

Macroelement Model for the Progressive-Collapse Analysis of Infilled Frames

Original

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(Article begins on next page)

35 **Introduction**

36 Progressive collapse analysis and robust design of structures and infrastructures are emerging as hot
37 topics in the last years for both researchers and practitioners. This is not only highlighted by the recent
38 increase of numerical and experimental studies, but also the general directions of the latest updates
39 of most of the technical codes indicate robustness as a trait of new construction (**Marchand and**
40 **Stevens, 2016, Adam et al., 2018**), following the basic principle of robustness-based design
41 according to which, the effect of an accidental damage suffered by a structure, would not be
42 disproportionate with respect to the cause that generated it. For what concerns reinforced concrete
43 frame building structures, the most critical condition inducing progressive collapse is generally
44 related to the loss of a base column due, for instance, to impacts or explosions. In these condition
45 vertical loads, and the arising inertial forces, are rapidly redistributed to the adjacent structural
46 elements, the damage mechanism involves the beams converging to the column, which develop a
47 resisting mechanism evolving in three sequential steps: a) a flexural resistant mechanism; b) a post-
48 cracking arching mechanism; c) a catenary mechanism, triggering a large displacements regime
49 (**Izzuddin et al., 2008, Weng et al., 2017**). The damage evolution is accentuated by the effect of
50 gravity inertial forces arising and can be stopped within one of these three phases only if adequate
51 resistance supply is available to achieve a new equilibrium configuration. Otherwise, collapse of the
52 spans above the collapsed column occurs, also undermining the overall stability. Several authors have
53 carried out studies concerning theoretical and practical assessment of this kind of progressive collapse
54 response for frame structures. Among these **Izzuddin et al. (2008)** and **Vlassis et al., 2008** provided
55 an effective method for the simplified evaluation of dynamic load-displacement demand, through a
56 pseudo-static approach based on the energy balance between internal and external work for the
57 substructural levels involved in the collapse mechanism. More recently experimental tests were
58 carried out on scaled size structural elements (**Ren et al., 2016**) and structures (**Xiao et al., 2015**).
59 Other experimental studies, accompanied by numerical interpretations (**Yu and Tan (2013), Pham**
60 **et al. (2015), Pham et al. (2017)** and **Weng et al. (2017)**) have investigated the fundamental

61 geometrical and mechanical aspects influencing the progressive collapse response of RC structures,
62 providing also effective modelling strategies. A comprehensive state of the art illustrating the latest
63 advances on gravity-induced collapse modeling for reinforced concrete structures has been also
64 provided by **Kunnath et al. (2018)**. The major findings of the aforementioned studies report that the
65 ductility of the beam end cross-sections plays a prominent role on the activation capacity of the
66 catenary mechanism, but also that this is conditioned by many other factors such as the horizontal
67 constraint degree as well as the real capacity of elongation of steel.

68 On the other hand, more recent studies, have demonstrated from the experimental and numerical point
69 of view that the gravity-induced progressive collapse mechanism of frame structures is significantly
70 altered by the presence of masonry infills (**Farazman et al., 2013, Xavier et al., 2015, Shan et al.,**
71 **2016, Qian et al., 2017, Li et al., 2019, Di Trapani et al 2020a**). Therefore, the interaction between
72 frames and infills due to lateral loads, which has been widely investigated in the past by experimental
73 quasi-static tests (e.g. **Mehrabi et al. 1996, Colangelo 2005, , Kakaletsis, Karayannis 2009, Calvi**
74 **and Bolognini 2001, Morandi et al. 2018**) and numerical studies addressing micro-modelling (e.g.
75 **Mehrabi et al. 1997, Stavridis and Shing 2010, Calì & Pantò 2014**) and equivalent single or
76 multi-strut macro modelling (e.g. **Panagiotakos and Fardis 1996, El-Dakhakhni et al. 2003,**
77 **Crisafulli and Carr 2007, Furtado et al., 2015, Di Trapani et al. 2020b**) also occurs in the case of
78 gravity-induced progressive collapse. This has noticeable impact to the overall resistance increase,
79 accompanied by a less prominent post-peak response and reduced propension in developing the
80 catenary mechanism under the large displacement regime in comparison with bare frames. As a
81 secondary effect, the increase in strength is also associated with a stiffness increment, causing an
82 increase of the dynamic amplification factor (**Di Trapani et al., 2020a**). Therefore, the infilled frame
83 has more resistance but, at the same time, the dynamic load demand increases too.

84 These premises make evident that progressive collapse of infilled frames cannot be assessed properly
85 without an adequate modeling of frame-infill interaction. In particular, a simplified modelling
86 strategy comes out as fundamental need to effectively assess progressive collapse response of infilled

87 frames, although available experimental tests are still limited. Also for this reason, the proposed
88 simplified macro-modeling approaches are theoretically based (e.g. **Farazman et al., 2013**) or not
89 entirely generalizable, being validated on single experiments (e.g. **Li et al., 2019**). In a recent paper,
90 **Di Trapani et al. (2020a)**, proposed the idea of three-strut fiber-section model, highlighting the
91 importance of defining the inclination of the eccentric struts as a function of the geometrical and
92 mechanical properties of the infilled frame. Despite this, no specific rules were given to define the
93 inclination of the struts, and the predictive capacity of the model was tested only with a limited
94 number of reference specimen tests, although with promising results.

95 Within this background, the paper formalizes the definition of a three-strut macro-element model
96 based on the observation of the collapse mechanism of available experimental tests and refined
97 numerical simulations of one-storey and multi-storey infilled frames. The model is defined using the
98 *OpenSees* (**McKenna et al., 2000**) software platform. The major novelty is the definition of two
99 parameters modulating the inclination of the struts and the resistance provided by the infill to the
100 overall response. These parameters are easily evaluated through the analytical correlations provided
101 in the paper, which put in relation the geometrical and mechanical characteristics of an infilled frame,
102 with the geometric configuration and mechanical response of the equivalent strut model. In this way,
103 the proposed model, can adapt its geometry and the stress-strain response of the struts to each infilled
104 frame arrangement, providing a specialized simulation of the collapse mechanism. The
105 aforementioned analytical correlation laws are obtained from an empirical dataset, specifically built
106 by merging experimental pushdown test results and results from pushdown test simulation made with
107 a refined finite element model. The validation of the model is carried against additional pushdown
108 tests not previously considered for the definition of the dataset. A practical application of the proposed
109 procedure is finally proposed for a six-storey building.

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111

112

113 **Proposed macro-element model formulation**

114 The proposed modelling approach is based on the experimental observations of the damage mechanisms
115 activating in single-storey and multi-storey infilled frames subject to pushdown tests. The model is
116 implemented in OpenSees and is based on the adoption of distributed plasticity fiber-section elements
117 for the RC frame. The model is easily reproducible with any finite element analysis program handling
118 fiber-section elements. The experimental tests by **Li et al (2019)** and **Qian and Li (2017)** and the
119 numerical investigations by **Di Trapani et al. (2020a)**, have clearly evidenced the behaviour of an
120 infilled frame under progressive collapse conditions (**Fig. 1a-1b**), basically showing that:

- 121 • masonry portions enclosed at the corners undergoing a reduction of the original 90° angle in
122 proximity of beam-column joints receive a confining action from the frame and because of
123 this, undamaged corner regions form;
- 124 • confined masonry corner regions behave as rigid parts, inducing a migration of the position
125 of the plastic hinges towards the inner part of the beam. In multi-storey infilled frames this
126 occurs following a more or less linear pattern (**Fig. 1b, 2b**), so that for the external corner
127 regions, the major movement of the plastic hinge position is observed at the highest floor
128 (while no migration is observed at the lowest floor). This condition is inverted in
129 correspondence of the internal corner regions. As a consequence of this, for the beams having
130 intermediate position plastic hinges form in a position that can be approximately determined
131 by intercepting the beam axis with the line joining the hinges at the upper and the lower floors
132 (**Fig. 1b**).
- 133 • masonry at central zones undergoes significant damage due to diagonal compression. This
134 induces sliding of bed joints and diagonal crushing in some cases.

135 A consequent schematization of the damage framework expected for building undergoing column
136 losses is shown in **Fig. 2** for one-storey infilled frame buildings (**Fig. 2a**) and multi-storey infilled
137 frame buildings (**Fig. 2b**). The major difference between these cases is related to the compression
138 fields (depicted in blue in **Fig. 2**). In fact, as already mentioned, the masonry confined by the frame

139 is subjected to significant compression stresses making these parts of the infill working as
140 compression struts. The latter have a unique slope which connects the positions of the plastic hinges
141 forming at different places in the beams. This means that, in multi-storey infilled frames, the position
142 of the plastic hinges depends on the storey level, while for the one-storey cases the damage
143 configuration is fixed. The proposed model is conceived to reproduce the experimentally observed
144 mechanical response of the single-storey and multi-storey infilled frames. The schematization of the
145 model is proposed in **Fig. 3** for both the cases. The model is composed of 3 struts per each infilled
146 frame. Struts S2 have infinite stiffness and strength and are used to effectively simulate the observed
147 damage mechanism in which masonry at corners remains almost intact. The product $\alpha_b l_b$ ($\alpha_b < 1$)
148 represents the position of the connection node of the beam with the S2 strut. This consequently defines
149 the angle of inclination of the struts and the location where the plastic hinge will form along the length
150 l_b because of the interaction with the infill. As it can be observed in **Fig. 3**, the distance $\alpha_b l_b$ is defined
151 once, both for the one-storey and multi-storey configurations, therefore the S2 struts of each infilled
152 frame are parallel. For the multi-storey case, the connection points of S2 struts with the intermediate
153 floor beams are obtained as the intersections of the straight lines connecting the origin points with the
154 distances $\alpha_b l_b$ at the top and bottom beams with the intermediate beams themselves. S2 struts at the
155 different floors are therefore aligned. The parameter α_b has a key role to the overall response, as it
156 modulates the net length of the beams when the plasticization of the frame occurs. Therefore, high
157 values of α_b are associated with a major contribution of the frame to the overall strength, while on
158 the contrary, low α_b values induce minor frame resistance. More generally, the coefficient α_b
159 considers the degree of coupling between the infill and the frame and therefore it defines the actual
160 influence of the infill to the overall response of the frame. The definition of α_b is therefore of crucial
161 importance as it defines the geometry of the entire resisting mechanism (**Fig. 3**) although its definition
162 depends on a quite large number of factors as it will be described in the subsequent sections.

163 The S1 strut models the shear behaviour of the infill in the inner part, which is subjected to diagonal
 164 compression forces. The identification of S1 strut cross-section dimensions is performed starting from
 165 the procedure proposed by **Di Trapani et al. (2018)** as modified in **Di Trapani et al. (2020a)**. The
 166 thickness of the struts (t) is the same as the actual thickness of the infill, the equivalent strut width (w)
 167 is evaluated as:

$$168 \quad w = \frac{h_c}{l_b} \frac{c^*}{\lambda^{*\beta^*}} d \quad (1)$$

169 where, with respect to the original formulation, the effect of vertical loads on columns is ignored in
 170 **Eq. (1)**, while the parameter λ^* is evaluated by means of the formula initially proposed by **Papia et**
 171 **al. (2003)**, where the positions of l_b and h_b are inverted:

$$172 \quad \lambda^* = \frac{\tilde{E}_m}{E_c} \frac{t l_b'}{A_b} \left(\frac{l_b'^2}{h_c'^2} + \frac{1}{4} \frac{A_b}{A_c} \frac{h_c'}{l_b'} \right) \quad (2)$$

173 The symbols A_b and A_c represent the area of the cross-sections of beams and columns, E_c is the elastic
 174 modulus of the concrete frame, \tilde{E}_m is the conventional elastic modulus taking into account both the
 175 Young's moduli of masonry along the two orthogonal directions (E_{m1} and E_{m2}) as:

$$176 \quad \tilde{E}_m = \sqrt{E_{m1} \cdot E_{m2}} \quad (3)$$

177 while the other symbols are deducible from **Fig. 3**. Parameters c^* and β^* are evaluated by the
 178 following equations:

$$179 \quad \begin{aligned} c^* &= 0.249 - 0.0116\nu + 0.567\nu^2 \cong 0.254 \\ \beta^* &= 0.146 - 0.0073\nu + 0.126\nu^2 \cong 0.147 \end{aligned} \quad (4)$$

180 depending on Poisson's ratio (ν) of the infill along the diagonal direction. For the sake of simplicity,
 181 the average Poisson's ratio can be used, otherwise the value $\nu=0.1$ is assumed, providing the values
 182 resulting in **Eq. (4)**. S1 struts are no-tension inelastic fiber-section trusses modelled with a
 183 *Concrete02* material (parabolic with linear softening) available in *OpenSees*, defined by the four
 184 parameters \tilde{f}_{md0} (peak-strength), \tilde{f}_{mdu} (ultimate-strength), ε_{md0} (peak-strain), ε_{mdu} (ultimate strain)

185 **(Fig. 4a)**. The equivalent strut is a phenomenological model which summarizes the complex response
 186 of the infill as a function of the boundary condition (e.g. aspect ratio, masonry orthotropy, influence
 187 of vertical stresses, frame-infill relative strength and stiffness). Therefore, the uniaxial stress-strain
 188 response of S1 strut will not be coincident to the masonry nominal one but will be related to it by the
 189 empirical relationships provided by **Di Trapani et al. (2018)**:

$$190 \quad f_{md0} = \tilde{f}_m \cdot 26.9 \cdot \alpha^{-0.28} \quad (5)$$

$$191 \quad f_{mdu} = f_{md0} \cdot (0.043 \cdot \beta - 0.06) \quad (6)$$

$$192 \quad \varepsilon_{md0} = \varepsilon_{m0} \cdot 3.024 \cdot \gamma^{0.347} \quad (7)$$

$$193 \quad \varepsilon_{mdu} = \varepsilon_{md0} \cdot 0.0184 \cdot \delta^{-1.166} \quad (8)$$

