STRUCTURAL FIELD STUDIES AFTER THE ERZINCAN TURKEY EARTHQUAKE OF 13TH MARCH 1992

13 MART 1992 ERZINCAN DEPREMINDEN SONRA YERINDE YAPISAL INCELEMELER

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ABSTRACT

Studies are reported of three aspects of the Erzincan earthquake of 1992, based on field work carried out in the first week of April 1992, using simple but quantitative methods. The first study investigated the variation of damage intensity along a 3km long north-south transect through Erzincan, with the intention of finding if any evidence existed of basin effects on ground motion in the city. Although not conclusive, the studies suggested that any such basin effects were probably weak. The second study investigated the influence on seismic performance of two aspects of construction practice in Erzincan. These related to gable end walls and to basement wall construction. Marked differences were found between different construction practices, from which clear lessons emerge. The third study attempted to relate simple indices of structural adequacy emanating from Japan to the performance of two very recent buildings affected by the earthquake. The Japanese indices appear to have value as a very simple measure of vulnerability of reinforced concrete buildings, for assessment or checking purposes.

INTRODUCTION

The UK post earthquake field investigation EEFIT mounted a 5 man field mission to Erzincan during a 2 week period from the 30 March 1992. The full findings of the mission are contained in the EEFIT report (Williams, 1993). A general investigation was carried out in order to present an overview of the earthquake, primarily to the UK engineering community. However, much of the evidence of structural engineering interest was contained within the limited area of Erzincan itself. The earthquake therefore afforded a rare opportunity to study in some detail the effect of a large earthquake on recent construction, essentially within an area that could be travelled on foot. Accordingly, several detailed studies were performed using simple but quantitative methods which it is hoped will add to the international effort to draw lessons for earthquake engineers from the evidence of the earthquake. One such study related to a very detailed damage survey around the strong motion instrument in Erzincan to add to the database at the Martin Centre, University of Cambridge; the findings are given in the EEFIT report. Three other studies, carried out by the author within Erzincan, are given in this paper. They required only a tape measure, a notebook and pencil, a camera; a strong pair of boots and the hospitality and kindness of the citizens of Erzincan.

NORTH SOUTH TRANSECT STUDY THROUGH ERZINCAN

A simple damage survey was made to try and establish whether ground motions in Erzincan had varied systematically with distance from the edge of the Erzincan basin. The survey was conducted along a north south line across the city, just to the west of the city centre (figure 1). Based on a hydrogeological survey map dated 1981, it appears that the depth of sedimentary material varied considerably along the survey line (Figure 2).

Two types of damage were recorded, as follows.

1) The percentage of boundary walls between 1m and 1.5m in height which had collapsed. The boundary walls were all to domestic properties adjoining the line of the survey and were all aligned in a north south direction.

2) The percentage of chimneys on buildings not exceeding 3 storeys which had collapsed, and were visible from the line of the survey.

The intention was to adopt a measure of damage which was easily recorded and was sufficiently abundant to provide some statistical measure of confidence. Generally, the type of housing and the nature of the boundary walls appeared similar throughout the survey, so it was hoped that a reasonably uniform damage measure would result.

The results of the survey are plotted in figure 3. The results are not conclusive; the chimney survey shows no significant variation of damage with distance from the edge of the basin while the boundary fence survey suggests some increase in motion at the southern end of the survey line. It is likely that more reliance can be placed on the chimney survey because the high damage level of walls to the south may be a result of softness in the uppermost layers of soil, in lower lying areas, which would not necessarily affect chimney response. Certainly, the absence of trend in the chimney damage suggests that any tendency of the motions to increase with distance from the edge of the basin was low.

For comparison, the percentage of damaged (light, heavy and total collapsed) buildings recorded by Yuzugulu et al (1992) in Erzincan is shown in Figure 1. No clear trend can be seen, with the damage level probably reflecting type of construction rather than distance from the northern edge of the basin.

INFLUENCE OF CONSTRUCTION PRACTICE ON STRUCTURAL PERFORMANCE

Performance of gable ends

Failure of gable ends was widespread. Three types of gable end were noted; their performance was quite different. Where the gable end extended vertically to eves level without lateral restraint (plate 1) failure was almost universal, probably exceeding 80%. Vertical gables which were restrained by the roof purlins (plate 2) suffered very substantially less damage, though cracking around the purlin can be seen in the plate. The third type of gable end was provided with a pitched end to the roof which bore onto the end wall (see extreme right of plate 1); no failures were observed with this arrangement.

