints while the needs for 3D modelling are essential in practice.

Phenomenological constitutive laws are also developed for masonry material, considered as homogenized medium and it is called macroscopic model. In macro-model or macroscopic model, there is no distinction between brick, mortar and brick-mortar interfaces. As a result, the masonry panel is modelled as a homogeneous-continuum-anisotropic-or-orthotropic material. Masonry stress-strain relationships are derived by performing tests on masonry, without distinguishing between bricks and mortars behaviour. As a result, a relationship between average stresses and strains in masonry material is established by macro-models. Macroscopic model would not be suitable to describe accurately some of the micromechanics occurring during the damage evolution but it is very effective from computational point of view. Some phenomenological models based on plasticity, damage theory, and non-tension assumptions are proposed in the vast body of literature.



Experimental results also show that masonry-infilled frames can experience a wide variety of the failure mechanisms such as those suggested by [8], [9], [10], [11]. More detail can be found in [11] and [12].

Dynamic and seismic assessment of MIRCFS is a challenge due to the nonlinearity of the constituents. This challenge can also become worst by considering local or global damage into the structure. This research aims to show the previous and recent contributions in the field of computational modelling of masonry material and MIRCFS. It contains seven sections concentrating on different main approaches and providing a novel and short literature review on existing methods.

2. Micro-modelling of mortar joints and brick-mortar interface

Tension and shear are governing mode of failure in mortar joints and brick-mortar interface so they need to be discussed more than compression mode. Combinations of above failure modes was discussed by Mann and Mueller ([13]). Van der Pluijm ([14]) presented the most complete experimental study of tension and shear behaviour carried out at the University of Eindhoven.

Tensile failure in mortar joints and brick-mortar interfaces is preceded by micro-cracking that increases in size and number. The micro-cracks are localized as the deformation increases and finally merging to one actual crack. As a result, crack is developed between the aggregate and the cement paste by stress concentration on the weakened part.

In order to obtain an insight into the response of mortar joints and brick-mortar interfaces under shear or distortional loading, a microscopic representation of micro-mechanical constitutive model requires the knowledge of inter-particle cementing. Hence inter-particle cementing in the context of cementation of granular materials are firstly required and then the behaviour of mortar joints and brick-mortar interface under distortional loading is needed. Shear behaviour of bed joints was being the subject of most of the researches and can be found extensively in the literature.

Inter-particle cementing supplies extra strength and stiffness to bonded geo-material, allowing them to withstand at much higher void ratio than the equivalent reconstituted geo-material at a given stress ([15]). This allows geotechnical engineers to consider reconstituted geo-material as a benchmark for the comparison of mechanical behaviour of corresponding bonded geo-materials (e.g. [16]). Another important consequence of inter-particle cementing is that the influence of confining pressure on the maximum shear modulus decreases as the level of cement-sand ratio increases. If there is sufficient inter-particle bonding generated by higher amount of cement such as those in mortar, confining stress does not have a significant effect on the shear modulus, unless it exceeds the stress necessary to break the inter-particle cementing bonds ([17]). On the other hand, density has a larger effect on small strain shear modulus of cemented sand than the confining pressure in reconstituted sands.

Owing to the inter-particle cementing in the vast body of literature, it is derived that the volume of the sample increases after yielding point in distortional loading (so called delayed dilatancy) of mortar joint or brick-mortar interface. It is derived that above state may be the onset of in-plane failures for a mortar joints or brick-mortar interface, or starting point of additional loading mechanism into other mortar joints (e.g. head joints and bed joints) surrounding the bricks.

A direct shear testing is a technique for evaluating the shear failure behaviour of mortar joints and brick-mortar interfaces, (see for example: [18], [19], [20], [21],[22], [23], [24]). In couplet test and triplet tests, a masonry element containing brick and mortars are loaded by a combined normal force perpendicular to the bed joint plane and shear force parallel to the bed joint plane. The triplet test is the standard test for the evaluation of the shear strength of masonry joints ([25]). Experimental results carried out by Atkinson et al. ([19]) and van der Pluijm ([20]) showed that dilatancy is considered as a function of the plastic relative tangential displacement and the normal stress. It was demonstrated that, by increasing the compressive stress and the total plastic strain, a transition from the joint dilation to joint compaction takes place, which would require a complex formulation to express the dependence of the dilatancy on the compressive stress ([26]) and on the strain history [3].

As derived from aforementioned notes, a computational model to capture this phenomenon accurately is essential for numerical simulations. Meanwhile due to the dilation (volume increase) of the mortar joints, diagonal cracking of the infill panel usually goes through the bed joints and head joints and, as

a result, cracking sometimes occurs through the bricks. To capture this mechanism in numerical analysis, some special techniques are required for modelling crack in masonry units ([2]).

Available constitutive relations, for mortar joints and brick mortar interface, in the literature can be categorized into four general methods as (i) plasticity ([1], [4], [27], [21], [28]), (ii) fracture energy ([29], [30], [31], [32]), (iii) damage theory ([3], [33], [5]), and (iv) thermodynamic laws, and coupling of adhesion, friction, and unilateral contacts ([34], [22], [35], [36], [37]). There are evidently compromises to be made between number of material parameters, number of experimental data required for calibration, and accuracy of the model for different loading paths.

3. Macro-modelling of masonry panel

When the masonry is subjected to progressive deformation, the crack may growth in mortar, brick, or mortar-brick interface depending on stress, stiffness and the initial micro cracks. The failure of a masonry panel may be either brittle or ductile depending on confinement pressure. Typically, the brittle failure is mostly observed on the masonry panel subjected to tension, compression, or shear stresses. Tensile failure has been known as the growth of micro cracks in mortar, bricks and mortar-brick interfaces, while the compression failure is a function of boundary conditions and size effects. Degradation of cohesion and friction angle is also specified as two reasons for shear failure.