194 where, neglecting the influence of vertical loads on columns, and inverting the position of l_b and h_b ,
 195 parameters α , β , γ and δ have the expressions:

$$196 \quad \alpha = \frac{\tilde{f}_m^2 \cdot w \cdot t}{f_{vm}^{0.2} (h_c / l_b) \cdot \lambda^{*0.2}} \quad (9)$$

$$197 \quad \beta = \frac{f_{md0}^{0.7} \cdot w \cdot t}{\tilde{E}_m^{0.2} d} \quad (10)$$

$$198 \quad \gamma = \left(\frac{f_{mdu}^2}{f_{md0}^2} \right) \left(\frac{E_c}{\tilde{E}_m^{1.5}} \right) \quad (11)$$

$$199 \quad \delta = \tilde{E}_m^{0.20} \cdot \varepsilon_{md0} \quad (12)$$

200 in which it is conventionally assumed $\varepsilon_{m0} = 0.0015$ and $\tilde{f}_m = \sqrt{f_{m1} \cdot f_{m2}}$, is the conventional strength
 201 of the masonry, which takes into account the strengths along the two orthogonal directions (f_{m1} and
 202 f_{m2}). The peak and ultimate stresses \tilde{f}_{md0} and \tilde{f}_{mdu} are related with f_{md0} and f_{mdu} by a strength
 203 reduction coefficient ξ (≤ 1) taking into account a potential strength reduction associated with the
 204 different expected failure modes for the infilled frame subjected to vertical loads (instead of
 205 horizontal), as a function of the possible combination of geometric and mechanical characteristics of
 206 the infilled-frame system, therefore :

207
$$\begin{aligned}\tilde{f}_{md0} &= \xi \cdot f_{md0} \\ \tilde{f}_{mdu} &= \xi \cdot f_{mdu}\end{aligned}\tag{13}$$

208 The peak and ultimate strains are instead assumed as those evaluated by **Eqs. (7-8)** respectively.

209 Considering the above listed equations, the equivalent strut model is defined based on the two

210 parameters α_b and ξ . Given the relevance that they assume on the overall response and the multitude

211 of variables on which they depend, the definition of α_b and ξ is carried out using an empirical

212 approach rather than a mechanics-based one as described in the subsequent section.

213 As regards the reinforced concrete frame, this is modelled using distributed plasticity fiber-section

214 elements. A *Concrete02* model is used for confined core and unconfined cover concrete fibers (**Fig.**

215 **4b**). Confined ($f_{cc0}, f_{ccu}, \varepsilon_{cc0}, \varepsilon_{ccu}$) and unconfined ($f_{c0}, f_{cu}, \varepsilon_{c0}, \varepsilon_{cu}$) concrete parameters are evaluated

216 according to the model by **Razvi and Saatcoglu (1992)**. As regards rebars, given the large

217 displacements and damage achieved by the frame during the pushdown analyses, it was necessary to

218 include advanced material damage state. Rebars were modelled using the *Hysteretic* material

219 backbone curve in order to simulate fracture in tension, in correspondence of the ultimate stress-strain

220 capacity point (f_t, ε_{su}), and buckling in compression (if any) in correspondence of the stress-strain

221 buckling point (σ^*, ε^*) evaluated according to the **Dhokal and Maekawa (2002)** model. Finally, the

222 beam-columns intersection zones are modelled with rigid links. The definition of the rigid link

223 extensions can be alternatively carried out using ASCE/SEI 41-17 provisions.

224

225 **Framework and data-set for the empirical evaluation of coefficients α_b and ξ**

226 The determination of parameters α_b and ξ is not easy to perform using a mechanical approach, in fact

227 this would require strong assumptions about specific collapse modes for the frame-infill system,

228 which are not simple to predict a-priori. The major unknown regards the masonry infill failure mode

229 which can alternatively be due to mortar joints sliding, diagonal cracking or mixed modes, and that

230 depends each time on the different combination of frame and infill mechanical and geometrical

231 properties and lateral constraint conditions. The procedure here followed, provides instead the

232 definition of two empirical correlations between the parameters α_b and ξ and the geometrical and
233 mechanical characteristics of the frame-infill system.

234 In order to derive the analytical correlation, a dataset of reference experimental and numerical
235 pushdown tests has been defined. The tests were simulated in *OpenSees* using the proposed equivalent
236 strut model. Parameters α_b and ξ were initially assigned as trial values. The simulations were then
237 repeated by iterating these parameter values up to the achievement of the best matching with the
238 reference pushdown responses. The finally obtained values for α_b and ξ parameters were then stored
239 and associated with the geometrical and mechanical data of the respective infilled frame. The
240 procedure was repeated for all the reference tests. Since the availability of experimental tests is limited,
241 the robustness of the data-set was increased using the also the results from refined finite element
242 simulations of pushdown tests carried out by **Di Trapani et al. 2020a**. The steps of the framework
243 are below summarized:

- 244 1. Acquisition of experimental test data (geometric and materials data, pushdown curves);
- 245 2. Modeling of each experimental test with the proposed multi-strut model;
- 246 3. Iterative calibration of parameters α_b and ξ ;
- 247 4. Acquisition of numerical test data from a refined, experimentally validated, FE model;
- 248 5. Modeling of each experimental test with the proposed multi-strut model;
- 249 6. Iterative calibration of parameters α_b and ξ ;
- 250 7. Definition of two analytical correlation laws between the obtained parameters α_b and ξ and the
251 most representative geometrical and mechanical characteristics of the infilled frame;
- 252 8. Blind validation tests with experimental and FE specimens not included in the calibration phase.

253 A flow-chart of the framework is also illustrated in **Fig. 5**. The experimental tests used were those by
254 **Qian and Li (2017)** (specimens WSS and WNL) and **Li et al. (2019)**. The formers are 1/4 scale two-
255 storey reinforced concrete frames with brick masonry infills horizontally restrained from one-side
256 only. The second is a 1/3 scale one-storey multi-bay reinforced concrete frames with brick masonry

257 infills. The test set-up did not provide lateral constraints, but the frame had one bay at each side of
258 the central bays involved in the pushdown tests.

259 The numerical specimens were those by **Di Trapani et al. (2020a)**. The latter are full scale one-storey
260 one-bay infilled frames having different aspect-ratios, lateral constraint degree (free or fixed), beam
261 cross-section aspect ratio and reinforcement arrangement (seismic or non-seismic). These specimens
262 have been modelled in the Atena 2D Version 5 (**Cervenka et al. 2017**) software platform, which is
263 based on a smeared cracking formulation for concrete and also supports frictional interfaces to be
264 used for modeling mortar joints. The typical assembly of the FE model is illustrated in **Fig. 6a**.
265 Concrete frame elements and masonry blocks are modeled by using CCIsoQuad nonlinear finite
266 elements. The constitutive model associated with concrete elements and masonry units having a
267 failure surface (**Fig. 6b**). Mortar joints are simulated using 2D interfaces governed by a Mohr-
268 Coulomb failure surface depending on friction coefficient (μ) and cohesion (c) (**Fig. 6c**). The
269 validation of the finite element modelling was performed by the authors in **Di Trapani et al. (2018)**.
270 Details about the specimens and the test type are reported in **Table 1**. The identification the equivalent
271 strut models is performed using **Eqs. (1-4)** for the determination of the S1 strut width and **Eqs. (5-**
272 **12)** for the determination the nominal stress-strain response. Geometric and mechanical details of the
273 experimental and numerical specimens together with the associated parameters identifying the
274 equivalent struts are listed in **Tables 2-3**. Parameters α_b and ξ are assigned as trial values in the first
275 step. Before starting the iterative calibration of α_b and ξ , the bare frame pushdown responses of the
276 reference specimens and those provided by the models are compared. This preliminary comparison
277 is carried out to check the reliability of the frame fiber-section model to reproduce the bare frame
278 response, so that the subsequent calibration of parameters α_b and ξ will not be affected by a
279 compensation of predictive errors of the frame model. For the sake of space only three sample
280 comparisons between the bare frame reference specimens' responses and the bare frame model
281 predictions are shown in **Figs. 7**. The latter confirm the reliability of the frame model in reproducing
282 the pushdown tests.

283 Numerical simulation of the infilled frames pushdown tests is then carried out. Parameters calibration
284 process is performed by comparing the reference pushdown responses and the model predictions
285 iteratively varying parameters α_b and ξ to get the best matching of the curves. The basic matching
286 criterion followed was an energetic criterion, that is the minimization of the difference between the
287 areas below the reference pushdown curve of the specimen A_{exp} and the predicted one A_{pred} , so that
288 $|A_{exp} - A_{pred}|$ is minimized. The adequacy of the results obtained by this approach was finally checked
289 by the authors at the end of each test. The ranges of variation assumed were 0.15-0.5 for α_b and 0.25-
290 1.0 for ξ . Results at the end of the calibration process are shown in **Fig. 8-9**, where numerical
291 predictions and reference pushdown curves are depicted. It can be generally observed that, despite
292 the large geometric and mechanical nonlinearity achieved during the tests, an adequate calibration of
293 α_b and ξ allowed a sufficiently accurate prediction of the overall pushdown response, giving a
294 positive feedback on the reliability of the proposed modelling approach. Some comparisons between
295 the deformed shapes of the experimental and numerical specimens and those of the multi-strut models
296 are shown in **Fig. 10**, demonstrating the consistency of the actual damage mechanisms with the
297 predicted ones. Optimal results obtained for parameters α_b and ξ at the end of the calibration process
298 of each specimen are finally listed in **Table 4**, together with the dimensional parameters resulting by
299 their application. As regards α_b , the values range between 0.15 and 0.50, meaning that the plastic
300 hinges in beams can form in a region comprised between 15% and 50% of the internal length l_b . This
301 is definitely consistent with the experimental and numerical evidence from the considered specimens
302 (**Fig. 10**). Regarding ξ values, the range is 25%-100% of the diagonal resistance f_{md0} . This wide range
303 should not be surprising, but highlights the fact that the infill contribution to the overall resistance is
304 extremely variable and depends on the resistance associated with the actual failure mechanism
305 developing, as well described in the following section.

306

307

308 **Empirical correlation laws for α_b and ξ**

309 The experimental and numerical data-sets containing the calibrated parameters α_b and ξ together with
310 the associated geometrical and mechanical features of the respective infilled frames are merged into
311 a unique hybrid data-set. Results are then analyzed in order to understand the dependence of these
312 parameters in a more general framework. An interpretation of the obtained α_b and ξ values can be
313 given by recognizing the actual role of these parameters to the overall response. In fact, while ξ
314 modulates the contribution of the infill in terms of strength, α_b modulates the shape of the collapse e
315 mechanism, namely the amplitude of the intact portion of masonry enclosed at the corners.
316 Consequently, α_b has effect on the vertical strength of the frame, as it defines the net shear span of
317 the beams involved in the collapse mechanism (**Fig. 11**). The modulation by means of coefficients α_b
318 and ξ allows considering the reciprocal influence of the frame to the final resistance of the infill and,
319 vice versa, the influence of the infill to the final resisting mechanism of the frame. Consistently with
320 this consideration, it can be observed the lowest ξ values are associated with specimens having
321 reduced strength and stiffness of the frame with respect to that of the infill, so that a lower confining
322 action is exerted and a sliding failure mode is more likely. On the contrary, a stronger confining action
323 by a stiffer frame, will orient failure to a crushing mode which is typically associated with a larger
324 strength. At the same time, the coefficient α_b has shown to be more influenced by the strength ratio
325 between the frame and the infill, therefore the lower α_b values have been obtained for the frames
326 having low strength with respect to the infill strength.

327 In this framework a crucial role is exerted by the lateral constraint degree, which influences the
328 vertical stiffness of the frame and even more its strength. In fact, in the case of rigid or semi-rigid
329 lateral contains, the axial force reaction to the pushing action of beams after the first cracking is
330 relevant and significantly increases the resisting end moments (M_p^+ , M_p^-). On the other hand, more
331 effective lateral constraint degree increases the frame stiffness and the associated confining action to

332 the infill with major exploitation of the infill strength. The scheme of the typical collapse mechanism
 333 is depicted in **Fig. 11** for the one-storey infilled frame.