Influence of basement construction on performance

A study was made of an estate of 4 storey apartment buildings in the Fatih district in the north east of Erzincan (Plate 3). The apartments consisted of two apparently very similar types of apartment, which had however performed rather differently. There were about 18 older apartments of the first type (plate 4) to the south of the site. They had four above ground storeys and a semi-basement about half of which was above ground level. The structure consisted of concrete frames with clay tile infill; some apartments had their major axes oriented east-west and others north-south. There was little evidence of damage externally in any of these older apartments; internally there was extensive severe cracking to the infill panels (but no collapses were noted) and hairline cracking could be seen in some beams and columns. The occupants were not sleeping in these apartments but used them during the day for washing etc.

To the north were 8 more recent apartments (Plate 5) of similar construction to those discussed above. 2 of the 4 apartments with their long axis in a north south direction had collapsed and had been demolished; the 2 others in the same direction had suffered a failure in their semi-basement (Plate 6). There was less apparent damage in the blocks oriented at right angles.

The structural difference between the two types of blocks was that the older blocks were provided with concrete walls between basement and ground floor level (Plate 7), whereas in the newer blocks, the concrete frame continued to foundation level and was infilled with concrete blockwork. Plate 8 shows an adjacent block under construction with this detail. The newer (and presumably cheaper) construction practice had apparently resulted in the creation of a stiff but weak and brittle storey with columns whose shear strength was less than their flexural strength. This detail should therefore be avoided.

INDICES OF SEISMIC VULNERABILITY

A dimensional survey was carried out on a 6 storey residential block with a single basement level in the Bahcelievler district of Erzincan. It was structurally complete, but not fitted out; the roof slab had been cast but the shuttering had not been struck. Blockwork infill walls appeared mainly complete but no finishes had been applied.

Figure 4 shows the first floor plan, based on a site survey using only a tape measure. The vertical structure started at basement level and continued essentially undiminished to roof level. Some reduction in column sizes were noted at upper levels, but a detailed survey was not carried out, due to the unsafe state of the building. Storey heights were 3m, except for the ground floor and basement which had storey heights of 3.6m. The infill blockwork comprised 180mm thick hollow clay tile, except in the basement, where hollow concrete block was used. The columns at ground floor level had about 1% vertical and .3% transverse steel, with no indication of closer spacing at column ends. The shear wall (gridlines 2, D-E) appeared to have about .2% steel vertically and .15% horizontally in each face, with no evidence of special confinement to the vertical edges. These estimates are based on the exposed steel visible on the surface of some of the members.

The state of the building appeared perilously close to collapse. The major damage was at ground floor level, with none visible in the basement and little at higher levels (though a detailed inspection was not made). The central shear wall was very heavily damaged with the concrete reduced essentially to rubble and the reinforcement clearly visible. There was severe spalling at many of the top and bottoms of the ground floor columns between gridlines B and H (which provided the resistance to east-west motions); little damage could be seen in the other columns which provided north-south resistance. The stair slab had fractured at ground floor level, where the single mat of steel had pulled completely out of the concrete. There was also heavy damage to the stair slab at first floor level.

It can be seen from Figure 4 that the structure was reasonably uniform on plan; an analysis by EEFIT has shown that the ratio of torsional to translational stiffness conformed to the limits required for regular structures in the Japanese Building Standard Law (IAEE, 1988). The vertical regularity was also reasonable, although the ground floor height was 20% greater than that of upper floors, and the building damage was concentrated at ground floor, indicating a weak storey at that level. The vertical regularity was therefore checked by performing a response spectrum analysis of the building, using the 5% damped spectrum recorded in Erzincan, and was also found to comply with the Japanese limits (Aoyama, 1981) for variation in storey drift with height of structure .