To distinct failure modes of a masonry panel, principle stresses and orientation of principle stress in respect to the bed joints or head joints are usually evaluated. Owing to experiments carried by [38], [39] and [40]; when the masonry panel is subjected to uniaxial compression, failure occurs in a plane normal to the plane of the panel. Depending on the orientation of the bed joints to the applied loading, failure may occur by cracking, and by sliding on the orientation of the bed and/or header joints; or in a combined mechanism involving both brick and mortar ([40]). In the case of biaxial compression, failure typically occurs by splitting in a plane parallel to the free surface of the specimen at mid-thickness, regardless of the bed joint angle. On the other hand, bed joint orientations play a significant role when principle stress ratio is high enough. In this case, failure occurs in combined mechanism involving both joint failure and lateral splitting. In biaxial tension-compression, both bed joint angle and principle stress ratio considerably influence the failure mode ([40]).

Based on the published data on the failure of masonry under plane stress condition, failure modes are generally categorized in Figure 1. To summarize them, possible failure patterns in a masonry panel would be categorized as: (1) straight-vertical crack along head joints and bricks (i.e. Figure 1-a Figure 1-h), (2) straight-horizontal crack along the bed joints due to tension, compression or shear loads (i.e. Figure 1-c, Figure 1-e, Figure 1-g, Figure 1-j), (3) tooting crack along bed and head joints (i.e. Figure 1-b), (4) stepped-diagonal crack with shear along the bed joints (i.e. Figure 1-i, Figure 1-k), (5) straight-diagonal crack (i.e. Figure 1-l), (6) combination of stepped and straight-diagonal cracks (i.e. Figure 1-f), (7) compressive crack (i.e. Figure 1-d), (8) splitting crack parallel to the external surfaces (i.e. Figure 1-m).

The most complete set of strength data of biaxially loaded masonry is given by [38] and [39]. The strength of masonry panel varies in different principle stress ratio and bed joint orientations with respect to principle stress. According to [41], the compressive strength of masonry, neglecting tensile strength, can be deduced by: (a) tensile failure of masonry, (b) compression failure of masonry, (c) shear failure of masonry, (d) sliding along the bed joints, and (e) tensile failure along the bed joints. Mojsilovic ([42]) extended the Ganz's proposed mechanisms by considering: (f) slip failure along the head joints. This helps to obtain the compressive strength of masonry as a function of bed joints orientation (θ) and principle stress ratio (σ_1/σ_2). Hence the uniaxial compressive strength can then be used to check the ultimate limit state in a shear wall by the lower bound theorem of plasticity.

Failure criteria for unreinforced masonry subjected to in-plane loading have been subjected to previous researchers. One of early developed failure criteria was suggested by [13] who applied the modified Mohr-Coulomb failure criterion on in-plane loaded masonry and distinguished four different failure types.

Developing a general failure criterion for masonry panel has been the subject in the vast body of literature. The early studies for taking account micro-modelling approach into obtaining the homogenized failure criterion was carried out by [43] and [44]. In above studies, a failure criterion for

the brick masonry panels was derived in terms of the normal, parallel and joint shear stresses. As suggested, the failure surface consisted of three elliptic cones in the normal, parallel and joint shear stress space in the form of following:

$$C_1\sigma_x^2 + C_2\sigma_y^2 + C_3\tau^2 + C_4\sigma_x\sigma_y + C_5\sigma_x + C_6\sigma_y + 1 = 0 \quad (1)$$

Where C_i , $i = 1, \dots, 6$, are material constants.

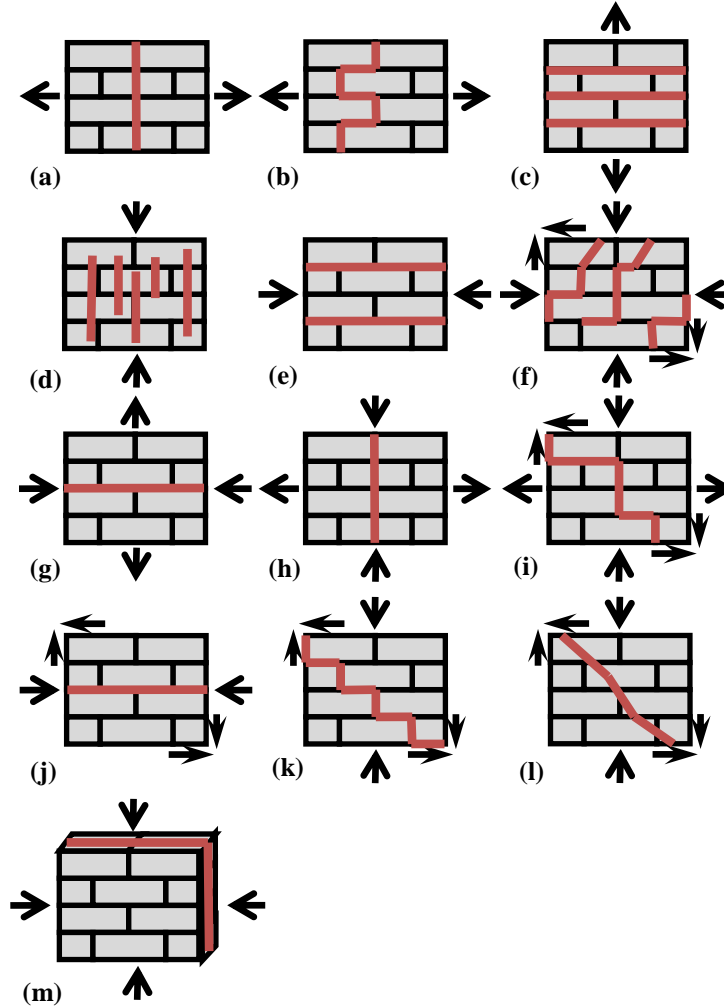


Figure 1. Failure patterns of masonry in plane stress state.

