THE 1ST OF MAY 2003, BINGOL, TURKEY, EARTHQUAKE, A STUDY OF THE PERFORMANCE OF THE BUILDING STOCK

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SUMMARY

The May 2003 earthquake in Bingol, Turkey, resulted in the loss of 177 people and occurred in a rapidly expanding town of 70,000 inhabitants, wherein the gross majority of the population is housed in buildings constructed using reinforced concrete infilled masonry frames of medium height. This is a common scenario in many countries where these constructions were built relatively recently, and should have been designed to conform to a seismic building code. However, they have proven to be the most highly vulnerable building type in today's large towns and cities and the cause of major loss of life. The observations from a post earthquake reconnaissance survey conducted in Bingol is presented, and the survey methodology explained. The effect of thirty years of construction of such buildings vis-à-vis the towns' vulnerability is also outlined by comparing with the damage effects from a previous event. It is concluded that the major shortcoming of many constructions is not only due to the general poor quality of materials and construction details, but the occurrence of irregularity in masonry infill distribution and layout.

INTRODUCTION

Scope

As part of ongoing research at the University of Bath into the vulnerability of low-engineered masonry infilled reinforced concrete framed buildings (LE-MIRCF), a post earthquake visit was organized to document the effects of the event that occurred on the 1st of May 2003, at Bingol, Turkey. The damage caused by events such as the Bingol earthquake, of relatively low magnitude, allows one to identify collapse mechanisms in structures before they are fully developed, thus allowing for the clear identification of strengths and weaknesses. The scope of the mission was therefore primarily to report on the performance of the LE-MIRCF buildings, by collecting enough information that would highlight the major issues involved in determining their response. Furthermore, sufficient data on individual constructions was collected in order to ensure that their subsequent numerical simulation would largely reflect the amount and distribution of damage observed.

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The Event
On the 1st of May 2003, an earthquake measuring 6.4 on the Richter magnitude scale (Mw=6.4[1]) occurred at a depth of circa 10 km, in the south Anatolian high-mountainous province of Bingol, in eastern Turkey. Many seismological centers [1,2,3,4,5] located the epicentral region at about 15 km North West of the city of Bingol (**Fig. 1**), the province's capital, 665km East of Ankara. In all, 177 fatalities were reported and around 519 people were injured [6]. However, the event will mostly be remembered for the tragedy occurring at the Celtiksuyu school were 84 people, nearly all children, lost their lives in the collapse of the dormitory block. Another 70 people died in collapses throughout Bingol town itself, whilst a further 13 perished in the neighboring villages [6].

Fig. 1 Map of Turkey and event location bounded by 100 km radius

Location
Bingol province is circa 8,125km^2 in area and is further subdivided into eight districts. It is composed of mountainous terrain with peaks over 2500 meters in elevation, connected by plateaus of tectonic origin and deep valleys. The province is rural with over half the population (130,269) living in more than 300 small villages and hamlets [7], while 123,470 live in urban centers. With around 40,000 households in the year 2000, it is one of the least populated and poorest provinces in Turkey, averaging a population density of 31 people per km^2 compared to Turkey's national average of 88.