334 Based on the aforementioned assumptions, the best correlation between ξ and the geometric and
 335 mechanical properties of the frame-infill system is found with the parameter κ so defined:

$$336 \quad \kappa = \frac{K_f}{K_s} \sqrt{\frac{l_b}{h_c}} \sqrt{\frac{\tilde{f}_m}{f_{vm}}} \quad (14)$$

337 where K_f is the conventional vertical stiffness of the frame obtained as the sum of the shear stiffnesses
 338 of the n_b beams involved in the mechanism:

$$339 \quad K_f = \sum_{i=1}^{n_b} \psi_i \cdot \frac{E_{ci} I_{bi}}{l_{bi}^3} \quad (15)$$

340 and K_s is the conventional vertical stiffness contribution provided by the n_s infills involved in the
 341 mechanism at all the stories:

$$342 \quad K_s = \sum_{j=1}^{n_s} \lambda_j \frac{\tilde{E}_{mj} \cdot t_j \cdot w_j}{d_j} \cdot \sin \theta_j \quad (16)$$

343 In the previous equation $i=1 \dots n_b$ is an index referred to the generic beam and $j=1 \dots n_s$ is an index
 344 referred to the generic strut. The coefficients ψ_i and λ_j are used to consider the influence of the lateral
 345 constraint degree on the real stiffness of the beams and of the struts. The following boundary values
 346 are assumed depending on the type of constraint:

$$347 \quad \psi_i = \begin{cases} 12 & \text{(rigid)} \\ 6 & \text{(free)} \end{cases} \quad \lambda_j = \begin{cases} 1 & \text{(rigid)} \\ 0.5 & \text{(free)} \end{cases} \quad (17)$$

348 In **Eq. (16)**, θ_j is the angle of inclination of the S1 strut. For the one storey infilled frame this is
 349 depending only on the geometry of the frame, while in the case of a multi-storey infilled frame, this
 350 depending on α_b (**Fig. 3b**) which is still depending on θ_j . The determination of θ_j can be thus
 351 performed iteratively, or in a simpler way, this can be assumed as the average between the maximum
 352 and minimum slopes (θ_{max} and θ_{min} respectively) resulting by assuming $\alpha_b = 0.5$ and $\alpha_b = 0.15$ (**Fig.**
 353 **12**):

354
$$\theta_j = \frac{\theta_{max} + \theta_{min}}{2} \quad (18)$$

355 where supposing that all the stories have the same interstorey height (h_c'), it can be easily found:

356
$$\theta_{max} = \arctan\left(\frac{nh_c'}{nl_b' - (n-1)(0.5l_b + 0.5h_{wc})}\right); \theta_{min} = \arctan\left(\frac{nh_c'}{nl_b' - (n-1)(0.15l_b + 0.5h_{wc})}\right) \quad (19)$$

357
 358 Resulting values of K_f , K_s and κ for the different specimens are reported in **Table 5**. The best
 359 correlation between ξ and κ , fitting experimental and numerical data can be established by the
 360 following exponential correlation law (**Fig. 13a**):

361
$$\xi = 0.212 \cdot e^{1.45\kappa} \quad (R^2 = 0.89) \quad (20)$$

362 which results in a determination coefficient of 0.89. The values of ξ resulting from **Eq. (20)** can be
 363 reasonably assumed in the range $0.25 \leq \xi \leq 1.0$.

364 The correlation of the parameter α_b with the characteristics of the infilled frame can be expressed as
 365 a function of the frame-infill strength ratio ρ , defined as:

366
$$\rho = \frac{R_f}{R_s} \quad (21)$$

367 in which R_f is the conventional vertical strength of the frame, obtained as:

368
$$R_f = \sum_{i=1}^{n_{Mp}} \frac{M_{p,i}^- + M_{p,i}^+}{l_b} \quad (22)$$

369 where, with reference to the simplified limit equilibrium scheme of the generic double span in **Fig.**
 370 **14a**, the subscripts i denotes the generic plastic negative and positive moments at the ends of each
 371 beam involved in the collapse mechanism, n_{Mp} is the number of end cross-section where the plastic
 372 moments form, while l_b is the net span length. The plastic moments have positive sign and are
 373 evaluated considering the maximum axial forces acting on the beams during the pushdown analysis
 374 ($N_{b,max}$). R_s is the conventional vertical resistance provided by the infills, obtained considering the
 375 limit equilibrium of the hinged simplified scheme of the generic storey in **Fig. 14b** as:

376

$$R_s = \sum_{j=1}^{n_s} \tilde{f}_{mdo,j} \cdot t_j \cdot w_j \cdot \sin \theta_j \quad (23)$$

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where the subscript j denotes the terms related to the generic S1 strut involved in the collapse mechanism. For the correlation between ρ and α_b two possible expressions are provided (**Fig. 13b**):

379

$$\alpha_b = -0.09\rho^3 + 0.54\rho^2 - 1.05\rho + 0.86 \quad (R^2 = 0.93) \quad (24)$$

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$$\alpha_b = 0.28\rho^{-0.58} \quad (R^2 = 0.87) \quad (25)$$

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The two expressions have similar determination coefficients (0.93 and 0.87 respectively) and can be used alternatively or to get an average value for α_b . Consistently with the actual experimental and numerical observations, the values of ξ resulting from **Eqs. (24-25)** can assumed be in the range $0.15 \leq \alpha_b \leq 0.5$. Resulting values of R_f , R_s , ρ and $N_{b,max}$ for the different specimens are reported in **Table 5**. It is noteworthy observing that the determination of the strength of the frame (R_f), and therefore of the coefficient ρ , is subject to the evaluation plastic moments at the ends of the beams, which in turn are depending on the maximum axial forces ($N_{b,max}$) developing because of the pushing action of the beams on the lateral supports. The latter are not straightforward to evaluate in practice, since they vary at each step of the pushdown analysis, therefore, in order to use **Eq. (22)**, two strategies can be adopted. The first is running a pushdown analysis supposing a trial value of α_b to get an estimation of the maximum axial forces acting on beams. The second is using the analytical correlation laws defined as described in the following.

The maximum compression axial force achieved by the beams in a pushdown tests basically depends on two factors. One is the lateral constraint degree, the more the constraints are stiff, the more the axial lateral reaction increases and so the axial force. The second is the beam height-to-length ratio ($\chi=h_{wb}/l_b$), as this conditions to the arching action developed by the beams after the first cracking. This is clear from **Fig. 15a**, which shows the distribution of maximum dimensionless axial forces ($v_{b,max}=N_{b,max} / f_c b_{wb} h_{wb}$) recorded for the previously analyzed experimental and numerical specimens as a function of χ and the lateral restraint conditions. In **Fig. 15b**, the trend of the average

401 dimensionless axial forces in beams is reported as a function of the vertical displacement and for the
402 different constraint conditions.

403 It is found that the two following simple polynomial correlation can be defined between $v_{b,max}$ and χ
404 for the two cases of rigid and free lateral constraints:

$$405 \quad v_{b,max} = \begin{cases} -8.27\chi^2 + 2.74\chi + 0.017 & \text{(rigid)} \\ -5.79\chi^2 + 1.36\chi - 0.041 & \text{(free and rigid-free)} \end{cases} \quad (26)$$

406 As previously mentioned, expressions in **Eq. (25)** can be used as a simple way to predict the
407 maximum axial force acting on the beams and therefore estimating the associated plastic moments to
408 introduce in **Eq. (22)**. An interpolation between the two functions can be used to estimate $v_{b,max}$ the
409 cases having semi-rigid connections.

410

411 **Blind validation tests**

412 Based on the obtained formulation, the procedure for the complete analytical definition of the
413 proposed infilled frame multi-strut model follows the below reported steps:

- 414 1. Evaluation of the equivalent strut width w (**Eqs. 1-4**);
- 415 2. Evaluation of the nominal stress-strain parameters of S1 strut ($f_{md0}, f_{mdu}, \varepsilon_{md0}, \varepsilon_{mdu}$) (**Eqs. 5-12**);
- 416 3. Determination of ξ (**Eqs. 14-20**);
- 417 4. Determination of \tilde{f}_{md0} and \tilde{f}_{mdu} (**Eqs. 13**);
- 418 5. Determination of α_b (**Eqs. 21-26**);

419 This procedure has been validated by applying it for the simulation of four reference tests not
420 belonging to previously defined data-set used to build the empirical correlations. The specimens
421 consisted of one experimental test by **Qian and Li (2017)** (Specimen WNS) and three numerical tests
422 of the refined FE models by **Di Trapani et al (2020a)** Details about the specimens and the test types
423 are reported in **Table 6**, while **Tables 7-8** collect all the fundamental geometric and mechanical data
424 for the definition of the equivalent strut and the nominal stress-strain parameters. Parameters κ and

425 ρ , evaluated according to **Eqs. (14)** and **(21)** are reported in **Table 9**, together with the dimensionless
426 axial forces ($v_{b,max}$) estimated by **Eq. (26)**. Parameters α_b and ξ finally resulting are reported in **Table**
427 **10**. The comparison between experimental force-displacement curves and numerical predictions are
428 depicted in **Fig. 16**. It can be observed that for all the considered cases, numerical predictions resulted
429 in a very good agreement with the test data, providing a sufficiently reliable prediction of the peak
430 resisting capacity and the post-peak branch, despite the large uncertainty associated with the advanced
431 damage state of materials during the tests. This result is also confirmed by the consistency between
432 actual and predicted deformed shapes as illustrated in **Fig. 17**.

433 434 **Application of the proposed method to a case-study structure**

435 The proposed modelling methodology is applied to perform the progressive collapse analysis of
436 infilled frames case-study structure. A multi-bay, six-stories infilled frame reinforced concrete
437 structure (**Fig. 18**) is used as example. The structure is supposed undergoing the central columns loss.
438 The frame is constituted by concrete having compressive strength $f_c=25$ MPa and steel rebars with
439 yielding stress $f_y=450$ MPa. The structure is designed according to the Italian design regulation (NTC
440 2018). Geometric and mechanical details of the frame and of the infills are reported in **Table 11**.
441 Detail of reinforcement of the central spans are illustrated in **Table 12**. As regards the infills outside
442 the progressive collapse mechanism, these are modelled with a standard equivalent strut model (e.g.
443 **Di Trapani et al., 2018**). A semi-rigid lateral constrain degree is assumed for the first five stories,
444 while for the last storey the constraint degree is supposed as free. The structure is subjected to a
445 pushdown tests, comparing the results with those obtained supposing the bare frame configuration in
446 the central bays. The detailed calculation steps are exposed in Appendix. The resulting geometry for
447 the equivalent strut model is illustrated in **Fig. 19**.
448 Results of the pushdown tests are shown in **Fig. 20a**. The latter show a strength increment of +40%
449 for the model having the infills in the central bays and noticeable stiffness increase. The post-peak

450 behaviour is, as expected, characterized by a rapid strength loss associated with the failure of the
451 infills. On the contrary, the model with bare central spans shows a more ductile behaviour up to a
452 displacement of 200 mm followed by a rapid strength loss. The two models show similar residual
453 resistance in the last stages of the test. A test of the adequacy of the hypothesis of considering a semi-
454 rigid behaviour of lateral constrains by **Eq. (25)** has been also carried out. **Fig. 20b** shows a good
455 agreement between the average maximum value of the dimensionless axial force predicted by **Eqs**
456 **(25)** and the actual dimensionless axial force trend during the analysis. The deformed shapes at the
457 same vertical displacement (300 mm) of the two models are finally shown in **Fig. 21**.

458
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Conclusions

460 The simulation of progressive collapse response of frame structures due to accidental column losses
461 is complicated because of the high mechanic and geometric nonlinearity. The complexity of the
462 collapse mechanism is accentuated by the influence of masonry infills which significantly interact
463 with primary RC structures. From the observation of experimental and numerical pushdown tests it
464 is evident that the infill-frame interaction induce: a) a migration of the plastic hinges towards the
465 inner part of the beams; b) the formation of regions without damage for the masonry portions enclosed
466 at the corners because of the confining action exerted by the frame in the collapsed central column
467 configuration; b) the sliding of bed-joints or diagonal crushing in the central zones of the infills.
468 These mechanisms can influence in a more or less relevant way the overall resistance, and this
469 depends each time on the coupling between the frame and the infill.

470 The paper presented a simplified multi-strut macro-model to be used for progressive collapse
471 simulations. The major novelty is just the capability of the model to adapt its geometry and
472 mechanical properties of the struts as a function the characteristics of the infilled frame, to match as
473 much as possible the mechanisms above described. The model provides different definition of the
474 geometry in the case of multi-storey frames. The definition of the model is regulated by two
475 fundamental parameters, α_b and ξ , modulating the slope of the external struts and the stress-strain of

476 the central strut respectively. These parameters can be evaluated by the analytical correlations
477 provided in the paper and basically depend on the strength and stiffness ratios between the frame and
478 the infill. The analytical correlations have been obtained from an empirical hybrid dataset, defined by
479 merging test data from real experimental tests and refined numerical simulations. The validation of
480 the model, carried out against four external blind tests, gave confirmation of a sufficient predictive
481 reliability of the model, even in consideration of the significant uncertainties affecting the response
482 of infilled frame systems. Concluding, the proposed model results a flexible and computationally
483 effective instrument to be used for the design and the assessment of robustness of new and existing
484 reinforced concrete infilled frames. Considering the noticeable influence of the infills to the overall
485 progressive collapse response, future works could exploit this modelling methodology to investigate
486 and compare the progressive collapse responses of bare and infilled frames under dynamic regime.

