Recent Greek Provisions For Rc Structures with Urm Infills

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Abstract: A new Greek Code is already approved and in force, covering structural assessment, interventions (repair or/and strengthening) and redesign of existing reinforced concrete (RC) structures, in line with the relevant provisions of Euro-Codes, and especially of EC 8-1 : 2004 and of EC 8-3 : 2005(for new and existing structures, respectively).

Among the various aspects covered by this extensive Code, admittedly far beyond and more detailed than EC 8, is that of masonry partitioning-infilling walls (made mainly of perforated clay bricks), already existing (plain/unreinforced-URM, with one or two leafs-wythes, previously damaged or not) or enhanced or arranged on purpose for seismic upgrading of old or/and inadequate RC buildings, consisting of engineered masonry panels, unreinforced or even reinforced.

According to this newGreek Code (nGCI), a lot of additional (to those of the EC 8) related problems and aspects are at least shortly covered (in a code-like format) and presented/discussed in this paper, such as :

Basic principles, i.e. reliability aspects, interaction of URM infills and RC elements or structures, quantitative global and local influence for frames or quasi-frames, possibly adverse local effects, assessment, repair or/and strengthening;

Technological and geometrical aspects, i.e. types of infills, existing (non-engineered) or new, geometrical data, presence of one or of two leafs (connected or not), panel's thickness and slenderness, influence of openings and of wedging;

Mechanical behavior, i.e. out-of-plane and in-plane response, macro-models based on shear panels or onequivalent compression diagonals (struts), mechanical characteristics and typical (default) mean values for design and redesign, influence of past damage and residual characteristics, as well as

Methods of analysis, assessment and redesign, i.e. linear and non-linear approaches, static or dynamic ones,verifications in terms of force (global or local behavior factors) or displacement, based on specific performance requirements and levels (no-collapse, significant damage, limited damage).

The rationalism, the methodology and the application rules of this new Greek Code on (Structural) Interventions (nGCI) are expected to influence EC 8 as well as the provisions for seismic design of even new framed or quasi-framed common RC structures of low to medium height (i.e. up to max. 10 storeys).

Keywords: Reinforced concrete (RC) structures, unreinforced masonry (URM) infills, shear panels, equivalent struts, behavior models, skeleton (back-bone) curves, assessment, redesign.

1. INTRODUCTION

It has long been recognized (see, for example, the pioneered work by S.V. Polyakov and others, [1-12]), that the influence of even unreinforced and non-engineered partitioning-infilling masonry walls in the response of framed (or quasi-framed) RC structures could be significant Fig. (**1**), covering almost all aspects of seismic behavior, including redundancy, possible period shift and gradual or abrupt resistance degradation under inelastic cycling (seismic) actions.

Thus, ignoring such an influence and interaction (related with global or local effects, main or side ones), as it is the case for most common and conventional structural designs and redesigns – even nowadays, may not always result in realistic and reliable predictions or even safe ones, not to mention the major problem regarding "open" ("soft" or "weak") ground storeys-pilotis [13-15].

In recognition of this fact, not to mention lessons learnt in past earthquakes (see, for example, a lecture by M.N. Fardis, [20]), and for several decades now, the interaction of frames and infills has been the subject of numerous theoretical and experimental investigations, in many countries, including large scale and shaking table tests [21, 22]. Of course, many of the earlier tests and studies (in the '50s up to the'70s) devoted to infilled frames for resisting blast loads or for stabilizing/restraining of tall buildings, or to steel frames with infills, made of rather strong concrete units-blocks (hollow or solid ones) or of micro-concrete.In addition, several attempts to model analytically and predict reliably the behavior of infilled frames have been reported in the rich technical literature, with models elaborated or even sophisticated. Nevertheless, a rather old statement (J.W. Axley and V.V. Bertero, [23]) still holds true – "infilled frame structural sys-

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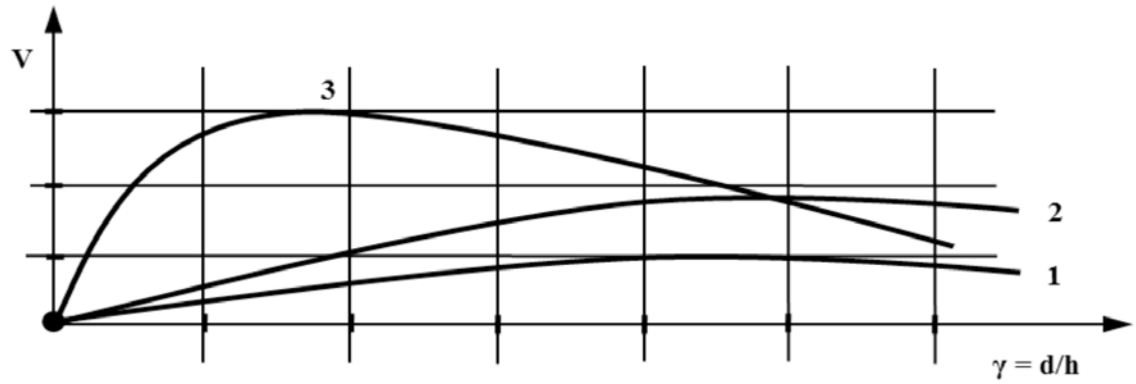


[1 : Bare frame, 2 : Frame and a set of multiple URM piers, 3 : Frame and an URM panel.
To this end, the larger the increase in stiffness and strength, the smaller the corresponding maximum displacement.]

Fig. (1). Qualitative (schematic) shear force-angular distortion (storey drift ratio) curves for infilled RC frames, [16-19].

tems have resisted analytical modelling", as a consequence of a lot of difficulties and uncertainties, interrelated or not ([24] and § 2e here below).

Certainly, most of the current (national or international) structural design codes and recommendations produced all over the world do contain a lot of principles, provisions or even application rules (quantitative and qualitative) regarding masonry infills in framed (or quasi-framed) RC structures (new or existing ones) of high or medium (or even low) overall ductility [25-29].To this end, the new Greek Code on assessment and upgrading of existing RC structures [30] contains a lot of provisions and application rules for masonry infilled frames or quasi-frames (in line with the general principles of EC 8 : 2004 and 2005), which are expected to influence even the seismic design of new concrete structures.

In this paper, the basic additional provisions and rules of this Code are presented and discussed, as well as calibrated by means of comparisons to other relative international approaches and design methodologies for URM infills. Nevertheless, infilled structures are still treated with scepticism in modern seismic codes, not to mention the relative reliability aspects regarding their behavior during the earthquake (EQ) itself [31-33].

2. GENERAL ASPECTS BASED ON EC 8

Euro-Code 8 [29] contains certain principles and provisions for masonry infills which contribute significantly to the resistance of the building (EC 8-1, § 4.3.1 (8)) and should be properly taken into account. These additional measures apply only to frame or frame equivalent dual concrete systems (and to steel or composite steel-concrete resisting frames) of high ductility class (DC H), with interacting non-engineered masonry infills that fulfill a set of conditions (EC 8-1, § 4.3.6.1), as follows :

a) Frame or frame equivalent dual concrete systems (or steel or composite resisting frames) are the structural systems in which both the gravitational and the seismic loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% or 50%, respectively, of the total shear resistance of the whole structural system.

For wall or wall equivalent dual concrete systems (or braced steel or composite systems), with similar percentages of base shear resistances, any interaction with the masonry infills may be neglected, in general.

In the above definitions, the fraction of shear resistances may be substituted by the fraction of acting shear forces in the design seismic situation.

b) Masonry infills, which are considered in principle as non-structural elements, are non-load bearing elements, constructed after the (assembly of the steel frame, in the case of steel or composite systems, and) hardening of the concrete frame, while they should be in contact with the surrounding frame elements (i.e. non-isolated, w/oany special gaps or separation joints) but w/o any structural connection to the frame (e.g. through posts, belts, ties or shear connectors).

On the other hand, if engineered masonry infills constitute part of the seismic resistant structural system (and the load bearing one), their design should be carried out in accordance with the principles, criteria and rules given for confined or quasi-confined masonry (see the relevant clause of EC 8).

c) It is assumed that no change in the structure and the masonry infills will take place during the construction phase or during the subsequent life and use of the building, unless proper justification and verification is provided.

Due to the specific nature of the seismic response, this applies even in the case of a change that leads to a favorable effect and an increase of resistance (EC 8-1, § 1.3 (2) P).

d) Although the scope of this and the subsequent clauses is limited to DC H, the provided criteria for good practice may be advantageous to be adopted for other ductility classes as well (medium – DC M and low – DC L). In particular, for masonry shear panels that might be vulnerable to out-of-plane damage or failure (especially at upper storeys of the building), the provision of ties can reduce the hazard of falling masonry (see § h).

e) Account shall be taken of the high uncertainties related to the characteristics and the behavior of masonry infills, namely (EC 8-1, § 4.3.6.2 (3) P):

— The variability of their mechanical characteristics and properties and of their contact with or attachment to the surrounding/bounding frame;

— Their possible modification (even unintentional) in-time or wear or degradation or damage during the life and use of the building, as well as

— Their non-uniform or "non-organized" degree of damage or failure suffered at various storeys of the building during the earthquake itself.

f) The consequences of additional (non-structural) irregularities due to masonry infills in plan as well as in elevation, even unintentional, shall be properly taken into account (EC 8-1, § 4.3.6.2, (1) P and (2) P), § 4.3.6.3), see APPENDIX **A**.

g) The possibly adverse local effects on the boundary RC members due to the frame-infill interaction (e.g. shear failure of columns or of beams under local shear forces induced by infills) shall be properly taken into account (EC 8-1, § 4.3.6.2 (4) P and § 5.9 for concrete buildings), see APPENDICES **B** to **D**.

h) For frame or frame equivalent dual structural systems, belonging to all ductility classes (DC H, M or L), except in the cases of low seismicity (EC 8-1, § 3.2.1 (4)), appropriate measures (damage limitation ones) should be taken to avoid brittle failure and premature disintegration of the infill walls, in particular of panels with large openings or of friable or of degraded materials, as well as to avoid partial or total out-of-plane collapse of rather slender panels, EC 8-1, § 4.3.6.4.Particular attention should be paid to masonry infills with a slenderness ratio (ratio of the smaller of clear length or height to effective thickness) of greater than 15.

Examples of such appropriate measures, to improve both in-plane and out-of-plane integrity and behavior, include (among others) concrete posts and belts across the panel and through the full thickness of the wall, wall ties cast into the bed joints of the masonry and fixed to the columns and light wire-meshes well anchored on the wall (at least on its one face) and the bounding frame.If there are large openings or perforations in any of the masonry infill panels, their edges should be trimmed with posts and belts.

In addition, the "damage limitation requirement" is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than that corresponding to the "no-collapse requirement" (i.e. under a more frequent and less severe earthquake), the interstorey drifts are limited to $d_r \cdot \nu \leq 0{,}005\ h$ (for non-structural elements of brittle materials "attached" to the structure, as URM), see EC 8-1, § 4.4.3, where $\mathbf{d_r}$ is the design interstorey drift ($d_r = q_d \cdot d_{r,e\ell}$, with the displacement behavior factor $q_d \geq q$ if $T \leq Tc$ – short period range, in the cases of linear-elastic analyses) and **h** is the storey height, with$\boldsymbol{\nu}$ an appropriate reduction factor.

The $\boldsymbol{\nu}$ factor takes into account the lower return period of the associated seismic action, the seismic hazard conditions and the degree of protection of property objective, with recommended values of 0,5 for lower and 0,4 for higher importance classes, respectively, see EC 8.

Additional damage limitation verifications might be required in the case of buildings important for civil protection or of monumental value or containing "sensitive" or "valuable" objects, equipment etc.

To this end, and before presenting the additional and more detailed relevant provisions of the new Greek Code, especially for existing RC structures, the following comments are made :

(i) How the "significant contribution" of masonry infills could be assessed, in a quantitative (and not only qualitative) and straight forward way ?

By means of response models, mechanical characteristics and default values, given in the next clauses of this paper (at least for common masonry infills in Greece), the in-plane shear strength of infills could be estimated for each storey and in each one of the two main orthogonal horizontal directions of the building. If this shear strength of infills, in any storey and in any direction, exceeds approx. 15% of the corresponding total shear resistance of the RC vertical elements, then the influence of infills could be considered as "significant" (see also APPENDIX **A**, (ii)), unless isolation (and additional) measures are taken (and maintained).

(j) Admittedly, the principle of non-engineered, non-structural and non-load bearing masonry infills, in a"simple" contact with the surrounding concrete frame elements (§ b), is in contradiction to the measures and rules associated with damage limitation of infills (§ h), especially those regarding the arrangement of posts and belts (usually made of concrete), or of ties or of shear connectors, of various types.

3. ADDITIONAL PROVISIONS OF THE NEW GREEK CODE

The main general additional principles of the new Greek Code on (Structural) Interventions [30] on existing RC structures (damaged or undamaged) are those related to a) inspection, investigation and documentation (leading to certain data reliability levels – DRLs, with an impact on almost all phases of redesign), b) performance levels and requirements (associated with the target behavior and degree of acceptable damage), c) elastic analyses based on global behavior or local ductility factors (**q** or **m**, respectively), and d) additional particularities related to URM infills in RC frames or quasi-frames.

a) Before any structural assessment, redesign or intervention is carried out, it is needed to investigate and document the existing structure to a sufficient extent and depth so as to obtain maximum data reliability on which to base any relevant action, taking into account that any alteration of or intervention on the URM infills also constitutes a relevant action on the existing structure itself (see also §2c). This involves inspection and surveying of the building, its structure and its condition, gathering of reliable information, compilation of the structure's "history" and maintenance, recording of any wear, deterioration or damage as well as conducting on-site and in-lab investigation works, tests and measurements, in a detailed and well specified manner (based on a plan prepared by the Structural Engineer), for both the foundation (and basement (s), if any) and the superstructure, separately for RC slabs, beams, columns and walls, as well as URM (or other) infills. To this end, and besides minimum requirements for investigation of and data on materials characteristics and strengths, there are minimum requirements regarding a set of "geometrical" data as well, including the following :

- Type and geometry of the foundation, the basement(s), if any, and the superstructure, general dimensions, lengths, heights, cross-sections etc., with a set of detailed drawings;
- Type and geometry, arrangement, thicknesses, degree of wedging, connections (if any), construction details etc. of the URM infills, shown on the same structural drawings;
- Thickness and weights of cladding, finishes, coverings, coatings, architectural or functional elements, other dead weights etc., and
- Reinforcement details, including reinforcement layout, number and diameter of bars, anchorage lengths, lap and starter bar lengths, detailing and closing of stirrups etc.