Bingol Town and its Building Stock
Bingol town is located on a plain in the upper Murat River plateau, around 1150 meters above mean sea level, and straddles the Capakcur River flowing east-west through an entrenched alluvial valley. With a population of circa 70,000 Bingol is a small developing town with few large buildings and some light industry. It is administratively composed of 13 districts (**Fig. 2**) however it is topographically divided in two by the river Capakcur. The southern bank includes the districts of Bahcelievier, Mirzan and Yenimahalle to the west, with predominantly traditional buildings of mostly 1 storey height, possibly 2, including himis type constructions built from timber frames with adobe infills and un-reinforced masonry buildings. A few reinforced concrete infilled frame apartment buildings up to 5 storeys are springing up gradually. The districts of Inonu, Yesilyurt and Kultur, are effectively the town's centre, housing the governor's office, mayor's office, telecommunication's block and various banks, with a predominant majority of reinforced concrete framed construction. On the other hand, the south eastern area is earmarked for development in the Aydinsu district but is still relatively sparsely populated and thus has few modern buildings.
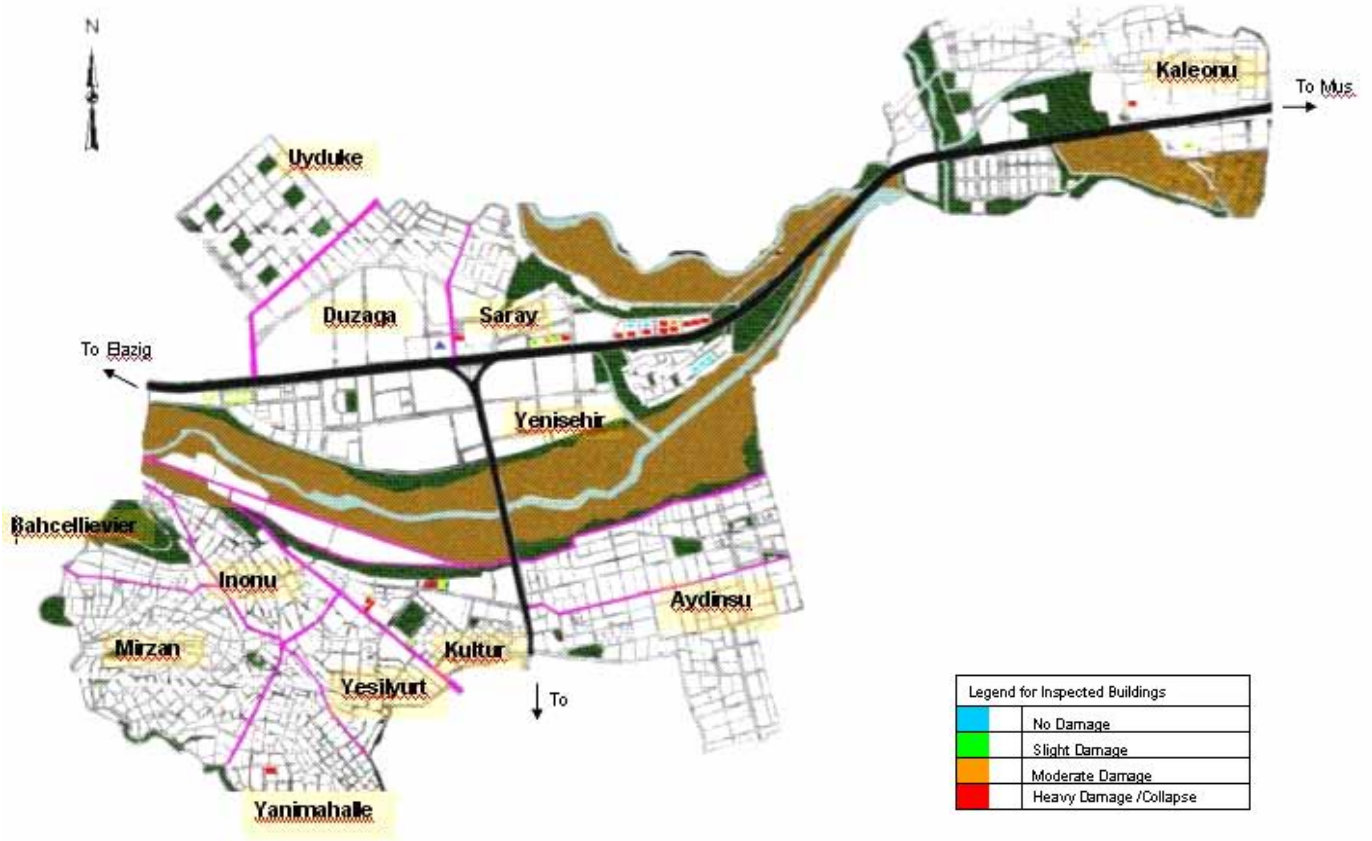
Fig. 2 Map of Bingol town

On the northern bank, the town is essentially composed of a recent settlement area which has been largely established after the 1971 earthquake, and is built upon deep alluvial deposits composed of coarse gravel to boulder size material within a stiff plastic clay matrix [8]. This part of the city includes the districts Uydukent and Duzagac to the west, and Saray and Yensihir to the east, all of which have a gross predominance of in situ cast reinforced concrete framed buildings with hollow clay brick infills, generally up to 5 storeys high. Finally the district of Kaleaonu lies to the north east of the city centre on the outskirts of Bingol and on the road to the town of Mus. The latter has a building stock of mostly 1 or 2 storey constructions, with traditional building types interspersed with newer reinforced concrete houses of 3 storey maximum height. Throughout the town, reinforced concrete is the material of choice for all new buildings, with a typical height of no more than 6 storeys, generally 5. The highest building structure is the main mosque in the town centre. No steel construction could be identified, apart from a bridge over a roadway. Traditional construction, though common is fading in the town itself. The surrounding villages and hamlets still have the himis type construction as their prevalent building type of up to 2 storeys high, however in the larger settlements reinforced concrete apartment buildings of 3 storeys or more were noted.

Fig. 3 The town of Bingol and its building stock, note the mosque in the centre
and the Capakcur river to the left.

Historical Seismicity

Various earthquakes have struck the region in the last century and a couple of documented records are also available of events in previous eras. Two historical earthquakes occurring in the vicinity of the region are the events of Kovancilar in 1789 and Karliova in 1875, both with a calculated Intensity of MMI VIII [25]. At least eight events having a magnitude above 5.0 Richter and within a radius of 100km from the current calculated epicentral location have occurred in the region during the last 50 years [10, 11]. The most notable event to have affected the town of Bingol in living memory, is that of the 22nd of May 1971, when a magnitude 6.7 event with an epicentral distance of just 17 km from the present earthquake, resulted in the death of around 878 people and injured 1,500. The building stock at the time was vastly different, and no more than 50 or so reinforced concrete buildings existed, mostly in the form of institutional buildings [12, 13]. Considerable damage was caused to the town, where a total of 1,571 buildings collapsed or were heavily damaged, 898 were moderately damaged and 535 were slightly damaged, resulting in a calculated MMI intensity of VIII-IX in Bingol town and VI, 40km from the event [14].

Building Code Requirements

The gross majority of the LE-MIRCF buildings have been built within the last 30 years, during which two revisions of the Turkish Seismic code have been issued. The 1975 [15] update which was the first to account for ductility and the current 1998 revision which is essentially based on capacity design procedures. In the current code Bingol province is all contained within the highest risk band of Zone 1 with a design earthquake identified as having a probability of exceedance of 10% in 50 years, and a corresponding PGA of 0.4g.