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APPENDIX – Details of calculations for the sample structure

489 With reference to the geometric and material details reported in **Table 11** for the sample structure,
490 the definition of the multi-strut model provides the following calculation steps:

491 1. Calculate $\lambda^*, c^*, \beta^*, w$ for S1 strut

$$\lambda^* = \frac{\tilde{E}_m}{E_c} \frac{t l_b'}{A_b} \left(\frac{l_b'^2}{h_c'^2} + \frac{1}{4} \frac{A_b}{A_c} \frac{h_c'}{l_b'} \right) = \frac{5675}{31476} \frac{300 \times 6300}{150000} \left(\frac{6300^2}{3400^2} + \frac{1}{4} \frac{150000}{320000} \frac{3400}{6300} \right) = 7.94 \quad (27)$$

$$c^* \cong 0.254; \quad \beta^* \cong 0.147 \quad (28)$$

$$w = \frac{h_c}{l_b} \frac{c^*}{\lambda^* \beta^*} d = \frac{2900}{5500} \frac{0.254}{7.94^{0.147}} 7159 = 707 \text{ mm} \quad (29)$$

492 2. Calculate the nominal stress-strain parameters for S1 strut $f_{md0}, f_{mdu}, \varepsilon_{md0}, \varepsilon_{mdu}$ (**Eqs. 5-12**)::

$$\alpha = \frac{\tilde{f}_m^2 \cdot w \cdot t}{f_{vm}^{0.2} (h_c / l_b) \cdot \lambda^{*0.2}} = \frac{6.02^2 \times 707 \times 300}{1.07^{0.2} (2900 / 5500) 7.94^{0.2}} = 9.50 \times 10^6 \quad (30)$$

$$\beta = \frac{f_{md0}^{0.7} \cdot w \cdot t}{\tilde{E}_m^{0.2} d} = \frac{1.61^{0.7} \times 707 \times 300}{5675^{0.2} 7159} = 7.34 \quad (31)$$

$$\gamma = \left(\frac{f_{mdu}^2}{f_{md0}} \right) \left(\frac{E_c}{\tilde{E}_m^{1.5}} \right) = \left(\frac{0.41^2}{1.61} \right) \left(\frac{31476}{5675^{1.5}} \right) = 0.00772 \quad (32)$$

$$\delta = \tilde{E}_m^{0.20} \cdot \varepsilon_{md0} = 5675^{0.20} \times 0.000839 = 0.00473 \quad (33)$$

$$f_{md0} = \tilde{f}_m \cdot 26.9 \cdot \alpha^{-0.287} = 6.02 \times 26.9 \times (9.50 \times 10^6)^{-0.287} = 1.61 \text{ MPa} \quad (34)$$

$$f_{mdu} = f_{md0} (0.043 \cdot \beta - 0.06) = 1.61 (0.043 \times 7.34 - 0.06) = 0.41 \text{ MPa} \quad (35)$$

$$\varepsilon_{md0} = \varepsilon_{m0} \cdot 3.024 \cdot \gamma^{0.347} = 0.0015 \times 3.024 \times 0.00772^{0.347} = 0.000839 \quad (36)$$

$$\varepsilon_{mdu} = \varepsilon_{md0} \cdot 0.0184 \cdot \delta^{-1.166} = 0.000839 \cdot 0.0184 \cdot 0.00473^{-1.166} = 0.00794 \quad (37)$$

493 3. Calculate ξ (Eqs. 14-20)

494

$$\begin{aligned} \theta_{\min} &= \arctan \left(\frac{nh'_c}{nl'_b - (n-1)(0.15l_b + 0.5h_{wc})} \right) = \\ &= \arctan \left(\frac{5 \times 3400}{5 \times 6300 - 4(0.15 \times 5500 + 0.5 \times 800)} \right) = 32.6^\circ \end{aligned} \quad (38)$$

$$\begin{aligned} \theta_{\max} &= \arctan \left(\frac{nh'_c}{nl'_b - (n-1)(0.5l_b + 0.5h_{wc})} \right) = \\ &= \arctan \left(\frac{5 \times 3400}{5 \times 6300 - 4(0.5 \times 5500 + 0.5 \times 800)} \right) = 42.0^\circ \end{aligned} \quad (39)$$

$$\theta_j = \frac{\theta_{\min} + \theta_{\max}}{2} = \frac{32.6^\circ + 42.0^\circ}{2} = 37.3^\circ \quad (40)$$

495 The beams belonging to the first five spans are considered having a semi-rigid lateral constrains at

496 the ends ($\psi_i=9, \lambda_j=0.75$). The beams belonging to the last span are considered having no lateral

497 constrains ($\psi_i=6, \lambda_j=0.75$). Therefore:

$$K_f = \sum_{i=1}^{n_b} \psi_i \cdot \frac{E_c I_{bi}}{l_{bi}^3} = (9 \times 10 + 6 \times 2) \times \frac{31476 \times 3125000000}{5500^3} = 60303 \text{ kN} / \text{m} \quad (41)$$

$$K_s = \sum_{j=1}^{n_s} \lambda_j \frac{\tilde{E}_{mj} \cdot t_j \cdot w_j}{d_j} \cdot \sin \theta_j = (0.75 \times 8 + 0.5 \times 2) \times \frac{5675 \times 300 \times 707}{7159} \sin(37.3^\circ) = 713208 \text{ kN} / \text{m} \quad (42)$$

$$\kappa = \frac{K_f}{K_s} \sqrt{\frac{l_b}{h_b}} \sqrt{\frac{\tilde{f}_m}{f_{vm}}} = \frac{60303}{713208} \sqrt{\frac{5500}{2900}} \sqrt{\frac{6.02}{1.07}} = 0.28 \quad (43)$$

$$\xi = 0.212 \cdot e^{1.45\kappa} = 0.212 \cdot e^{1.45 \times 0.28} = 0.32 \quad (44)$$

498 4. Calculate \tilde{f}_{md0} and \tilde{f}_{mdu} (**Eqs. 13**):

$$\tilde{f}_{md0} = \xi \cdot f_{md0} = 0.32 \times 1.61 = 0.51 \text{ MPa} \quad (45)$$

$$\tilde{f}_{mdu} = \xi \cdot f_{mdu} = 0.32 \times 0.41 = 0.13 \text{ MPa} \quad (46)$$

499 5. Calculate α_b (**Eqs. 21-26**):

$$\chi = \frac{h_{wb}}{l_b} = \frac{500}{5500} = 0.091 \quad (47)$$

500 Consistently with the previous assumptions the dimensionless axial forces of the beams belonging to
 501 the first five spans are evaluated as average ($v_{b,max(semi-rigid)}$) between the values obtained from the two
 502 expressions in **Eq. (26)**. The second expression in **Eq. (26)** is used for the top beams ($v_{b,max(free)}$).

$$v_{b,max(rigid)} = -8.27\chi^2 + 2.74\chi + 0.02 = -8.27 \times 0.091^2 + 2.74 \times 0.091 + 0.02 = 0.20 \quad (48)$$

$$v_{b,max(free)} = -5.79\chi^2 + 1.36\chi - 0.04 = -5.79 \times 0.091^2 + 1.36 \times 0.091 - 0.04 = 0.036 \quad (49)$$

$$v_{b,max(semi-rigid)} = v_{b,max(rigid)} + v_{b,max(free)} = \frac{0.20 + 0.036}{2} = 0.12 \quad (50)$$

503 The resulting axial forces used to evaluate the plastic end moments of the beams are then:

504

$$N_{b,max(semi-rigid)} = v_{b,max(semi-rigid)} f_c h_{wb} b_{wb} = 0.12 \times 25 \times 500 \times 300 = 450000 \text{ N} = 450 \text{ kN} \quad (51)$$

$$N_{b,max(free)} = v_{b,max(free)} f_c h_{wb} b_{wb} = 0.036 \times 25 \times 500 \times 300 = 135000 \text{ N} = 135 \text{ kN} \quad (52)$$

It follows:

$$R_f = \sum_{i=1}^{n_{M_p}} \frac{M_{p,i}^-}{l_b} + \sum_{i=1}^{n_{M_p}} \frac{M_{p,i}^+}{l_b} = 2 \left(\frac{368.5 + 368.5 + 368.5 + 327.8 + 327.8 + 273.3}{5.5} \right) + \quad (53)$$

$$+ 2 \left(\frac{265.2 + 265.2 + 265.2 + 224.6 + 224.6 + 154.7}{5.5} \right) = 1249 \text{ kN}$$

$$R_s = \sum_{j=1}^{n_s} \tilde{f}_{mdo,j} \cdot t_j \cdot w_j \cdot \sin \theta_j = 10 \times 0.51 \times 300 \times 707 \times \sin(37.3^\circ) = 655503 \text{ N} = 655.5 \text{ kN} \quad (54)$$

$$\rho = \frac{R_f}{R_s} = \frac{1249}{655.5} = 1.90 \quad (55)$$

Calculation of α_b using **Eq. (24)**

$$\alpha_b = -0.09\rho^3 + 0.54\rho^2 - 1.05\rho + 0.86 = -0.09 \times 1.90^3 + 0.54 \times 1.90^2 - 1.05 \times 1.90 + 0.86 = 0.20 \quad (56)$$

$$\alpha_b l_b = 0.20 \times 5500 = 1100 \text{ mm} \quad (57)$$

Alternative calculation of α_b using **Eq. (25)**

$$\alpha_b = 0.28\rho^{-0.58} = 0.28 \cdot 1.90^{-0.58} = 0.19 \quad (58)$$

505

Data Availability Statement

506

Some or all data, models, or code generated or used during the study are available from the

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corresponding author by request. (OpenSees models).

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613 TABLES

614 Table 1. Experimental and numerical specimens and test set-up details.

Author	Specimen code	Type	Sotrey no.	Bays no.	Scale	Masonry type	Lateral constraint	Seismic detailing	Aspect ratio (l_b/h_c)
Li et al. (2019)	(no code)		1	2	1/3	Brick	two-sides (partial)	Yes	1.58
Qian and Li (2017)	WSS	Experimental	2	1	1/4	Brick	one-side (full)	Yes	2.41
Qian and Li (2017)	WNL		2	1	1/4	Brick	one-side (full)	No	3.28
Di Trapani et al. (2020a)	FEM-S1F-B500						no	Yes	1.02
	FEM-S1R-B500						two-sides (full)	Yes	1.02
	FEM-nS1F-B500						no	No	1.02
	FEM-nS1R-B500						two-sides (full)	No	1.02
	FEM-S2R-B500	Numerical	1	1	1/1	Clay hollow	two-sides (full)	Yes	2.04
	FEM-nS2F-B500						no	No	2.04
	FEM-nS2F-B200						no	No	1.83
	FEM-nS2F-B300						no	No	1.90
	FEM-nS2R-						two-sides (full)	No	1.90

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616 Table 2. Geometric and mechanical details of experimental and numerical specimens and parameters for the
617 identification of the equivalent strut width of S1 struts

Specimen reference	t (mm)	h_c (mm)	h_c' (mm)	l_b (mm)	l_b' (mm)	d (mm)	b_{wc} (mm)	h_{wc} (mm)	b_{wb} (mm)	h_{wb} (mm)	E_{m2} (MPa)	E_{m1} (MPa)	\tilde{E}_m (MPa)	E_c (MPa)	λ^* -	w (mm)
Li et al. (2019)	63.5	950	1100	1500	1700	2025	200	200	100	150	5376	7680	6426	35725	3.170	274.5
Qian and Li (2017) - WSS	65	685	825	1650	1800	1980	150	150	90	140	4410	6300	5271	32837	7.191	156.1
Qian and Li (2017) - WNL	65	685	825	2250	2400	2538	150	150	90	140	4410	6300	5271	32837	16.915	129.4
FEM-S1F-B500	300	2670	3170	2720	3440	4678	720	300	300	500	6401	5032	5675	31476	1.659	1080.8
FEM-S1R-B500	300	2670	3170	2720	3440	4678	720	300	300	500	6401	5032	5675	31476	1.659	1080.8
FEM-nS1F-B500	300	2670	3170	2720	3440	4678	720	300	300	500	6401	5032	5675	31476	1.659	1080.8
FEM-nS1R-B500	300	2670	3170	2720	3440	4678	720	300	300	500	6401	5032	5675	31476	1.659	1080.8
FEM-S2R-B500	300	2670	3170	5440	6160	6928	720	300	300	500	6401	5032	5675	31476	8.587	629.0
FEM-nS2F-B500	300	2670	3170	5440	6160	6928	720	300	300	500	6401	5032	5675	31476	8.587	629.0
FEM-nS2F-B200	300	2970	3170	5440	6160	6928	720	300	1000	200	6401	5032	5675	31476	6.490	729.0
FEM-nS2F-B300	300	2870	3170	5440	6160	6928	720	300	700	300	6401	5032	5675	31476	6.190	709.4
FEM-nS2R-B300	300	2870	3170	5440	6160	6928	720	300	700	300	6401	5032	5675	31476	6.190	709.4

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619 Table 3. Masonry infill material properties and parameters for the determination of the nominal stress-strain
620 response of S1 struts

Specimen reference	f_{vm} (MPa)	f_{m2} (MPa)	f_{m1} (MPa)	\tilde{f}_m (MPa)	α -	β -	γ -	δ -	f_{md0} (MPa)	f_{mdu} (MPa)	ε_{md0} -	ε_{mdu} -
Li et al. (2019)	1.08	8.96	12.8	10.71	2.47E+06	4.08	0.00390	0.003823	4.22	0.49	0.000662	0.00803
Qian and Li (2017) - WSS	1.10	7.35	10.5	8.78	1.25E+06	2.52	0.00271	0.003237	4.21	0.42	0.000583	0.00859
Qian and Li (2017) - WNL	1.10	7.35	10.5	8.78	1.19E+06	1.65	0.00366	0.003594	4.27	0.43	0.000647	0.00844
FEM-S1F-B500	1.07	4.18	8.66	6.02	1.07E+07	16.77	0.05008	0.009042	1.56	1.03	0.001605	0.00713
FEM-S1R-B500	1.07	4.18	8.66	6.02	1.07E+07	16.77	0.05008	0.009042	1.56	1.03	0.001605	0.00713
FEM-nS1F-B500	1.07	4.18	8.66	6.02	1.07E+07	16.77	0.05008	0.009042	1.56	1.03	0.001605	0.00713
FEM-nS1R-B500	1.07	4.18	8.66	6.02	1.07E+07	16.77	0.05008	0.009042	1.56	1.03	0.001605	0.00713
FEM-S2R-B500	1.07	4.18	8.66	6.02	8.93E+06	6.83	0.00658	0.004471	1.64	0.38	0.000794	0.00802
FEM-nS2F-B500	1.07	4.18	8.66	6.02	8.93E+06	6.83	0.00658	0.004471	1.64	0.38	0.000794	0.00802
FEM-nS2F-B200	1.07	4.18	8.66	6.02	9.84E+06	7.76	0.00878	0.004942	1.59	0.44	0.000877	0.00789
FEM-nS2F-B300	1.07	4.18	8.66	6.02	1.00E+07	7.53	0.00811	0.004808	1.59	0.42	0.000853	0.00792
FEM-nS2R-B300	1.07	4.18	8.66	6.02	1.00E+07	7.53	0.00811	0.004808	1.59	0.42	0.000853	0.00792

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625 **Table 4.** Parameters after the calibration α_b and ξ and resulting dimensional properties.