The desired reliability level of the above mechanical and geometrical data depends on several factors and affects all phases of assessment and redesign, including the determination of actions, action-effects and resistances, while uncertainties are covered by introducing the concept of "Data Reliability Level – DRL", far beyond the relevant provisions of EC 8-3 regarding "Knowledge Levels and Factors" (or of FEMA [27]).

Three DRLs are distinguished : High (H), Sufficient or Satisfactory (S) and Tolerable (T), corresponding roughly to "Knowledge Levels" – KLs 3 to 1 of EC 8-3 (Full, Normal, Limited), as far as "primary" seismic elements are concerned. For "secondary" seismic elements, a DRL less than Tolerable (T) could by permitted, while for URM infills a DRL H or S is imposed.

In addition, DRL is not necessarily the same for the entire building or even the same group of elements or of data; different DRLs for the various sub-categories of elements and of information could be determined. It is only for the selection of the proper method of analysis that the most unfavorable among the individual DRLs shall be used.

Depending on DRL (i) an appropriate method of analysis in chosen (since there is no point in the desired precision of any advanced method being heigher than the expected inaccuracy of the data which will be used), (ii) the appropriate safety factors $\boldsymbol{\gamma_f}$are selected for certain actions of higher uncertainty, combined with relevant $\boldsymbol{\gamma_{Sd}}$ factors (i.e. uncertainties of the models through which the effects of actions are assessed), and (iii) the appropriate safety factors γ_m for material properties are selected, combined with relevant γ_{Rd} factors (i.e. uncertainties of the models for resistances of all types and kinds).Generally, for DRL S the γ-factors are selected according to the provisions of the Codes for the design of new structures, with no modifications.

a) Three Performance Levels – PLs (target structural behaviors) are foreseen : Collapse Prevention(C) or no-collapse or near collapse (associated with extensive and severe/heavy structural damage, but w/o collapse), Life (and Property) Protection (B) or significant/substantial and extensive structural damage (a repairable one), and Immediate Use and Function (A) or limited structural damage (associated with no or minor damage and immediate occupancy and use w/o any restriction). In fact, these three PLs correspond (in general) to the three Limit States of EC 8-3, namely Near Collapse (NC), Significant Damage (SD) and Limited Damage (LD).

These PLs (strictly for the load bearing structure alone) are combined with the foreseen seismic action to give a "target" for the assessment or the redesign of the structure, not necessarily the same. To this end, and for a conventional life-time of 50 yrs (the same for new and existing buildings), the seismic action could be assessed on a probability of exceedance equal to (1) 10% (mean return period of approx. 475 yrs) - in general, or (2) 50% (mean return period of approx. 75 yrs) - after the approval of a Public Authority, leading to an overall seismic action of 100% or 60% compared to that of EC 8-1, respectively. The importance factor γ_I of EC 8-1 should be properly taken into account, allowing for the expansion of life-time beyond 50 yrs, or (equivalently) taking into consideration the generalized consequences of a potential failure.

The "targets" could be two, namely B1 and A2 or C1 and B2 or A2, depending on the use and importance of the building, while for new buildings the "target" according to EC 8-1 is in principle B1 (life and property protection, p_e = 10% in L_t = 50 yrs).

This foreseen "target" (a combination of PL and of the seismic action, in terms of p_e–if and when this is permitted) influences all phases of assessment and redesign, including methods of analyses (linear for PL A or B, non-linear for PL B or C), **q** and **m** factors, actions and action-effects, resistances, detailed provisions, verifications etc.

b) When linear (or pseudo-linear) analyses are to be used for existing structures, two methodologies are foreseen according to the new Greek Code, namely :

- The use of an overall (global) ductility factor **q**, for the entire structure, being in fact a product of the overstrength ($\mathbf{q_o}$) and the ductility ($\mathbf{q_d}$) factors of the building as a whole, i.e. $\mathbf{q = q_o \cdot q_d}$, or
- The use of local "displacement" ductility factors $\mathbf{m_i}$ (directly interrelated to $\mathbf{q_d}$, i.e. $\mathbf{m_i \leftrightarrow q_d}$), for individual structural elements (primary or secondary) or URM infills, based on their available ductility (their skeleton or back-bone curves).

The Code contains detailed criteria and application rules for estimating $\mathbf{q_o}$, $\mathbf{q_d}$ and $\mathbf{m_i}$ values, for existing elements (damaged or not) or for elements after repair/strengthening or for new (added) elements, as well as for the interrelation $\mathbf{m_i \leftrightarrow q_d}$, for assessment or redesign purposes, depending of course on PLs and DRLs.

To this end, two comments are made:

- The values of $\mathbf{m_i}$factors (**m** for member) are chosen and calibrated so that the value of the corresponding overall **q** factor of the structure as a whole does not deviate by more than 15% than the foreseen conservative default value according to the Code, and
- The value of $\mathbf{m_i}$ factor for an individual element is a good and reliable estimator of its seismic behavior; by convention, if $\mathbf{m_i \geq 2}$, i.e. if the behavior is quasi-ductile, verification is made in terms of "deformation" (based, in principle, in materials' properties represented by just their mean values, properly calibrated), while if $\mathbf{m_i < 2}$, i.e. if the behavior is quasi-brittle, verification is made in terms of "force" (based, in general, on materials' properties represented by their mean values minus one standard deviation, taking into account proper $\boldsymbol{\gamma_m}$ factors, depending on DRLs).

In general, the verification and the check of safety inequality, i.e. $E_d = \gamma_{Sd}.\ E\ (E_k\ .\ \gamma_f) < (1/\gamma_{Rd})\ .R(R_k/\gamma_m) = R_d$, is performed in terms of "force" for linear analysis or non-linear analysis and brittle members, or in terms of "displacement" for non-linear analylis and ductile members. In addition, linear modelling is meant to be used mainly for new buildings and non-linear is meant to be used primarily for the purposes of assessment and redesign of existing buildings, while for infilled structures dynamic analyses (of any type) are not recommended.

d) For URM infills, the following specific criteria and rules are foreseen according to the new Greek Code :

- Survey and documentation include exposing masonry walls at (at least) 2 locations on each floor, with an exposed area of approx. 0,7x0,7 m. When inspecting and surveying, reliable information is collected regarding:
  - The system and the quality of construction, the wedging between infills and bounding elements;
  - The type and the quality of materials (bricks and mortar);
  - Possible wear or deterioration, damage etc.;
  - The thickness of leafs-wythes, their possible connection;
  - The thickness of joints (volume of mortar) and the degree of filling with mortar, for both bed and head joints, and
  - The presence and the details of any posts, belts, connectors etc.

To this end, if differences and deviations are high, additional investigation is needed, e.g. at 4 locations on each floor.

- In order to determine the behavior of infills, compressive and shear strengths, as well as the corresponding moduli, are of interest.
  - When more precise data are not available, the above properties could be determined indirectly by semi-empirical relations or taken as equal to their foreseen default values; in this case, the DRL for the mechanical characteristics is considered Sufficient or Satisfactory (DRL S);
  - When the mechanical characteristics are calibrated by means of tests and measurements on-site or/and in-lab of a certain number of representative samples/specimens (according to the Structural Engineer's judgment), the DRL can be considered High (DRL H);
  - A Tolerable DRL (DRL T) is not allowed for URM infills to be taken into account in assessment or in redesign;
  - For DRL S or H, $\boldsymbol{\gamma_m}$ values for the strength of URM infills may be taken equal to 2,5 or 2,0 , respectively.
- Similar provisions are foreseen, regarding DRLs (S or H) of geometrical characteristics, i.e. mainly the number of leafs-wythes and the thicknesses.
- For URM infills (existing or built on purpose) only PLs A and B are allowed, while all PLs (including C, collapse prevention) are allowed only in the case of engineered and reinforced RM infills.

  In addition, and based on specific skeleton curves, URM infills could be checked in terms of "force" (**q** or **m** values) or of "displacement" (non-linear analysis), considering them as quasi-ductile thanks to the "confining" action of the surrounding framing RC elements.
- The **q** values (default ones) for RC frames (or quasi-frames) with URM infills, for assessment or redesign, depend on three main and decisive factors, namely (i) the standards applied for their design (and construction), (ii) their favorable presence or absence, or their generally (not locally) unfavorable presence, and (iii) the degree of damage (if any) in primary structural elements, not to mention PLs.

  As an example, for Greece, and for PL B (life and property protection), a building constructed in the '70s, with a substantial structural damage and unfavorable presence of URM infills on alarge scale (i.e. presence of many "short" columns), may be assessed for $q \approx 1{,}1$, but redesigned for $q \approx 1{,}3$ or even 1,7 , simply if damage is fully repaired or if a favorable presence of full height URM infills on a large scale is ensured as well, respectively. Also, a building constructed in the '90s, with aconsiderable structural damage and unfavorable presence of URM infills as in the previous case, may be assessed for $q \approx 1{,}3$, but redesigned for $q \approx 1{,}7$ or even 2,3 , simply if damage is fully repaired or if the unfavorable effects of infill walls are eliminated (e.g. by removal of infills or by lessening of their effects or by converting partial to full infilling) as well, respectively.
- Correspondingly, rather low **m** values of URM infills could be estimated, based on their skeleton curves (see §6 of this paper) and their deterioration or damage, if any and if not fully repaired (see § 7 of this paper).
- Finally, additional criteria and rules are provided, as in the following clauses and paragraphs of this paper, while, as a general principle, URM infills could be taken into account only if (i) they are in a "simple" contact with RC

framing elements at (at least) 3 out of their 4 sides, (ii) they do not present large or multiple openings or perforations, and (iii) they are not prone to premature out-of-plane damage (depending on their slenderness).

In addition, infill walls shall not be taken into account selectively, e.g. from storey to storey or from planar frame to planar frame or from place to place etc., not to mention that only the wythes in full contact with the boundary frame elements shall be considered when computing in-plane response unless proper measures are provided (e.g. by anchoring all sides of the walls).

4. PROVISIONS REGARDING THE INFLUENCE OF OPENINGS

The influence of openings on the behavior of URM infills depends on certain geometrical and mechanical characteristics, most of which have been investigated for several decades now, analytically or/and experimentally (see, for example, [24, 27, 34-42]).

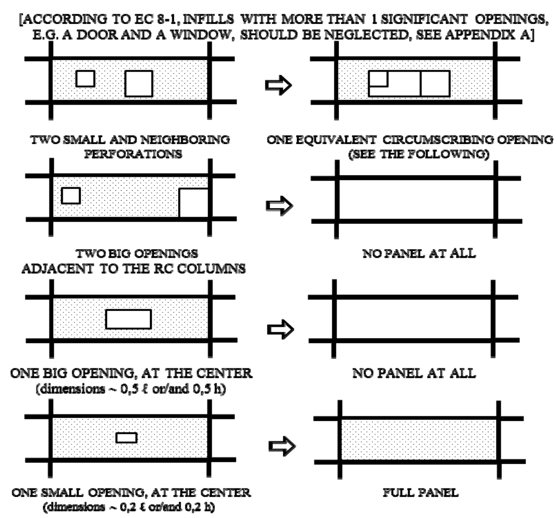


Fig. (2.1). "Black" or "White" decisions regarding openings of URM infills.

The ultimate influencing but qualitative factor is the potentiality for a reacting intact shear panel or a set of struts (in both directions), which in turn depends on (i) the boundary conditions of the infill panel, (ii) the size and location of openings or perforations, and (iii) the existence and function of any trimming or boundary elements along the edges of the openings (e.g. posts, belts etc.).

Admittedly, analytical modelling of infills with openings is cumbersome and laborious, especially as far as common and conventional structural design and redesign is concerned, not to mention increased and disproportionate uncertainties, which could invalidate all relevant efforts. Experimental data and theoretical work (based even on photo-elasticity), however, are not sufficient to establish reliable guidelines, while the use of various models requires increased engineering judgment on a case-by-case basis.

To this end, various approaches could be used, probably different for the global or the local effects, based on micro-models (finite element methods, FEMs) or macro-models (sets of struts, or of struts-and-ties, if reinforcement and connectors are provided), including models based on "semi-rigid end-segments or offsets" or "equivalent framing elements" for the bounding RC elements or on "rigid arms" or "equivalent strut width" for the perforated panels themselves, by modifying the relevant properties. For single and central openings, a practical approach is that of a reduced strut width [40, 41], based on the (perimeter or) the area ratio, with the reduction coefficient equal to 1,25 $(1 - A_o/A_p) \leq 1$, with $\mathbf{A_o}$ the area of the opening and $\mathbf{A_p}$ the area of the panel.

In recognition of these facts, the new Greek Code contains certain quantitative criteria (in line with all the above), completed with a set of only 5 practical rules, for a "Black" or a "White" decision, i.e. a decision of "no panel at all" or "full panel" (neglecting openings), respectively; to this end, the following Fig. (**2**) contains an attempt (by the authors) to present these rules in a practical and "visual" way (see also [43]).

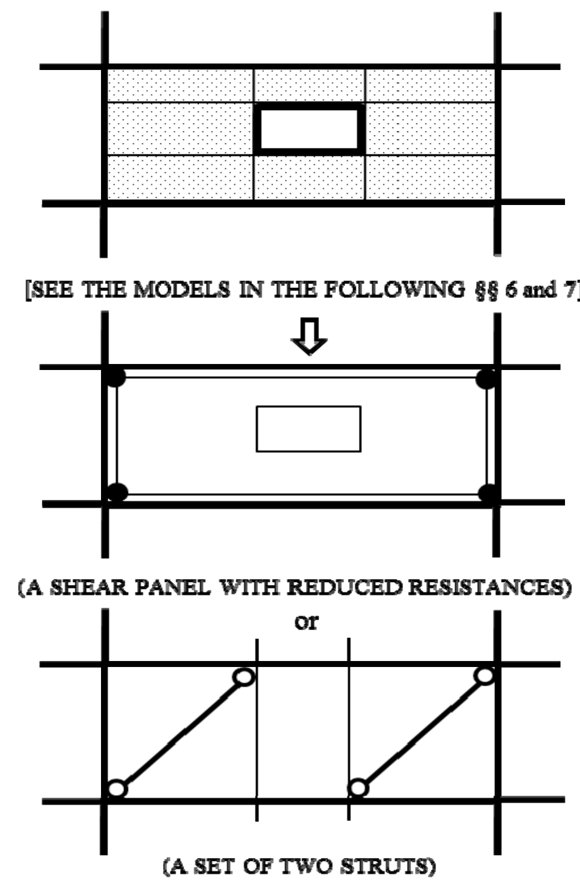


Fig. (2.2). One approx. central opening with dimensions between 0,2 and 0,5 of those of the panel (especially in the case of any trimming elements, posts, belts etc.).