To check the applicability of smeared crack models into the homogenous unreinforced and reinforced masonry, Lotfi and Shing ([45]) combined non-hardening von Mises (J2) plasticity, for un-cracked masonry, with orthotropic model for cracked masonry incorporating single crack model of fixed, coaxial rotating and non-coaxial rotating crack representations. Elastic-plastic plane stress model was based on the von Mises yield criterion and an associated flow rule, combined with a Rankine type tension cut-off. Owing to this model, tensile cracking occurs when the tension cut-off surface is reached; this transforms the material behaviour from elastic-plastic to nonlinear orthotropic with the axes of orthotropy parallel and perpendicular to the crack. For the case of fixed crack, both the crack direction and the axes of orthotropy remain constant while, for rotating crack, the crack direction and the axes of orthotropy rotate with the principle axes of strain in such a way that the crack remains normal to the direction of the maximum principle strain. Post crack shear modulus (G_{nt}), which is generally intended to model the residual cohesion and aggregate-interlock forces, is modeled by a

shear retention factor that varies from one to zero. The shear retention factor is assumed constant for fixed crack model and as a nonlinear function of tensile strain for coaxial rotating crack model.

The performance of the finite element model may also be evaluated with experimental data obtained by diagonal compression test ([46]) on a masonry panel. The load-displacement curve obtained from the numerical analysis and experiment were used for the assessment. It was shown that both the fracture strength and failure mode were well captured in the numerical solution. Lotfi and Shing ([45]) showed that the smeared-crack model can accurately capture the flexural failure of a reinforced masonry wall. However, they showed that the brittle shear behaviour of the wall coming from the diagonal cracking cannot be captured properly for lightly reinforced wall panels using this method.

A practical method for capturing all possible modes in masonry ensures that the orthotropic constitutive laws must be developed, and, as a result, different failure criteria are used for shear, compression and tension behaviour. One of early successful macro models for modelling masonry is the plasticity model presented by [2]. A composite yield criterion suitable for modelling of anisotropic materials under plane stress conditions were presented, in which, the tension and compression failure mechanisms were associated with different yield criteria related to cracking and crushing of the material, respectively. It is also called as an extension of conventional formulations for isotropic quasi-brittle materials to describe orthotropic behaviour. Hence the presented composite yield criterion by [2] can be thought as an extension of the composite plasticity model developed by [47]. This constitutive model requires 12 material parameters which are divided into two categories as: (1) strength parameters; containing 4 uniaxial tension and compression strengths and 3 auxiliary parameters controlling shear stress contributions to tension and compression failure and coupling between normal stress values, (2) inelastic parameters; containing 4 independent fracture energies and one peak strain in compression. To calibrate the aforementioned model and create the composite yield surface, Lourenco et al. ([48]) suggested 7 experiments containing 4 uniaxial tension and compression tests and 3 biaxial tests. Lourenco ([2]) assessed the credibility of the proposed composite model, on generating the strength yield surface, with different experimental data carried out by (i) Page ([38], [39]) on biaxially loaded masonry panel containing solid clay brick masonry, (ii) Ganz and Thurliman ([49]) for hollow clay brick masonry, and (iii) Lurati et al ([50]) for hollow concrete block masonry. The results of reproducing the composite yield surface presented promising conclusions.

A macro-model (homogeneous) finite element model was also developed by [51] to represent local response of masonry such as cracking, crack propagation, and crushing. The uniaxial constitutive model was used in a biaxial stress state by using an equivalent uniaxial strain concept which was originally developed for concrete ([52]). It is assumed that the masonry is regarded as isotropic material under biaxial tension-compression or uniaxial tension and the nonlinearity is not taken into account, but an anisotropic material model were developed for biaxial compression-compression or uniaxial compression. The failure surface for compression failure mechanisms was assumed by equation presented by Naraine and Sinha ([53]) considering principle stress invariants obtained by normalized principle stresses and uniaxial compression strengths. The drawbacks of the model presented by [51] are counted as: (1) it is only available for biaxial failure criterion and the shear failure in masonry joint is ignored, (2) biaxial tension-compression range of failure criterion is ignored.

To obtain a general failure surface (criterion) for masonry, some attempts have also been made by other researchers. Because of difficulties in developing a representative biaxial tests and the large number of tests involved, there have been few research. Elliptic cones were suggested by [54], [55], [56], and [57], while a sophisticated convex shaped failure surface was developed by [58] using the cubic tensor polynomials and least square approach over the experimental data. In another attempt, [59] adopted a critical plane approach ([60]), which is also called as an extension of nonlinear Coulomb failure, to formulate a macroscopic failure criterion for structural masonry. This approach involves the notation of the existence of a critical plane, or the localization plane, on which the failure function reaches a maximum.

A simple micro-mechanical model used by [61] to obtain the homogenized failure surface. Adopting a polynomial expansion for 2D stress field, linear optimization problem is derived by an iterative procedure on the elementary cell. Different mechanical characteristics for the mortar and actual

thickness of the joints were considered to evaluate differences in the homogenized failure surface. As noted by [61], mechanical characteristics adopted for the mortar joints play significant role on the resultant failure surface.

Kawa et al. ([62]) assessed the strength of brick masonry based on homogenization theory and lower bound analysis, and found the material functions for utilizing in the critical plane approach ([60]) and predicting the directional strength characteristics of masonry panel.

In another attempt made by [63], a simple rigid-plastic homogenization model was developed for obtaining out-of-plane anisotropic masonry failure surfaces as bricks and mortar joints were assumed rigid perfectly plastic and postulating an associated flow rule.

4. Other simplified methods for modelling masonry infilled frames

Summarizing experimental observations carried out on MIRCFs can help us to have a better understanding of the behaviour of MIRCFs in seismic area. On the other hand, actual manifestation regarding response prediction of the MIRCFs in the future earthquakes can be observed in the full scale or small scale tests. A large number of monotonic, cyclic, dynamic and shaking table experiments have been carried out in the last 60 years. Comparative evaluations of commonly used methods for masonry panels and masonry infilled frames subjected to seismic loading was reported by [64], [65], and [66].

Polyakov ([67]) performed some experiments and he proposed a compression strut model (Figure 2.a) in order to compute the maximum strength and stiffness, and deflection at failure. Similar simplified concepts have been used by many other experimental studies. This simplified, yet applicable, concept was evaluated by full scale or small scale experiments and in some cases proposed for some modifications into its original concept by multi-struts for better modelling. As derived from previous experiments, masonry infilled frames will have higher stiffness, strength, damping ratio and lower ductility and natural period than bare frames, hence the response of the system will be influenced by above changes and it is expected that MIRCFs experience lower performance points in the concept of performance based design (PBD). This has also been concluded by [12] using numerical analysis by the method of incrementally dynamic analysis (IDA) as; bare frames showed the better collapse performance compared to the partially and fully infilled frames. Well-accepted issue related to diagonal strut models is related to the weak interaction between infill masonry and bonded frame.