Specimen reference	α_b	ξ	$\alpha_b \cdot l_b$	$\tilde{f}_{md0} = \xi \cdot f_{md0}$	$\tilde{f}_{mdu} = \xi \cdot f_{mdu}$
	-	-	(mm)	(MPa)	(MPa)
Li et al. (2019)	0.500	1	750	4.22	0.49
Qian and Li (2017) - WSS	0.200	0.75	330	3.16	0.32
Qian and Li (2017) - WNL	0.500	0.85	1125	3.63	0.36
FEM-S1F-B500	0.250	0.90	680	1.40	0.93
FEM-S1R-B500	0.200	1.00	544	1.56	1.03
FEM-nS1F-B500	0.500	0.85	1360	1.32	0.87
FEM-nS1R-B500	0.300	1.00	816	1.56	1.03
FEM-S2R-B500	0.200	0.60	1088	0.98	0.23
FEM-nS2F-B500	0.200	0.50	1088	0.82	0.19
FEM-nS2F-B200	0.200	0.25	1088	0.40	0.11
FEM-nS2F-B300	0.150	0.25	816	0.40	0.10
FEM-nS2R-B300	0.150	0.40	816	0.63	0.17

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627 **Table 5.** Parameters κ and ρ and associated data for their determination.

Specimen reference	K_f	K_s	κ	N_{bmax}	R_f	R_s	ρ	θ
	(kN/m)	(kN/m)	-	(kN)	(kN)	(kN)	-	(°)
Li et al. (2019)	10717.5	45079.7	0.94	34.51	39.70	79.86	0.50	32.91
Qian and Li (2017) - WSS	8123.5	36781.2	0.97	17.31	67.03	58.13	1.15	27.00
Qian and Li (2017) - WNL	3203.7	22151.3	0.74	9.82	26.61	51.56	0.52	25.00
FEM-S1F-B500	117309.0	266586.3	1.05	80.53	478.24	615.59	0.78	42.66
FEM-S1R-B500	234618.0	533172.5	1.05	1084.00	977.65	683.99	1.43	42.66
FEM-nS1F-B500	117309.0	266586.3	1.05	35.00	214.12	581.39	0.37	42.66
FEM-nS1R-B500	234618.0	533172.5	1.05	943.00	697.06	683.99	1.02	42.66
FEM-S2R-B500	29327.3	141476.0	0.70	871.00	453.42	169.68	2.67	27.23
FEM-nS2F-B500	14663.6	70738.0	0.70	104.18	218.80	141.40	1.55	27.23
FEM-nS2F-B200	3128.2	81981.7	0.12	41.61	164.46	79.68	2.06	27.23
FEM-nS2F-B300	7390.5	79771.9	0.30	84.92	222.35	77.17	2.88	27.23
FEM-nS2R-B300	14780.9	159543.8	0.30	1040.00	375.07	123.48	3.04	27.23

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630 **Table 6.** Experimental and numerical blind test specimens and test set-up details.

Author	Specimen code	Type	Storey no.	Bays no.	Scale	Masonry type	Lateral constraint	Seismic detailing	Aspect ratio (l_b/h_c)
Qian and Li (2017)	WNS	Experimental	2	1	1/4	Brick	one-side (full)	No	2.41
Di Trapani et al. (2020a)	FEM-S2F-B500	Numerical	1	1	1/1	Clay hollow	no	Yes	2.04
	two-sides (full)						No	2.04	
	two-sides (full)						No	1.83	

631 **Table 7** Geometric and mechanical details of experimental and numerical blind test specimens and parameters

632 for the identification of the equivalent strut width of S1 struts

Specimen reference	t	h_c	h_c'	l_b	l_b'	d	b_{wc}	h_{wc}	b_{wb}	h_{wb}	E_{m2}	E_{m1}	\tilde{E}_m	E_c	λ^*	w
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	-	(mm)
Qian and Li (2017) - WNS	65	685	825	1650	1800	1980	150	150	90	140	4410	6300	5271	32837	7.191	156.1
FEM-S2F-B500	300	2670	3170	5440	6160	6928	720	300	300	500	6401	5032	5675	31476	8.587	629.0
FEM-nS2R-B500	300	2670	3170	5440	6160	6928	720	300	300	500	6401	5032	5675	31476	8.587	629.0
FEM-nS2R-B200	300	2970	3170	5440	6160	6928	720	300	1000	200	6401	5032	5675	31476	6.490	729.0

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Table 8. Masonry infill material properties and parameters for the determination of the nominal stress-strain response of S1 struts for the blind test specimens.

Specimen reference	f_{vm} (MPa)	f_{m2} (MPa)	f_{m1} (MPa)	\tilde{f}_m (MPa)	α -	β -	γ -	δ -	f_{md0} (MPa)	f_{mdu} (MPa)	ε_{md0} -	ε_{mdu} -
Qian and Li (2017) - WNS	1.1	7.35	10.5	8.78	1.25E+06	2.52	0.00271	0.003237	4.21	0.42	0.000583	0.00859
FEM-S2F-B500	1.07	4.18	8.66	6.02	8.93E+06	6.83	0.00658	0.004471	1.64	0.38	0.000794	0.008018
FEM-nS2R-B500	1.07	4.18	8.66	6.02	8.93E+06	6.83	0.00658	0.004471	1.64	0.38	0.000794	0.008018
FEM-nS2R-B200	1.07	4.18	8.66	6.02	9.84E+06	7.76	0.00878	0.004942	1.59	0.44	0.000877	0.007885

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Table 9. Parameters κ and ρ and associated data for their determination.

Specimen reference	K_f (kN/m)	K_s (kN/m)	κ -	$v_{b,max}$ -	$N_{b,max}$ (kN)	R_f (kN)	R_s (kN)	ρ -	θ (°)
Qian and Li (2017) - WNS	8123	39377	0.90	0.034	14.36	38.40	66.38	0.58	29.08
FEM-S2F-B500	14663	70738	0.70	0.036	135.33	256.91	164.02	1.57	27.23
FEM-nS2R-B500	29327	141476	0.70	0.20	757.41	396.03	164.02	2.41	27.23
FEM-nS2R-B200	6256	163963	0.12	0.11	547.79	215.66	79.68	2.71	27.23

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Table 10. Parameters α_b and ξ and resulting dimensional properties.

Specimen reference	α_b -	ξ -	$\alpha_b \cdot l_b$ (mm)	$\tilde{f}_{md0} = \xi \cdot f_{md0}$ (MPa)	$\tilde{f}_{mdu} = \xi \cdot f_{mdu}$ (MPa)
Qian et al. (2017) - WNS	0.42	0.800	693	3.37	0.34
FEM-S2F-B500	0.19	0.580	1033.6	0.95	0.22
FEM-nS2R-B500	0.20	0.580	1088	0.95	0.22
FEM-nS2R-B200	0.18	0.250	979.2	0.40	0.11

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Table 11. Geometric and mechanical properties of the infilled frames of the sample strut.

t (mm)	h_c (mm)	h_c' (mm)	l_b (mm)	l_b' (mm)	d (mm)	b_{wc} (mm)	h_{wc} (mm)	b_{wb} (mm)	h_{wb} (mm)	E_{m2} (MPa)	E_{m1} (MPa)	\tilde{E}_m (MPa)	f_{vm} (MPa)	f_{m2} (MPa)	f_{m1} (MPa)	\tilde{f}_m (MPa)
300	2900	3400	5900	6300	7159	400	800	300	500	6401	5032	5675	1.07	8.66	4.18	6.02

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Table 12. Reinforcement details for the central span beams.

Section	Stories 1-2-3		Stories 4-5-6	
	Top reinforcement	Bottom reinforcement	Top reinforcement	Bottom reinforcement
1-1	7 ϕ 16	4 ϕ 16	6 ϕ 16	3 ϕ 16
2-2	7 ϕ 16	4 ϕ 16	6 ϕ 16	3 ϕ 16

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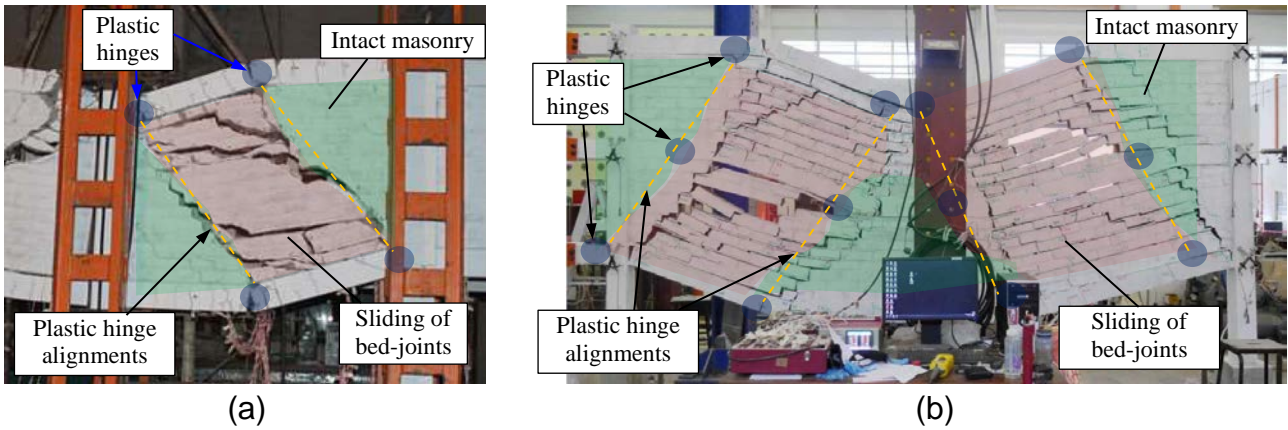


Fig. 1. Interpretation of the damage mechanism in pushdown tests of infilled frames: a) One-story infilled frame (Li et al. 2019); b) Two-story infilled frame (Qian and Li 2017).

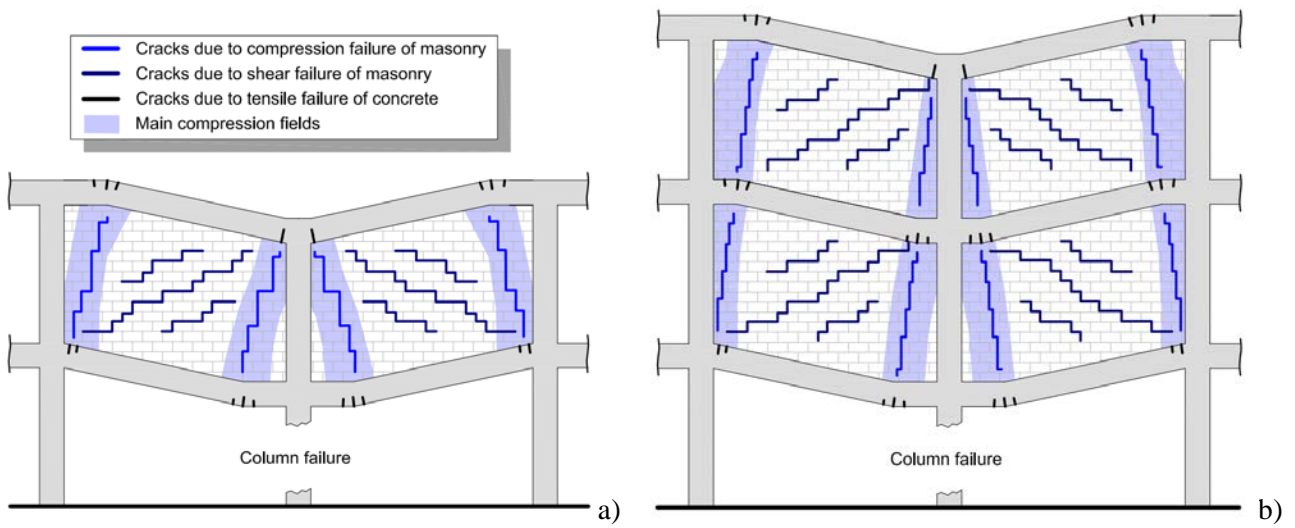


Fig. 2. Typical damage mechanisms associated with progressive collapse of infilled frames: a) One-story infilled frames; b) Multi-story infilled frames.

