13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 880

ON THE VULNERABILITY ASSESSMENT OF MODERN LOW TECHNOLOGY ENGINEERED RESDIENTIAL CONSTRUCTION

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SUMMARY

In the past five years, modern low technology engineered residential construction, typically represented by medium height reinforced concrete infilled frames in developing countries, have been responsible for very high death tolls during seismic events. It is therefore apparent that there is a pressing need to evaluate and improve the seismic performance of these buildings in order to reduce society's existing earthquake risk. However, any mitigation measure should be arrived at without adopting methods which are either too simplistic, thus running the risk of being overtly conservative, nor too taxing on the information required and therefore difficult to implement. Thus a major concern is how to estimate the realistic performance of these structures and consequently be able to determine any possible weaknesses. From data collected following post earthquake surveys a database of typical structures and damage patterns has been identified. It is apparent that current procedures for seismic assessment do not adequately account for the majority of the constructions observed. The need therefore exists for a tool that closely predicts the damage mechanisms to be expected in such buildings, hence the concepts of a tool being developed for the analysis of such structures is presented.

INTRODUCTION

Problem Background

The collapse of numerous low engineered masonry infilled reinforced concrete frame (LE-MIRCF) buildings, designed to resist only gravity loading or nominal seismic forces have been the cause of widespread loss of life in many recent earthquakes throughout the world. In particular, the events of Turkey 1999, where thousands of reinforced concrete frame buildings collapsed [1] and the Bhuj earthquake in India 2001, where more than 13,800 people lost their lives [2], stand out. Most recently, the substantial death toll in the Algerian earthquake of 2003 [3] and the Morocco 2004 event has revealed the global scale of the problem.

Rapid economic growth in many developing countries has given rise to a large city population in need of housing. This has resulted in a real estate boom, mostly driven by property speculators keen to make profit

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by providing the cheapest possible means of accommodating large numbers of people. Reconnaissance literature evidences that such social and economic changes together with increasing globalization has given rise to the abandonment of traditional building techniques and their replacement, in most parts of the world, by concrete framed construction which is fast and economical. A largely privately constructed building stock that has not been adequately designed to resist earthquake forces is the result. Furthermore, a substantial portion of the existing building stock in Europe has been developed prior to the introduction of second generation seismic codes and employs a similar form of construction. Although these structures will slowly be replaced by more reliable construction, they will remain the single greatest source of existing seismic risk for the foreseeable future. In 1999, even a low magnitude event such as the one to hit the northeast of Athens, Greece, pointed out the inherent vulnerability of a large part of the existing LE-MIRCF buildings in Europe, which were mainly built during the period from the 1950's to the 1980's, and lacking any provision for ductility [4]. Similarly the Bingol, Turkey 2003 event is an example of the effect of LE-MIRCF constructions on the vulnerability of a developing town, where in the 30 years between the last event to affect the town, local traditional constructions have been replaced by LE-MIRCF buildings, with the result that the majority of the casualties was caused by the collapse of the latter as against the former [5].

The problem facing society at large is therefore to quantify the risk posed by these constructions, which can only be done by first quantifying the available seismic resistance of typical buildings in an accurate and un-conservative manner. Though it is instructive to understand the behavior of LE-MIRCF buildings through experimental testing, it is clear that given the testing limitations [6], it is not possible to fully replicate and test enough building typologies to draw any specific conclusions as to the vulnerability of the existing building stock. Indeed the very variability of such constructions in geometrical configuration, materials employed and constructional standards adopted, preclude the possibility of embracing gross generalizations which might lead to un-conservative and un-economic predictions of future losses. On the other hand analytical seismic vulnerability assessment can be used as a first and fundamental tool in any repair and strengthening procedure, where decisions on the latter are based on the outcome of the former.

Building Typology

Post earthquake reconnaissance literature identifies such residential construction as being typically five storeys high, though heights up to eight storeys are not uncommon. They house residential, office and commercial facilities, often in the same building, with residential units forming stiff box-like rigid storey structures, whilst commercial space or vehicle parking is often provided at ground floor level, resulting in relatively large open spaces without infill partitions and occasionally with a higher storey height than for the residential upper floors **(Fig. 1)**. Typical conditions reported [7] indicate that up to four apartments per storey can be accommodated, thus implying the potential for a high casualty rate with the loss of a single building. In town centers most buildings adjacent to each other do not have any gap separating them, however in the suburbs they tend to be detached from one another.

The structural framework is either a series of two dimensional frames or a three dimensional frame, with a monolithic cast in-situ reinforced concrete slab. Buildings having an irregular three-dimensional frame grid owing to complex functional requirements are prevalent, whilst sometimes infills of partial heights create short captive columns mainly around openings and ventilators. The size and orientation of the columns are determined by architectural considerations, such as location of infill walls, and are therefore generally arranged haphazardly in plan with irregular spacing, and having widths matching that of the infill masonry thickness. Very often a large percentage of the columns are oriented with their greater cross sectional dimension, and hence stiffness in one direction, thus creating weaker frames and making buildings much weaker in one lateral direction. Beams sometimes frame into the columns eccentrically and where internal masonry partitions do not intersect the frame, underlying beams spanning in between main beams are provided for their support. Reinforced concrete shear walls are sometimes encountered around lift shafts when present, however the staircases around lift cores usually prevents good connection between the lift shafts and the floor slabs.