According to the 1975 revision, the Seismic coefficient for an ordinary reinforced concrete building with a Structural coefficient K of 1.0, therefore for non-ductile moment resisting frames with unreinforced masonry partition walls, gives a coefficient of 0.15g. It is to be noted that this value implicitly associates a factor of circa 2.5 as a reduction factor of the spectral amplification due to plastic behavior and the inherent dissipation capability of reinforced concrete frames [16]. On the other hand, applying the 1998 revision [17], and using the lowest available ductility factor possible in the code, i.e. of 4, the seismic

coefficient is 0.25g, resulting in a 60% increase when compared to the previous code revision. However, this is associated with particular provisions and construction details which are more stringent than rules from the earlier code and ensure ductile behavior.

In order to compare possible demand and supply, the non-linear spectrum for the N-S component of the only available record is compared in **Figure 4** with the design seismic spectra as defined in both codes. Both Code spectra are shown with their respective minimum reduction factors as detailed above. The record spectra have been calculated with differing ductility values ranging from 2 to 4, derived by using an elasto-plastic model with kinematic strain hardening, having a strain hardening parameter of 0.05, together with a damping value of 5%. As the soil condition underlying the accelerogram is a medium to stiff dense sand and gravel, it pertains to a local site class 2 or 3, as defined by the 1998 code, therefore the design spectra are shown bounded at each end by either extreme of the possible underlying soil class.

The majority of reinforced concrete buildings up to 5 storeys and with lightweight infill frames have a fundamental natural period of under 0.7s. Clearly therefore they would have been subjected to forces much higher than those anticipated in the 1975 code. As regards the 1998 revision, the spectrum envelopes buildings having a fundamental period of over 0.7s, therefore the five storey LE-MIRCF constructions would still have been subjected to higher forces than designed. It is important to note however that local site effects might have played a part in the recorded motion due to its location as explained in [18]. Nevertheless, it appears that even for a relatively moderate event, a significant part of the building stock in the town was subjected to forces higher than those prescribed by the codes. Thus emphasizing the inherent vulnerability of such constructions without ductile detailing, which rely mostly on over-strength to resist additional seismic forces to those specified in the codes.
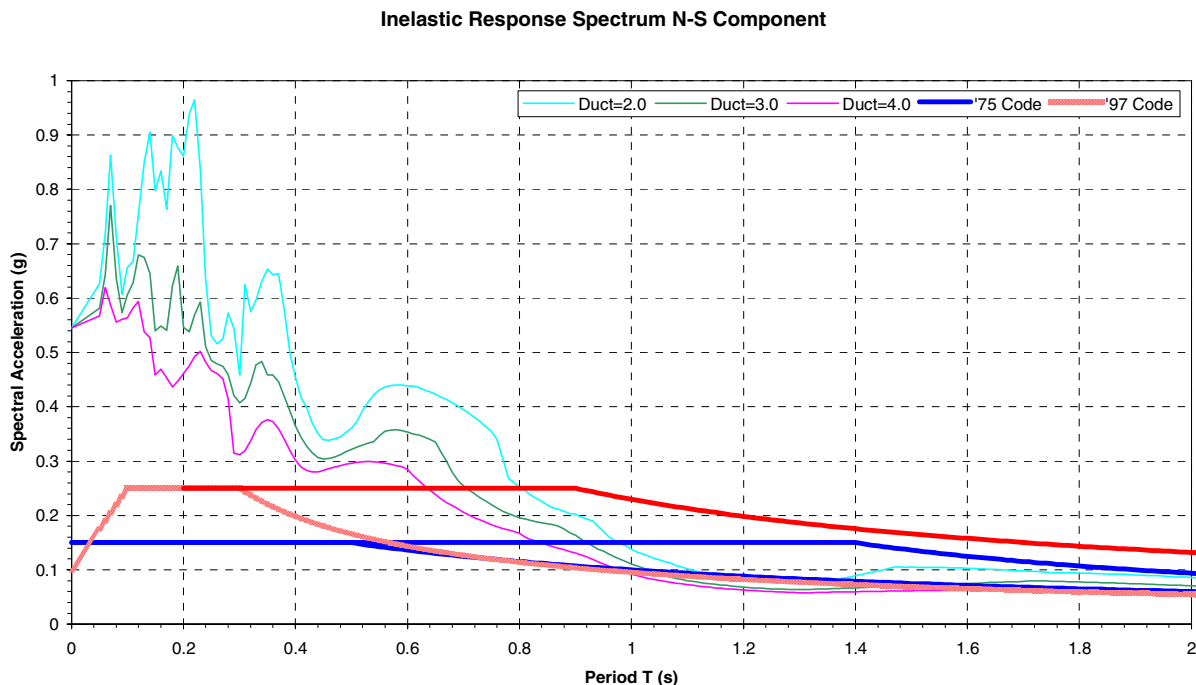

Inelastic Response Spectrum N-S Component

Fig. 4 Comparison between inelastic spectra of N-S component of strong motion record and design spectra of the two Turkish Seismic codes (5% damping)