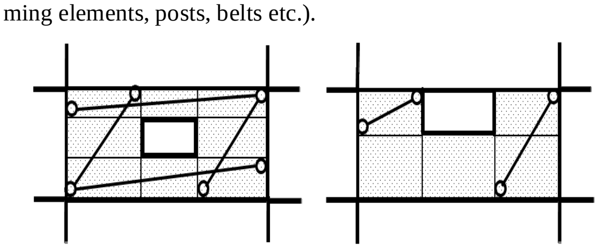


Fig. (2.3). Other relevant proposals, suitable for local analyses ([27, 38]).

5. PROVISIONS REGARDING THE SLENDERNESS OF INFILLS

In general, infill walls suffer, during the earthquake itself, from out-of-plane damage at upper storeys (depending mainly on their slenderness) and in-plane damage at lower storeys (depending mainly on interstorey drifts).

Therefore, premature out-of-plane damage, leading to a drastic reduction of in-plane resistance (as well as to instability situations), should be minimized, based on panel slenderness ratioλ = L/t, where **L** is the "clear" length of the diagonal strut, $L = \sqrt{\ell^2 + h^2}$, and **t** is the "equivalent" effective thickness of the panel, t = t_{eff}, depending on construction details as follows (see Figs. **3** and **4**):

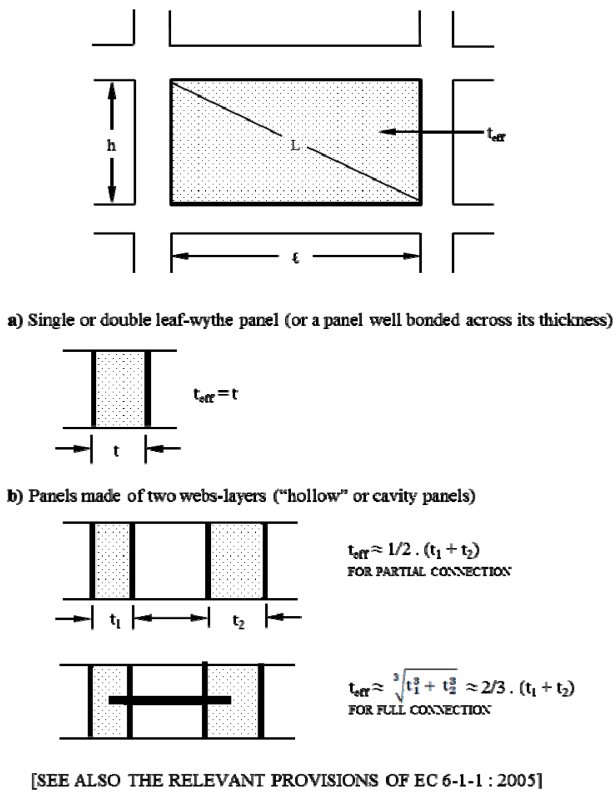


Fig. (3). Geometry of panel(s).

In the case of a "simple" contact (w/o any connectors) along the perimeter of the panel, i.e. along all its 4 sides, the following simplified approach is used :

- For $\lambda \le 15$ (or $\ell/t \le 15$ and $h/t \le 15$), the expected reduction of resistance is practically zero and the panel is fully taken into account in the design.
- For $\lambda \ge 30$ (or $\ell/t \ge 30$ and $h/t \ge 30$), the resistance is almost zero (i.e. the reduction is almost 100%) and the panel in not taken into account at all.
- For intermediate values of**λ**, the reduction could be assessed based on **Φ**factor ($\le$1), according to EC 6.

The new Greek Code allows a simpler approach as well, based on semi-empirical data [43-45], according to the diagram here below. To this end, it is pointed out that the definition of **λ**is somehow different according to EC 8-1, that of λ = min. (h;ℓ)/t, while for infill panels with λ> 15 particular attention should be paid (see § 2h of this paper).

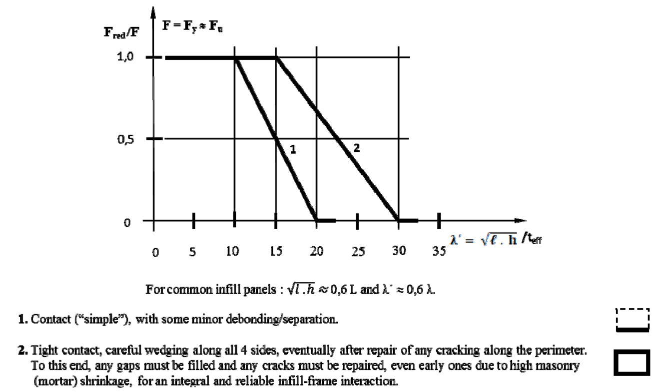


Fig. (4). Reduction of out-of-plane resistance(s).

NOTE

Many tests and studies have been devoted on face-loading or out-of-plane loading of URM infills and their compression membrane or arching action in resisting such an EQ loading, see, for example, [27, 46], or the extensive work of R. Angel *et al.* [47].

According to FEMA [27], URM infills need not to by analyzed for face-loading (during an EQ) meeting certain requierements for membrane or arching actions, i.e. if (i) the RC frame components have sufficient stiffness and strength to resist thrusts from such an action, (ii) the infills are in full contact with the bounding elements, and (iii) their slenderness ratio h/t is lower than 8 for high seismic zones and PL A up to 15 for low seismic zones and PL B.

6. MODELS AND RESISTANCES OF URM INFILLS

6.1. General Aspects

The models adopted by the nGCI are those of shear panel(s) (§ 6.2) and ofequivalent strut(s) (§ 6.3), together with the related resistance characteristies, which in turn depend on :

- Both the constituent materials (perforated clay bricks and low strength mortars), the bonding and the construction itself or any damage (§ 7), as well as
- The "contact" lengths between the RC framing elements and the infill panels, which in turn depend on interstorey drift and possible damage.

Therefore, geometrical data entering and formulating resistances, are, in fact, related to the foreseen degree of damage, i.e. the Performance Level (only PL A or B for URM infill panels, see §§ 3b and 3d). The simplified models account for post cracking and cyclic seismic actions and hysteretic behavior, i.e. for 3 full load reversals (3 full cycles) for any imposed deformation, and they are meant for linear and push-over analyses. Nevertheless, it should be kept in mind that URM infills is a "material" with widely ranging properties and characteristics.

In the following paragraphs the relevant models are shortly presented and discussed, based mainly on greek data, while in APPENDIX **E** additional and more detailed data are given for greek URM infills. To this end, emphasis is given on the fact that the foreseen "deformation" of masonry (for both models) is generally higher than that anticipated for plain URM, thanks to the "confinement" offered by the RC elements which in turn is higher the stronger the frame.

In principle, mean resistances given for the 2 models are meant for PL B, i.e. life and property protection, while for PL A, i.e. immediate use and function (of the building), resistances could be increased by 50% (if more precise and reliable data are missing).

NOTE

A state-of-the-art on models for URM infills could be found in [24, 48], as well as in [49]; almost all of them belong in 2 main categories, that of local-or micro-models and that of simplified- or macro-models.

Micro-models (even complex non-linear ones) are based on finite element methods (FEMs) for the infill panel itself as well as for the interface (mortar joint) between the panel and the bounding RC frame, see, e.g., [23, 50-56].

Macro-models, generally simplified and suitable for global analyses, are based on shear panels (with simple or complex nodes, isotropic or orthotropic, with infills as homogeneous materials or under smeared cracking), on shear beams or shear springs, or on trusses or struts (single or double or triple, or in sets), see, e.g. [10, 11, 25, 27, 38, 57-60].

6.2. Model Based on Shear Panel(s)

The relevant general aspects have been presented in § 6.1, as well as in §§ 4 and 5, while remarks about the compatibility of this model with that of equivalent strut(s) are discussed in §§ 6.3 and 6.4. The effect of any damage is presented in § 7. The model is presented in the following Fig. **5**, with **ℓ** and **h** theclear panel dimensions and $\alpha = h/\ell$ the aspect ratio of the infill panel ($\alpha < 1$).

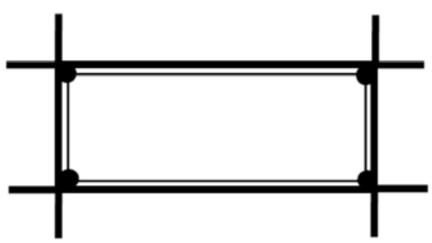


Fig. (5.1). Orthotropic (or "equivalent" isotropic shear panel.

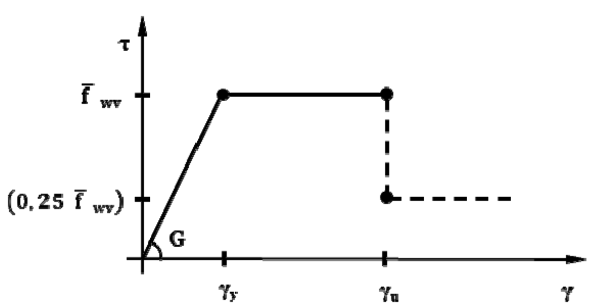


Fig. (5.2). The corresponding bilinear skeleton curve (PL B).

Mean shear strength $\bar{f}_{wv}$ (along bed joints) could be assessed according to the provisions of EC 6-1-1: 2005 (for a practically zero normal stress, around the center of the panel, due to its self weight only) and certain additional rules of the nGCI; alternatively, use could be made of practical recommendations or default values (see §6.4 and APPENDIX **E**).

Angular distortion or storey drift ratio**(γ)** values are taken equal to :

$$\gamma_y = (1,0 \text{ to } 1,5) \,.\, 10^{-3} \,.\, (\ell/h + h/\ell), \text{ and} \quad (1)$$

$$\gamma_u = (2,0 \text{ to } 3,5) \,.\, 10^{-3} \,.\, (\ell/h + h/\ell), \quad (2)$$

where $(\ell/h + h/\ell) = L^2/h \,.\, \ell = (1 + \alpha^2)/\alpha$.

To this end, the **γ** values shall be taken into account in full correspondence, i.e. lower (or higher) $\boldsymbol{\gamma_y}$ values and lower (or higher) $\boldsymbol{\gamma_u}$values, respectively, with $m \approx 2,0$ to 2,5.

For PL A the resistances are 50% higher, i.e. $1,5\bar{f}_{wv}$ and $1,5\ \gamma_y$.

NOTES

a) Among others, a relevant model, based on a4-node isoparametric plane stress element, proposed by A.J. Kappos [61], seems promising.

b) According to FEMA ([27]), the diagram of Fig. 5.3 could be used for URM infills, with **γ** values (storey drift ratios) finally multiplied by **κ** – the knowledge factor (κ = 0,75 to 1,00).

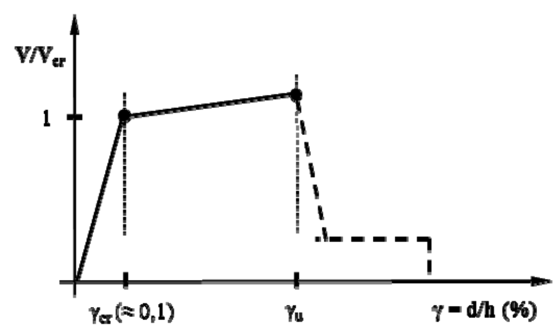


Fig. (5.3). V-γ diagram for shear panel(s), FEMA [27].

Values of $\boldsymbol{\gamma_u}$ could be assessed based on the panel aspect ratio ($\alpha = h/\ell$) and on the relative strength between the RC frame (V_{RC}) and the URM infill (V_{URMI}), as follows :

- $\gamma_u \approx 0,2$ to 0,3 %, for $\alpha = 0,5$ and $V_{RC}/V_{URMI} < 0,7$, up to
- $\gamma_u \approx 1,0$ to 1,5 %, for $\alpha = 2,0$ and $V_{RC}/V_{URMI} \geq 1,3$.

As it is obvious, the γ_{cr} ($\approx \gamma_y$) values according to FEMA are lower than those given by the nGCI, leading to a post-cracking (or post-yielding) plateau much longer than that foreseen by the nGCI; therefore, much higher **m** values are expected according to FEMA, see§ 6.4 here below. In addition, "hardening" is not taken into account by the nGCI.

6.3. Model Based on Equivalent Strut(s)

The relevant general aspects have been presented in § 6,1, as well as in §§ 4 and 5, while remarks about the compatibility of this model with that of shear panel(s) are discussed in a Note here below as well as in § 6.4. The effect of any damage is presented in § 7. The model is presented in the following Fig. (**6**), with **ℓ** and **h** the clear panel dimensions and $\alpha = h/\ell$ the aspect ratio of the infill panel ($\alpha < 1$).

Mean diagonal compression strength$\bar{f}_{wc,s}$ (along the strut) could be assessed according to the following formula, or, alternatively, use could be made of practical recommendations or default values (see § 6.4 and APPENDIX **E**):

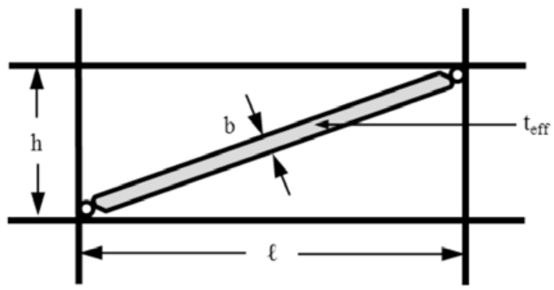


Fig. (6.1). Equivalent (compression) strut(s).

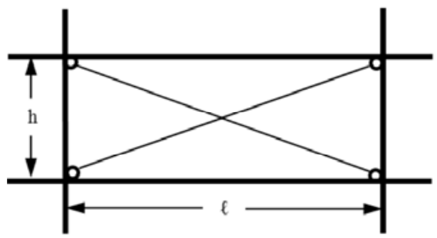


Fig. (6.2). A set of strut(s)-and-tie(s), with bars of half or full stiffness, for linear (compression and tension bars) or non-linear (compression bars only) analyses, respectively.

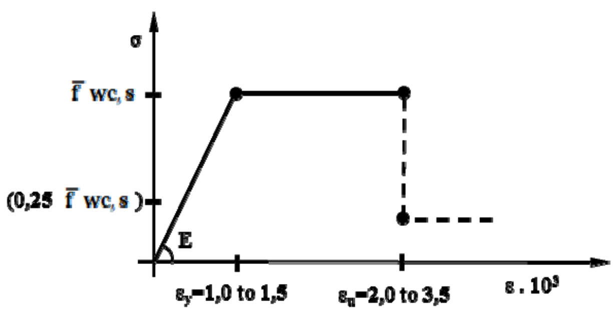


Fig. (6.3). The corresponding bilinear skeleton curve (PL B).

$$\bar{f}_{wc,s} = \lambda_m \cdot \lambda_c \cdot \lambda_s \cdot \kappa \cdot f_{bc}^{0,7} \cdot f_{mc}^{0,3} \approx 1,25\kappa \cdot f_{bc}^{0,7} \cdot f_{mc}^{0,3}\,, \qquad (3)$$

where

λ_m : conversion factor relating mean to characteristic (acc. to the Code) strength, $\lambda_m \approx 1,5$.