There is common practice to study the seismic performance of masonry buildings and masonry-infilled frames using macro-elements instead of modelling the detailed constituents (micro-modelling). One of simplified macro-elements was suggested by [68] using rigid body spring model ([69]) and a simplified heuristic approach (Figure 2.b). Above method has one drawback which is related to model implementation difficulties ([66]).

Another simplified method for analysing masonry walls and MIRCFs is provided by combination of advantages of finite element and discrete element methods ([70]). In this technique, masonry units are connected to each other using springs. Constitutive models related to the behaviour of the masonry panel are addressed by normal and tangential springs (Figure 2.c). Most advantage of this method is its ability to simulate large displacements the may take place between units through successive failure.

Another promising technique for simulating the response of masonry panel is so called 'equivalent frame model' in which the masonry cell/panel is represented by equivalent nonlinear frame structure (Figure 2.d and see for example: [64], [65], [71]). Accordingly, each masonry panel is modelled by macro-elements representing piers, spandrels and rigid zones. One of the most important issues related to equivalent frame model is addressed by modelling of spandrels, in which strength criteria may be underestimated. On the other hand, inaccurate interaction between macro-elements would also be addressed by equivalent frame model. The readers are referred [72] in which a critical review on advantages and limitations of equivalent frame model was made. In addition to above simplified solutions, plane discrete elements (see for example [73]) are used in which macro elements consist of articulated quadrilateral with rigid edges including diagonal and shear springs.

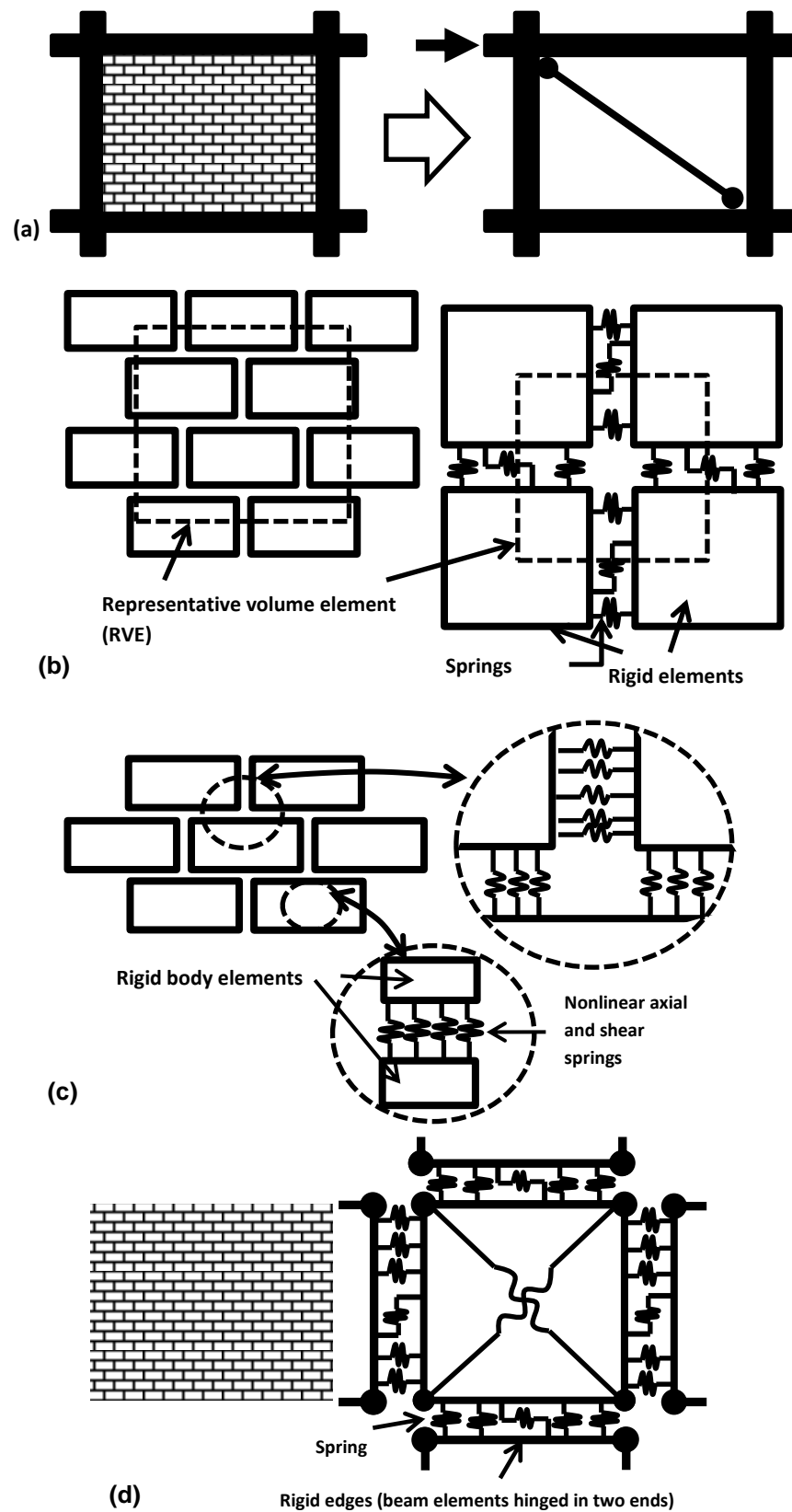


Figure 2. Alternative approaches; (a) diagonal strut model, (b) macro-element-based rigid body spring model, (c) micro-element-based rigid body spring model (applied element method), (d) equivalent frame model and plane discrete element.

