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Behaviour of foundations over surface fault rupture: Analysis of case histories from the Izmit (1999) earthquake

Le comportement des fondations par-dessus la rupture de faille superficielle: Analyse des case–histories du tremblement de terre de Izmit (1999)

Ioannis Anastasopoulos
Candidate, National Technical University, Athens

ABSTRACT

The 1999 Mw7.4 Izmit (Turkey) earthquake was triggered by reactivation of the North Anatolian Fault. Although the faulting mechanism was strike–slip, the paper focuses on the normal fault that nucleated within the pull-apart basin of Gölcük, and its effect on overlying residential structures. The 4 km normal rupture, caused maximum vertical displacements of 2.3 m. Several structures were crossed by the rupture. As expected, many of them either collapsed or were severely damaged. Surprisingly, some structures survived the dislocation with no damage, while in some cases the rupture deviated to "avoid" the structures. Luckily, the foundation of the involved structures comprised a variety of foundation types, ranging from simple separate footings to piled foundation. The paper provides a comprehensive description of the observed fault-foundation interaction patterns, accompanied by the results of soil exploration and geological trenching. Each structure is analyzed through the use of finite element modelling to reveal the main aspects of Fault Rupture—Soil–Foundation–Structure Interaction (FR–SFSI).

RÉSUMÉ

Le 1999 Mw7.4 Izmit (à la Turquie) tremblement de terre a été déclenché par la réactivation de la Faille Anatolienne du Nord. Bien que le mécanisme était strike-slip, ce papier se fixe sur la rupture normale qui a regroupé dans le bassin de force à part de Gölcük, et son effet sur les structures résidentielles. La rupture normal, 4 km longue, a causé déplacements verticales de 2.3m. Plusieurs structures ont été traversées par la rupture superficielle. Comme prévu, beaucoup d'entre eux ont été sévèrement endommagées. Etonnamment, certaines de ces structures ont survécu la rupture avec absolument de dommage, et dans certains cas la rupture été déviée à "évité" ces structures. Heureusement, la fondation des structures impliquées a compris un assortiment de types de fondation, étendant des fondations séparées simples à la fondation entassée. Le papier présente une description complète de l'interaction observée, accompagnée des résultats de l'exploration de sol et du trenching géologique. Chaque structure est analysée par l'usage du méthode d'éléments finis (FEM) pour révéler les aspects principaux effectuent Fault Rupture—Structure–Fondation Interaction (FR–SFSI).

1 INTRODUCTION

The disastrous 1999 Mw7.4 Izmit earthquake was triggerered by reactivation of a 125 km portion of the North Anatolian Fault (NAF). With its epicenter 5 km southwest of Izmit, it struck the industrialized corridor around the Marmara Sea, causing more than 30,000 fatalities. The earthquake caused tectonic surface rupture over an area exceeding 110 km in length, with maximum offset of 5 m. General overviews of the behaviour of numerous structures in various locations can be found in Earthquake Spectra (2000). The differential displacement of the Gölcük segment relative to the Sapanca segment produced a 4 km NW-SE (110^o) normal fault east of the city of Gölcük, crossing the small community of Denizevler, with maximum vertical displacement of 2.4 m. The geometry of the ruptures, the geomorphology, in combination with palaeo-seismicity studies confirm the tectonic origin of the event (Tutkun et al., 2001; Pavlides et al., 2003).

The dislocation crossed several residential structures. As expected, many of them collapsed or were severely damaged. Surprisingly, several structures survived, essentially unharmed, with the rupture path seeming to have deviated, as if to "avoid" them. In other cases the damage was substantial even though the dislocation was "masked" by the near-surface soil, not creating a distinct scarp. The rigidity of the foundation appears to have been one of the crucial factors affecting the performance. The involved structures were supported on a variety of foundation types, ranging from isolated footings, to rigid box-type foundations, and piles. The paper outlines the reconnaissance of the area, providing a documented description of the observed performance, along with the results of soil exploration and geological trenching. Each structure is analysed nymerically to reveal the main aspects of FR-SFSI.