λ_c : factor accounting for the favorable "confining" effect, $\lambda_c \approx 1,2$.

λ_s : factor accounting for the adverse effect of transverse tension, $\lambda_s \approx 0,7$.

κ : coefficient depending on the types of bricks and mortars, according to EC 6-1-1 : 2005 , with $\kappa \approx 0,35$ to $0,55$.

In addition, reduction coefficients should be considered, accounting:

- For bed joints thicker than 15 mm, with $\kappa_1 \approx 0,85$, and
- For head joints not fully filled with mortar, with $\kappa_2 \approx 0,6$ to $0,9$, depending on the findings of investigation/documentation (§§ 3a and 3d).

To this end, the values of the normalized axial deformation **ε** (=ΔL/L) shall be taken into account in full correspondence, i.e. lower (or higher) $\boldsymbol{\varepsilon_y}$ values and lower (or higher) $\boldsymbol{\varepsilon_u}$ values, respectively, with $m \approx 2,0$ to $2,5$.

For PL A the resistances are 50% higher, i.e. $1,5\,\bar{f}_{wc,s}$ and $1,5\,\boldsymbol{\varepsilon_y}$.

NOTES

a) According to the nGCI, the estimation of the width **b** of the "equivalent" strut as well as the interrelations between the 2 models should be based on structural rather than on "elastic" approaches, as follows:

- **<u>Force Analysis</u>**

$N = V : \cos a$ and $L = \ell : \cos a$

$N = (t \cdot b) \cdot \bar{f}_{wc,s}$ and $V = (t \cdot \ell) \cdot \bar{f}_{wv}$

Therefore : $b \approx L \cdot (\bar{f}_{wv} : \bar{f}_{wc,s})$,

while for mean strengths the width is :

$$\Rightarrow \mathbf{b \approx 0,15\,L} \text{ for } \bar{f}_{wv} : \bar{f}_{wc,s} \approx \qquad (4)$$

- **<u>Displacement Analysis</u>**

$\sigma = \varepsilon \cdot E \Rightarrow N : (t \cdot b) = (\Delta L : L) \cdot E,\ \ \Delta L = d \cdot \cos a,\ L = h : \sin a$

$\tau = \gamma \cdot G \Rightarrow V : (t \cdot \ell) = (d : h) \cdot G,\ \ V = N \cdot \cos a$

Therefore : $E \cdot b \approx G \cdot \ell : \cos^2 a \cdot \sin a$,

and the compatibily interrelation between the models is :

$$\Rightarrow (\mathbf{E \cdot A_s}) \approx (\mathbf{G \cdot A_p}) : \mathbf{\cos^2 a \cdot \sin a}, \qquad (5)$$

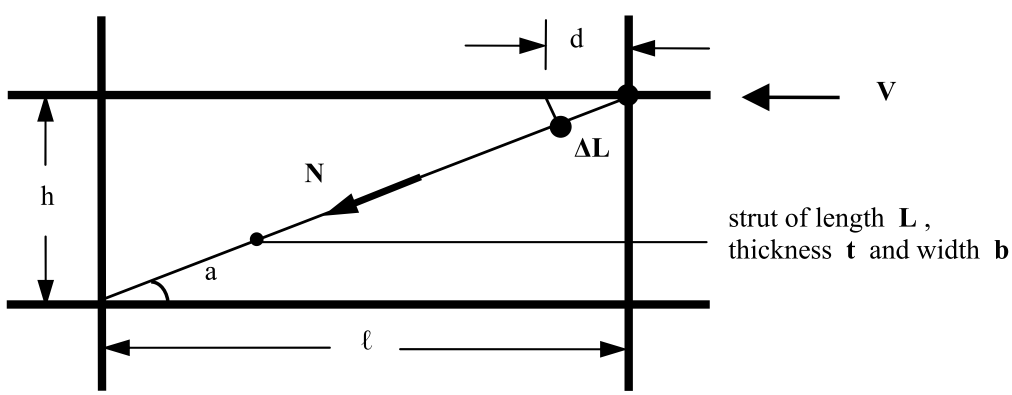


Fig. (6.4). Forces and displacements.

where $E.A_s$:strut's stiffness (with $A_s = t \cdot b$) and

$G.A_p$:panel's stiffness (with $A_p = t \cdot \ell$).

Nevertheless, certain aspects are ignored according to these analyses, i.e. that of vertical deformation or of the interrelation of strengths between the infill and the frame.

b) In the revelant rich technical literature (see, in addition, [62-66], a variety of similar expressions for the equivalent strut width **b** (in the case of full infilling) could be found, ranging from 0,10 **L** up to 0,35 **L**, based mainly on elastic approaches (i.e. on a "beam on elastic foundation" approach, see M. Hetenyi / 1946). Among them, the following are mentioned here below (for brickwork infills) :

– M. Holmes [5, 6]: $b \approx (1/4 \text{ to}) \ 1/3 \ . \ L$

– B.S. Smith [7, 62, 63]: $b \approx 0{,}10$ to $0{,}25$ L

– R.J. Mainstone [8, 9]: $b \approx 0{,}10$ L

– T.P. Tassios ([17], [18], [25]) : $b \approx 0{,}25$ L (± 50%)

– T. Paulay and M.J.N. Priestley [46]: $b \approx 0{,}25$ L

– M.N. Fardis [48] : $b \approx 0{,}10$ to 0,15L for PLA up to 0,20 L for PL B.

To this end, FEMA [27] proposals are based on the work of R.J. Mainstone [8, 9], which in fact does not take into account the effect of the panel aspect ratio.

c) In the case of partial infilling or of infills with openings or perforations (see § 4 of this paper), each sub-panel or pier between adjacent openings or an opening and a column (or a beam), could be substituted by an "equivalent" (or effective) strut with "equivalent" dimensions (height and length).

6.4. Compatibility of the Models and Additional Aspects

- It is known that the geometry and the properties of the infill panel and of the RC elements influence the response of the total; therefore, differences are expected between the proposed models and those based on other analytical studies, not to mention the increased uncertainties of URM infills themselves.

  Nevertheless, the 2 models should be considered as simplified but rational and practical ones, fully compatible and certainly conservative.

- Based on NOTE**a** of § 6.3 (see also APPENDIX **E**), the following are valid :

  $\cos a = \ell/L = \Delta L/d \ (=V/N)$

  $\sin a = h/L$

  $1/\cos a \ . \ \sin a = L^2/\ell \ . \ h = \ell/h + h/\ell$

  $\gamma = d/h$ and $\varepsilon = \Delta L/L$

  $\gamma/\varepsilon = 1/\cos a \ . \ \sin a = \ell/h + h/\ell$

  (see **γ** and **ε** values of the 2 models)

  $E \ . \ (tb) / G \ . \ (t\ell) \approx 1/\cos^2 a \ . \ \sin a$, see Equ. (5)

  $E/G = (\ell/0{,}15L) / \cos^2 a \ . \ \sin a = (1/0{,}15) / \cos a \ . \ \sin a = (1/0{,}15).(\gamma/\varepsilon)$

  $E/G = (1/0{,}15) \ . \ (\ell/h + h/\ell)$, for $b \approx 0{,}15$ L, see Equ. (4)

  For common infill panels, with $h \approx 2{,}5$ to 3,0(or even 3,5) m, a relation could be found as follows :

  $\gamma/\varepsilon = 1/\cos a \ . \ \sin a = \ell/h + h/\ell \approx 2{,}5 \ (2{,}0^+ \text{to } 3{,}5^+)$

  $E/G = (1/0{,}15) / \cos a \ . \ \sin a \approx (1/0{,}15) \ . \ 2{,}5 \approx 16{,}5 \ (\pm)$.

  Therefore, due to compatibility needs, the relation of moduli E and G is totally different than that based on elastic approaches (i.e. $E = 2(1+\nu) \ . \ G \rightarrow E \approx 3G$, even for $\nu \approx 0{,}5$, or $E \approx 2{,}5$ G, as it is widely accepted)..

  Both models lead to certain member ductility factors m ($= \gamma_u/\gamma_y = \varepsilon_u/\varepsilon_y$), see also §§ 3c and 3d.

  For PL A : $m_A = 1{,}0$ (to 1,1), combined with increased resistances, while for PL B : $m_B = m_{model}/\gamma_{Rd} \approx (2{,}0$ to $2{,}5)/1{,}2 \approx 1{,}5$ to 2,0.

  According to FEMA [27], see also NOTE**b** of § 6.2, the relevant **m** values are rather high, i.e. $m_A = 1{,}0$ to 1,5 and $m_B = 3$ to 8 (!), before any modification by the knowledge factor.

- Finally, and based on the above, the 2 proposed models are those of Fig. (**7**) (for common low aspect ratios of infills).

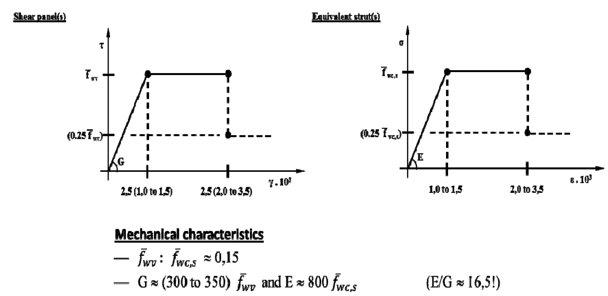


Fig. (7). The 2 equivalent models according to the nGCI (PL B).

  For PL A resistances should be increased by 50%.

- Residual response characteristics are not given, since URM infills are taken into account only for PL A or B, while for PL C they are considered fully damaged (with zero resistances).

  Only engineed masonry infill panels, generally reinforced ones (with diffused vertical and horizontal reinforcement), could be taken into account for PL C, generally by means of non-linear analyses. For such infills, their residual horizontal branch could be represented by F_{res}/F ($F = F_y \approx F_u$) $\approx 0{,}25$ and $d_{max}/d_u \approx 1{,}5$, as it is the case of RC elements (see the nGGI). To this end, similar provisions are foreseen by FEMA [27].

7. DAMAGED URM INFILLS

Wear, deterioration or pre-existing damage of infill panels should be taken into account, if not fully repaired, based on their model characteristics (§ 6) and on appropriate "resistance" reduction coefficients **r**, according to their "Damage Level – DL" (as it is the case of RC elements), Fig. (**8**).

Of course, $r \rightarrow 1$ for undamaged elements (or for minor damage with practically zero consequences) and $r \rightarrow 0$ for fully damaged elements (with practically zero response and ductility).

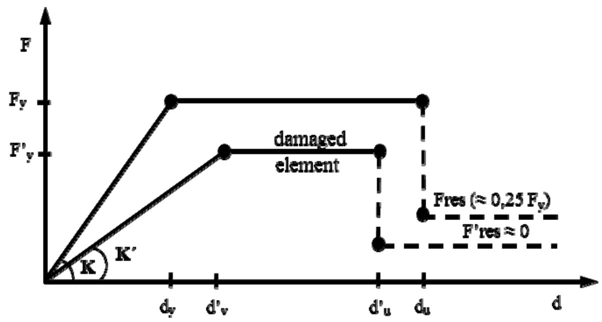


Fig. (8). Degraded skeleton curves of damaged URM infills.

In general, the **r** factors (**r** for residual) follow the trend:

$K'/K = r_K \leq F'_y / F_y = r_R \leq d'_u/d_u = r_{du}$. (6)

To this end, the nGCI contains (in an informative rather than a normative appendix) default values of **r** factors, as in the following Table **1**. If wear or deterioration is present simultaneously, i.e. combined with the "mechanical" damage (according to the Table **1**), a Δr value sould be subtracted, with $\mathbf{\Delta r} \approx 0{,}05 \div 0{,}15$ (depending on the deterioration), leading to $r_f = r - \Delta r$ (subscript **f** for final).

Similar approaches (and **r** values) could be found elsewhere as well, see, e.g., [40, 41], or even in FEMA [27], where reduced default values are given for the mechanical characteristics of URM infills depending on their "condition" (good, fair, poor), see APPENDIX **E**.

Finally, existing URM infills could (or should) be repaired or even enhanced for seismic rehabilitation; common or "conventional" methods could be applied, such as infilling of openings, filling of gaps (between the frame and the panel), deep repointing/rejointing, application of coatings or of shotcrete layers (with light wire-meshes), not to mention strengthening by means of externally bonded fiber reinforced polymers (FRPs).

External strengthening layers could be applied in full coverage of the panel or in an arrangement of "strips" in various orientations, e.g. **X** or **H** or other frames, while FRPs could be made of E-glass or carbon or aramide fibers, in uni- or bi-directional composites (with a linear or a bi-linear σ-ε constitutive law, for $\pm 45^o$ fiber orientation).

The strengthening scheme could include various shear connections between the frame and the panel or it could be limited on the masonry panels themselves (in the case of an intact contact between the panel and the frame).

8. CONCLUDING REMARKS

The effect of URM infills in RC frames (or quasi-frames) could be significant, globally or locally, favorable or not. Modern Codes contain certain principles, criteria and application rules for a reliable and safe estimation of the real response of such "hybrid" structures during the earthquake itself, based on an extensive international research and study, both analytical/theoretical and experimental, not to mention lessons learnt in past earthquakes.

To this end, EC 8-1 and EC 8-3, as well as the detailed new Greek Code on (Structural) Interventions (nGCI, 2010/2011), fully harmonized with ECs, refer to almost all aspects of the seismic design of such structures, in a normative or informative manner. Most of these aspects and provisions or rules of these Codes, already in force, are presented and discussed in this paper (and its rather lengthy APPENDICES on specific relative issues).

The operationality of the nGCI (regarding URM infills) has been checked (by means of a limited number of seismic designs and redesigns, till now) and found satisfactory [75], although it is seemingly complex and rather lengthy¸ not to mention that there are still some problems to be solved. Additional studies and calibrations are underway regarding the applicability of the Code, while certain modifications or corrections (regarding URM infills) are expected.