Fig. 1 Typical masonry infilled reinforced concrete apartment housing. Left - Turkey, Center - Greece, Right - India. [7]

Once the frame is partially or totally complete, the infill partition walls are mortared against the narrow side of the column, between the cast in-situ frame without any positive connection to the latter. Non-load bearing masonry infills offer a simple means of providing external cladding and internal partitions to a frame building. This system represents a cheap, easy and quick method of construction, built with low-tech procedures and relatively unskilled labor, affording good weather, thermal and acoustic insulation. Room partitions are built from one leaf of hollow brick masonry whilst external walls may be constructed in two leaves, separated by an insulation layer without any structural connection between them. Foundations are usually isolated reinforced concrete pad footings either connected or unconnected by relatively lightly reinforced shallow ground beams.

Materials Used in Construction

Various reconnaissance reports [1, 2, 5] and interviews with local practitioners such as those undertaken in Bingol [5], reveal that frequently in-situ concrete is batch mixed on site. Aggregate and sand are not washed or sieved and any water source which is at hand is used in the mix. Such methods result in mixing by volume and not by weight and therefore no account for moisture content is made. The resulting concrete is generally of poor quality, with a weak compressive strength. Additionally, it is poorly graded, compaction on site being inadequate, having high water content and aggregates over 30mm in size. Segregation and honeycombing are common, whilst concrete cover to the reinforcement varies widely, though generally less than 25mm. All reinforcement is generally smooth mild steel. Steel reinforcement ratios and details are usually only adequate for gravity loading considerations, and include 90 degree hooks at longitudinal bar ends and lap slice locations not suitable for laterally loaded frames. Volumetric ratio of transverse steel is often less than 0.3% and therefore does not provide the necessary tri-axial state of stress for the concrete core. The masonry infills vary in form and material ranging from hollow or solid clay bricks, cement and concrete blocks, and hewn stones amongst others, generally laid in a cement mortar.

Structural Behavior Under Seismic Load

As most of these buildings were constructed relatively recently many should have been designed to conform to a seismic building code. However, post earthquake reconnaissance reports continually indicate that whilst some buildings perform adequately during an event, other similar constructions close by suffer irreversible damage even in relatively low magnitude events. In many cases this poor performance is related to the distribution of infill layout both in plan and elevation. Clearly, unless adequately separated from the frame, the structural interaction of the frame and infill panels plays a fundamental role on the overall seismic response of the building and its individual members. In fact, numerous examples of earthquake damage can be traced to the modification of the structural response of the basic frame by the coupling with the non-structural masonry partitions and infill panels. This is because even if they are relatively weak, masonry infills can drastically alter the intended structural response, attracting forces to parts of the structure that have not been designed to resist them. Recent earthquakes have merely highlighted the structural deficiencies of this building stock with respect to seismic loads.

Inherent weaknesses in such structures usually include the presence of weak storeys triggered by an uneven distribution of the infills in elevation which result in an irregular stiffness distribution. Also, significant unexpected torsional response of the structure may originate from a non-uniform plan distribution of infill walls. Another common source of failure is at the tops of captive columns where the masonry infill does not extend over the full height of the storey. The short unsupported length of column above the infill induces a high ratio of shear force to bending moment. These features are all directly related to the interaction between the frame and the infills and lead to brittle modes of failure. Nonetheless when coupled with regular layouts, the presence of non-engineered infill walls is often positive [8], mainly due to the large amount of extra lateral capacity which can be offered by the infills, even at large imposed deformations, as well as to the increased stiffness of the infilled structure leading to smaller displacements during the earthquake. Thus it can be argued that infills, absorbing and dissipating large quantities of seismic energy, act as a first line of seismic defense in a building.

In spite of the aforementioned, many seismic building codes still do not account for frame infill interaction in design and hence this leads to a general lack of appreciation of their seismic behavior. Furthermore the lack of suitable analytical tools exasperates the situation. The general poor quality of materials and construction details adopted, only leads to the further endangerment of life. Therefore as Fardis et al [9] stated, although there is still no consensus in the international community regarding the implications of masonry infills on the earthquake resistance and seismic safety of reinforced concrete buildings, there is wide agreement that negative effects are often associated with irregularities in the distribution of the infills in plan or elevation, however, if the infilling is uniform in all storeys, drifts and structural damage are dramatically reduced.