DAMAGE ASSESSMENT

Building statistics from the Year 2000 Census [7], show that there were a total of 17,209 building units in Bingol town, of which the gross majority, 14,348 were classed as being residential, 763 as mostly residential and 1,481 as commercial establishments, together with various other building classes each with no more than 150 units. It is the norm in Turkey to report the number of construction damage on the basis of the number of dwellings or individual building units, and not the number of separate buildings, hence the three tier subdivision. Furthermore the figures indicate that in Bingol, less than 20 reinforced concrete buildings actually collapsed, or 0.4% of the city's total building stock, 11% of all buildings suffered heavy damage, 9% suffered moderate damage, 30% had slight damage, and the remaining 49% were undamaged (**Fig 5**). Separate damage statistics for both residential and commercial premises are also shown, wherein the higher vulnerability of commercial units, most of which are located at ground floor, is clear, as 45% suffered moderate damage and above, whereas only 31% of residential units suffered a similar damage level. This is just but a first indication of the increased vulnerability of the weak ground storey configuration which mostly house commercial premises.
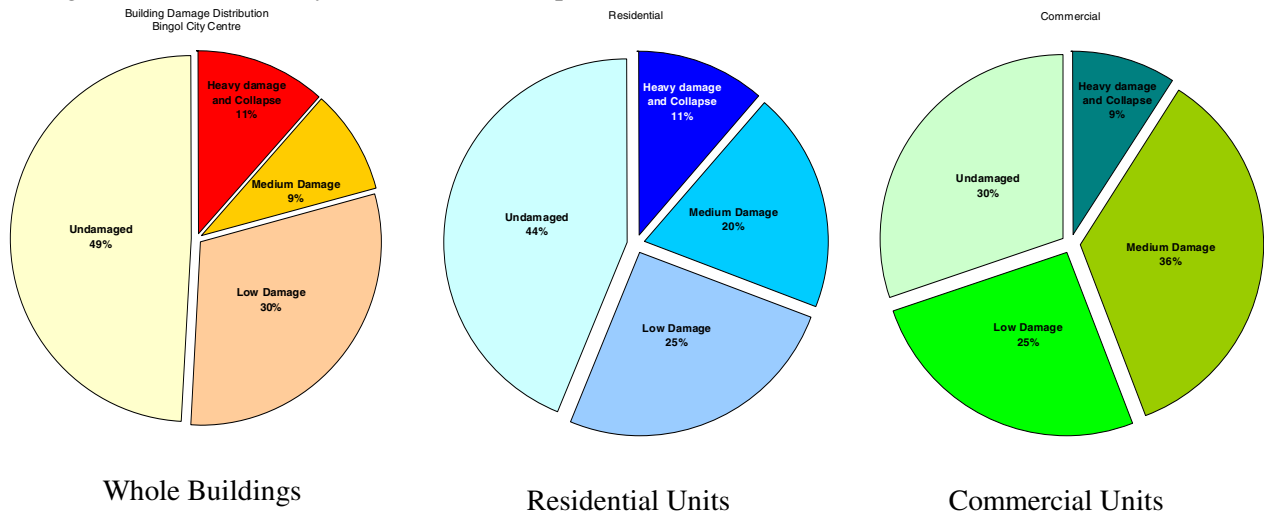

Whole Buildings   Residential Units   Commercial Units

Fig. 5 Damage Assessment Data [6]

The dire performance of institutional buildings is observed when the above figures are compared with the data available for damage to the schools which are all based on a standard design, where no less than 48% of all schools suffered moderate or worse damage levels. As a matter of fact, out of a total number of 27 schools in the area, 4 completely collapsed or were heavily damaged, 9 were moderately damaged, 11 were slightly damaged and only 3 were undamaged [6]. This was also reflected in other government buildings which also suffered severe damage, such as police stations and hospitals, which are also generally based on template designs.

Effect of Thirty Years of Development
The effect of 30 years development on the town's vulnerability can be briefly gauged by comparing the effects of the earthquake that struck Bingol town in 1971 to the damage caused in the latest event. The somewhat larger preceding event caused a total of 294 deaths in the town itself, with the remaining fatalities being caused in the surrounding villages. During this time it is reported that there were about 46 reinforced concrete framed buildings at Bingol town centre [13]. The figures also indicate that 12 (26%) of these constructions suffered heavy damage or collapse, another 12 suffered moderate damage, 20 (43%)

suffered slight damage and 2 (4%) did not exhibit any damage, implying that LE-MIRCF buildings have not undergone any substantial improvement, as reflected by their similar behavior in both events.

OBSERVED BUILDING PERFORMANCE OF LE-MIRCF STRUCTURES

Survey Methodology

In order to enable eventual comparisons between structures exhibiting different response traits, it was imperative to include in the building sample constructions exhibiting the full range of damage levels clustered around the same area of town. The range of building damage was defined as having a five tier classification, specifically including for collapsed, heavily damaged, moderately damaged, lightly damaged and undamaged constructions. Over a six day period, fifty buildings were surveyed in detail, some of which having identical designs thus allowing for speedier investigations. The strategy adopted for inspection was centered around the completion of a post-event evaluation pro-forma [18] for each surveyed construction, developed specifically for LE-MIRCF buildings. The survey form caters for a large amount of data, which however could not always be completely gathered, as some buildings were internally inaccessible. Nevertheless, such an approach facilitated the inspection of the buildings by standardizing the visual examination procedure.

Following the identification of a sample construction on a detailed town map, complete planimetric measurements of the overall building dimensions were carried out, together with measurements detailing the layout of the exterior frame spans, individual column sizes and orientation wherever possible. These were then supplemented with building orientation and photographic records. Furthermore detailed internal inspections were also carried out in many instances and included internal measurements to enable the determination of the building configuration, together with a description of any internal damage present. The latter included reporting on the presence of a variety of local damage mechanisms, including amongst others, concrete spalling, buckling of the longitudinal bars, stirrup rupture, and captive column effects. The location, size and disposition of a few major cracks was also noted and supplemented by Schmidt hammer readings taken from various reinforced concrete elements, with at least 3 readings from each tested element. Important information relating to the structure type was always gathered, particularly noting the type of load bearing structure, the method of construction and its quality, materials used and the presence of any visible irregularity in infill layout or framing arrangement. Additionally local residents were frequently interviewed in order to establish details such as the location of visible damage in particular buildings, building use at the time for each storey, the reporting of any casualties, and building completion dates. Depending on the overall damage level, each survey lasted up to three hours. Supplementing the surveys, interviews with local engineers were also conducted in order to determine and familiarize with local building practices.