Of course, this Code, in line with all modern ones, is in favor of non-linear (inelastic) analysis (static one), more relaxedthan that interms of forces; nevertheless, it is "pro-

Table 1. Values of Rfactors for Damaged URM Infills

DAMAGE LEVEL	SHORT DESCRIPTION	r_K	r_R
DL 1 Light	Light cracks, generally isolated ones, with a width< 2÷3mm, in particular around openings, or debonding/separation cracks. Multiple cracks, generally light ones, interconnecting or not,especially on masonry infill panels with multiple or large openings/perforations.	0,90 0,70	0,90 0,70
DL 2 Significant	Substantial cracks, diagonal or bidiagonal ones, with a width > 5mm, debonding/separation cracks, cracks on posts or belts, w/o significant displacement out-of-plane (< 5mm).	0,50	0,50
DL 3 Heavy	Heavy/severe cracks, generally bidiagonal ones, failure, wide debonding/separation, substantial damages on posts or belts, significant displacement out-of-plane (but < 15mm).	0,20	0,20

[Values for r_{du} factors are not given; engineering judgement is needed.Substantial damage, i.e. that with r or $r_f \leq 0{,}85$, shall be fully repaired, in any case.]

moting" an "intermediate" level of linear (elastic) analysis based on member ductilities $\mathbf{m_i}$ (including URM infills), finally and overall calibrated by means of a global (and modified) behavior factor **q**, suitable for infilled RC frames (or quasi-frames) as well.

Nevertheless, it has to be mentioned that certain relative aspects are not duly covered by the technical literature or the Codes themselves, as follows :

(i) The increased ability of infilled frames to absorb energy even after their max. resistance should be properly taken into account in the seismic design, e.g. by means of an increased viscous damping (based e.g. on their global influence regarding period values and on their residual characteristics), compared to those of the RC structure.

(ii) Infilled frames, even with non-engineered and non-structural URM infills, could be taken into account not only in PLs A and B but in PL C as well (i.e. collapse prevention), if a detailed analysis proves that the bounding RC frame remains fully stable following the failure (or loss) of an infill panel.

(iii) Possible eccentricities between the infill panels and the surrounding in contact RC framing elements should be considered. Of course, EC 8-1 contains a relative and rather strict rule for ductile RC structures (DC H or M):

The eccentricity of the beam axis relative to that of the column into which it frames shall be limited, to enable efficient transfer of cyclic action-effects between "primary" elements to be achieved, while to enable this requirement to be met the eccentricity **e** (i.e. the distance between the centroidal axes of the 2 members) should be limited to less than $b_c/4$, where b_c is the cross-sectional dimension of the column normal to the longitudinal axis of the beam (and to the planar frame).

Due to the facts that (1) some RC members in infilled structures could be regarded as "secondary" seismic elements and (2) the need for cyclic transfer between RC members themselves is "blunted", a more relaxed rule is proposed by the authors, that of $e < b_c/3$ instead of $e < b_c/4$. Of course, the full thickness of infill panels should be "contained" within the width of the beam and of the column.

(iv) It seems that the biaxial in-plane behavior and strengths of URM panels-infills, "contained" (→ "confined") or not, unfortunately **DO NOT** follow any of the well known constitutive laws or approaches.

A set of strengths (and mechanical characteristics) depend on the "composite" (and its construction details), while another set of strengths depend primarily on the mortar itself (with a limited overall influence of the "composite").

In fact, this is true for masonry in general (load bearing or not), with a very low relative strength ratio of the constituent materials, as it is the case of URM panels-infills, with $f_{bc}/f_{mc} > 2$ to 3 and $f_{bt}/f_{mt} > 5$ to 8 (see APPENDIX **E**).

Therefore, there is a need of additional studies and calibration of models and resistances, as well as of the interaction of URM panels-infills and of modern RC structures, designed and constructed according to modern seismic codes.

APPENDIX A

Additional Irregularities Due to Masonry Infills

For structural systems and masonry infills as per §§ 2a to 2e of this paper, the consequences of any additional irregularities especially due to the infills shall be properly taken into account in the design or redesign (see § 2f), as follows (EC 8-1, § 4.3.6.3) :

(i) Irregularities in plan

- Strongly irregular, non-uniform or non-symmetrical arrangements of infills in plan, taking into account the extent of wedging or of openings or perforations in infill panels, should be avoided.
- In the case of severe in plan irregularities due to the infills (e.g. existence of infills mainly along two consecutive faces of the building), spatial models should be used for the analysis.

  Infills should be included in the model and a parametric sensitivity analysis should be performed, regarding their position and their properties, e.g. by disregarding 1 out of 3 or 4 panels in a planar frame, especially on the more flexible sides.

  Special attention should be paid to the verification of structural elements on the more flexible sides of the plan of the building (i.e. furthest away from the side where infills are concentrated) against the effects of any, even accidental, torsional response caused by the infills.

  To this end, infill panels with more than 1 significant openings or perforations (e.g. a door and a window) should be disregarded in such models for analyses (in accordance with the previous paragraphs).
- When masonry infills are not regular, but not in such a way as to constitute a strong irregularity in plan, these irregularities may be taken into account by increasing by a factor of 2 the effects of the accidental torsional eccentricity of storey mass from its nominal location (i.e. $e_a = \pm 0{,}10$ L instead of $\pm 0{,}05$ L, where **L** is the floor dimension perpendicular to the direction of the seismic action), in accordance with the rules for linear-elastic analyses.

(ii) Irregularities in elevation

- As a basic principle, if there are considerable irregularities in elevation (e.g. drastic reduction of infills in 1 or more storeys compared to the others, pilotis etc.), the seismic action-effects in the vertical elements of the respective storeys shall be increased, as a counterbalance measure against the lack of increased resistance due to infills.
- If a more precise and detailed approach is not used, a relative deemed to satisfy rule is the amplification of calculated seismic action-effects (axial forces, bending moments and shear forces) by a magnification factor $\eta = (1+\Delta V_{Rw}/\Sigma V_{Ed}) \leq q$, where **q** is the behavior factor, ΔV_{Rw} is the total reduction of the resistance of masonry infills in the storey concerned, compared to the more infilled storey above it, and ΣV_{Ed} is the sum of the seismic shear forces acting on all vertical seismic members of the storey concerned, and especially the primary ones – i.e.

practically those contributing more than 85% to lateral stiffness of the building (or more than 75% for an existing one).

To this end, if the above magnification factor **n** is lower than 1,1, there is no need for modification and amplification of the seismic action-effects (N, M and V values for columns).

<u>NOTES</u> (by the Authors)

1) Irregularities due to infills may be imposed not only due to non-uniformity or non-symmetry, as far as their arrangement is concerned, but due to mechanical particularities as well, e.g. due to differences in panels aspect ratio or thickness or in their degree of "active" connection with the frame, not to mention possible problems due to their varying degree of damage suffered during the earthquake itself (see APPENDICES **B** to **D**).

2) In fact, EC 8 does not provide rules (or at least principles or criteria) for modelling of URM infills or for their verification.

APPENDIX B

Adverse Local Effects Due to Masonry Infills (in General)

For structural systems and masonry infills as per §§ 2a to 2e of this paper, the possibly adverse local effects due to their interaction (e.g. premature formation of unstable mechanisms or brittle shear failure of primary or even secondary columns under concentrated shear forces induced by infills), shall be properly taken into account and avoided (see § 2g) by specific design or redesign verifications, according to EC 8-1, § 5.9 (for concrete buildings), as follows:

- Because of the particular vulnerability of infill walls of ground floors (mainly under in-plane actions), a seismically induced irregularity is to be expected there and appropriate measures should be taken. If a more precise method is not used, the entire length of columns of the ground floor should be considered as a critical length/region (i.e. dissipative zone) and be detailed/confined accordingly.

  In addition, where the masonry infills extend to the entire length of adjacent columns, and there are masonry walls/panels on only one side of the column (e.g. corner or other columns), the entire length of the relevant column should be considered as a critical dissipative zone and be detailed/reinforced accordingly.

- If the height of masonry infills is equal to the clear length of the adjacent concrete columns (full infilling), the "contact" length ℓ_c of columns (i.e. the short length over which the equivalent diagonal strut force of the infill is assumed to be applied), should be verified and detailed in shear, as in APPENDIX **C**.

  According to several studies (see for example [45]), RC beams are relieved while RC columns are overloaded in shear (close to their end-sections) under the seismic action in infilled frames (or quasi-frames). Thus, the EC 8 and the nGGI do not contain rules for infills and RC beams.

  In addition, problems close to frame joints are rather limited, with the exception of older structures containing "weak" RC elements and heavy well wedged URM infills, of higher strength (e.g. with f_{wv}>250 to 350 kPa).

- If the height of masonry infills is smaller than the clear length of the adjacent concrete columns, the consequences of the decreased shear ratio of those columns, due to the actual "naked" (or clear) column length ℓ_n, should be appropriately covered, among other additional measures, as in APPENDIX **D**.

<u>NOTES</u> (by the Authors)

1) In principle, EC 8 does not allow for a reduction of seismic action-effects on RC frames (or quasi-frames) due to the presence of interacting non-structural infills, of any type (except of "confined" ones).

On the contrary, the Code refers to their possible adverse effects (globally or locally) and contains certain provisions and rules for minimizing such effects.

2) The presence of infills in framed (or quasi-framed) structures could invalidate the whole "delicate" seismic design, by imposing concentrated inelastic deformations and ductility demands or leading to premature brittle failures (even at local level), unless proper and adequate measures are taken.

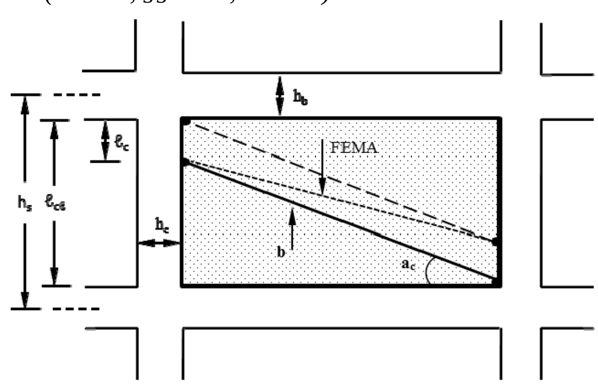
3) Some RC framing members could be taken into account as secondary (and not primary) seismic elements, with rather "relaxed" verification and detailing rules.

4) According to FEMA ([27]), the requirements for local checks of columns or beams shall be waived if the mean URM shear strength (based on tests) is less than appox. 140 kPaor 350 kPa, respectively.

APPENDIX C

Local effects due to full infilling, i.e. ifthe height of infill panels is equal to the clear length of the adjacent RC columns

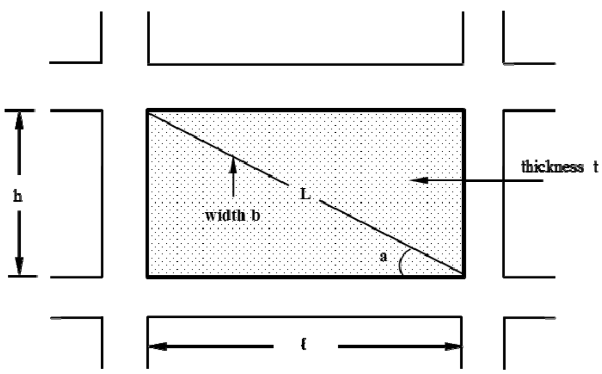
(EC 8-1, §§ 5.9.1, 3 and 4).



h_s : storey height, h_c : column's dimension, ℓ_{cl} : column's clear length
ℓ_c : column's "contact" length, b : strut's breadth – width (URM infill panel)

Fig. (C.1). Geometry of the frame and of the panel.

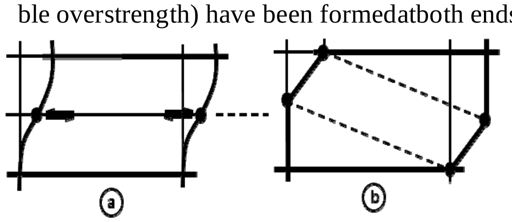
- The entire length of RC columns is considered as a critical region and should be detailed/reinforced accordingly. This rule is applied in any case, if $\ell_{c\ell}/h_c \leq 3$, for DC H or M (with $\nu_d \leq 0{,}55$ or 0,65, respectively).
- Unless a more accurate estimation is made, taking into account the geometry and the elastic properties (?) of the masonry infill panels and of the RC framing elements (beams and, mainly, columns), equivalent strut's breadth-width **b** may be assumed to be a fixed fraction of the length of panel's diagonal.
- The column's "contact" length$\boldsymbol{\ell_c}$ shouldbe assumed to be equal to the full vertical breadth-width of the diagonal strut of the infill, i.e. $\ell_c \approx$ b/cosa.
- This "contact" length$\boldsymbol{\ell_c}$should be verified in shear for the smaller of the following two shear forces :
  a) The horizontal component of the infill's strut force, assumed to be equal to the horizontal shear strength of the URM panel, as estimated on the basis of the full section of the panel and the shear strength of mortar's bed joints, or



$L = \sqrt{\ell^2 + h^2} = h/\text{sina} = \ell/\text{cosa}$, $b \approx \kappa \,.\, L$ ($\kappa \approx 0{,}15$, see §§ 6.3 and 6.4).

Fig. (C.2). Geometry of the strut.

  b) The shear force computed in accordance with the shear capacity design criterion, depending on the ductility class, assuming that the overstrength flexural capacity of the (primary or secondary seismic) column, γ_{Rd}. M_{RC}, develops at the two ends of this length $\boldsymbol{\ell_c}$, i.e. assuming that plastic hinges (with their possible overstrength) have been formedatboth ends of$\boldsymbol{\ell_c}$.



V_{Ed} : min (V_a ; V_b), where
$Va = Aw \,.\, f_{wv} = (t \,.\, \ell) \,.\, f_{wv}$ and
$Vb = 2\, \gamma_{Rd} \,.\, M_{RC} / \ell_c$, with
γ_{Rd} = 1,3 or 1,1 for DC H or M, respectively.

Fig. (C.3). Failure modes and shear forces.

In these expressions, M_{RC} is the design value of column's bending moment of resistance (corresponding to the axial force in the design seismic situation), in infill's plane, and γ_{Rd} is the overstrength factor, accounting for steel strain hardening and concrete confinement in the column's compression zone.

<u>NOTES</u> (by the Authors)

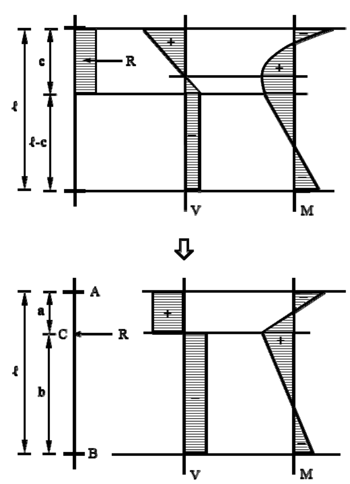
1) Concentrically braced frames are more suitable for global analyses, but forces on columns (and beams) are not represented. On the other hand, eccentrically braced (knee-braced) frames yield infill effects on "critical" columns directly and an overall sideway mechanism, controlled by the RC columns' plastic regions as well as the "residual" infill resistances.