2 OVERVIEW OF THE DENIZEVLER CASE-HISTORIES

In Denizevler, within an area of 1 km, five residential buildings, a mosque, a basketball stadium, an automobile factory, and a high-voltage electricity pylon were crossed by the outcropping dislocation. Although the vertical differential displacement exceeded 2 m, only few of these structures collapsed. Four buildings survived with minor or no damage, with the surface rupture being diverted. Soil conditions do not differ significantly from point to point, and therefore differences in the behavior can be attributed to the foundation, in addition to the location of the rupture relative to the building. A detailed investigation of the area can be found in Anastasopoulos & Gazetas (2004a). From east to west, a first impressive success was that of the high-voltage electricity pylon, that did not collapse, sustaining only minor damage despite the "loss" of two of its four supports. Further west, a 4-story building (Bldg. 1) on the hanging wall, sustained no damage at all, with the rupture deviating around it. To its west, a Mosque was heavily damaged. Next to it, a 1-story building (Bldg. 2) was literally cut by the fault. Building 3 (2 stories + attic) remained on the "footwall", without any damage, avoiding a direct hit thanks to diversion of the fault rupture. Finally, a newly built Basketball Court, founded on piles, sustained severe damage. Due to space limitations we will focus only on Buildings 1, 2, and 3.

2.1 *Building 1*

As depicted in Figure 1 the surface rupture diverted and just avoided the 4-story reinforced-concrete structure, leaving it totally un-harmed. The downward settlement reached 2.3 m, accompanied by a strike component of 1.1 m. The only apparent damage was the flooding of the basement, due to the local modification of the water table. The owners were inside the house during the earthquake and felt no vertical falling. Evidently, the vertical displacement was of a quasi-static nature. The foundation of the 9 x 10 m building consists of strip footings ~0.6 m x ~0.3 m (height x width) transversely connected through tie beams of similar dimensions.

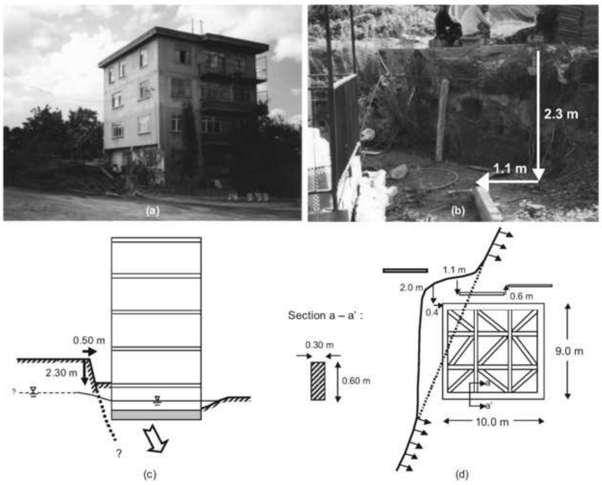

Figure 1. Building 1, four stories plus basement : Minor Damage

2.2 *Building 2*

Building 2 was a simple 1-story structure. Its wooden tile-roof was supported on cinder block walls. The walls were practically founded directly on the soil, without any foundation. This poor building could not have been expected to perform well subjected to a differential displacement of 1.5 m, and indeed it was torn apart by the rupture (Fig.2). However, it did not collapse completely, not causing fatalities. The rupture crossed its north-east corner tearing it apart from the rest.

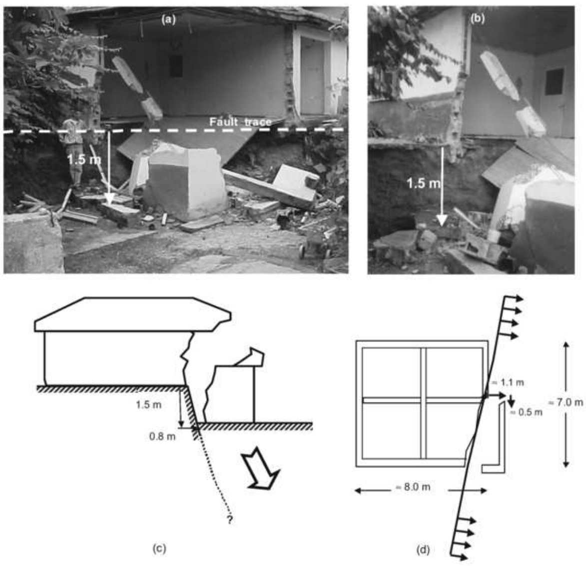

Figure 2. Building 2, one story cinder-block structure : Collapse

2.3 *Building 3*

Building 3 managed to survive without any visible damage. Most importantly, the rupture was diverted, as in the case of Bldg. 1, but since Bldg. 3 is founded on the footwall, the rupture was diverted to the North, towards the hanging-wall (Fig.3). The vertical displacement was 2.1 m. The 2-story (+attic) reinforced concrete building is founded on a rigid box-type foundation, comprising stiff concrete beams, ~0.5 m x ~0.8 m (width x height) sandwiched between a mat and a top slab, both ~0.3 m thick. The thickness of the whole box reaches 1.4 m, and the voids are filled with soil. It appears that this box foundation is quite common in the provincial regions of Turkey with poor soils. In Adapazari, where most of the failures were of the bearing capacity type, although many buildings toppled, foundation and superstructure remained un-harmed, confirming the ability of such foundations to safeguard vulnerable superstructures.