Observed Construction

By far the gross majority of all new buildings in Bingol are reinforced concrete frame structures with weak hollow clay blocks as the infilled masonry. Many of these constructions fall under the LE-MIRCF typology, and pertain to the broader class of non-ductile structures and gravity load only designed constructions. Most people in Bingol live in such buildings, which are termed 'beskats' because of their five storey height. From the survey carried out, two to four apartments per storey is the norm, resulting in a total of 10 families per building. With each family having an average 4 members, each block houses 40 people at least. The potential for a high casualty rate in these constructions is therefore obvious [19]. Whereas in the central part of the town most buildings adjacent to each other did not have any gap between them, nearly all the constructions in the newer districts are completely isolated from each other.

All such buildings observed, consisted of reinforced concrete slabs cast monolithically with reinforced concrete beams and columns, with a notable absence of pre-cast concrete flooring. Floor to ceiling height

throughout the building was generally found to be between 2.6 to 3 meters, but ground storey heights varying up to 4.5 meters were measured when used for commercial purposes. Furthermore, nearly all the buildings had an irregular three dimensional frame grid owing to complex functional requirements. The orientation of the columns is haphazard in plan and depends on the location of the infill walls. Thus resulting in irregular column spacing and orientation, with a large number of columns oriented in the same direction, making buildings much weaker in one lateral direction. The gross majority of the columns observed were of a rectangular constant section throughout the building height with a minimum width of 230mm, thus matching 2 wythes of infill masonry, and a depth that varied up to 1000mm. Detailed member observations from collapsed buildings, damaged members and buildings under construction, revealed that longitudinal reinforcement in the columns is usually of eight or ten 16-18mm diameter smooth round bars, terminating either with straight lap splices or occasionally in 180 degree hooks. Laps in the longitudinal steel were always noted at the bottom of the column sections and when measured were more than 40 times lap diameter in length. Transverse reinforcement in the columns was noted as comprising 6-8mm diameter smooth steel stirrups at 200 to 300 mm centers and employing 90 degree hooks at the ends resulting in a transverse steel volumetric ratio of often less than 0.3% even for the smallest column sections.

Due to the irregular column spacing, beam spans are also generally irregular but seldom greater than 5 meters. Minimum measured beam dimensions were of 230 mm width, thus matching the column width, together with a 400 to 500 mm deep web, as measured from the ceiling soffit. Added to the typical 125 mm thick in-situ reinforced concrete floor the beams have a total depth of over 600 mm. Observed longitudinal reinforcement was of up to five 16mm diameter smooth steel bars in a single layer for either top or bottom reinforcement. Typically the central one or two bars, being bent diagonally near the gravity load inflection points, serve as sagging bottom bars near mid-spans and as hogging top bars near supports, whilst providing extra shear reinforcement. Transverse steel was measured as single stirrups of 6-8 mm diameter smooth steel at 250 to 300 mm centers. Beams sometimes frame into the columns eccentrically and are prevalent in either direction. Occasionally foundations were observed in new constructions, and consisted of reinforced concrete pad foundations connected by relatively lightly reinforced shallow ground beams or of a continuous pad footing over a line of columns.

All reinforcement is generally smooth mild steel, however evidence was seen of increasing use of deformed high tensile steel bars in some of the most recently constructed buildings, especially for the ones built after the year 2000. Following interviews with local practitioners it was established that buildings are constructed in-situ and each floor is cast in one pour. The aggregate and sand is obtained from the river bed, is not washed or sieved and any water source which is at hand is used in the mix. Mixing of concrete is done by volume and not by weight, using either portable concrete mixers or even simple manual mixing. However, ready mixed concrete from the local batching plant is available though considerably more expensive. The resulting concrete was observed to be of generally poor quality, being poorly graded, with a rather high water and sand content, and having rounded aggregate pebbles some of which are well over 30 mm in size. Segregation and honeycombing were quite common, whilst concrete cover to the reinforcement varied widely, though generally less than 25 mm. Schmidt hammer readings thus indicated a weak compressive strength below the 20MPa minimum allowable cylinder strength in Zone 1, as required by the latest revision of Turkish Seismic Code [17]. Nonetheless, instances of a good quality concrete with a grade of around 25MPa were also measured.

Once the frame is partially or totally complete, the masonry infill walls are constructed against the narrow side of the column, forming partition walls. In all cases the infills were simply mortared in place without any positive connection to the reinforced concrete members. Room partitions are built from one wythe of masonry and, where these do not intersect the frame, underlying beams spanning in between beams are provided for their support. The outer walls are usually constructed in two wythes separated by a 20mm

polystyrene insulation layer with no structural connection either between the wythes or to the reinforced concrete frames. The masonry infills are constructed from hollow clay bricks laid in a cement mortar, having typical dimensions of 130x190x200 mm. A coat of cement-lime plaster circa 10 mm thick was generally noted on all exposed surfaces. Short or captive columns created by infills of partial heights around window openings were observed in many buildings which had a floor or two used for commercial purposes, in apartment buildings with a basement used for housing services and in all the institutional buildings such as schools and hospitals.