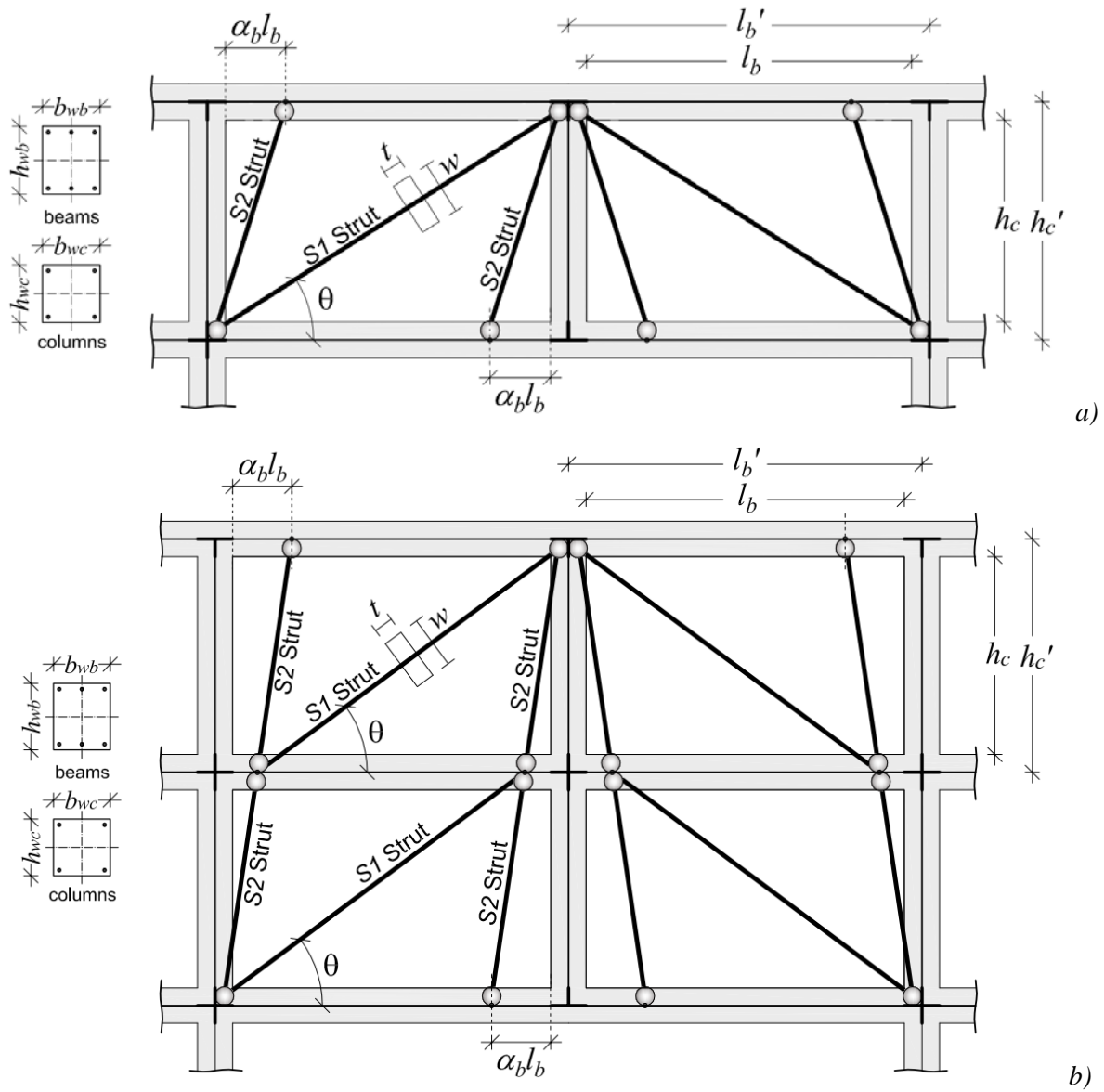


Fig. 3. Proposed equivalent strut model for: a) One-storey infilled frames; b) Multi-storey infilled frames.

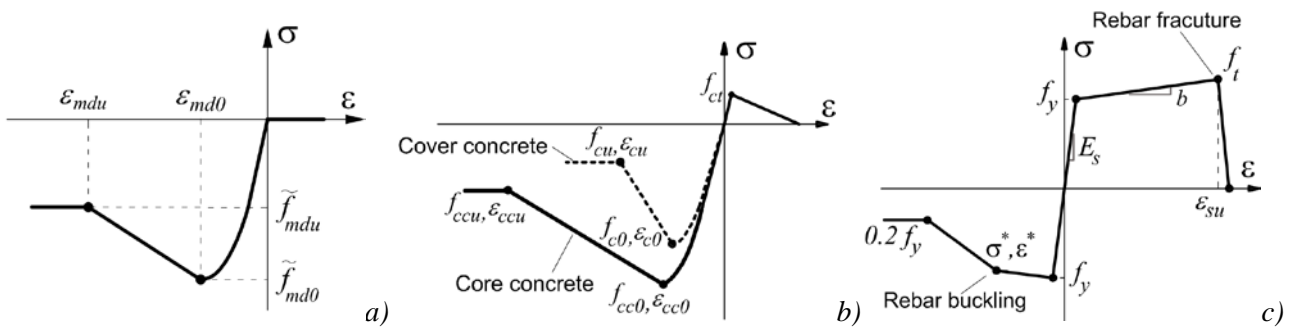


Fig. 4. Adopted stress-strain model for: a) S1 struts; b) Confined and unconfined concrete; c) Rebars.

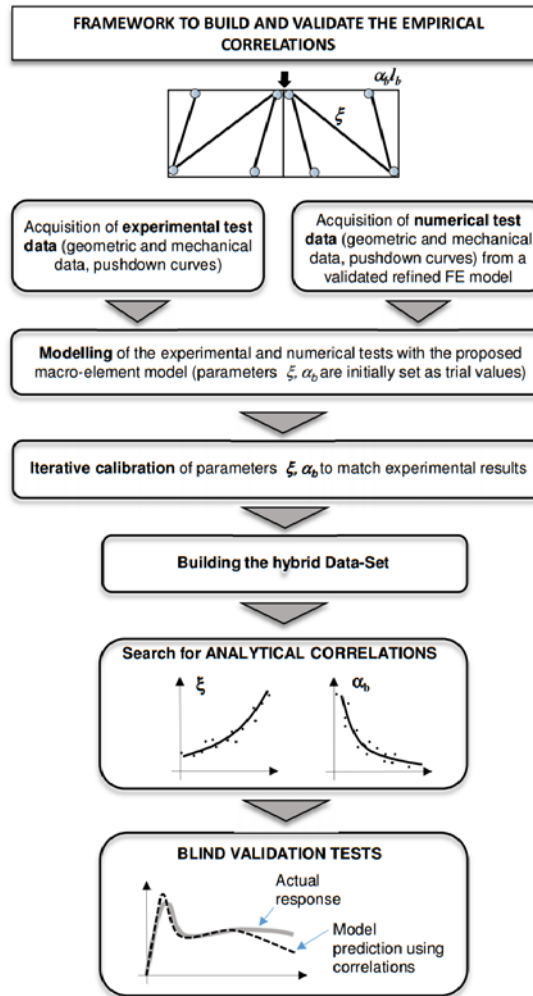


Figure 5: Framework to build and validate the empirical correlations.

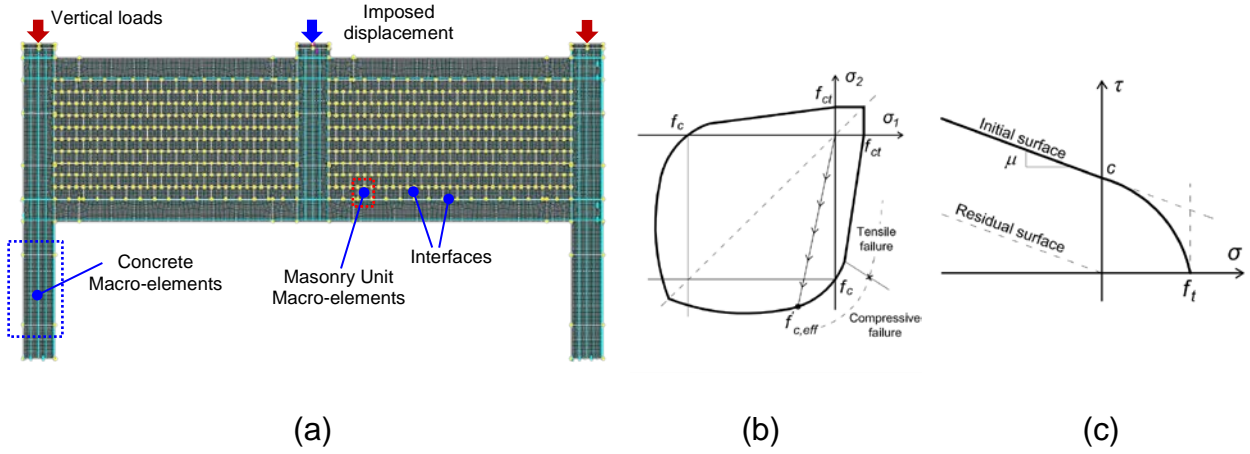


Figure 6: Refined FE model (Di Trapani et al. 2020): a) Overall assembly of the model; b) Concrete and masonry macro-elements failure surface; c) interfaces failure surface.

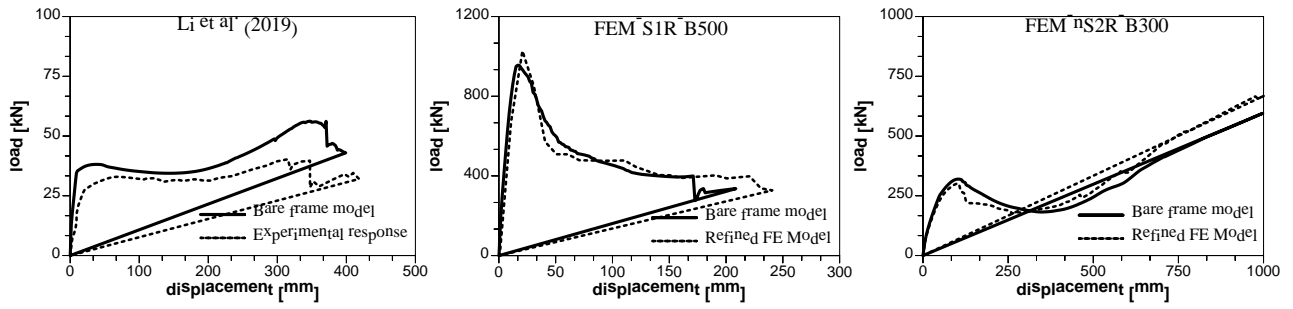


Fig. 7. Bare frame experimental and numerical pushdown curves and model predictions.

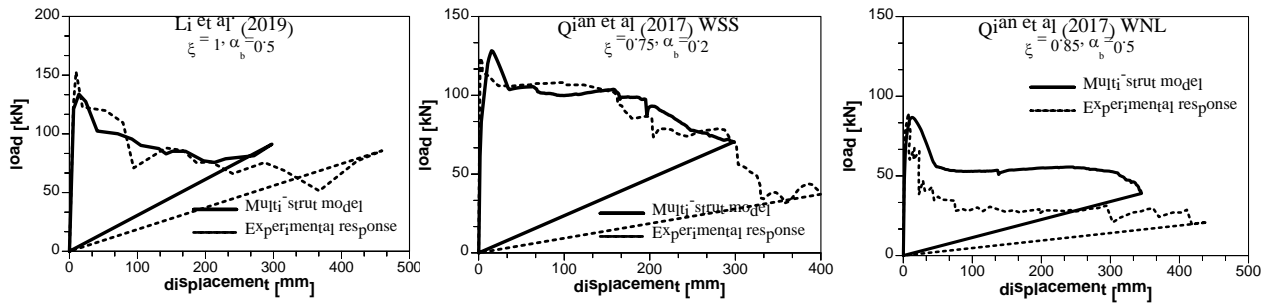


Fig. 8. Experimental pushdown curves and proposed model pushdown curves at the end of the calibration.

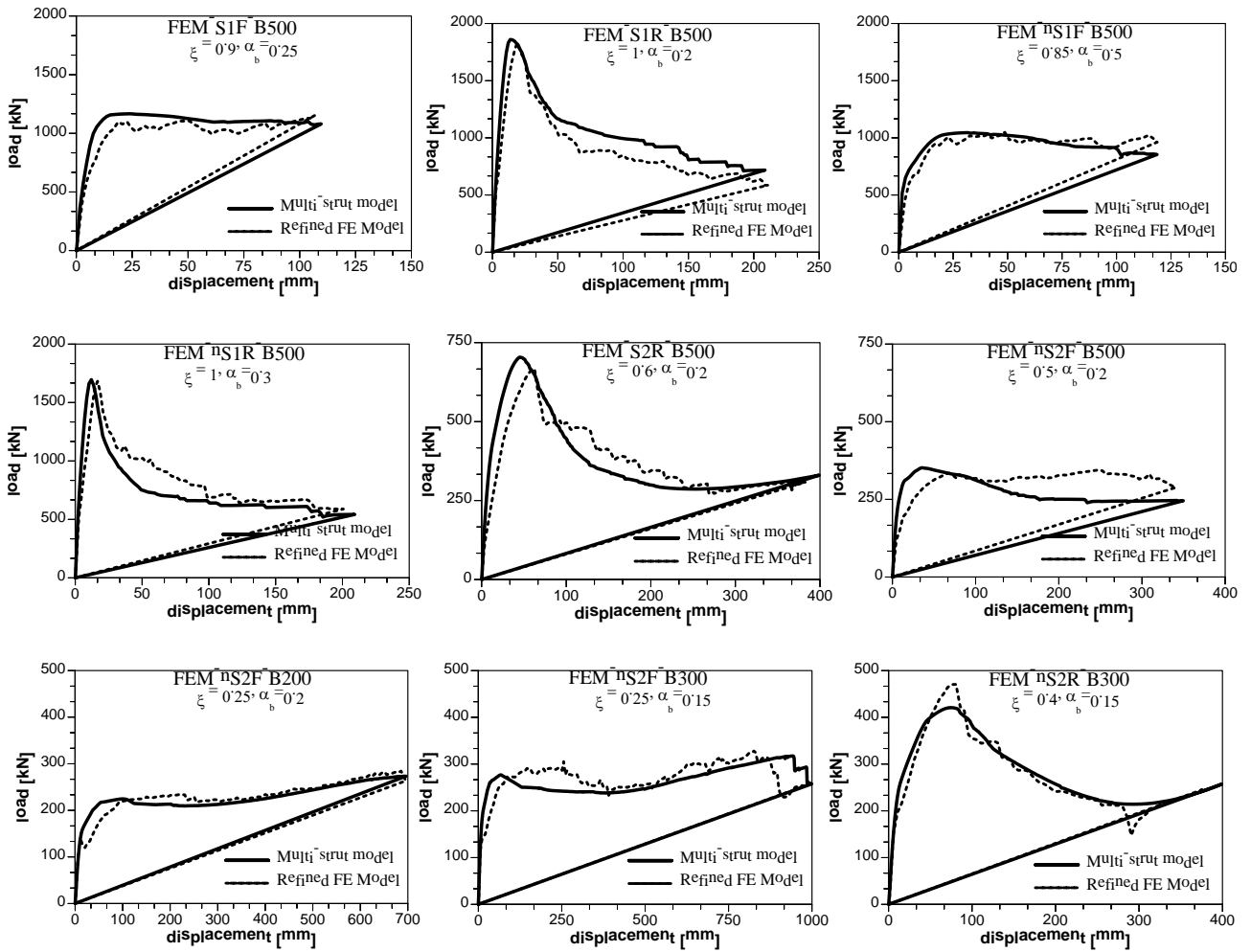


Fig. 9. Numerical pushdown curves from the refined FE model and proposed model pushdown curves at the end of the calibration.