5. Reinforced concrete frames

Previous studies have shown that the accuracy of a model incorporating infill masonry and surrounding RC frames depends very much on the finite-element scheme. Hence numerical tools to simulate reinforced concrete frames are indispensable for simulating the response of MIRCFs during an earthquake. Fracture of the concrete and mortar joints can be modelled by several approaches. One of the most promising approaches for implementation in a finite element is the smeared crack approach which the fracture is described within a continuum. The constitutive model developed via smeared crack approach can be embedded in different concepts as (i) decomposed strain concepts and (ii) total strain concepts ([74]). While the smeared crack model is an elegant approach for modelling crack and fracture in continuum, it naturally possesses some drawbacks. Numerous problems inherent to smeared crack models, e.g. the mesh size dependency of numerical solution ([75]), failure to capture diagonal shear cracks ([76]), directional bias, spurious kinematic modes, and stress locking ([77]) have been identified for quasi-brittle material such as mortar, brick and concrete. Also Fixed smeared crack may produce over-stiff behaviour while rotating smeared cracks do not ([77]). Another approach for modelling crack in a quasi-brittle material is discrete crack approach. It was generated based on the theory of fracture mechanics and fictitious crack model ([78]). It is well accepted that the discrete crack approach does not exhibit the strong mesh sensitivity as the classical smeared crack models ([75]). Zero thickness interface models are results of aforementioned theory. The only disadvantage of discrete crack approach is the need to modify the finite element mesh at each crack increment. This modification is avoided in some special case such as introducing interface element along all conceivable paths at the beginning of the analysis (see for example: [79]).

To reduce the aforementioned shortcomings related to both smeared and discrete crack approaches, Stavridis and Shing ([6]) combined both approaches to capture the different failure modes of infilled frames, including the mixed-mode fracture of mortar joints and the shear failure of RC members. Without the prior knowledge of the locations of the cracks, this discretization scheme has advantages over the traditional smeared crack or discrete crack methods but it considers the orientation of cracks as three value; 0, 90, and a value between 0 and 90 degrees. This methodology has also been validated with the experimental data of [80] and [11].

Flexibility-based inelastic beam-column elements ([81]) are the models used for the RC columns and beams. In this case, beam-column element is divided into fibers so that the steel reinforcement can be modelled. The uniaxial behaviour of the steel fibers may also be represented by the Menegotto and Pinto ([82]) steel model, while for the concrete it may be modelled by [83] and [84]. This simple solution can be built in OpenSEES ([85]). Micro and Macro modelling approaches of masonry panel cannot be addressed by OpenSEES as the aforementioned flexibility-based inelastic beam-column element is not able to transfer shear between fibres and from masonry panel. Also above fiber-section model is only valid for the case in which small deformation is addressed and plane section remains plane during the loading. Diagonal strut model presented in previous section is the only possible method for simulation in OpenSEES.

Another simplified solution for modelling the reinforced concrete and infill panel was presented by [86]. They modelled MIRCFs by discretizing the masonry units and beam-column elements into components interconnected with tensile and shear springs.

6. Single-scale finite element method

The effectiveness of numerical simulation tools for analysing masonry structures and masonry infilled frames have become more important in the light of potential difficulties of large scale physical models used in experimental simulations. The equilibrium equations and constitutive relations are two constituents of sophisticated numerical methods for boundary-value problems. In statically indeterminate problems, the equilibrium equations must be solved simultaneously with the constitutive relations. When the latter are represented by nonlinear rate equations, the resulting problem is one of nonlinear partial differential equations, and must be solved numerically, that is, the differential equations must be replaced by algebraic equations, and this is accomplished by means of a spatially discretized model of the continuum. Spatial discretization may contain finite element and finite difference methods. The most commonly used spatial discretization method nowadays is the finite

element method. In two-dimensional and three-dimensional domains, a particular division into subdomains is known as a finite element mesh, and the elements are typically polygons and polyhedral, respectively. Associated with each element are a number of points known as nodes, usually located on the boundary (but occasionally in the interior) of the element. The nodes usually include, at the very least, the vertices of the polygon. It is generally most convenient to use, as the generalized coordinates of the model, the values at the nodes of the functions describing the displacement field; these may simply be the nodal displacements. A finite element is characterized, then, by its shape, by the nodes associated with it, and by the assumed variation of displacement within the element, which is described by means of shape functions or interpolation functions. These functions are ordinarily taken as polynomial in terms of a local set of rectilinear coordinates.

Using the finite element method for spatial discretization, the discrete equilibrium equation for the single-scale solid can be written as following:

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \int_V \mathbf{B}^T \cdot \boldsymbol{\sigma} \cdot dV + \mathbf{f} = 0 \quad (2)$$

Where, \mathbf{M} , \mathbf{C} , and \mathbf{f} are mass, damping, and force matrices, respectively. \mathbf{B} is strain-displacement matrix. \mathbf{u} is the displacement discretization parameter.

Numerical integration of the above-mentioned equation is carried out using different algorithms depending on the type of analysis, static or transient analyses. Newmark algorithm ([87]) is known as an oldest and most extensively used algorithm for integration of the equations for transient analysis. Some other techniques have also been suggested in literature such as Hilber-Hughes-Taylor algorithm ([88]), generalized alpha algorithm ([89]), and the converging energy and momentum algorithm ([90]). To improve the procedure of convergence, it is also necessary to incorporate a procedure for incrementing the load. There are different explicit or implicit techniques in the literature. Sub-stepping technique ([91]) or load-factor technique ([92]) may be used in this regards as explicit solutions.

In the finite element solution, stress invariants are related to the scale of the material, creating single-scale finite element model. For example; macro modelling and micro modelling have their own finite element formulations and constitutive models. Orthotropic constitutive model is applied on the masonry panel while isotropic constitutive models are used for constituents of a masonry panel such as for brick, mortar and brick-mortar interface. In the last 50 years, extensive numerical approaches using single-scale finite element solution to predict the failure of masonry panels and to obtain the response of masonry panel for different loading conditions have been presented by many researchers. Haseltine et al. ([93]) and Page ([43]) are early researchers who attempted using finite element solution for masonry design and analysis.