Reinforced concrete shear walls are the exception in the majority of the inspected buildings. From all the buildings surveyed only the mayor's building had a lift, and this was constructed from infill masonry, whilst all the apartment buildings had a single reinforced concrete staircase, simply connected to the floor slabs to provide access to the upper storeys. Sloping roofs are constructed from a rough timber framework and steel sheathing over a horizontal reinforced concrete slab. Occasionally column reinforcement was also visible at this level for eventual addition of another storey. In the majority of buildings surveyed, it was noted that the steel reinforcement was correctly fixed, at least in so far as complying to gravity loading considerations, with adequate lap lengths. Moreover, constructions built after the year 2000 did evidence more attention to detail, such as closer stirrup spacing and the use of high tensile deformed bars, whilst when available architectural plans were accompanied with corresponding structural detail drawings. However, from interviews conducted, it transpired that even if the approved plans comply with the current Code, it appears that no mechanisms are in place to ensure that construction conforms to these plans. Finally, government buildings such as schools, military and social housing projects often evidenced higher construction standards, especially through the quality of the concrete, although glaring exceptions were also noted.

Damage Observed in Surveyed Buildings
In all, about fifty buildings were surveyed in detail. Given the number of buildings inspected and the diversity of the damage types surveyed, a few considerations of general validity can be drawn. However the building sample inspected is still limited when compared to the town's total of LE-MIRCF building population. Therefore, the information extracted is not aimed at providing global generalizations but rather to extract factual information from constructions which were situated very close to each other and to report on their performance vis-à-vis their geometric disposition, structural layout and standard of construction, thus emphasizing the major points which determined the building's response.

A total of 15 collapsed buildings were surveyed, which amount to around 75% of the LE-MIRCF buildings which were reported to have collapsed [6]. The corresponding distribution of global collapse mechanisms for these buildings is represented in **Figure 6**. The most common cause of global collapse was by far precipitated by the presence of a weak ground storey in the form of a ground floor collapse or a first storey one, when the basement was semi-visible. Pancake type collapse, usually triggered when a more even distribution of stiffness in elevation is present, was identified as the other global failure mechanism, whilst the unidentified refer to the four buildings that were demolished prior to inspection. No instances of overall soil failure, building overturning or structural roof failure were observed. From these buildings only two were internal developments and thus had buildings adjacent to them, the rest were either completely isolated or, as in a couple of instances, at the end of a row of buildings.

Members exhibiting some form of discernable localized damage were prevalent in 35 of the surveyed buildings and are represented in **Figure 7**. The varying local failure types were identified individually in constructions, or were frequently present in varying combinations. Diagonal cracks in the infill panels were noted in all the constructions which were classified as lightly damaged or above. No evidence of horizontal cracking in the mortar joint was observed, though in 26% of the 35 buildings, the infill panels

actually had crushed masonry units, whilst in 9% overturning of a wythe or whole panel was observed. This fact reinforced the feeling acquired on site that the mortar used in the gross majority of cases was relatively strong when compared to the highly perforated individual masonry units. As regards the reinforced concrete elements, damage caused by shear effects in columns (including compressive type x-cracks) was prevalent in 40% of the buildings, the short column effect in 34% and clearly visible column hinging in 20%. What became apparent from the surveyed building stock was that few buildings exhibited elements which could be classed as being moderately damaged and consequently, if reinforced concrete elements were damaged, they were either heavily damaged beyond repair, or had caused the building to collapse. This is attributed to the brittle failure modes prevalent in these buildings due to the lack of ductile detailing
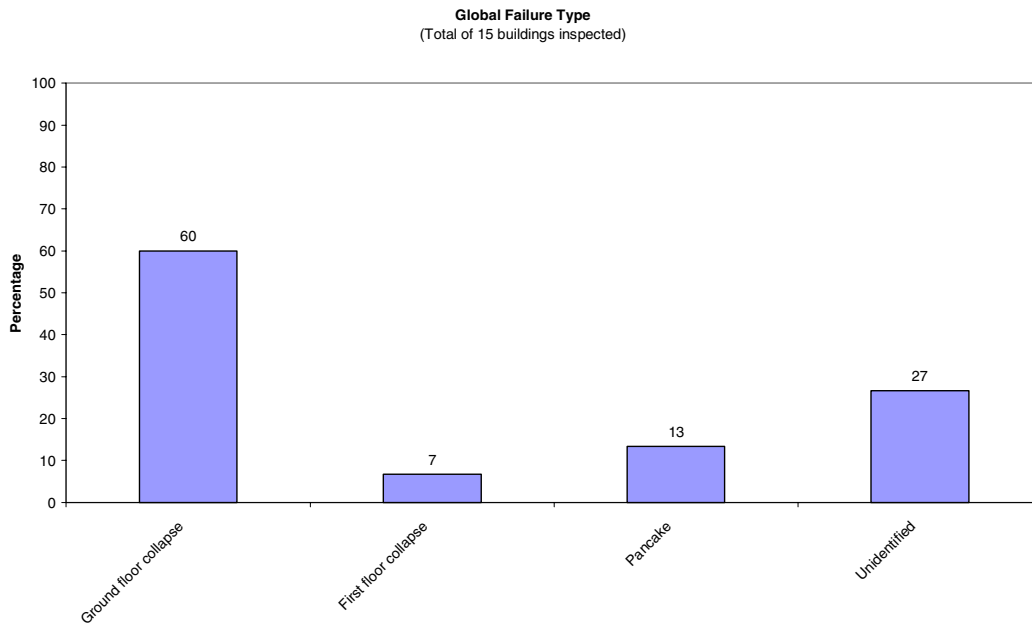
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Global Failure Type
(Total of 15 buildings inspected)