According to several authors (see, e.g., [46]), the consequences of full (or even partial) infilling could be based, as a simplification, on a columns' length $\ell' \approx \ell_{c\ell}/2$, for both the windward upper and the leeward lower part of the columns, disregarding "actual" "contact" (or "naked") lengths.

2) According to FEMA [27], a slightly different approach is foreseen, with a less inclined strut and $\ell_c \approx$ b/cosa$_c$, withtana$_c \approx (\ell_{\ell c}-\ell_c)/\ell$, see the sketch at the beginning of this APPENDIX **C**.

This approach is used for partial infilling (and captive columns) as well, see **NOTE 3** of the following APPENDIX **D**.

3) In the case of full infilling, the following seismic action-effects are expected on framing columns, based on an elastic approach, while similar expressions may be found based on a plastic approach (e.g. based on plastic hinges):



$V_A = (R \,.\, b/\ell) \,.\, (1 - a^2/\ell^2 + ab/\ell^2)$
$V_B = (R \,.\, a/\ell) \,.\, (1 - b^2/\ell^2 + ab/\ell^2)$

$M_A = R \,.\, ab^2/\ell^2 = M_B \,.\, b/a$
$M_B = R \,.\, a^2b/\ell^2 = M_A \,.\, a/b$
$M_C = 2R \,.\, a^2b^2/\ell^3$

[For $a = \ell/5$: $V_A \approx 0{,}9\,R$, $V_B \approx 0{,}1\,R$, $M_C \approx 0{,}05\,R \,.\, \ell$, $M_A = M_C \,.\, \ell/2a$, $M_B = M_C \,.\, \ell/2b$]

Fig. (C.4). Local effects on RC columns due to infills.

4) According to an early parametric analysis [28] of URM infills, with an aspect ratio $\alpha = h/\ell$ = 0,5 to 1,0 (h = 2,5 m), and rather thick and strong (t = 0,2 m, f_{wc} = 2 to 10 MPa), the following findings are valid :

A kind of knee-joint is formed; it is assumed that almost 40% of the total strut's force F ($F = t \,.\, b \,.\, f_{wv,s} \approx t \,.\, b \,.\, 0{,}25\, f_{wc}$) is acting on the RC column at a small distance ℓ_o from its end-section, with $\ell_o \approx$ (b/3) cos a.

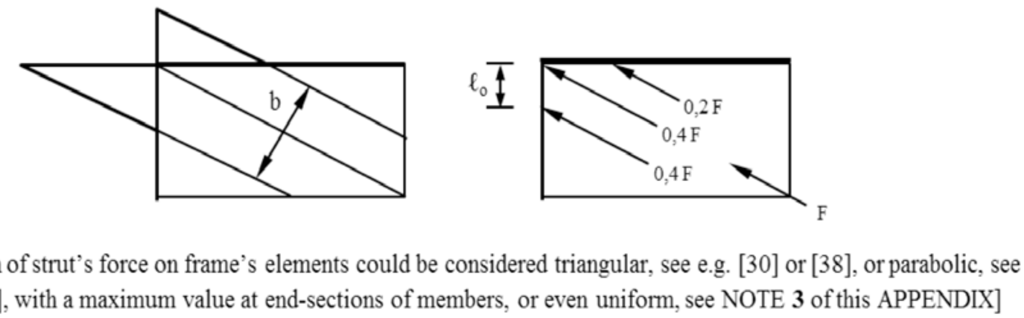


Fig. (C.5). Distribution of strut's force [28].

Based on geometrical and mechanical data, it was found [28] that $b \approx 0{,}15$ L, $\ell_o \approx 0{,}3$ m and $V_i \approx 0{,}015\ f_{wc}$. (t . ℓ).

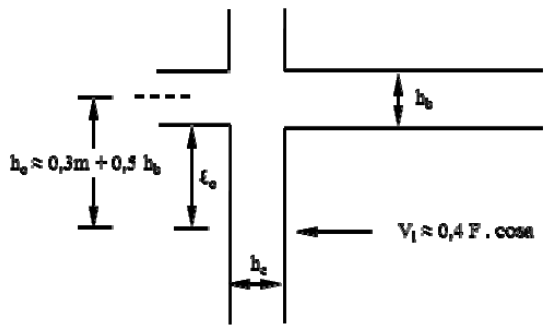


Fig. (C.6). Additional RC column's shear force ([28]).

Thus, the "additional" column's design shear force (to be carried by the RC column, by means of "additional" transverse reinforcement) could be estimated as follows :

$V_d=(h_o/2h_c).\ V_i = [(0{,}15\ \text{m} + 0{,}25\ h_b)/h_c]\ .\ 0{,}015\ f_{wc}.\ (t.\ \ell)$.

In this expression, a proper reduction of the shear force is foreseen, due to a direct strut action for loads near direct supports (see EC 2-1-1: 2004, §§ 6.2.2 (6) and 6.2.3 (8), with a reduction coefficient $\beta = a_v/2d_c$, $a_v = h_o$ and $d_c \approx h_c$, $0{,}5\ d_c \leq a_v \leq 2\ d_c$ and $0{,}25 \leq \beta \leq 1{,}00$, for fully anchored longitudinal reinforcement).

APPENDIX D

Local Effects Due to Partial Infilling, i.e.if the Height of Infill Panels is not Equal to the Clear Length of the Adjacent RC Columns (EC 8-1, § 5.9.2).

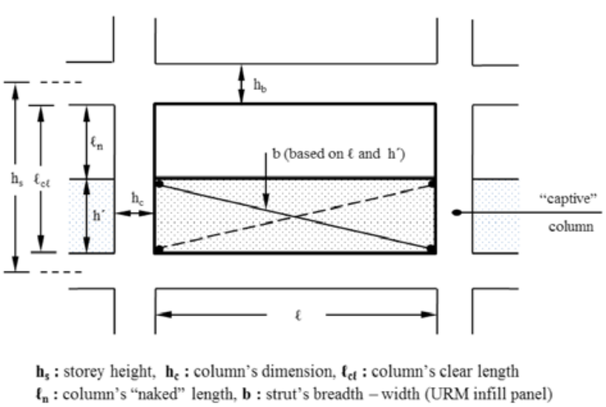


Fig. (D.1). Geometry of the frame and of the panel.

- The entire length of RC columns is considered as a critical region and should be detailed/reinforced accordingly. This rule is applied in any case, if $\ell_{c\ell}/h_c \leq 3$, for DC H or M (with normalized axial force $\nu_d \leq 0{,}55$ or 0,65, respectively).
- The "naked" length $\boldsymbol{\ell_n}$ should be verified in shear for the shear force computed in accordance with the shear capacity design criterion, depending on the ductility class, assuming that plastic hinges (with their possible overstrength) have been formed at both ends of $\boldsymbol{\ell_n}$**,** as it is the case **b** of full infilling (previous APPENDIX **C**, **b**,with $\boldsymbol{\ell_n}$ instead of$\boldsymbol{\ell_c}$**,** i.e.$V_{Ecd} = 2\ \gamma_{Rd}\ .\ M_{Rc}/\ell_n$).
- The transverse reinforcement to resist this shear force V_{Ecd} should be placed along $\boldsymbol{\ell_n}$(non-contact length, "naked") andextend a length $\mathbf{h_c}$(column's dimension in infill's plane) into the column's part in contact with the infill,i.e. for a length$\ell_n + h_c$.
- If $\ell_n \leq 1{,}5h_c$, the shear force V_{Ecd} should be resisted entirely by bidiagonal reinforcement.

  To this end, EC 8 does not contain rules for such a reinforcement for columns; therefore, use could be made of similar provisions for DC H beams or coupling beams, when an almost full reversal of shear forces is expected :

  For an algebraic value of the ratio ζ = min. V_E/max. $V_E \approx$ - 1, the area of reinforcement in each diagonal direction, crossing the column end-sections at an angle a to the axis of the element, should be $A_S \geq 0{,}5\ V_E/f_{yd}$. sina, with a = 45°, or tana ≈ $0{,}8\ h_e/l_n$.

  The anchorage length of bidiagonal reinforcement should be 50% greater than that required by EC 2-1-1: 2004.

<u>NOTES</u> (by the Authors)

1) In the case of "short" columns (due to partial infilling), with a height (see the previous sketch) approx. equal to $h_n \approx \ell_n + 0{,}5\ (h_b + h_c)$, combined with regular-"free" columns (with no infilling at all), with a height equal to h_s $(=\ell_{c\ell} + h_b) > h_n$, a first (elastic) consequence is that these "short" columns (with higher stiffnesses) attract higher shear forces, multiplied by $(h_s/h_n)^3$, as well as higher bending moments, multiplied by $(h_s/h_n)^2$.

In fact, after the formation of plastic hinges at both end-sections of both columns (with their possible overstrength, $M_{Ro} = \gamma_{Rd}\ .\ M_R$), shear forces could be estimated as $V_{1,p\ell} = 2M_{Ro}/h_s$ and $V_{2,p\ell} = 2M_{Ro}/h_n = V_{1,p\ell}\ .\ (h_s/h_n)$.

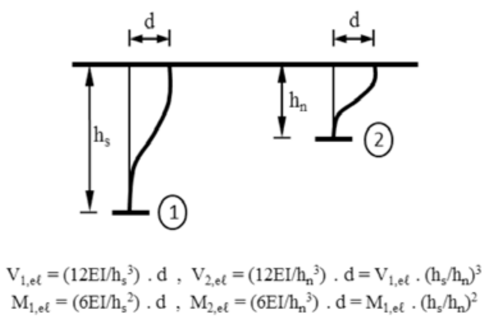


Fig. (D.2). The elastic approach for RC columns with different heights.

Of course, the drastic consequence for such "short" columns is their reduced shear span ratio ($\alpha_s = \ell_s/h_c = M/V.h_c$), with a relevant adverse M-V interaction and reduction of strengths, and their reduced ductility.

2) The provisions for partial infilling (according to this APPENDIX **D**), cover other "accidental" cases andinstability mechanisms as well, e.g. those due to premature failure and falling down of some infills in the case of full infilling (see APPENDIX **C**).

To this end, it is recommended [18] that RC columns should be checked according to the provisions of APPENDICES **C** and **D**, with a length ℓ_c (≈ b/cosa) or $\ell_n \approx 0{,}4\ \ell_{c\ell}$ ("accidentally" "naked" length), whichever is smaller.

3) According to FEMA [27], a slightly different approach is foreseen (see also **NOTE 2** of the previous APPENDIX **C**), as follows :

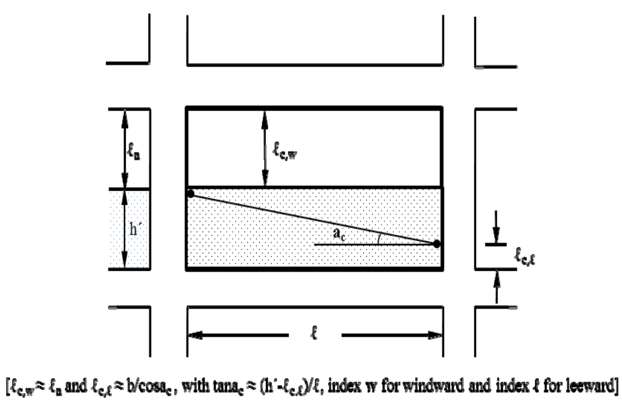


Fig. (D.3). The relevant approach according to FEMA ([27]).

4) In the case of very short "naked" lengths (i.e. if $\ell_n \leq 1{,}5\ h_c$), the requirement for bidiagonal reinforcement capable of resisting entirely the shear force ± V_E is very difficult to be met; other alternatives should be examined, e.g. that of "isolation" of the infills and full check against out-of-plane effects.

APPENDIX E

Data for Greek URM infills

i) Specifically for the purpose of the nGCI, default mean values of strengths of URM infills could be used contained in the following Table, if more precise data are not available (see § (iv) here below), with $\bar{f}_{wc,s}$ (in kPa) the mean diagonal compression strength(along the strut) and $\bar{f}_{wv}$ (in kPa) the mean shear strength (along the bed joints).

Table E.1. Default Strength Values for Greek URM Infills [45]

	INFILL PANEL	CONDITION AND WEDGING		
		GOOD	FAIR	POOR
$\bar{f}_{wc,s}$	DOUBLE LEAF, t_{eff} ≈ 0,2 m	2000	1500	1000
	SINGLE LEAF, t_{eff} ≈ 0,1 m	1500	1000	750
$\bar{f}_{wv}$	DOUBLE LEAF, t_{eff} ≈ 0,2 m	250	200	150
	SINGLE LEAF, t_{eff} ≈ 0,1 m	200	150	100

These default values are valid for :

- Common greek ifills of the last 30 to 50 yrs, with an aspect ratio α< 1;
- Clay units-bricks with horizontal perforations (more than 35% voids);
- Poor lime-cement mortars;
- Almost fully filled bed joints (with a thickness of 10 to 15 mm);
- Partially (~ 50%) filled head joints (with a similar thickness), and
- Infill panels under practically zero normal stress (i.e. σ_o≈o), except that due to their own self weight.

Based on the Table, and for URM infills not in a poor condition and wedging, the overall mean values of strengths are :

$\bar{f}_{wc,s}$ ≈ 1,50 MPa and $\bar{f}_{wv}$ ≈ 0,20 (to 0,25) MPa, with $\bar{f}_{wv}/\bar{f}_{wc,s}$ ≈ 0,15.

For older RC structures, with thicker, heavier and stronger infill panels, under a considerable σ_o (at their middle), there is evidence that $\bar{f}_{wc,s}$ values could be 1,5 times higher (up to 2,50 or even 3,00 MPa) and $\bar{f}_{wv}$ values could be 2,0 times higher (up to 0,50 MPa).

NOTE

According to FEMA [27], a similar "condition" of URM infills is foreseen, as follows :

- Good : Intact panels, with no "visible" cracks,
- Fair : Minor cracks only, and
- Poor : Degraded materials, significant cracks,

while default values of strengths are provided, with mean values equal to 1,3 times the lower-bound ones, which in turn are equal to the mean values minus one standard deviation, i.e. $f_m = 1{,}3\ (f_m - s)$, or $s/f_m \approx 0{,}20$ to $0{,}25$ (a rather low normalized standard deviation for comnon greek URM infills, see §§ (ii) and (iv) here below).

To this end, the foreseen default mean values of strengths (in kPa) are given in the following Table; finally, they have to be modified by the κ- factor (0,75 to 1,00), depending on the knowledge level (minimum, usual or comprehensive).