ANALYTICAL ASSESSMENT OF REINFORCED CONCRETE INFILLED STRUCTURES

Current Assessment of Reinforced Concrete Infilled Structures

Due to the vast scope of assessment procedures and the many uncertainties involved, generalization of rules for the assessment of building structures is rather difficult. Therefore, guidance documents are more common and provide general criteria and methodologies, as opposed to application rules. However, with the appearance of documents such as FEMA-356 [10], performance based assessment techniques have now gained general acceptance. Indeed contemporary assessment methods tend to favor displacement based approaches, wherein the deformation capacity of the structure and of the individual members needs to be determined for the given dimensions, reinforcement and material properties.

Thus the availability of recent documents dealing specifically with assessment and the popularity of non-linear approaches to evaluating the behavior of structures, provide the capability to identify and quantify the risks to many existing structures such as many bare reinforced concrete frames. Nonetheless, closed form approaches and current analytical techniques inevitably cater for regular constructions only. Yet as evidenced from numerous post–earthquake reports the gross majority of the building structures pertaining to the LE-MIRCF building classification are of irregular layout, both in their framing disposition and layout of non-structural elements. Hence, there appears to be a knowledge gap when determining the deformational capacity of many of these buildings. Indeed as D'Ayala and Charleson [11] stated "This is an area that will benefit from further research, and certainly will need full inclusion in any evaluation or strengthening and repair codified document." Thus the need to confidently assess such irregular structural systems in their inelastic range to withstand seismic loads is called for.

The selection of an assessment approach depends on its purpose, however if the intention is to clearly identify weaknesses and hence the necessity or other of an appropriate retrofit scheme for a specific building, the use of at least some form of simplified analysis becomes almost obligatory. Practically all methods of analysis could and have been used for assessment purposes and all else being equal, the level of sophistication involved in the modeling and subsequent analysis is a measure of the reliability of the results. Code-type analysis has been used to provide a rough indication of weak components of the structure, where damage is expected to concentrate. Such methods are based on a force based approach whilst employing predefined code criteria and hence have the disadvantage of being regional. On the other hand inelastic static analysis has known significant popularity in recent years, and is now one of the leading techniques being applied to assessment. In fact many contemporary seismic design guidelines encourage the use of push-over analysis, including ATC-40 (1996) [12], FEMA 356 (2000) [10], New Zealand (1996) [13], Japan AIJ1990 (1990) [14]. However, the most comprehensive method would be the use of inelastic dynamic analysis for a variety of recorded earthquake motions, even though as Elnashai (2002) [15] states, the "necessity domain" of nonlinear dynamic analysis as against static inelastic analysis is ever decreasing.

The Necessity for a 3D Modeling Approach

Whereas in design a large amount of unknowns are actually catered for in any final scheme, doing so in assessment might lead to unpredictable consequences leading either to conservative or un-conservative estimates of the response. In the case of the former an economic cost which might reduce a scheme to the improbable might result whilst for the latter undesirable level of damages could occur. Such a fundamental difference between design and assessment invariably requires the use of different analytical techniques. Thus while 2D analysis in either orthogonal direction with subsequent action effect superposition provides adequate results for the design of many structures, the extension of such a procedure to assessment of LE-MIRCF buildings invariably leads to approximations and assumptions which might defeat the whole purpose, especially when one considers the delicate socio-economic implications in seismic rehabilitation.

Thus, as a first and fundamental step the irregularity inherent in these constructions can only be reliably reproduced by complete 3D numerical models, especially so when one considers the intrinsic spatial nature of the earthquake loading itself. Such modeling clearly needs to account not only for the idealization of the reinforced concrete members but also for the modeling of the diaphragm effect due to the slabs and the representation of the infill masonry. Following this, decisions on the direction of earthquake loading can be undertaken with the respective capacities of the structure in a few main orientations being calculated, thus implying the ignorance associated with the determination or other of the possible angle that the earthquake hits the structure. Ideally complete dynamic time-history analysis would then be performed for all possible combinations, but clearly for the time being the mere post-processing of the results would prove an insurmountable challenge.

In spite of this, the literature presents various methodologies in order to predict the seismic response of 3D irregular constructions by simpler methods other than using complete time history analysis. However, they are invariably developed on the basis of single storey structures with results subsequently extended to multistorey buildings. Clearly important simplifying assumptions are relied upon and as appealing as they appear, it is apparent that any simplification of the complete dynamic time history analysis can never achieve a complete description of the response when the very nature of the load itself remains unknown up to the time of the event. What is required then in a seismic assessment procedure is to determine the displacement capacity of the structure, thus harmoniously integrating with a performance based assessment philosophy. A technique that has the makings of such a procedure is to adopt the 2D pushover analysis and apply it to the 3D configuration.