For reinforced concrete structures up to 31m in height with a minimum degree of ductility and lateral strength and which conform to the regularity limits referred to above, the Japanese Building Standard Law specifies that buildings which satisfy the following inequality do not require 'phase 2' checks for resistance to an extreme earthquake. The inequality applies to structures mainly relying on beam-column frames for lateral resistance amd is based on observations of building performance in damaging Japanese earthquakes (Aoyama, 1981).

$$\frac{\Sigma 1.8A_w+\Sigma 1.8A_c}{1.0ZWA_i} \geq 1 \tag{1}$$

Where

- A_w = area of shear walls in the direction of seismic force being considered (mm^2)
- A_c = area of columns (mm^2)
- Z = zone factor = 1.0 for the most seismic area of Japan
- W = weight of building above the level under consideration (N)
- A_i = vertical distribution factor = 1.0 at ground floor level

The Bahcelievler building satisfies both regularity and height requirement, but is unlikely to conform to the minimum ductility requirement of the Japanese code. Nevertheless, it is instructive to calculate the ratios of equation 1 at ground floor level. The zone factor was taken as 1 on the basis that both Japan and this area of Turkey are regions of high seismicity. Since the columns are very much stiffer in one direction that the other, only the columns aligned with the direction under consideration were included in the calculation of A_c. In the east/west direction, (the likely direction of maximum shaking) the ratio was calculated at 0.55, and in the north/south it was 0.78.

Similar ratios were calculated for the southern end of the new Town Hall in Erzincan, a five storey building which was structurally complete but unoccupied at the time of the earthquake. The building also had a moment resisting reinforced concrete frame with limited concrete shear walls and infill blockwork. The building appeared to have suffered no structural damage but there was extensive cracking to finishes and infill walls. The ratios calculated from equation 1 were 0.79 east-west (the likely direction of maximum excitation) and 0.51 north-south. Note that these values would not in themselves be unacceptable in Japanese practice but would necessitate a further check to be carried out.

A very simple calculation has therefore demonstrated for a severely damaged but still standing building that inadequate provision of seismic resisting structure was probably provided in the direction of maximum shaking. A building with no structural but extensive non-structural damage has a provision of lateral resisting structure in the direction of maximum shaking which, by this crude calculation, is 44% greater. The simple formula therefore appears to have some merit, provided it is combined with simple but conservative rules for ensuring vertical and horizontal regularity (perhaps based on those proposed in Eurocode 8, 1988), provision of minimum main and transverse steel (based for example on the Turkish seismic code) and limits on beam dimensions to encourage 'strong column/weak beam' structures. It is suggested that such checks may be useful for rapid assessment of the seismic adequacy of low to medium rise buildings. Such simple rules may be more effective than more sophisticated measures which could be misapplied, and can certainly be helpful as independent supplementary checks.

CONCLUSIONS

1) Simple but quantitative methods of structural survey can yield valuable insights during post earthquake reconnaissance missions.

2) Any tendency for ground motion intensity to vary with distance from the northern edge of the Erzincan basin has been shown as likely to have been weak, based on a 3km long north south transect through Erzincan.

3) The widespread failure in Erzincan of gable ends at roof level was confined to blockwork gables which were unrestrained at roof level. Vertical gable ends onto which bore roof purlins were very much less prone to damage, as were roofs with a pitched end gable. These latter details are to be preferred, since falling masonry creates a serious hazard.

4) Semi basements in concrete frame buildings, where the basement walls are created by infill blockwork were prone to collapse. This was attributed to the creation of a brittle storey between ground level and the first suspended floor. Similar buildings with concrete walls at basement level performed much better.

5) Simple vulnerability indices used in Japanese practice may be valuable elsewhere, including Turkey, in providing rapid methods of assessing or checking the seismic adequacy of low to medium rise concrete buildings.