Table E.2. Default Strength Values, FEMA [27]

	CONDITION		
	GOOD	**FAIR**	**POOR**
f_{wc}	~ 8.000	~ 5.400	~ 2.700
$\bar{f}_{wv}$	~ 240	~ 180	~ 120

[Compression strengths are rather high, while shear strengths – depending mainly only on the mortar – are almost the same with those of greek infills, see Table 2, with an overall mean value of approx. 150 to 200 kPa]

(i) According to the nGCI, the following are provided for the strengths of common URM greek infills (in an informative appendix):

- f_m = mean "accredited" or measured value, depending on the criteria for investigation/documentation
- s = standard deviation, with $s/f_m \approx 0{,}2$ to 0,4 (i.e. highly increased uncertainties)
- f_d = design value = f_κ/γ_m, with
- f_κ = characteristic value
- For linear analysis, $f_\kappa = f_m - s$ and $\gamma_m = 2{,}0$ or 1,5 , for DRL S or H, respectively.

  Recommended f_κ = min (0,65 f_m ; f_m – f), with f ≈ 0,50 MPa or 0,05 MPa for diagonal compression or shear, respectively.
- For non-linear analysis, $f_\kappa = f_m$ and $\gamma_m = 1{,}1$ or 1,0 for DRL S or H, respectively.

(ii) Certain reliable and calibrated models for greek URM infill shear panels (in terms of shear stress-angular distortion, **τ-γ**) have been proposed, suitable for monotonic and cyclic actions as well; a set of these models are presented and compared here below (see Table **4**), within the framework of the nGCI, while additional data are given in § (iv).

The general skeleton curve of these models is a 3-linear (up to a 5-linear) one, simplified as follows:

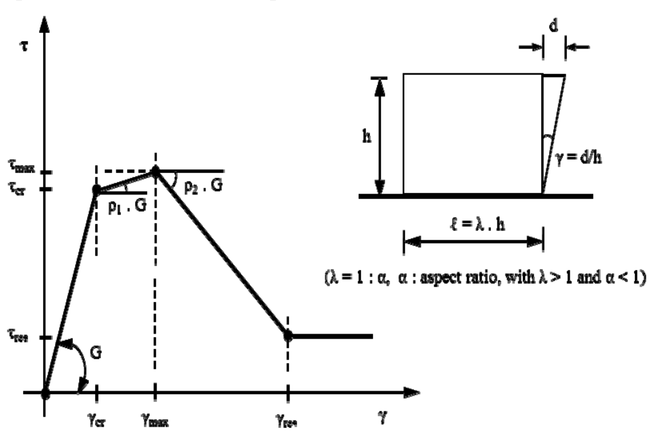


Fig. (E.1). The general model for URM infills.

(ii) Model proposed by T.P. Tassios, 1984 [18].

- $\tau_{cr} \approx 2/3\ f_{wt}\ .\ \sqrt{1 + (\sigma_o/f_{wt})} \approx 2/3\ f_{wt}$,

  with $f_{wt} \approx (0{,}15 \text{ to } 0{,}35)\ \sqrt{f_{wc}}$(MPa), depending on "confinement"
- $\gamma_{cr} \approx 0{,}5$ to 1,0 ‰ for $\lambda \geq 1$ or 1,0 to 2,0 ‰ for $\lambda \leq 1$,
- $\tau_{max} \approx 1{,}30\ \tau_{cr}$ and $\gamma_{max} \approx 1{,}30\ \gamma_{cr}$,$\tau_{res} \approx 0{,}40\ \tau_{cr}$ and $\gamma_{res} \approx 3{,}00\ \gamma_{cr}$.

  To this end, for $f_{wc} \approx 1{,}5$ MPa and $\lambda = \ell/h \approx 2{,}0$(i.e. $\alpha \approx 0{,}5$), it is concluded that :

  $\tau_{cr} \approx 0{,}2$ to 0,4 MPa / $\tau_{max} \approx 1{,}30\tau_{cr}$, and

  $\gamma_{cr} \approx 0{,}5$to1,0 ‰ / $\gamma_{max} \approx 1{,}30\ \gamma_{cr}$,

  with f_{wc} and f_{wt} the strengths along the diagonals (based on semi-empirical relations).

1) Model proposed by M. Fardis and T. Panagiotakos, 1996 [70, 71].

- $\tau_{cr} \approx f_{wv} \approx f_{wt}$ and $\tau_{max} \approx 1{,}30\ \tau_{cr}$
- $\gamma_{cr} \approx 1{,}5$ ‰ and $\gamma_{max} \approx \gamma_{cr}.\ [1 + (0{,}3/\rho_1)]$
- $\rho_1 \approx$ (0,05 to) 0,20 and $\rho_2 \approx 0{,}01$ to 0,10
- $\tau_{res} \approx 0{,}05$ to0,10 τ_{cr} and $\gamma_{res} \geq 2{,}00\ \gamma_{max}$.

  To this end, for $f_{wv} \approx f_{wt} \approx 0{,}25$ MPa and $\rho_1 \approx 0{,}15$, it is concluded that:
  $\tau_{cr} \approx 0{,}25$ MPa / $\tau_{max} \approx 1{,}3\tau_{cr}$, and
  $\gamma_{cr} \approx 1{,}50$‰ / $\gamma_{max} \approx 3{,}00\ \gamma_{cr}$,
  with f_{wc} and f_{wt} the strengths along the diagonals (based on tests, on-site or in-lab/on wallettes).

2) Model proposed by A. Kappos and K. Stylianidis, 1998 [72, 73].

- $\tau_{cr} \approx 0{,}70\tau_{max}$ $\begin{cases} 0{,}70.\ 0{,}22\ \sqrt{f_{wc}}\ ,\ N = 0 \\ 0{,}70\ .\ 0{,}35\sqrt{f_{wc}},\ N \neq 0 \end{cases}$
- $\gamma_{cr} \approx 0{,}22\ \gamma_{max}$ $\begin{cases} 0{,}09\ /(80 + h/t).\ \sqrt{f_{wc}}\ ,\ N = 0 \\ 0{,}11\ /(80 + h/t).\ \sqrt{f_{wc}}\ ,\ N \neq 0 \end{cases}$

  (N: axial load on RC framing columns, values in MPa)
  To this end, for $f_{wc} \approx 1{,}50$MPa, $h \approx 2{,}50$ m and t = 0,1 or 0,2 m, it is concluded that :
  $\tau_{cr} \approx 0{,}2$to 0,3 MPa / $\tau_{max} \approx 1{,}45\ \tau_{cr}$, and
  $\gamma_{cr} \approx 1{,}0$ ‰ / $\gamma_{max} \approx 4{,}50\ \gamma_{cr}$.

- Model proposed by M.P. Chronopoulos, 2004 [45], for $\alpha < 1$.
  - At initial cracking
    $\tau_{cr} \approx$ (0,75 to) 1,00 . $f_{wt,s}$ [higher values for higher σ_o]
    $\gamma_{cr} \approx 1{,}0$ to 3,0 ‰ [0,5 to 4,0 ‰, increased sensitivity]
  - At maximum strength
    $\tau_{max} \approx$ (1,0 to 1,5) . τ_{cr}[higher values for higher σ_o]
    $\gamma_{max} \approx$ (2,0 to 4,0) . γ_{cr}[1,0 to 8,0 ‰, increased sensitivity]
  - Residual characteristics
    $\tau_{res} \approx$ (0,15 to 0,35) . τ_{max}[depending on the damage]

Table E.3. Typical values of the 4 relevant models.

	τ_{cr} (MPa)	γ_{cr} (‰)	τ_{max}/τ_{cr}	γ_{max}/γ_{cr}	τ_{res}/τ_{max}	$\gamma_{res}/\gamma_{max}$
[18] 1984	0,30	(0,75)	1,30	1,30	0,30	2,25
[70], [71] 1996	0,25	1,50	1,30	3,00	0,10	≥ 2,00
[72], [73] 1998	0,25	1,00	1,45	4,50	–	–
[45] 2004	0,20	2,00	1,25	3,00	0,25	2,50
RECOMMENDED VALUES	0,25	1,50	1,25	3,00	0,25	2,50

$\gamma_{res} \approx (2,0to3,0) \,.\, \gamma_{max}$[depending on the damage]

To this end, one could conclude that :

$\tau_{cr} \approx f_{wt,s}and\gamma_{cr} \approx 2,0$ ‰ ,while $G \approx 500\, f_{wt,s}$.

Index **s** is valid for diagonal strengths, with $f_{wv} \approx f_{wt,s} \approx 0,15\, f_{wc,s}$ (overall mean values).

The basic characteristics and the relevant values according to these 4 models are compared in the Table here below.

To this end, differences are not that high, taking into account the variety of related (or even interrelated) uncertainties, not to mention that the response of infills is influenced by their geometry, i.e. their aspect ratio (α=h/ℓ), their slenderness (λ=L/t_{eff}), and the surrounding RC framing elements. Nevertheless, γ_{cr} values are lower than those of the nGCI by a factor of 2.

(i) Based on [43] to [45], as well as on relevant calibrations (see, e.g., [74, 75]), the following analytical data are given for common greek URM infills :

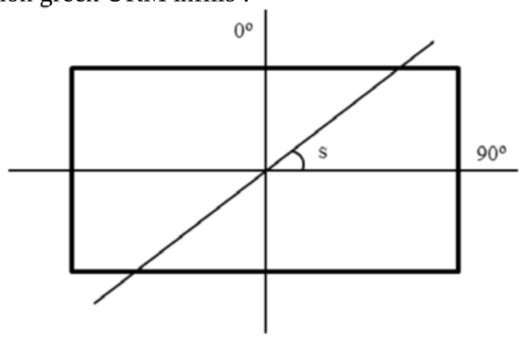


Fig. E.2: Explanation of subscripts used in the following text.

- Highly increased uncertainties are encountered, even higher than those associated with plain masonry itself (load bearing one); therefore, large scattering of mechanical (and other) characteristics is expected.
- Common clay units-bricks (**b**for blocks) are used, with approx. dimensions 60x85x185 mm, with 6 horizontal perforations (more than 35 and up to 50 % voids) and with webs of 5 up to 10 mm in thickness.

The compressive strengths of bricks are : $f_{bc,0} \approx$ 2,0/4,0 to 7,0/9,0 MPa (overall mean 4,0 MPa)

$f_{bc,90} \approx$ 6,0/9,0 to 15,0/22,0 MPa.

- Common poor lime-cement mortars (**m** for mortars) are used, of low strengths and characteristics, depending on a lot of (construction) parameters.

Their strengths are : $f_{mc} \approx$ 1,0 to 5,0/7,0 MPa (overall mean 1,5 MPa)

$f_{mt} \approx$ 0,1 to 0,4 MPa (overall mean ≈ 0,2 $f_{mc} \approx$ 0,3 MPa).

- Infills are made with a running bond, with almost fully filled bed joints (with a thickness of 10÷15 mm) and partially (~50 %) filled head joints (with a similar thickness).

Three types of URM infills are common, namely :

– Single leaf, with a nom. thickness of 100 to120 (140) mm and an effective one of 100 mm (nom. weight ~2,0 kN/m^2);

– Double leaf, with a nom. thickness of 180 to 220 (240) mm andan effective one of 200 mm (nom. weight ~3,5 kN/m^2),and

– Cavity or "hollow" panels, made of 2 wythes, mostly unconnected, to facilitate insulation or other (architectural) needs.

In what follows, mean values of strengths of the two main greek types of infills are given, while higher or lower values (up to ±20 %) are expected for double or single leaf panels, respectively; cavity panels (with an actual thickness of each skin equal approx. to 70 up to 100 mm) are not considered at all regarding in-plane behavior.

- Compression strengths :

  $f_{wc,0} \approx$ 1,5 to 5,0 MPa (overall mean 2,75 MPa)

  $f_{wc,90} \approx$ 0,4 to 0,9 $f_{wc,0}$

  $f_{wc,s} \approx$ 0,5 to 0,7 $f_{wc,0}$ (overall mean 1,50 MPa).

To this end, $f_{wc,0}$ could be found based on the relative strengths of the constituent materials (see, also, T. Paulay and M.J.N. Priestley, 1992, [46], based on the work of H.K. Hilsdorf, 1969), as follows :

$f_{wc,0} \approx \xi\, (0,65\, f_{bc,0} + 0,1\, f_{mc})$, with $\xi \approx 1,00$ for $t_{joints} \approx$ 10 to 15 mm or $\xi \approx 0,85$ for t_{joints}> 15 mm.

Other relevant "characteristic" are : $\varepsilon_{max} \approx 2{,}0/3{,}0$ to $4{,}0/9{,}0$ ‰ ;

E at ~ $0{,}5\ f_{wc,0} \approx 500$ to $900\ f_{wc,0}$, and

E at ~ $0{,}9\ f_{wc,0} \approx 100$ to $500\ f_{wc,0}$.

- Tensile strengths :

$f_{wt,0} \approx 0{,}5$ to $0{,}8 f_{wt}$

$f_{wc,90} \approx 1{,}7$ to $2{,}0\ f_{wt,0}$

$f_{wt,s} \approx (f_{wt,0} \approx) 0{,}75 f_{mt}$ (overall mean 0,25 MPa).

- Shear strengths :

– Horizontal sliding

$f_{wv}S = f_{wvo} + \mu \cdot \sigma_o$, $f_{wvo} \approx 0{,}1$ to $0{,}3$ MPa , $\mu \approx 0{,}3$ to 0,9 (0,5),

while for $\sigma_o \approx 0 \rightarrow \mathbf{f_{wv}S \approx 0{,}75\ f_{mt}\ (\approx f_{wt,s})}$.

Alternatively, $f_{wv}S \approx 0{,}15$ (to 0,25) $(f_{wc,0})^{1/2}$.

– Diagonal cracking

$f_{wv}C = (0{,}6 \text{ to } 1{,}3)\ f_{wt,s} \cdot (1+\sigma_o/f_{wt,s})^{1/2}$,

while for $\sigma_o \approx 0 \rightarrow \mathbf{f_{wv}C \approx f_{wt,s}\ (\approx 0{,}75\ f_{mt})}$.

Therefore, both shear failure mechanisms are almost equally probable.

- Regarding the biaxial behavior of masonry see also [74] or the "classical" works by A.W. Page and A.W. Hendry during the '70s and the '80s.
- Regarding horizontal sliding under shear, the following are foreseen by others :

– $f_{wv}S \approx 0{,}5/(1+5/f_{wc,0}) \approx 0{,}1\ (f_{wc,0})^{1/2}$, in MPa, for $f_{wc,0} \leq 5$ MPa,

$f_{wv}S \approx 0{,}25$ MPa for $f_{wc,0} \geq 5$ MPa, for older structures, [3];

– $f_{wv}S \approx f_{wvo} + \mu \cdot \sigma_o \leq 0{,}15$ (to 0,20) $f_{wc,0}$, with $f_{wvo} \approx 0{,}1$ to 1,5 MPa (0,04 $f_{wc,0}$)

and $\mu \approx 0{,}3$ to 1,2 (0,5), as a simplication for uncracked masonry, [46].