EXPLORATORY ASSESMENT STUDY OF LE-MIRCF BUILDINGS

Objectives

In order to explore the possibilities, strengths and limitations afforded by current analytical environments a pilot study was conducted through a three dimensional simulation of a regular, gravity only designed non ductile reinforced concrete frame building with varying masonry infill panel distribution. The three different configurations analyzed included a bare frame, a regular infilled frame and a pilotis infilled frame, which represents irregularity through the omission of the ground floor infill masonry panels. Therefore, a prototype construction was adopted, with dimensions similar to those reported in post-earthquake reconnaissance reports [1,2,5]. However, it was important to adopt a highly regular framework the geometry of which is shown in **Fig. 3,** in order to be able to extrapolate any outcome. The structure was then designed to satisfy EUROCODE 2 [16] requirements to resist vertical loads only. Even though most of the constructions under consideration would probably have been designed to older codes, this does not mean that the structure adopted is not representative of these constructions. Indeed, many of these buildings have dubious origins and hence the basic model is but a lower bound minimum requirement as regards member dimensions and steel quantities. All columns were dimensioned as 300x300mm with different steel percentages for the outer and inner columns but retaining the same details throughout the building height. Main vertical load bearing beams were 500x250mm whilst transverse beams were 500x200mm with a different percentage of reinforcement.

Simulation Approaches

Finite element formulations that allow modeling at the material level using a point by point basis, with different elements used for the concrete, reinforcing steel and possibly for their interaction through bond would definitely be the ideal solution. Such modeling allows representation of even minor details of the geometry of the members, and allows the history of stresses and strains at every point of the structure to be followed. However, the computational requirements of such an approach restrict its application to the analysis of the response of individual members or sub-assemblages and therefore to date, the application of such detailed modeling for entire structures under non-linear dynamic effects has not been widely employed. However, given the hardware possibilities available nowadays fiber models have assumed a leading role, wherein a member model is used as a founding unit and the detailed stress-strain response at a large number of points over the cross-section of the member is followed during the response analysis.

Having ruled out the practicability of using a general purpose finite element program, a suitable platform had to be found that would allow the inelastic response of structures. Many computer programs for the seismic response analysis of structures are available amongst which DRAIN [17] originally developed for 2-D frames but with later versions capable of undertaking 3-D analysis, ANSR [18] a non linear program specifically for 3-D structures, IDARC [19], RUAUMOKO [20] developed in New Zealand, OPENSEES [21] and SEISMOSTRUCT [22] to name a few. Although the merits of each program is beyond the scope of this paper, in choosing a suitable analytical engine various parameters had to be kept in mind, which included the availability of the source code, the ability to analyze 3-D structures and finally the ability to model the spread of inelasticity along the member length and across the section depth, thus allowing for accurate estimation of damage distribution in reinforced concrete members. DRAIN-3DX [23] was finally chosen as a suitable platform given its widespread use by other researchers and most importantly the possibility of adding the required infill panel element through coding, whilst the post-processing approach of SeismoStruct [22] was adopted. Nevertheless, the subsequent adoption of a suitable modeling strategy for the infills can always be applied to any open source code.

Methodology

Non-linear push-over analysis for the bare frame was performed for a variety of loading conditions as detailed below, in order to determine the effect of the direction of the seismic action. The post-elastic behavior of the frame and the probable lateral seismic force capacity was determined using an inverted triangular static load distribution comparable to first mode behavior, and justified with the 85% modal weight participation factor for this mode. For each case the sum total of the seismic action, was kept equal and the analysis was conducted using a model with 4 elements per member for one third service load. The conventional pushover scheme adopted together with the loading profile was chosen in order to allow for easier comparison between each analysis and the corresponding normalized load factor versus displacement response are shown in **Fig. 2** for the values corresponding to the centre of mass position.

(a) loading only in the x-direction, representing a 0 degree loading angle,
(b) loading only in the y-direction, representing a 90 degree loading angle,
(c) 94.8% loading in the x-direction and 31.6% in the y-direction (3:1) for a 18 degree loading angle,
(d) 89.4% loading in the x-direction and 44.7% in the y-direction (2:1) for a 26 degree loading angle,
(e) 70.7% loading in the x-direction and 70.7% in the y-direction (1:1) for a 45 degree loading angle,
(f) 44.7% loading in the x-direction and 89.4% in the y-direction (1:2) for a 64 degree loading angle,
(g) 31.6% loading in the x-direction and 94.8% in the y-direction (1:3) for a 71 degree loading angle.