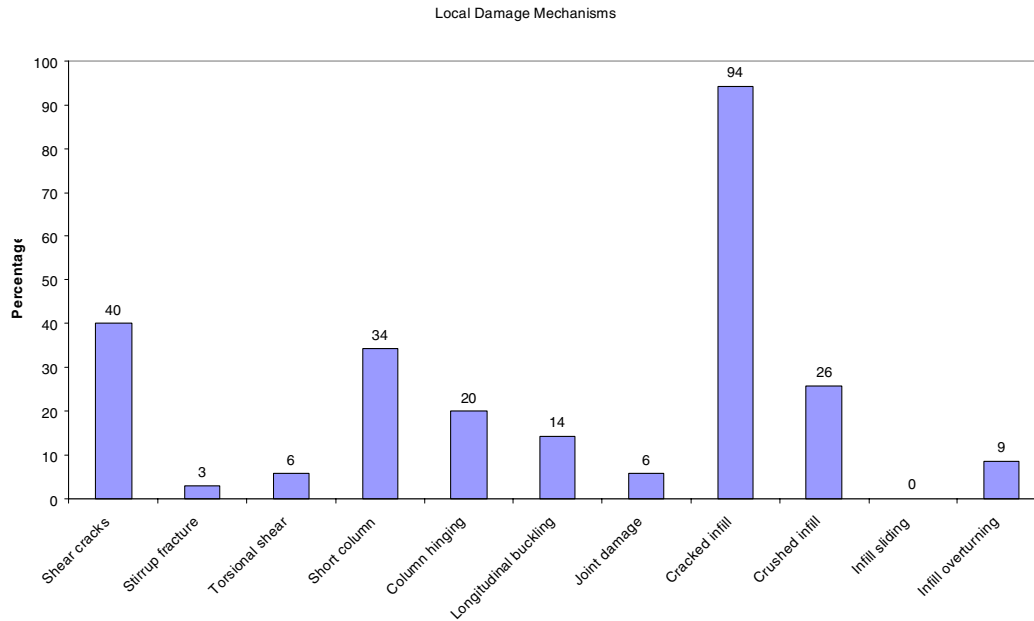**Fig. 6** Global collapse mechanisms from buildings surveyed.

Fig. 7 Local failure mechanisms from buildings surveyed.

Discussion

The most interesting fact about the Bingol event is the general uniformity of the building stock in Bingol town, with typically only slight variations to the basic 5 storey structure, such as the presence of a commercial ground floor, different storey height or the presence of a structural wall. Compliance with seismic provisions is mandatory, and investigations of documentation submitted for planning approval revealed that detailed structural designs accompany architectural plans. However, the general uniformity of member sizes and the similar reinforcement layouts in them, lead one to believe that many buildings are detailed simply through experience.

Collapses were few, though unfortunately tragic, as the occupancy of apartment buildings is invariably high. From the two identifiable global collapse modes observed, the pancake type collapse resulted in the greatest number of lives lost and occurred in at least one building, the Celtisuyuk dormitory block, and possibly also in an apartment building. However, the gross majority of the collapsed buildings failed due to a soft ground storey, with the remaining upper floors or underlying basement remaining practically intact. The causality rate in such structures was therefore commensurate with the usage of the ground floor at the time. As most of the soft ground storeys were caused by the presence of commercial establishments, which were closed during the time of the event, the death rates in these buildings were much lower.

Few buildings throughout the town with a soft storey at ground floor did not suffer significant damage and the ones that did not, were probably located in areas were ground shaking was not as intense as the main record available. Moreover, for the buildings in the town centre which abutted each other, consideration has to be given to the fact that the adjacent blocks provided mutual support during the event, and thus had reduced displacements and consequently less damage. Even here however, the majority of buildings with a soft ground storey did evidence significant damage in the ground storey columns, which suffered various shear type effects. These were accompanied by vertical hairline cracks between the beam column joints, and almost total masonry infill destruction in the internal partitions at ground floor. Damage to the reinforced concrete elements was nearly solely confined to the ground floor, apart from a few very narrow cracks in other storeys. The masonry infills in the first floor suffered moderate damage in most cases, but all semblance of damage was gone by the second storey with nominal frame infill separation in the upper storeys.

As expected, the reinforced concrete frames with masonry infills formed a relatively stiff and strong lateral load resisting system in the upper storeys, unlike the frames with few or no infill walls in the ground storey, in order to provide more space for commercial outlets. The smaller stiffness at ground floor thus induced increased deformation demand in the frame members of the soft ground storey and almost the entire lateral deformation was concentrated in the ground storey columns with the upper storeys moving laterally as a rigid block. Additionally, unlike the upper storey columns, the ground storey columns in such LE-MIRCF buildings could not share the lateral shears with the infill walls. Many of these then sustained brittle shear failure. The generally poor concrete strength, with a typical value of around 15MPa compressive cylinder strength as obtained from the Schmidt hammer readings, only made matters worse. Volume batching, which does not account for moisture in the aggregates, together with manual mixing techniques and placement, all resulted in a higher water content to ensure good workability, and in a weak porous concrete. The smooth round largish river pebbles further contributed towards a weak mix.

The lack of ductility of the constructions was evidenced by the general lack of significant cracks in the reinforced concrete elements. Fully developed plastic hinges were noted only sparingly, as they seemed to be preceded by shear cracking. This is also a direct consequence of the detailing practice adopted for the reinforcing stirrups in the columns, which offered only light confinement to the core concrete. These therefore failed in a brittle shear mode and were prone to catastrophic failure. Only in a few buildings did the ground storey columns sustain significant shear and flexural cracking together, such as the construction shown in **Figures 8-11**.