7. Multi-scale finite element method

In the previous section, the single-scale finite element method for studying the masonry at microscopic and macroscopic model was presented. However, masonry exhibits an internal structure (mortar, brick) at more than one length scale i.e. micro, meso and macro. As a result, masonry with internal constituents may also show multiscale features, i.e. they may be invariant with respect to scaling.

At the structural level a direct simulation of multiscale systems is usually rather complex and time consuming because the discretization has to be reduced to the lowest scale at which information is needed. This would allow one to obtain at the same time the macroscopic behaviour of the entire structure as well as all the information at the lower scales ([94]). The output of multiscale material modelling of the masonry is usually effective material properties which are used at the macro-scale presented in section 3. Hence we deal with multiple-scale bridging down to the micro-scale. The most common methods for scale bridging in the field of masonry is asymptotic analysis of masonry cell with periodic structure at the micro level, which is also called *homogenization theory*. In the last 20 years, a large number of researches have been carried out on multiscale modelling of masonry using homogenization theory (see for example: [2], [61], [95], [96], [97]). A comparative study carried out by [98] shows that global behaviour of masonry is possibly evaluated very well by multi-scale method and computational homogenization.

8. Conclusion

As can be deduced from previous experiments and earthquake observations, adding masonry infill panel into the ductile or non-ductile frames (reinforce concrete or steel) increases the complexity of system due to material properties, configurations of the infill panels and interaction between infill panel and encased frame. On the other hand, masonry panel is a complex unit containing brick and mortars and, as a result, material properties of these two elements and their response during loading mechanisms play significant roles on the in-plane or out-of-plane behaviour of the panel, and dynamics of the whole structure.

Simplified or alternative solutions have been developed in the last 30 years to reduce the computation time and cost for simulating the response of masonry wall and masonry-infilled frames for a seismic event or dynamic assessment. Multi-scale finite element solution is concluded as one of elaborated methods that consider both micro-structure and macro-structure of the masonry panel and is able to reduce the computation time and cost, and also the uncertainty incorporated in single-scale macro models.

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9. References

- [1] Lotfi H R and Shing P B 1994 Interface Model Applied to Fracture of Masonry Structures *J. Struct. Eng.*
- [2] Lourenço P B 1996 *Computational strategies for masonry structures* (Delf: Delft University of Technology)
- [3] Gambarotta L and Lagomarsino S 1997 Damage models for the seismic response of brick masonry shear walls. Part I: The mortar joint model and its applications *Earthq. Eng. Struct. Dyn.*
- [4] Mehrabi A B and Shing P B 1997 Finite Element Modeling of Masonry-Infilled RC Frames *J. Struct. Eng.*
- [5] Sacco E and Toti J 2010 Interface elements for the analysis of masonry structures *Int. J. Comput. Methods Eng. Sci. Mech.*
- [6] Stavridis A and Shing P B 2010 Finite-Element Modeling of Nonlinear Behavior of Masonry-Infilled RC Frames *J. Struct. Eng.*
- [7] Zhai C, Wang X and Kong J 2017 Numerical Simulation of Masonry-Infilled RC frames Using XFEM *J. Struct. Eng.* **143** 1–13
- [8] Mehrabi A 1994 *Behavior of Masonry-infilled reinforced concrete frames subjected to lateral loading* (University of Colorado-Boulder)
- [9] Cristafulli F J 1997 *Seismic behaviour of reinforced concrete structures with masonry infills* (University of Canterbury)
- [10] El-Dakhkhni W W, Elgaaly M and Hamid A A 2003 Three-Strut Model for Concrete Masonry-Infilled Steel Frames *J. Struct. Eng.* **129** 177–185
- [11] Stavridis A 2009 *Analytical and experimental study of seismic performance of reinforced concrete frames infilled with masonry walls*
- [12] Sattar S 2014 *Influence of Masonry Infill Walls and Other Building Characteristics on Seismic Collapse of Concrete Frame Buildings* (University of Colorado at Boulder)
- [13] Mann W and Müller H 1982 Failure of Shear-Stressed Masonry- an Enlarged Theory, Tests and Application to Shear Walls *Proceedings of the British Ceramic Society* pp 223–35
- [14] van der Pluijm, R. 1998 *Out-of-plane bending of masonry : behaviour and strength* (Technical University of Eindhoven)
- [15] Burland J B 1990 On the compressibility and shear strength of natural clays *Géotechnique*
- [16] Yu H S, Tan S M and Schnaid F 2007 A critical state framework for modelling bonded geomaterials *Geomech. Geoengin.* **2** 61–74