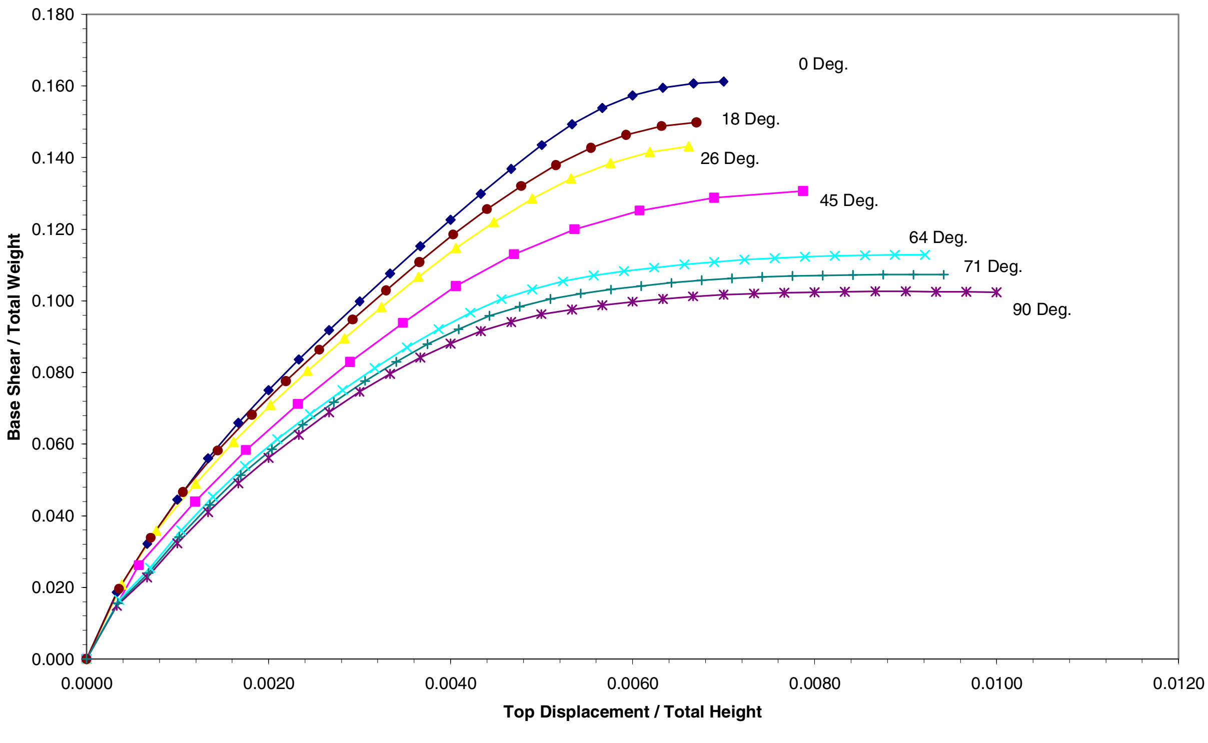

Fig. 2 Normalized capacity curves for varying load directions for the Bare Frame at the centre of mass position, with an equal load vector initially applied for all cases.

In order to determine the failure point of the structure, the deformation capacity of the individual members needs to be determined for the given dimensions, reinforcement and material properties. This is by no means an easy task, especially for LE-MIRCF constructions that suffer from non-ductile failure modes, where the amount of dependable displacements is limited not only by flexural mechanisms, but also shear mechanisms including both diagonal tension failure of the web and of the compressive struts. Furthermore, buckling of compression reinforcement also needs to be considered. However, due to the limitations of implementing such features in the tool employed, failure was assumed to occur at the onset of a concrete strain of 0.005 in the core of any column. Finally a check was also carried out in order to determine the column shear capacity as defined in EC2 [16], in order to establish whether shear failure preceded flexural failure, with all the results indicating that the columns do not fail in shear before failing in flexure.

The difference in response in either orthogonal direction is evident from the post peak behavior shown in **Fig. 2**, which is ductile for the weaker y-direction, and is striking when considering the very similar geometrical dimensions in either direction of the construction. In fact, the columns were square whilst the beams were 50mm narrower in the y-direction. Reinforcement distribution was substantially different however between the beams of the weaker direction and those of the main load-bearing beams in the x-direction. Hence, the strong beam-weak column effect in the x-direction resulted in a brittle response with a higher lateral peak capacity, whilst the weaker beams together with the weaker columns gave rise to a structural behavior which is ductile and more suitable for seismic loading. This difference in behavior between either direction stems directly from the design adopted, and a family of curves then exists in between either.

It is apparent therefore that even for a highly regular building significant differences in both strength and capacity depending on the direction of loading exist. Indeed when the direction of loading deviates from the stronger axis there is a substantial reduction in strength but not necessarily a corresponding increase in deformation capacity as evidenced from the 18 and 26 degree angle cases. Though the applicability of these preliminary results is arguable and need to be compared against results from actual recorded motions, it is evident that the simple extension of 2D design techniques to confident assessment approaches can be misleading. Whilst most Seismic Codes and Assessment guidelines require that the horizontal components of the seismic action should be considered as acting simultaneously in the two main directions by proposing a series of combination rules such as the square root of the sum of the squares (SRSS) or the 1:3 rule for concurrent action effects, few if any guidelines are encountered regarding the most probable onerous direction of loading especially when considering 3D nonlinear pushover. Earthquake loading is spatial in nature and hence any extension of the 2D pushover analysis to the 3D analysis domain of LE-MIRCF constructions, needs to account for this multi-dimensional effect in order to take advantage of the spatial model created, all the more so in lieu of the displacement based performance framework adopted.