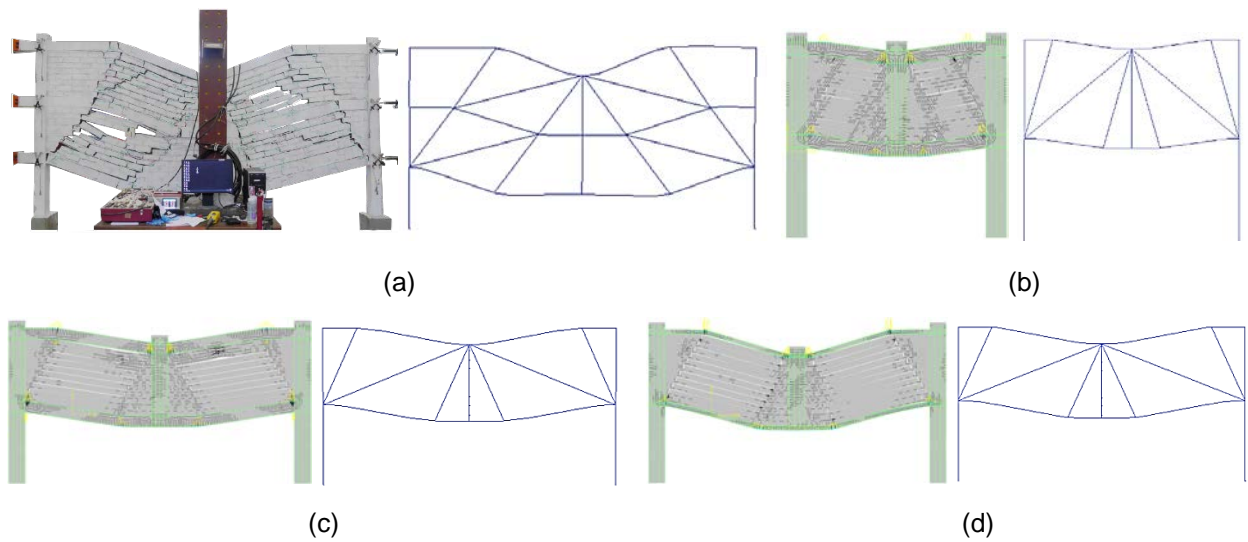


Fig. 10. Comparison between the deformed shapes of experimental and numerical specimens and those obtained after the equivalent strut models calibration: a) Qian and Li (2017) – WNL; b) FEM-S1R-B500; c) FEM-S2R-B500; d) FEM-nS2F-B200

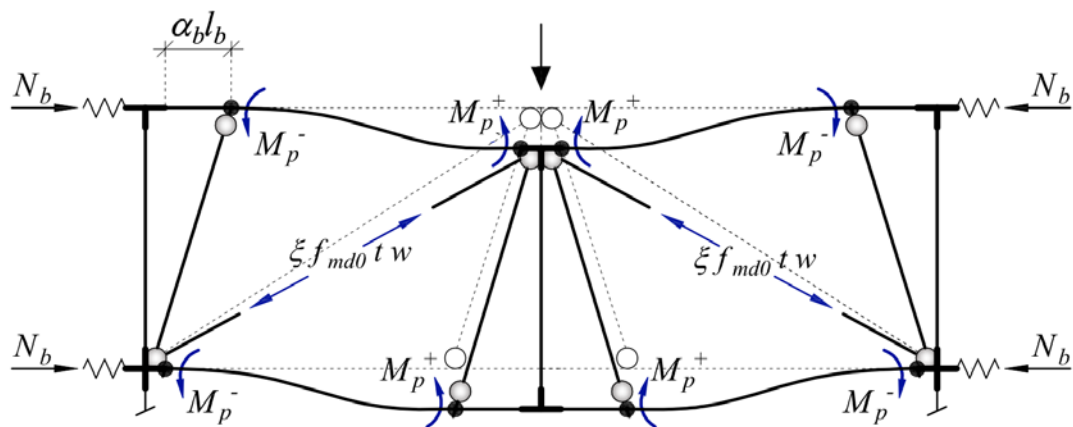


Fig. 11. Typical collapse mechanism of the proposed equivalent strut model (sample for a one-storey infilled frame).

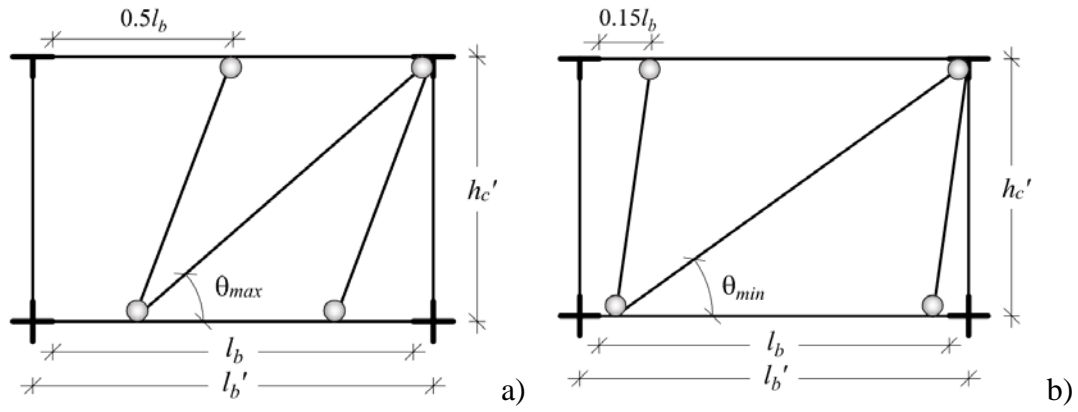


Fig. 12. Limit configurations of the equivalent strut model for the estimation of the average angle of inclination of S1 strut in multi-storey infilled frames: a) θ_{max} configuration; b) θ_{min} configuration.

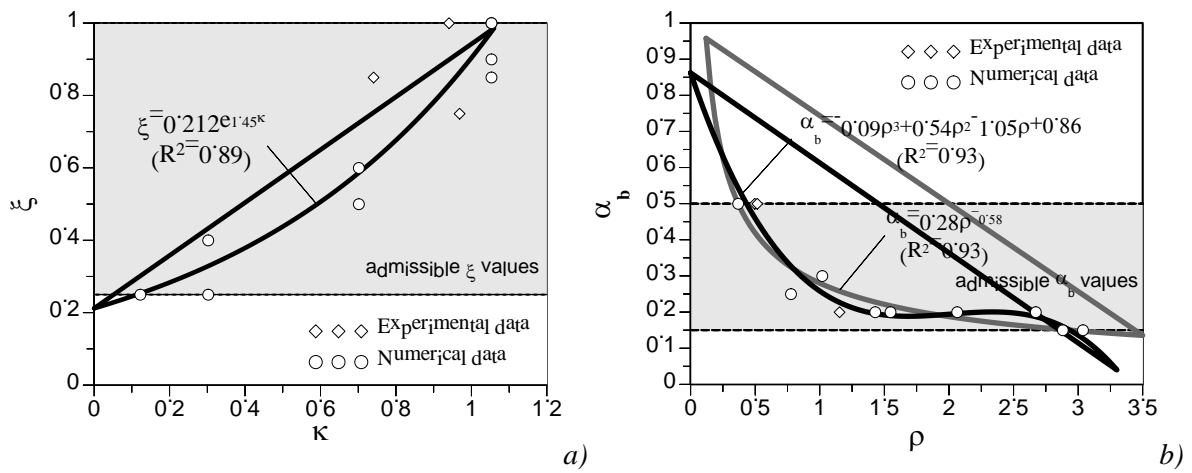


Fig. 13. Experimental and numerical data and analytical correlations: a) $\xi-\kappa$; b) $\alpha_b-\rho$.

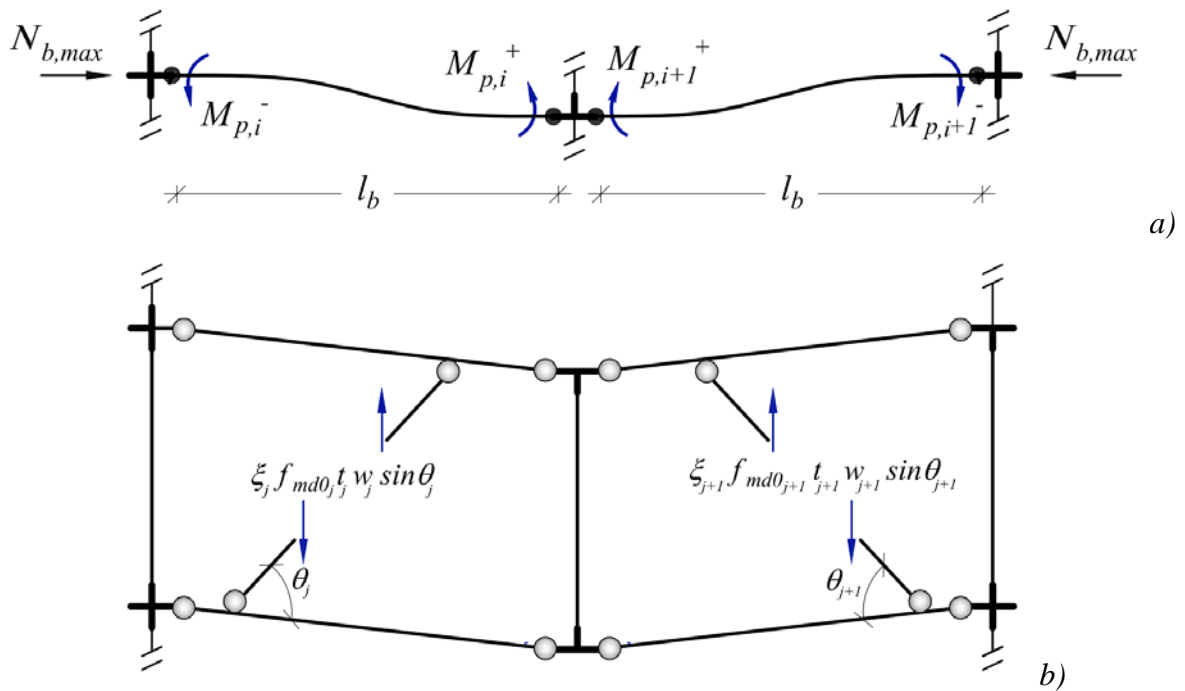


Fig. 14. Simplified limit equilibrium schemes for the evaluation of: a) R_f ; b) R_s .

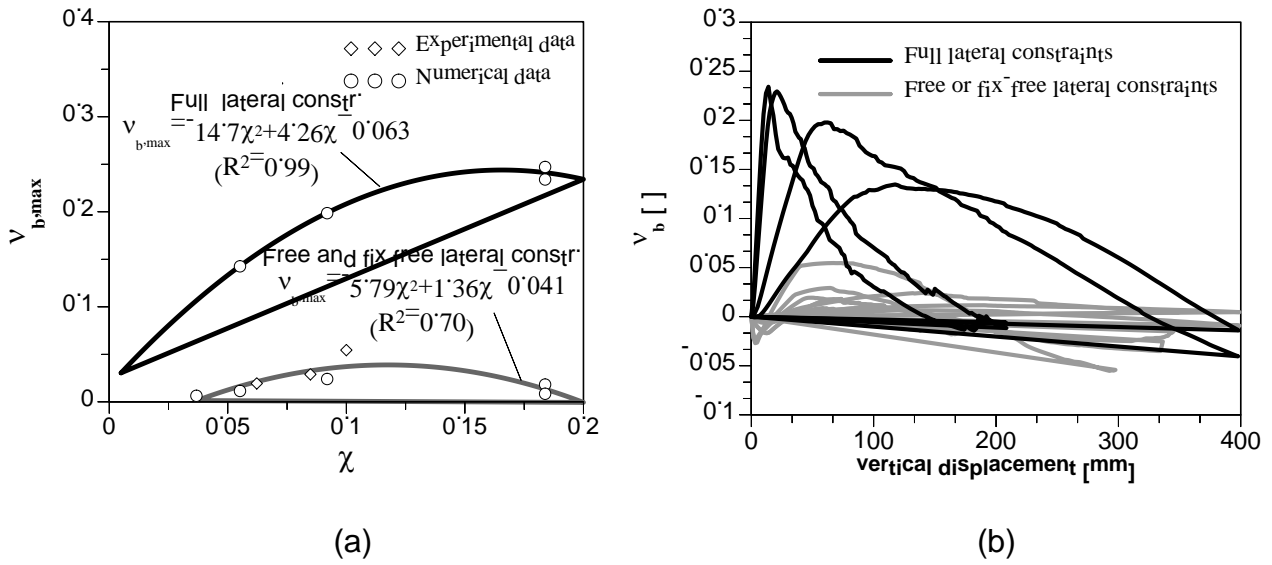


Fig. 15. Average dimensionless axial forces in beams: a) experimental and numerical data and analytical correlations with the and the height-to-length ratio (χ) for different lateral restraint conditions; b) variation with the vertical displacement.