- [17] Pestana J M and Salvati L A 2006 Small-Strain Behavior of Granular Soils. I: Model for Cemented and Uncemented Sands and Gravels *J. Geotech. Geoenvironmental Eng.*
- [18] Hofmann P and Stockl S 1986 Tests on the shear-bond behaviour in the bed-joints of masonry *Mas. Int.* **9** 1–15
- [19] Atkinson R H, Amadei B P, Saeb S and Sture S 1989 Response of Masonry Bed Joints in Direct Shear *J. Struct. Eng.*
- [20] Pluijm R Van der, Rutten H and Ceelen M 2000 Shear behaviour of bed joints *12th International Brick/Block Masonry Conference*
- [21] van Zijl G P A G 2004 Modeling Masonry Shear-Compression: Role of Dilatancy Highlighted *J. Eng. Mech.*
- [22] Fouchal F, Lebon F and Titeux I 2009 Contribution to the modelling of interfaces in masonry construction *Constr. Build. Mater.*
- [23] Lourenço P B, Barros J O and Oliveira J T 2004 Shear testing of stack bonded masonry *Construction and Building Materials*
- [24] Alecci V, Fagone M, Rotunno T and De Stefano M 2013 Shear strength of brick masonry walls assembled with different types of mortar *Constr. Build. Mater.*
- [25] CEN 1999 *Methods of tests for masonry, Part3. Determination of initial shear strength*
- [26] Fafitis A and Divakar M P 1992 A Constitutive Model for Aggregate Interlock Shear Based on Micromechanics *J. Energy Resour. Technol. - ASME* **114** 235–43
- [27] Lourenço P B and Rots J G 1997 Multisurface Interface Model for Analysis of Masonry Structures *J. Eng. Mech.*
- [28] Koutromanos I, Stavridis A, Shing P B and Willam K 2011 Numerical modeling of masonry-infilled RC frames subjected to seismic loads *Computers and Structures*
- [29] Cervenka J and Saouma V E 1995 Discrete crack modeling in concrete structures *Fract. Mech. Concr. Struct.*
- [30] Carol I, Prat P C and López C M 1997 Normal/Shear Cracking Model: Application to Discrete Crack Analysis *J. Eng. Mech.*
- [31] Puntel E, Bolzon G and Saouma V E 2006 Fracture Mechanics Based Model for Joints under Cyclic Loading *J. Eng. Mech.*
- [32] López C M, Carol I and Aguado A 2008 Meso-structural study of concrete fracture using interface elements. I: Numerical model and tensile behavior *Mater. Struct. Constr.*
- [33] Alfano G and Sacco E 2006 Combining interface damage and friction in a cohesive-zone model *Int. J. Numer. Methods Eng.*
- [34] Raous M, Cangémi L and Cocu M 1999 A consistent model coupling adhesion, friction, and unilateral contact *Comput. Methods Appl. Mech. Eng.*
- [35] Guíamatsia I and Nguyen G D 2014 A thermodynamics-based cohesive model for interface debonding and friction *Int. J. Solids Struct.*
- [36] Serpieri R, Alfano G and Sacco E 2015 A mixed-mode cohesive-zone model accounting for finite dilation and asperity degradation *Int. J. Solids Struct.*
- [37] Parrinello F, Marannano G and Borino G 2016 A thermodynamically consistent cohesive-frictional interface model for mixed mode delamination *Eng. Fract. Mech.*
- [38] PAGE A 1981 The biaxial compressive strength of brick masonry *Proc. Inst. Civ. Eng.* **71** 893–906
- [39] PAGE A W 1983 The strength of brick masonry under biaxial compression-tension *Int. J. Mason. Constr.* **3** 26–31
- [40] Dhanasekar M, Page A W and Kleeman P W 1985 The failure of brick masonry under biaxial stresses *Proc. Inst. Civ. Eng.* **79** 295–313
- [41] GANZ H R 1985 *Masonry walls under normal force and shear (in German)* (Zurich, Switzerland)
- [42] Mojsilović N 2011 Strength of masonry subjected to in-plane loading: A contribution *Int. J. Solids Struct.* **48** 865–73
- [43] Page A W 1978 Finite Element Model for Masonry *J. Struct. Div.* **104** 1267–85
- [44] Page A W, Kleeman P W and Dhanasekar M 1985 An in-plane finite element model for brick

- masonry *New Analysis Techniques for Structural Masonry* pp 1–18
- [45] Lotfi H R and Shing P B 1991 An appraisal of smeared crack models for masonry shear wall analysis *Comput. Struct.*
 - [46] Hegemier G A, Krishnamoorthy G, Nunn R O and Moorthy T V 1978 Prism Tests for the Compressive Strength of Concrete Masonry *Proceedings of the North American Masonry Conference, Boulder, USA* pp 18-1 to 18-17
 - [47] Feenstra P H and De Borst R 1996 A composite plasticity model for concrete *Int. J. Solids Struct.*
 - [48] Lourenço P B, Rots J G and Blaauwendraad J 1998 Continuum Model for Masonry: Parameter Estimation and Validation *J. Struct. Eng.* **124** 642–52
 - [49] Ganz H R and Thurlimann B 1984 Tests on the biaxial strength of masonry
 - [50] Lurati, F. and Thurlimann B 1990 *Tests in concrete masonry walls (in German)* (Zurich, Switzerland)
 - [51] Zhuge Y, Thambiratnam D and Corderoy J 1998 Nonlinear dynamic analysis of unreinforced masonry *J. Struct. Eng.* **124** 270–7
 - [52] Darwin D and Pecknold D A 1977 Nonlinear Biaxial Stress-Strain Law for Concrete *J. Eng. Mech. Div.* **103** 229–41
 - [53] Naraine K and Sinha S 1991 Cyclic behavior of brick masonry under biaxial compression *J. Struct. Eng.* **117** 1336–55
 - [54] Dhanasekar M, Kleeman P W and Page A W 1985 Biaxial stress-strain relations for brick masonry *J. Struct. Eng.* **111** 1085–100
 - [55] Scarpas A 1991 *Non-local plasticity softening model for brittle materials/experimental evidence/analytical modelling/preliminary results* (Athens, Greece)
 - [56] Andreus U 1996 Failure criteria for masonry panels under in-plane loading *J. Struct. Eng.* **122** 37–46
 - [57] Bortolotti L, Carta S and Cireddu D 2005 Unified yield criteria for masonry and concrete in multiaxial stress state *J. materials Civ. Eng.* **17** 54–62
 - [58] Symakezis C A and Asteris P G 2001 Masonry Failure Criterion under Biaxial Stress State *J. Mater. Civ. Eng.* **13** 58–64
 - [59] Ushaksaraei R and Pietruszczak S 2002 Failure Criterion for Structural Masonry based on Critical Plane Approach *J. Eng. Mech.* **128** 769–78
 - [60] Pietruszczak S and Mroz Z 2001 On failure criteria for anisotropic cohesive-frictional materials *Int. J. Numer. Anal. Methods Geomech.* **25** 509–24
 - [61] Milani G, Lourenço P B and Tralli A 2006 Homogenised limit analysis of masonry walls, Part I: Failure surfaces *Comput Struct* **84** 166–80
 - [62] Kawa M, Pietruszczak S and Shieh-Beygi B 2008 Limit states for brick masonry based on homogenization approach *Int. J. Solids Struct.* **45** 998–1016
 - [63] Milani G, Lourenço P B and Tralli A 2009 Homogenized rigid-plastic model for masonry walls subjected to impact *Int. J. Solids Struct.* **46** 4133–49
 - [64] Magenes G and La Fontana A 1998 Simplified nonlinear seismic analysis of masonry buildings *Proc. Br. Mason. Soc.* **8** 190–5
 - [65] Kappos A J, Penelis G G and Drakopoulos C G 2002 Evaluation of simplified models for lateral load analysis of unreinforced masonry buildings *J. Struct. Eng.*
 - [66] Lourenço P B 2002 Computations on historic masonry structures *Prog. Struct. Eng. Mater.*
 - [67] Polyakov S V 1960 *On the interaction between masonry filler walls and enclosing frame when loaded in the plane of the wall, in: construction in seismic regions* (Moscow)
 - [68] Casolo S and Pena F 2007 Rigid element model for in-plane dynamics of masonry walls considering hysteretic behaviour and damage *Earthq. Eng. Struct. Dyn.* **36** 1029–48
 - [69] Kikuchi A, Kawai T and Suzuki N 1992 The rigid bodies-spring models and their applications to three-dimensional crack problems *Comput. Struct.*
 - [70] Karbassi A and Lestuzzi P 2012 Fragility Analysis of Existing Unreinforced Masonry Buildings through a Numerical-based Methodology *Open Civ. Eng. J.* **6** 121–30
 - [71] Marques R and Lourenço P B 2014 Unreinforced and confined masonry buildings in seismic