Results

Following this initial analysis, the difference in behavior between the three chosen building configurations was studied by observing the collapse mechanisms in the main axis, with loading applied only in the x-direction. The failure criterion adopted was identical for all analysis and is as detailed previously. The lateral capacity of the bare frame structure was determined as being around 0.15W where W represents the entire weight of the structure together with a service load equal to one third the full live load and the ultimate deformation of 120mm. **Fig. 3** shows the progression of damage for the entire frame structure. First yielding of a member was calculated in the internal columns on the outer frame on the leeward side at a load level of 0.125W. Both internal columns on the leeward half of the frame also yield at this point as do the windward corner columns. The first two beams then yield at a load of 0.14W and are the internal ones located in the outer bays as shown in **Fig. 3**. Following the spreading of yielding throughout many of the members, spalling of the first members occurs in the inner columns at a load level of 0.155W. Peak load is subsequently reached at 0.157W with initial member crushing occurring at a post-peak load of 0.155W in the columns at the leeward building side, namely in all the columns on the outer frame and also the two internal columns. Failure is deemed to occur at a post peak load factor of 0.153W when crushing of the two internal columns on the leeward half occurs, as shown in **Fig. 3**.

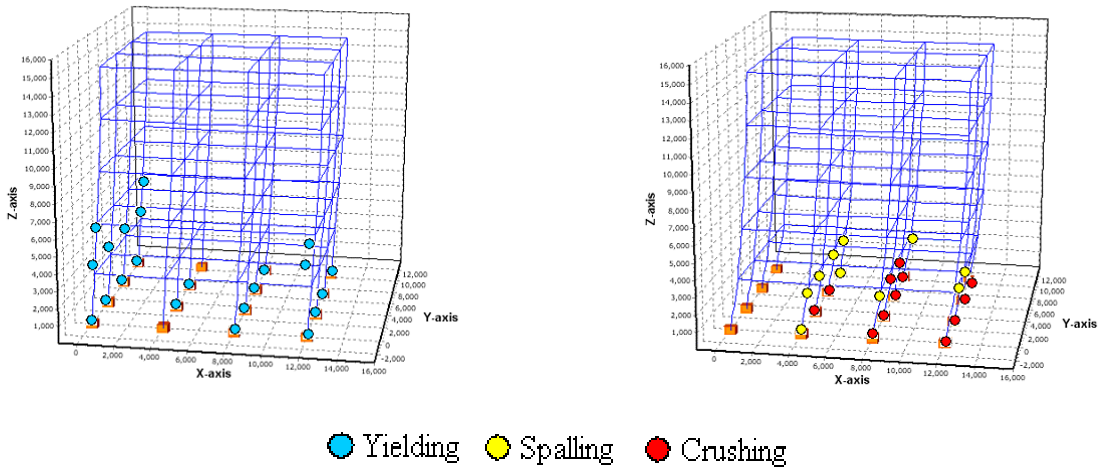

Fig. 3 (a) Frame displacement and member yielding at a load level of 0.140W.
(b) Frame displacement and locations of local member damage at a post-peak load level of 0.153W.

The second building configuration to be analyzed was a regular infilled frame in order to include the modeling of the infilled masonry. Currently, the program does not have a suitable element to model the masonry infills and therefore an inclined strut approach as suggested by Zarnic and Tomazevic [24] was adopted with an infill distribution as shown in **Fig. 4**. Naturally the amount of infills in each storey and direction determine the lateral capacity of the building so any generalities to other building plans are superfluous in this case. However, the objective was to determine the overall effect modeling of the infills would bring about. The masonry was then modeled as a weak concrete strut, allowing axial inelasticity and having a compressive strength of 4MPa with a width in plane of one third the length of the diagonal. These values are not of importance however, as the objective of this simulation was not to reproduce specific materials and conditions, and therefore in order to gauge the effect of varying the strut width, a further analysis was subsequently performed for twice the strut width. Results for the topmost floor of the load deformation response are shown in **Fig. 6**. The increase in capacity for the latter case is significant as is the increase in stiffness. This demonstrates the sensitivity of the response on the correct value of strut width and highlights the impracticality of the approach unless adequate experimental tests are at hand for the masonry material on site.

The progression of damage for the entire regular infilled frame structure is shown in **Fig. 4**. As cracking of the reinforced concrete members progresses, the infills at the centre of the building in the first storey crush at a load level of 0.16W and are soon followed by the infills at ground floor and second floor. Yielding commences in the leeward internal columns, and takes place at 0.17W, whilst the first beams to yield are located on the ground floor of the leeward side at a load level of 0.18W. Following the crushing of all the infills, including and up to the second storey, the spreading of yielding throughout the reinforced concrete members continues. The first members to spall are the internal columns on the leeward side of the structure, at a load level of 0.215W. Peak load is reached at 0.216W and member crushing initiates at a post-peak load of 0.214W, in the columns at the leeward building side. All the columns on the outer frame and also the two adjacent internal columns reach the crushing strain at the same load level. Subsequently, failure is deemed to occur at a post peak load factor of 0.211W, as shown in **Fig. 4(c)**.