Fig. 8 The Dunlop building appears undamaged from the façade

Fig. 9 Shear failure in a column partly due to inadequate reinforcement

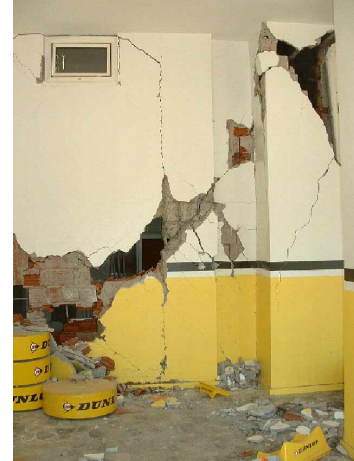**Fig. 10** Short column effect resulting in explosive shear failure

Fig. 11 Shear failure extending from the infills right across the column

Reinforced concrete flexural members detailed for ductile behavior include an under reinforced section, ensuring that flexural yielding precedes shear failure, and good confinement of concrete in the critical region. For confining concrete in the transverse direction, transverse stirrup reinforcement closely spaced to develop a tri-axial state of stress in the core concrete in the plastic hinge regions and having 135 degree hooks with adequate hook length are mandatory. Furthermore, column bars are to be spliced in the middle-half of the storey. Detailing practice included neither of these features and it was observed that the splice to all column bars is just above the floor slab, with no additional tie reinforcement provided over the lap length. Clearly therefore, the lowest level of ductility assumed by the 1998 Turkish Seismic provisions of 4 is optimistic at best, as most of the constructions investigated implied a ductility of 2, even for those buildings constructed after the implementation of this code. Nonetheless, newer constructions did appear to have a slightly higher standard of detailing.

Damage in beams was rare, as the weak column strong beam mechanism did not allow these elements to develop their full strengths and the columns failed before. Only in buildings without a dominant soft ground storey was concrete spalling and evidence of cracking noted in beams. Rarer still were beam-column joint failures. As a matter of fact, only in a bare frame building were these noticed, where the beam spans of 6.5 meters induced additional response requirements on these elements.

Many fully infilled apartment buildings, without commercial premises in their ground floor, suffered non-structural damage to the masonry infills, in the form of shear cracks at the ground storeys. Though such damage proved alarming to the local populace, the damage was generally slight, predominantly contained in the ground storey and gradually dying out to hairline cracks in the finishes of the upper storeys. The reinforced concrete elements of these constructions generally suffered only localized damage to particular details, such as at construction joints or intersecting beam junctions. Evidently the masonry infills supplanted the lack of ductile detailing in these constructions. Few out-of-plane collapses of the infill walls were visible, however complete corner collapses, where the infill walls formed the building corner, were noted as at Bingol Hotel in the town centre.

The presence of reinforced concrete shear walls generally ensured that buildings suffered much less damage than for constructions without them, especially when these were combined with small room spans and a regular structure. In Bingol Lisesi school (**Fig. 12, 13**), where the shear walls were used in conjunction with large beam spans and room layouts, they attracted a lot of damage and emphasized the

weak concrete used there, though probably they saved the rest of the building from added damage. Though many government buildings suffered severe damage, as evidenced by the number of schools and police stations rendered unusable, general concrete quality and on-site attention to detail did appear to be better than for the typical beskat.

Fig. 12 Bingol Lisesi school

Fig. 13 Damage to the shear wall at Bingol Lisesi

The very poor performance of irregular structural configuration was noted, such as short column effects and torsionally irregular structural systems. Buildings with plan asymmetry experienced significant torsional motions. As a result, the flexible side of the building experienced larger displacements than the stiff side. The excessive deformations caused considerable damage to the columns on the flexible side. Buildings with plan irregularities, such as those with re-entrant corners or L-shape plans, were uncommon as were those with elevation irregularities involving large vertical setbacks in elevation, whilst no floating columns were noticed in the constructions. The total absence of intermediate storey collapse was also observed.

CONCLUSION

The difference in damage between one district and another in Bingol town can be attributed to local site amplifications. However, even if the possibility exists of larger earthquake forces actually being imposed on the structure than those originally designed for, as shown by the comparison of the strong motion record with the design spectra for the region, regular structures in close proximity of irregular ones, both built to the same standards and employing the same level of structural proficiency and detailing and subjected to the same ground motions, only suffered slight damage whereas the latter suffered irreparable damage.

Therefore, the overriding lesson conveyed from the event is the need to have design professionals strongly conceptualize the structural behavior of the construction when subjected to lateral forces. Further improvement would be attained by the gradual education towards well detailed reinforced concrete structures. Given the level of expertise employed in the region in applying a modern material, such as reinforced concrete using low technology methods, heavy damage and collapse of the beskat was almost exclusively precipitated by "architectural" rather than structural design flaws. Furthermore, thirty years of urban development has resulted in the replacement of many traditional buildings with LE-MIRCF buildings. Nonetheless, the behavior of the latter does not appear to have improved much, since the first construction types of such buildings had been subjected to severe ground motions, even though the level of seismic forces they require to be designed against has been continuously increased through at least two seismic code revisions in the region. This is attributed to the fact that current design and assessment techniques which do not specifically cater for the majority of the available building stock in such regions, and do not include for infill participation are essentially flawed. Under the action of seismic loading, the

infills behave as structural elements, even when not vertically load bearing and hence need to be accounted for in analysis.

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