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13 MART 1992 ERZİNCAN DEPREMİNDEN SONRA YERİNDE YAPISAL İNCELEMELER

Edmund BOOTH

Bu çalışmada 1992 başında yapılan ve basit fakat sayısal yöntemlerin kullanıldığı arazi çalışmalarına dayanarak, 1992 Erzincan depreminin üç yönü üzerindeki incelemeler konusunda bilgi verilmektedir. Araştırılan birinci konu, eğer varsa tam kentin altındaki zeminde çanak etkisini gösteren bulgular elde etmek amacıyla Erzincan'ı Kuzey - Güney doğrultusunda kesen 3 km. uzunluğundaki bir kesitte hasar şiddetindeki değişimin incelenmesidir. Kesin olmamakla birlikte çalışmalar çanak etkisi olasılığının zayıf olduğunu düşündürmektedir. Üzerinde durulan ikinci konu Erzincan'daki inşaatların iki özelliğinin yapıların davranışına etkilerinin araştırılmasıdır. İnşaat uygulamaları arasında üzerinde durulması ve ders çıkarılması gereken farklar bulunmuştur. Araştırılmaya gayret edilen üçüncü konu depremden etkilenen iki yeni binanın dayanım göstergeleri ile Japonya'dan kaynaklanan basit yapısal uygunluk göstergeleri arasında ilişki kurmaktır. Japon kaynaklı göstergelerin, betonarme binaların deprem duyarlığını çok basit olarak ölçmek, değerlendirmeler yapmak ya da doğrulama amacıyla kullanılabilecek düzeyde oldukları görünmektedir.

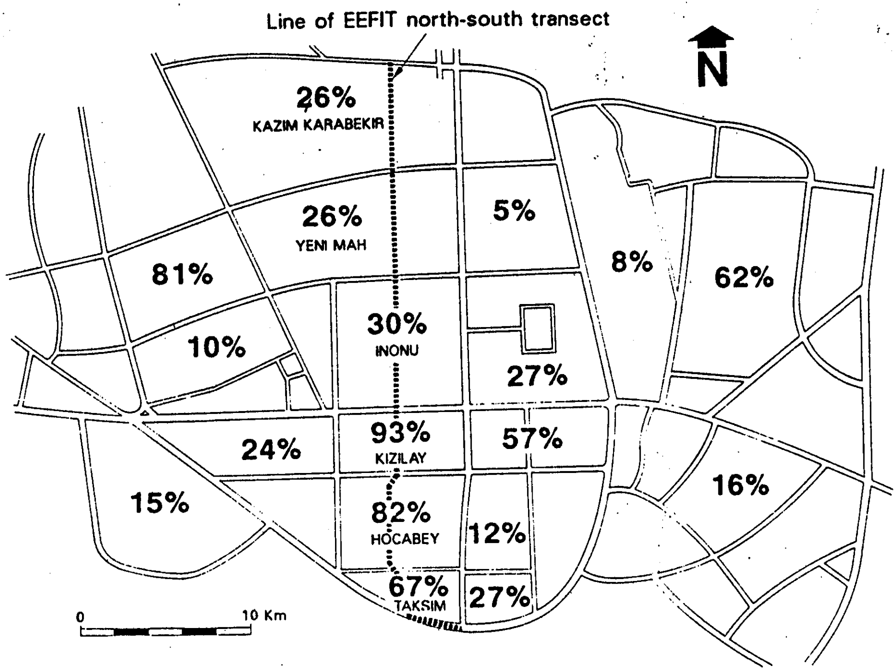


Figure 1 Line of north-south transect survey through Erzincan. Also shown are percentage of buildings damaged in each district (from Yuzugulu et al, 1992)