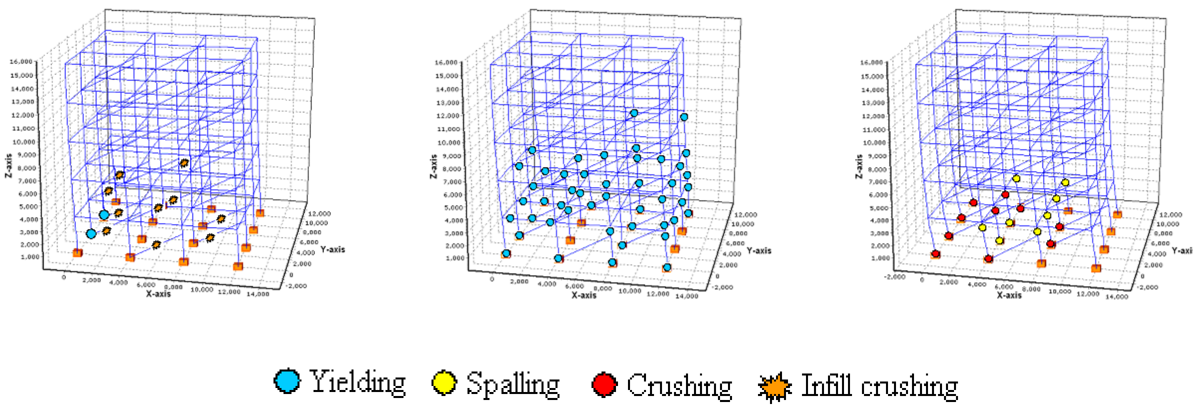

Fig. 4 Frame displacement and local member damage at (a) 0.178W. At this stage all infills at ground floor have suffered significant damage and most at first floor as well. (b) load level of 0.207W. (c) Post-peak load level of 0.211W. All the infills have been severely damaged at this point except for the ones in the upper two floors (omitted for clarity).

The third configuration analyzed was the pilotis frame, representing an irregular masonry infilled frame structure commonly observed in post earthquake field visits, as shown in **Fig. 5**. The masonry infills were modeled using the same approach as before with identical properties as for the regular infilled frame. The difference in behavior of this structure is apparent, especially in regard to the mechanisms witnessed between the other two structures. What is immediately apparent when viewing **Fig. 5**, which shows the damage experienced by the structure at failure, is the lack of yielding members above ground floor level. In fact all inelasticity is concentrated in the columns of the ground floor, with only two beams at ground floor yielding as well. Damage to the infills is also absent, except to the central infills at first floor. Such behavior was witnessed first hand in both the Molise [25] and Bingol [5] events. On the other hand, in the two previous configurations, yielding in the members had progressed in many columns throughout the height of the frame, before failure occurred, thus representative of post earthquake reconnaissance missions on regular buildings. The sudden failure of the weak ground storey for the irregular infilled frame structure is then clearly portrayed even by this preliminary analysis. The initial members to yield were the central columns on the leeward side of the structure at 0.118W and yielding then progressed to the other columns at the ground storey. The first and only infills exhibiting severe damage crushed at a load of 0.153W and were located at the first storey central bay, whilst spalling of the aforementioned columns was recorded at the same load level. Finally crushing of the columns occurred at 0.157W with failure defined at a post-peak load of 0.155W.

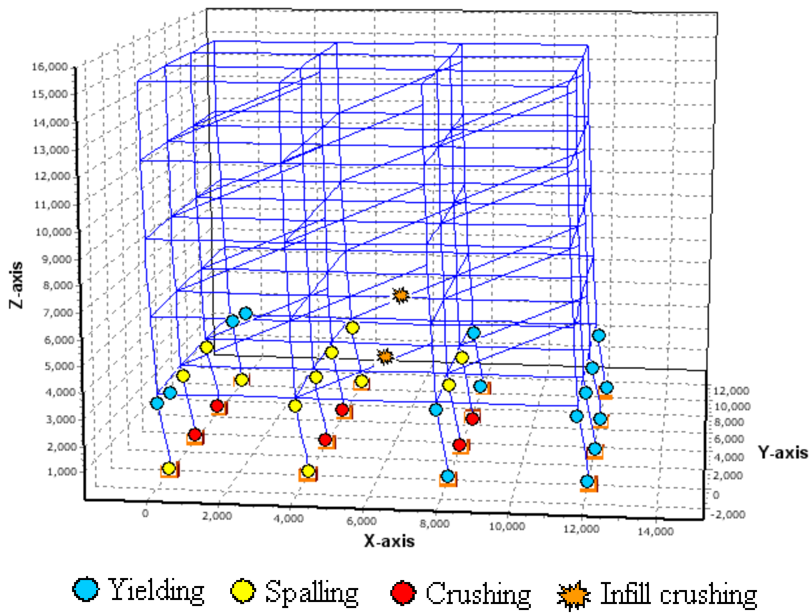

Fig. 5 Pilotis frame displacement and locations of member damage at a post-peak load level of 0.155W. Only two infills have been severely damaged at this point, and all non-linear behavior is concentrated in the columns at ground floor.