- For out-of-plane earthquake (EQ) loading, the bending (tensile) strengths of greek URM infills are :

– Approx. 0,30 to 0,40 MPa, for arching between beams, i.e. for horizontal cracking, or

– Approx. 0,50 to 0,70 MPa, for arching between columns, i.e. for vertical cracking.

- Finally, it should be mentioned that URM infills are favorably influenced (in terms of strength and deformation as well) by being "contained" in a RC frame (→ "confined"), while (at the same time) the area of joints and of end-segments of RC framing elements are almost equally "confined" by infills, in the case of full infilling.

CONFLICT OF INTEREST

The authors confirm that this article content has no conflicts of interest.

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REFERENCES

[1] S.V. Polyakov, *Masonry in Framed Buildings*, MIR Publishers: Moscow, 1956 (translated by G. Cairns).

[2] S.V. Polyakov, *On Interaction Between Masonry Fillerwalls and Enclosing Frame When Loaded in Plane of Wall*, Earthquake Engineering, EERI: San Fransisco, 1960.

[3] S.V. Polyakov, *Design of EQ Resistant Structures*, MIR Publishers: Moscow, 1974 (translated by A. Schwartz).

[4] S.V. Sachanski, Solution of the contact problem at the analysis of the interaction between a RC frame and the infilling at loading with horizontal forces, Proc. Bldg Research Institute: Sofia, 1960 in Bulgarian).

[5] M. Holmes, "Steel frames with brickwork and concrete infilling", *Proc. Inst. Civ. Eng.*, vol. 19, no. 4, p. 6501, 1961.

[6] M. Holmes, "Combined loading on infilled frames", *Proc. Inst. Civ. Eng.*, vol. 25, no. 1, p. 6621, 1963.

[7] B.S. Smith, "Lateral stiffness and strength of infilled frames", *ASCE J. Struct. Div.*, vol. 88, p. ST6, 1962.

[8] R.J. Mainstone, "On the stiffnesses and strengths of infilled frames", *Build. Res. Stn.*, vol. CP 2/72, p. 34, 1972.

[9] R.J. Mainstone, and G.A. Weeks, "The influence of a bounding frameon the racking stiffnesses and strengths of brick walls", *Build. Res. Stn.*, vol. CP 3/72, p. 14, 1972.

[10] A.E. Fiorato, M.A. Sozen, and W.L. Gamble, "*An Investigation on the Interaction of RC Frames With Masonry Filler-Walls*", Report no: UILU-ENG, Urbana Champaign, University of Illinois pp. 70-100, 1970.

[11] R.E. Klinger, and V.V. Bertero, "EQ resistance of infilled frames", *ASCE J. Struct. Div.*, vol. 104, p. ST6, 1978.

[12] M. Dhanasekhar, and A. Page, "The influence of brick masonry infill properties on the behavior of infilled frames", *Proc. Inst. Civ. Eng.*, vol. 81, no. 4, pp. 9061, 1986.

[13] M.N. Fardis, and M.G. Calvi, Effects of infills on the global response of RC frame, 10 ECEE, Vienna, 1994.

[14] M.N. Fardis Ed., Experimental and numerical investigation on the seismic response of RC infilled frames and recommendations for code provisions, ECOEST – PREC8 Rep., LNEC, Lisbon, 1997.

[15] A. Kappos, and F. Ellul, *Seismic design and performance assessment of masonry infilled RC frames, 12 WCEE, Auckland, 2000*.

[16] S. Sugano, *State-of-the-art in aseismic strengthening of existing RC bldgs, 7 WCEE*, Istanbul, 1980.

[17] T.P. Tassios, *Physical and mathematical models for redesign of damaged structures,IABSE Symp., Strengthening of Bld Structures – Diagnosis and Therapy*, Venice, 1983.

[18] T.P. Tassios, Masonry, Infill and RC walls under cyclic actions, CIB Symp., Wall Structures, Warsaw, 1984.

[19] E. Vintzeleou, and T. Tassios, "Seismic behavior and design of infilled RC frames", *J. Eur. Earthquack Eng.*, vol. 2, 1989.

[20] M.N. Fardis, Invited Lecture: Lessons learnt in past earthquakes,10ECEE, Vienna, 1994.

[21] P.G. Carydis, H. Mouzakis, J. Taflambas, and E. Vougioukas, *Response of infilled frames with brick walls to EQ motions, 10 WCEE*, Madrid, 1992.

[22] F.M. Mazzolani, G. Corte, L. Fiorino, and E. Barrechia, *Full-scale cyclic tests of a real masonry-infilled RC bldg for seismic upgrading,* COST Workshop, Prague, 2007.

[23] J.W. Axley, and V.V. Bertero, "Infill panels: Their influence on seismic response of bldgs" Univ. of Cal./Berkeley, EERC Rep.79-28, 1979.

[24] CEB, "RC frames under EQ loading, state-of-the-art report" Th. Telford/London, 1996.

[25] CEB, Assessment and redesign of concrete structures, Bull d' Info. N° 162, Lausanne, 1983.

[26] FEMA Publ. N° 273, 1997, NEHRP Guidelines for the seismic rehabilitation of bldgs (as well as the following Publ's N° 274 and 276/1997, 306 to 308/1998 and 310/1998).

[27] FEMA Publ. N° 356, Prestandard and Commentary for the seismic rehabilitation of bldgs 2000.

[28] T.Tassios, E. Vintzeleou, M. Chronopoulos Euro-Code 8 (EC 8), pre-Draft, Justification Note N° 9, RC frames filled by masonry walls 1988.

[29] Euro-Code 8 (EC 8)/EN 1998, Design of structures for EQ resistance, Part 1: General rules, seismic actions and rules for bldgs, 2004, Part 3: Assessment and retrofitting of bldgs, 2005.

[30] New Greek Code (nGCI) on (Structural Assessment,) Interventions (and Redesign) of existing RC structures, 2012 (In greek), see also [75].

[31] C. Dymiotis, A. Kappos, and M. Chryssanthopoulos, "Seismic reliability of RC frames with uncertain drift and member capacity", *ASCE J. Struct. Eng.*, vol. 125, no. 9, p. 10, 1999.

[32] C. Dymiotis, A. Kappos, and M. Chryssanthopoulos, "Seismic reliability of masonry-infilled RC frames", *ASCE J. Struct. Eng.*, vol. 127 no. 3, p. 10, 2001.

[33] C. Dymiotis, "*Probabilistic Seismic Assessment of RC Buildings with and W/O Masonry Infills*", Ph. D. Thesis, University of London, London 2000.

[34] D.V. Mallick, and R.P. Garg, "Effect of openings on the lateral stiffness of infilled frames", *Proc. Inst. Civ. Eng.*, vol. 49, p. 7371, 1971.

[35] T.C. Liauw, "An approximate method of analysis for infilled frames with or w/o openings", *Build. Sci.*, vol. 7, pp. 90004-90007, 1972.

[36] T.C. Liauw, and S.W. Lee, "*On the behavior and the analysis of multi-storey infilled frames subject to lateral loads*", *Proc. Inst. Civil Eng.*, vol. 63, p. 8052, 1977.

[37] V. Thiruvengadam, *On the natural frequencies of infilled frames, Earthquake Engeering and Structural Dynamics*, *13*, J. Wiley and Sons: Sussex, 1985.

[38] R. Žarnič, *Modelling of response of masonry infilled frames*, 10 ECEE, Vienna, 1994.

[39] V. Kodur, M. Erki, and J. Quenneville, "Seismic design and analysis of masonry-infilled RC frames", *Can. J. Civil Eng.*, vol. 22, pp. 576-587, 1995.

[40] G. Al-Chaar, "Evaluating strength and stiffness of URM infilled RC structures" US Army Corps of Engs, Rep. ERDC/CERL TR-02-1, 2002.

[41] G. Al-Chaar, "Design of FRP materials for seismic rehabilitation of infilled RC structures" US Army Corps of Engs, Rep. ERDC/CERL TR-02-33, 2002.

[42] P.A. Teeuwen, "Experimental and numerical investigation into the composite behavior of steel frames and precast concrete infill panels with window openings", *Steel Compos. Struct.*, vol. 10, no. 1, 2010.

[43] M.P. Chronopoulos, Infills according to the provisions of the EC 8 and the new Greek Code on (Structural) Interventions (in greek), Available at http://www.episkeves.civil.upatras.gr, 2010 and 2011.

[44] T. Tassios, and M. Chronopoulos, "Additions and interventions on small bldgs and masonry structures" (in greek), Ministry of Public Works, Research Reports, 1986.

[45] M.P. Chronopoulos, The influence of infills on RC structures (in greek), Ministry of Public Works, Committee for Building Collapses during Athens Earthquake (of 1999), 2001 and 2004.

[46] T. Paulay, and M.J.N. *Priestley, Seismic design of RC and masonry bldgs*, J. Wiley and Sons: NY, 1992.

[47] R. Angel, D. Abrams, D. Shapiro, J. Uzarski, and M. Webster, "Behavior of RC frames with masonry infills", Univ. of Il. Res. Rep. UILU-ENG-94-2005, 1994.

[48] M.N. Fardis, *Seismic design, assessment and retrofitting of RC structures (based on EC 8),* Springer: Dordrecht, 2009.

[49] G. Magenes, and S. Pampanin, Seismic response of gravity-load design frames with masonry infills, 13 WCEE, Vancouver, 2004.

[50] F. Pires, A. Campos-Costa, and S. Raposo, *Hysteretic behavior of RC frames infilled with brick masonry walls*, 10 ECEE, Vienna, 1994.

[51] P.G. Asteris, *A method for modelling of infilled frames (method of contact points*), 11 WCEE, Acapulco, 1996.

[52] P.G. Asteris, "Lateral stiffness of brick masonry infilled plane frames", *ASCE J. Struct. Eng.*, vol. 129, no. 8, pp. 1071-1079, 2003.

[53] P.G. Asteris, "FE micro-modelling of infilled frames", *Elect. J. Struct. Eng.*, vol. 8, pp.1-11, 2008.

[54] A.B. Mehrabi, P.B. Shing, and M.P.Schuller, "Performance of masonry-infilled RC frames unter in-plane lateral load" Univ. of Col., Res. Rep. CD/SR – 94/6, 1994.

[55] A.B. Mehrabi, and P.B. Shing, "FE modeling of masonry-infilled RC frames", *ASCE J. Struct. Eng.*, vol. 123, p. 604, 1997.

[56] A.N. Stavridis, and P.B. Shing, "FE modeling of non-linear behavior of masonry-infilled RC frames", *ASCE J. Struct. Eng.*, vol. 136, pp. 285, 2010.

[57] G. Manos, M. Triamatakis, and B. Yasin, *Experimental and numerical simulation of the influence of masonry infills on the seismic response of RC framed structures*, 10 ECEE, Vienna, 1994.

[58] I.N. Doudoumis, E.N. Mitsopoulou, and G.N. Nikolaidis, *A macro-element for the simulation of the infill panels in multi-storey frames under horizontal seismic actions*, 10 ECEE, Vienna, 1994.

[59] E. Smyrou, C. Blandon, S. Antoniou, R. Pinho, and H. Crowley, *Implementation and verification of a masonry panel model for non-linear pseudo-dynamic analysis of infilled RC frames, 1 Eur. Conf. EQ Engrg and Seismology*, Geneva, 2006.

[60] C.Z. Chryssostomou, P. Gergely, and J. Abel, *Non-linear seismic response of infilled steel frames, 10 WCEE*, Madrid, 1992.

[61] A.J. Kappos, "DRAIN – 2D/90: Program for the inelastic analysis of plane structures subjected to seismic input" User's Manual, ESEE Rep. No. 96/6, Imp. College, London, 1996.

[62] B.S. Smith, "Methods for predicting the lateral stiffness and strength of multi-storey infilled frames", *Build. Sci.*, vol. 2, pp. 247-257, 1967.

[63] B.S. Smith, and C. Carter, "A method of analysis for infilled frames", *Proc. Inst. Civil Eng*, vol. 44, p. 7218, 1969.

[64] T.C. Liauw, "Elastic behavior of infilled frames", *Proc. Inst. Civil Eng.*, vol. 46, p. 7285, 1970.

[65] E. Bazan, and R. Meli, *Seismic analysis of structures with masonry infills, 7 WCEE*, Istanbul, 1980.

[66] D.J. Kakaletsis, and C.G. Karayannis, "Influence of masonry strength and openings on infilled RC frames under cyclic loading", *J. Earthquake Eng.*, vol. 12, no. 2, pp. 197-221, 2008.

[67] T.C. Liauw, and K.H. Kwan, "Non-linear analysis of multi-storey infilled frames", *Proc. Inst. Civ. Eng.*, vol. 73, p. 8577, 1982.

[68] T.C. Liauw, and K.H. Kwan, "Plastic theory of non-integral infilled frames", *Proc. Inst. Civ. Eng.*, vol. 75, p. 8635, 1983.

[69] T.C. Liauw, and K.H. Kwan, "Plastic theory of integral infilled frames", *Proc. Inst. Civ. Eng.*, vol. 75, p. 8718, 1983.

[70] T.B. Panagiotakos, and M.N. Fardis, *Seismic response of infilled RC frame structures, 11 WCEE*, Acapulco, 1996.

[71] M.N. Fardis, and T.B. Panagiotakos, "Seismic design and response of bare and masonry infilled RC frames", *J. Earthquake Eng.*, vol. 1, no. 3, pp. 475-503, 1997.

[72] C.N. Michailidis, K.C. Stylianidis, and A.J. Kappos, *Analytical modelling of masonry infilled RC frames subjected to seismic loading, 10 ECEE*, Vienna, 1994.

[73] A.J. Kappos, K.C. Stylianidis, and C.N. Michailidis, "Analytical models for brick masonry infilled RC frames under lateral loading", *J. Earthquake Eng.*, vol. 2, no. 1, pp. 59-87, 1998.

[74] C.A. Syrmakezis, and P.G. Asteris, "Masonry failure criterion under biaxial stress state", *ASCE J. Mater. Civ. Eng.*, vol. 13, no. 1, pp. 58-64, 2001.

[75] New Greek Code (nGCI), Justification Notes (JNs) and Studies/Designs regarding the operationality and the applicability of the Code (in greek), 2004 to 2008.

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