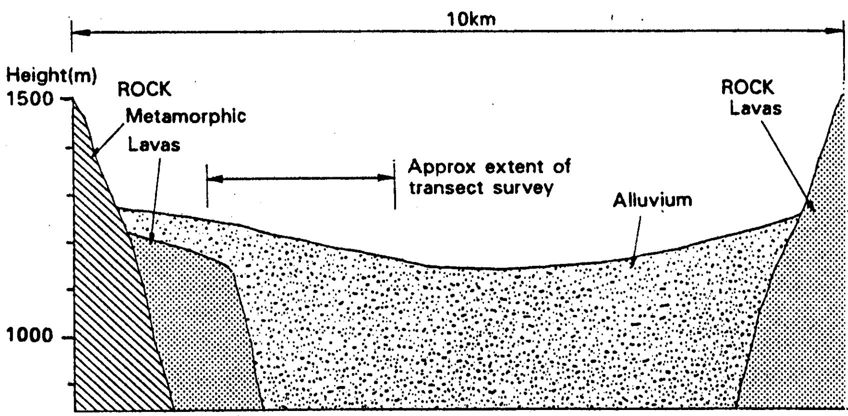
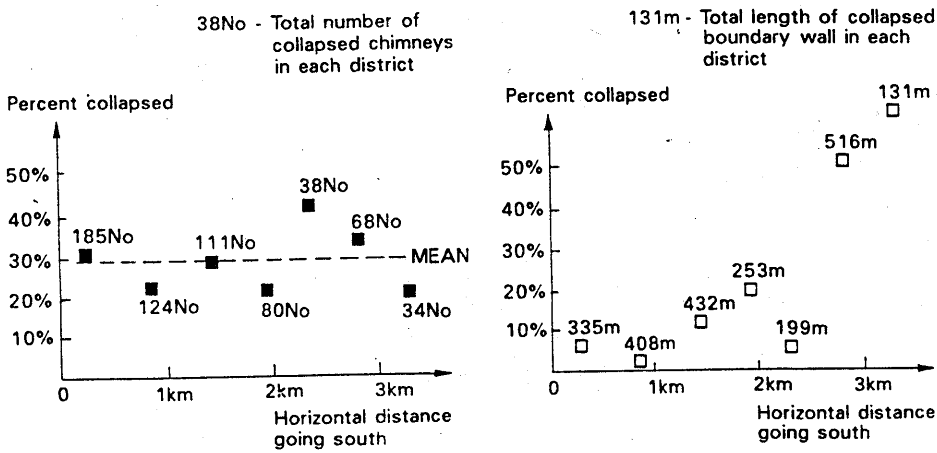
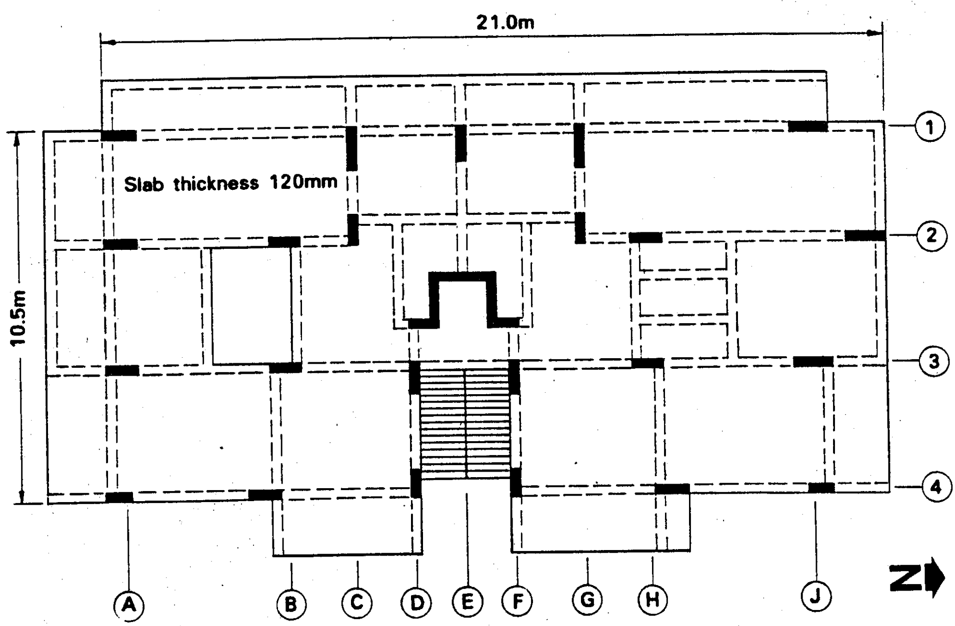


Figure 2 Cross section through Erzincan basin, at Erzincan, looking east. (Based on hydrogeological survey, 1981)



a) Chimney damage b) Boundary wall damage

Figure 3 Damage survey along north-south transect through Erzincan



Typical beam size 200 x 600 deep

Typical column 300 x 1000

Figure 4 First floor plan of 6 storey apartment building in Bahcelieuler district, Erzincan.