Discussion

The behavior of all three configurations to the static non-linear push over analysis is best represented in **Fig. 6**, by following the load deformation response of the topmost floor. It is apparent that the weakest configuration in terms of capacity is the pilotis structure. More damagingly and more important however, this frame also possesses the least displacement capacity, as failure was defined at circa 75mm, thus confirming the brittle behavior of such an irregular construction. On the other hand the bare frame structure, whilst possessing the same lateral capacity, reaches failure at around 120 mm, therefore possessing nearly twice the pilotis' displacement capacity. The regular infilled configuration undoubtedly has the highest peak capacity, with 50% extra strength over the bare frame. When the masonry infill strut width was doubled, the capacity increased even more. The displacement capacity of the regular infilled configuration was recorded at around 110mm, slightly less than for the bare frame. A marked increase in stiffness for both the infilled frames as against the bare frame is also apparent. Thus, the difference in behavior of these configurations is apparent even for very simple configurational differences under the most basic uni-directional applied loading. Whilst, the simple strut model is able to represent overall behavior and indicate the increased vulnerability of the irregular structure, however, it is apparent that such a simple formulation for the infills was not able to capture the shear type failures in the reinforced concrete columns so frequent in post earthquake damage surveys, especially for irregular constructions. Clearly, more powerful models are required to capture the effects witnessed in reconnaissance literature and tests on masonry infilled frames.

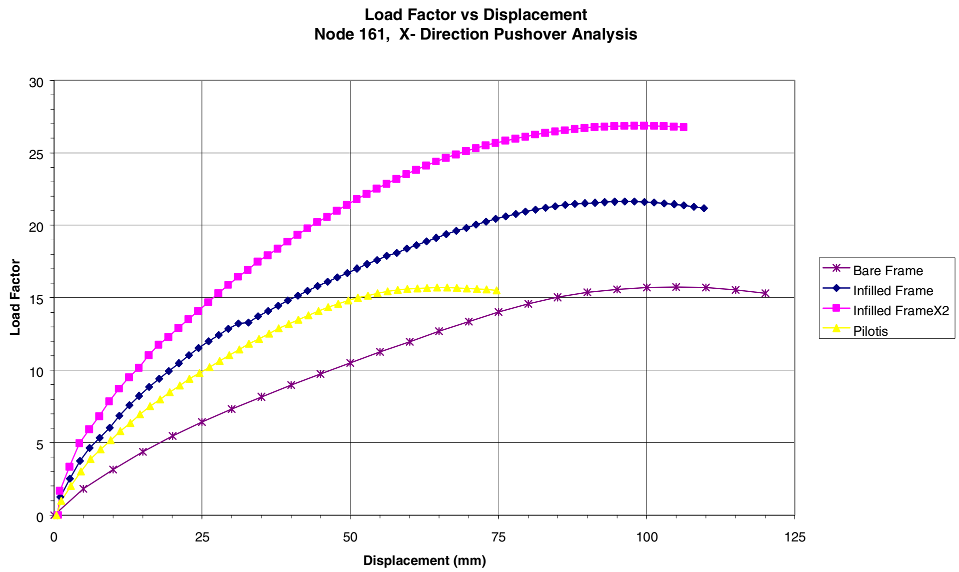

Fig. 6 Comparison of the displacement response of the different configurations, when subjected to an identical applied loading.

CONCLUSION

Post earthquake visits have highlighted common building typologies and corresponding damage suffered by these typologies. On the other hand, an environment has been identified which allows the spatial non-linear analysis of reinforced concrete structures. However, the latter lacks an intuitive and generally applicable representation for the masonry infills, whilst is unable to capture the shear failures prevalent in irregular constructions. The preliminary simulations carried out have highlighted the importance and necessity of modeling the infills in a 3D environment. Therefore, the need to implement an accurate yet efficient representation of the spatial characteristics of a construction having masonry infilled reinforced concrete frames has been established, and a tool capable of rationally accounting for the effects of the masonry infill panels needs to be developed in this context.

From the point of view of structural response of infilled frames, many aspects need to be considered. The variability of the mechanical properties of the infills and the frame-to-infill interface behavior is a major factor, be it in terms of the constituent materials and also on the construction details adopted. Furthermore, the influence of the detailing of reinforced concrete structural members, the location and dimensions of openings, the overall geometry, number of bays, number of storeys, and aspect ratio of infills, all have to be taken into account. A study is therefore underway were through the identification of various individual typologies the irregularity found in common constructions will be quantified and modeled using a modified version of the aforementioned tool, thus allowing for reliable fragility curves to be derived which will be of immediate use in seismic hazard studies, whilst also providing the environment for the assessment of single constructions. The realistic and unconservative vulnerability assessment of these structures will hopefully result.

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