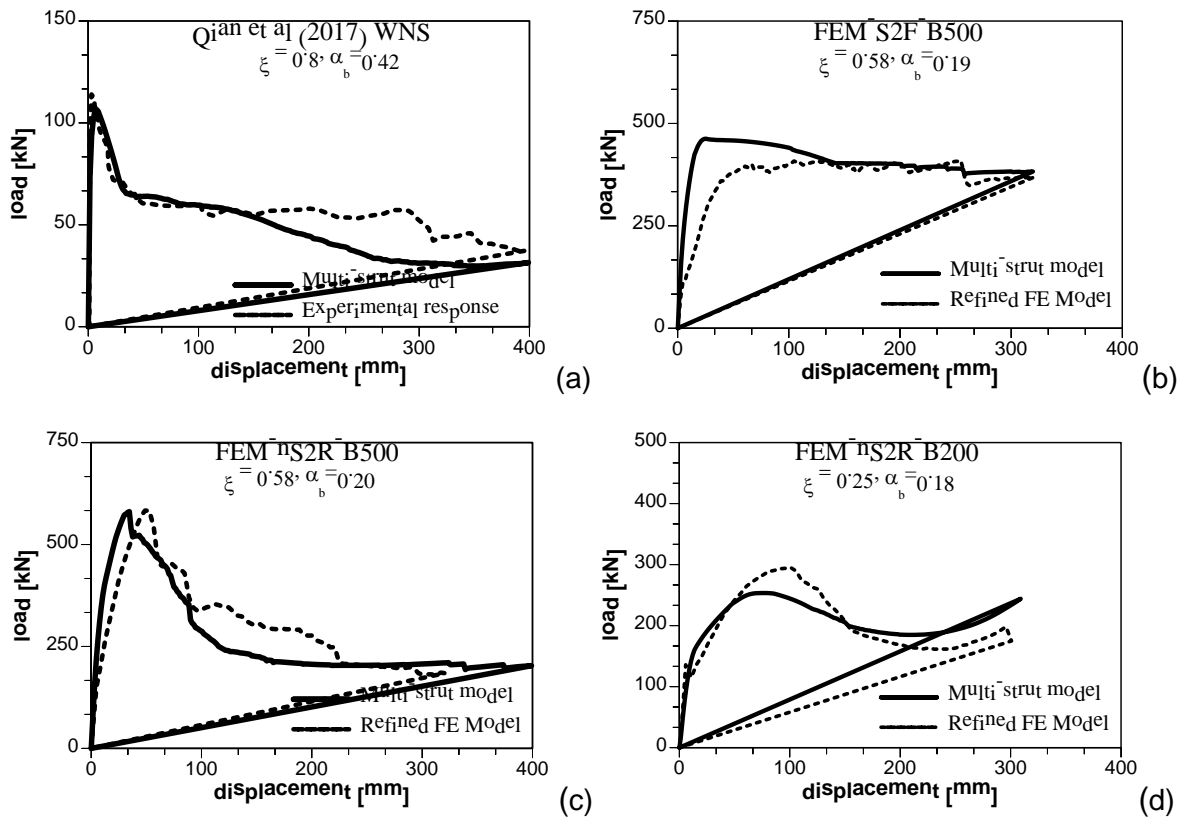


Figure 16. Blind validation tests of the proposed model with experimental and numerical test results.

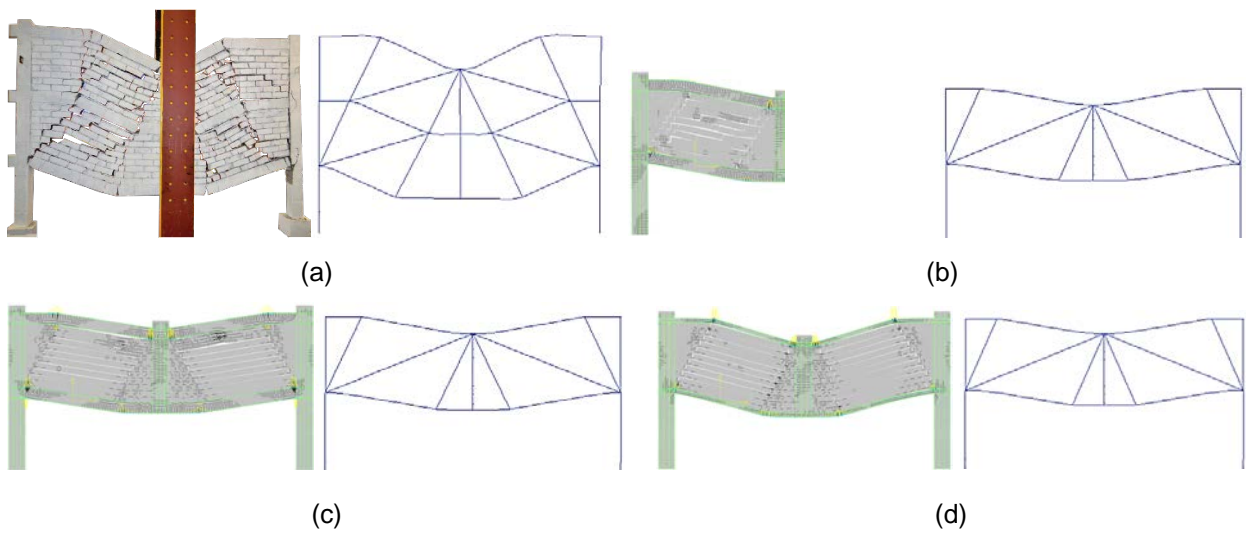


Fig. 17. Comparison between the deformed shapes of experimental and numerical blind test specimens and those predicted by the model: a) Qian and Li (2017) – WNS; b) FEM-S2F-B500; c) FEM-nS2R-B500; d) FEM-nS2R-B200

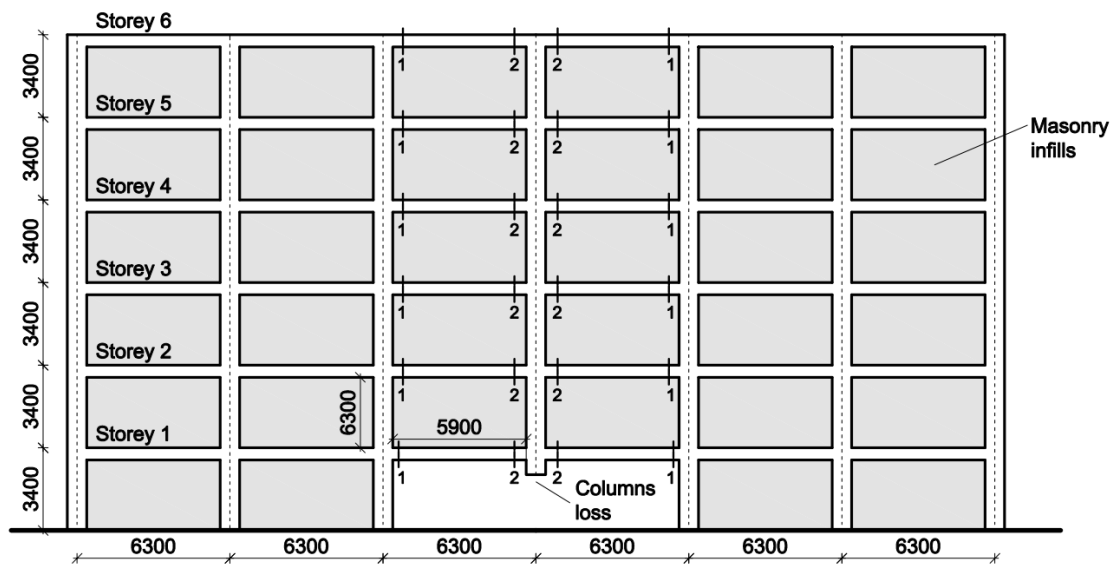


Fig. 18. Sample structure geometry (dimensions in mm).

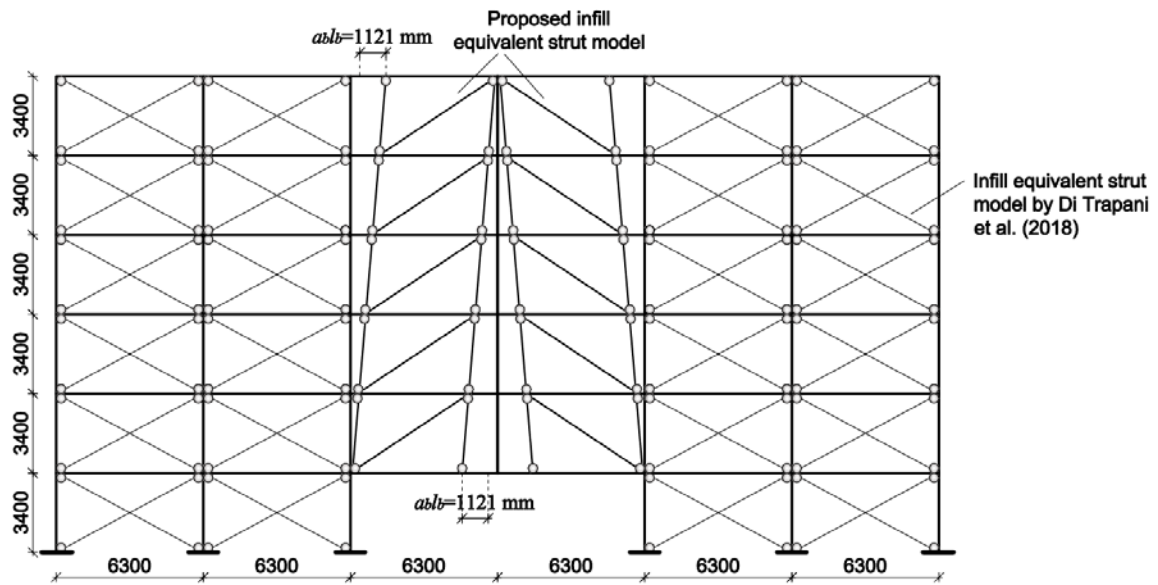


Fig. 19. Sample structure multi-strut progressive collapse model (dimensions in mm).

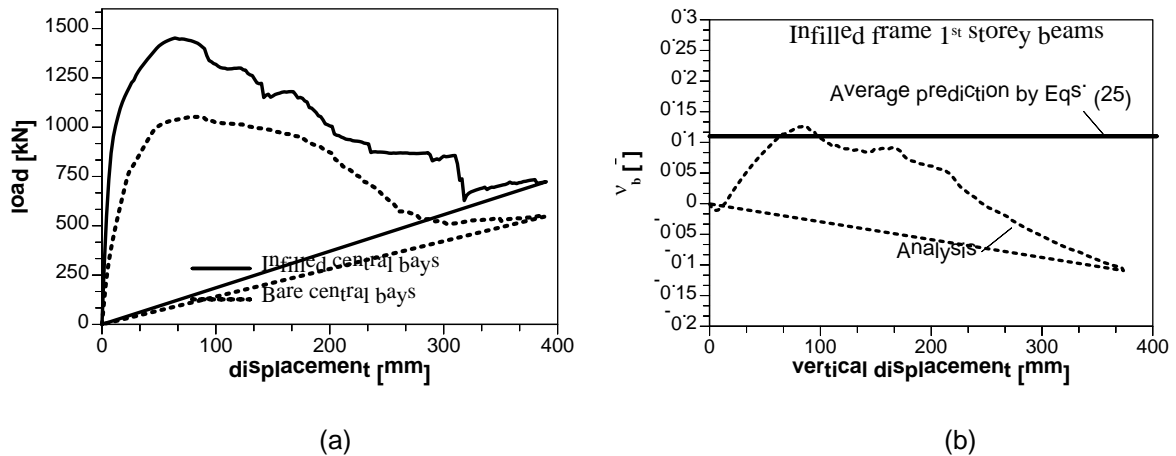


Fig. 20. Results of the pushdown tests: a) comparison between pushdown responses of the model with and without infills in the central bays; b) Dimensionless axial force in first storey beams for the model with the infills in the central bays.

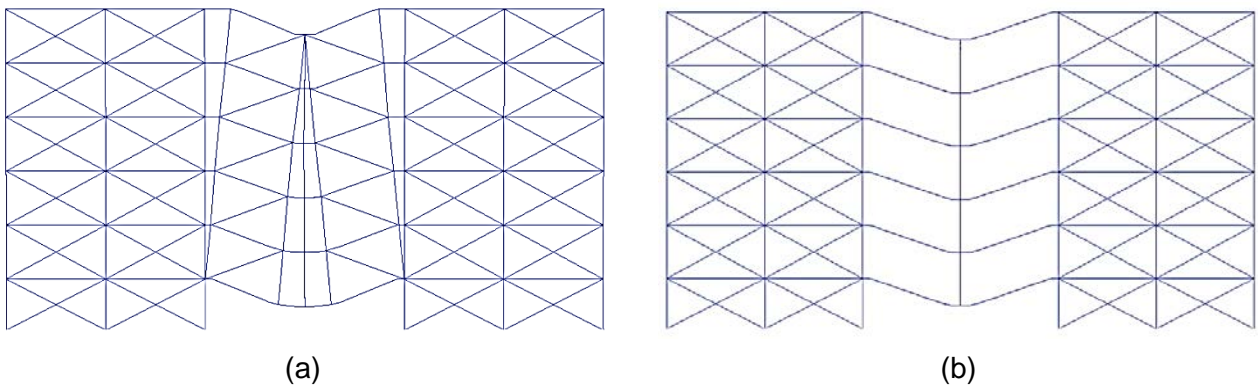


Fig. 21. Deformed shapes of the models during the pushdown tests: a) Infilled central bays; b) Bare central bays.