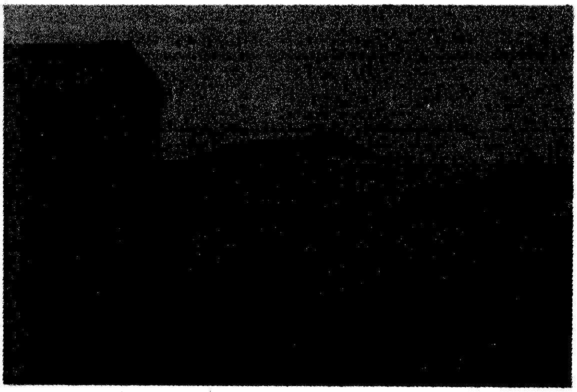


Plate 1: Failure of gable end wall

Plate 2: Gable end restrained by roof purlin

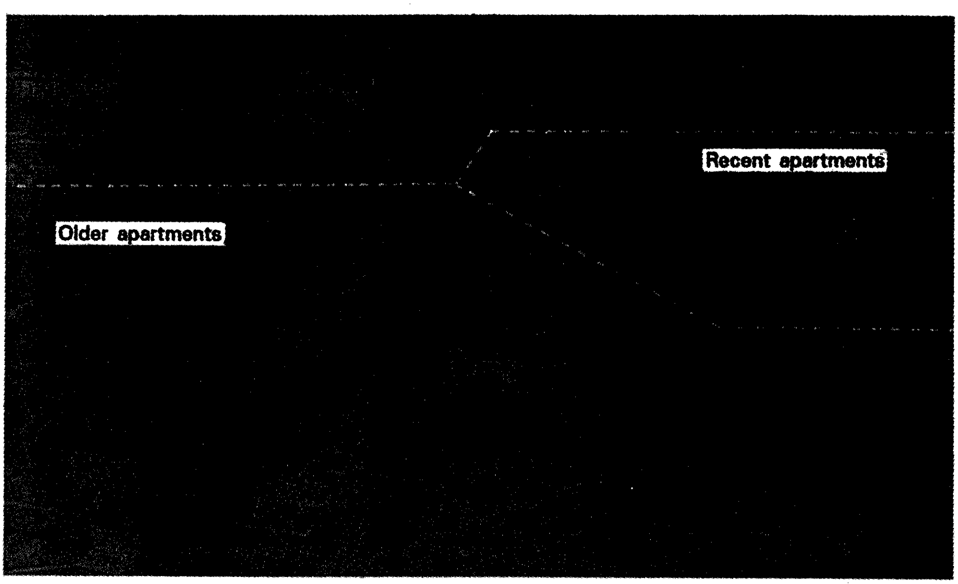


Plate 3: Aerial view of housing estate, Fatih district of Erzincan, before earthquake

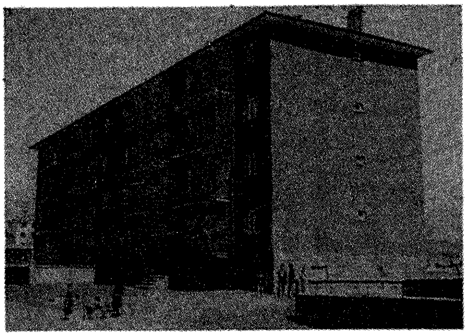


Plate 4: Older apartment building

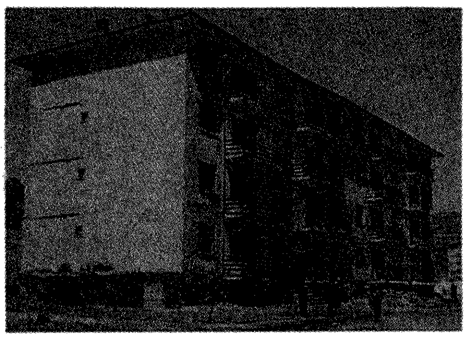


Plate 5: Newer apartment building

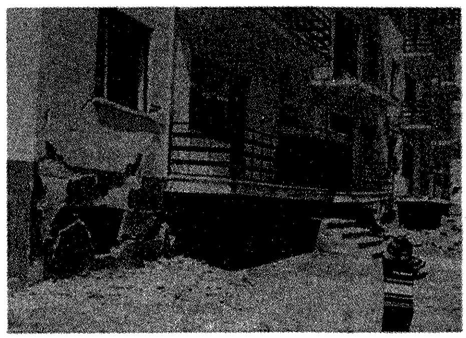


Plate 6: Close up of Plate 5 showing failure in semi-basement

Plate 7: Semi-basement wall construction in older apartment building

Plate 8: Semi-basement wall construction in recent apartment building