- regions: Validation of macro-element models and cost analysis *Eng. Struct.* **64** 52–67
- [72] Quagliarini E, Maracchini G and Clementi F 2017 Uses and limits of the Equivalent Frame Model on existing unreinforced masonry buildings for assessing their seismic risk: A review *J. Build. Eng.* **10** 166–82
- [73] Calio I and Pantò B 2014 A macro-element modelling approach of Infilled Frame Structures *Comput. Struct.* **143** 91–107
- [74] Feenstra P H, de Borst R and Rots J G 1991 Numerical study on crack dilatancy I: Models and stability analysis *Eng. Mech.*
- [75] Bazant Z P 1976 Instability, Ductility, and Size Effect in Strain-Softening Concrete *J. Eng. Mech. Div.*
- [76] Cervenka V and Gerstle K H 1972 *Inelastic Analysis of Reinforced Concrete Panels: Part II: Experimental Verification* vol 3
- [77] Rots J G and Blaauwendraad J 1989 Crack models for concrete: discrete or smeared? Fixed multi-directional or rotatin? *Heron*
- [78] Hillerborg A, Modéer M and Petersson P E 1976 Analysis of crack formation and crack growth in concrete by means of fracture mechanics and finite elements *Cem. Concr. Res.* **6** 773–82
- [79] Rots J G 1988 *Computational modeling of concrete fracture*
- [80] Mehrabi A B, Schuller M P, Noland J L and Foundation N S 1994 Performance Of Masonry-Infilled RIC Frames Under In-Plane Lateral Loads *Struct. Eng. Struct. Mech. Res. Ser. Rep. CD/SR-94/6, Univ. Color. Boulder*
- [81] Spacone E, Filippou F C and Taucer F F 1996 Fibre beam-column model for non-linear analysis of R/C frames: Part I. Formulation *Earthq. Eng. Struct. Dyn.*
- [82] Menegotto M and Pinto P E 1973 Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and nonelastic behavior of elements under combined normal force and bending *IASBE Symposium on Resistance and Ultimate Deformability of structures Acted on by Well-Defined Repeated Loads*
- [83] Mohd-Yassin M-H and Filipou F C 1994 Nonlinear analysis of prestressed concrete structures.pdf *Structures Conccress XII*
- [84] Ayoub A and Filippou F C 2000 Mixed formulation of nonlinear steel-concrete composite beam element *J. Struct. Eng. New York, N.Y.*
- [85] McKenna F and Fenves G L 2000 An Object-oriented software design for parallel structural analysis *Proceedings of the SEI/ASCE Structures Congress*
- [86] Chiou Y-J, Tzeng J-C and Liou Y-W 1999 Experimental and analytical study of masonry infilled frames *J. Struct. Eng.* **125** 1109–17
- [87] Newmark N M 1959 A Method of Computation for Structural Dynamics *J. Eng. Mech.* **85**
- [88] Hilber H M, Hughes T J R and Taylor R L 1977 Improved numerical dissipation for time integration algorithms in structural dynamics *Earthq. Eng. Struct. Dyn.* **5** 282–92
- [89] Chung J and Hulbert G M 1993 A Time Integration Algorithm for Structural Dynamics With Improved Numerical Dissipation: The Generalized- α Method *J. Appl. Mech.* **60** 371:375
- [90] Bathe K J 2007 Conserving energy and momentum in nonlinear dynamics: A simple implicit time integration scheme *Comput. Struct.* **85** 437–445
- [91] Potts, D.M. and Zdravković L 1999 *Finite Element Analysis in Geotechnical Engineering: Theory* (LONDON: Thomas Telford)
- [92] Filippou F C and Fenves G L 2004 Methods of analysis for earthquake-resistant structure *Earthquake Engineering From Engineering Seismology to Performance-based Engineering* ed Y Bozorgnia and V V Bertero
- [93] Haseltine B A, West H W H and Tutt J N 1977 Design of Walls to Resist Lateral Loads, *Struct. Eng.* **55** 422–30
- [94] Zienkiewicz O C, Taylor R L and Fox D 2013 *The Finite Element Method for Solid and Structural Mechanics: Seventh Edition*
- [95] Massart T J, Peerlings R H J and Geers M G D 2007 An enhanced multi-scale approach for masonry wall computations with localization of damage *Int. J. Numer. Methods Eng.* **69** 1022–

- 59
- [96] Bacigalupo A and Gambarotta L 2012 Computational two-scale homogenization of periodic masonry: Characteristic lengths and dispersive waves *Comput. Methods Appl. Mech. Eng.* **213** 16–28
 - [97] Di Re P, Addessi D and Sacco E 2018 A multiscale force-based curved beam element for masonry arches *Comput. Struct.* **208** 18–31
 - [98] Addessi D, Marfia S, Sacco E and Toti J 2014 Modeling Approaches for Masonry Structures *Open Civ. Eng. J.* **8** 288–300