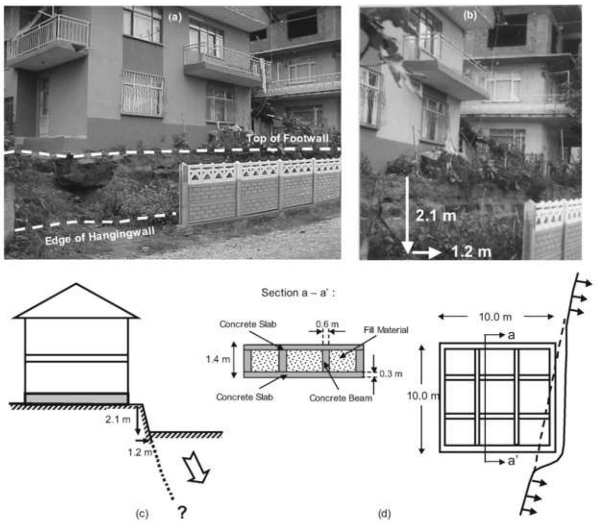

Figure 3. Building 3, two stories plus attic : No Damage

3 SOIL INVESTIGATION AND TRENCHING

In the area of study we conducted a limited soil investigation, comprising four boreholes and a 6 x 4 x 4 m (length x width x depth) trench. The soil exploration took place right beside Bldg.3 about 18 months after the earthquake, and regrettably, the fault scarp had been covered with fill. Two boreholes were located within the hanging wall, while two other were within the footwall. The first 6 to 8 m consist of relatively loose to medium soil layers with N_{SPT} ranging from 17 to 33, while deeper the soil becomes stiffer: $N_{SPT} \approx 50$, at depth of 15 m. The soil profile comprises alternating layers of silty to fine sand, and sand, while clayey materials are only limited to some thin layers. The water table was found to be at approximately -2 m.

The geological cross-section produced by the excavated trench revealed that besides the current dislocation, a second also exists. This older rupture is apparently the result of older seismic events (Pavlides, 2003) confirming the tectonic nature of the dislocation. In fact, the trench showed that the fault had been activated at least 3 times in the past. Our findings are in agreement with the recently published study by Klinger et al. (2003), confirming the tectonic origin of the normal fault.

4 ANALYSIS METHODOLOGY

Our goal is to present an in-depth analysis of fault rupture propagation from the bedrock to the ground surface, incorporating the interaction with the structure. To this end 2D plane-strain analyses are performed. However, the rupture is always longer than any structure, and their crossing is rarely exactly perpendicular; therefore, the plane-strain assumption is only a first practical approximation. The analysis is conducted in two steps. First, fault rupture propagation is analysed in the free field, ignoring the structure. Then, knowing the outcropping location, the model of the structure, consisting of beam elements, is placed on top of the soil model connected through contact elements, which are infinitely stiff in compression with no resistance in tension. In shear their behaviour follows Coulomb's friction law. Thus, the structure is not bonded to the ground, and both uplifting and slippage can realistically occur. By comparing the results, FR–SFSI is visualized and quantified.

The developed FE model is displayed in Fig. 4, referring to an H = 40 m soil layer at the base of which a normal fault, dipping at an angle α, ruptures and produces downward movement of vertical amplitude h. Our model is $B = 4H$ =160 m in width, following Bray's recommendation (1994) that a $B : H = 4 : 1$ ratio is sufficient to minimize boundary effects. The discretisation is finer at the medium, being sparser at the two edges. The differential displacement is applied to the left part of the model in small consecutive steps.

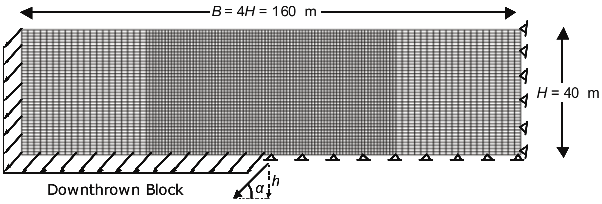

Figure 4. Problem geometry and finite element discretization

Several experimental and numerical studies have shown that soil behaviour after failure is decisive in rupture propagation. Scott & Schoustra (1974) utilizing the FEM and an elastic-perfectly-plastic constitutive soil model ended up with results contradicting both reality and experimental studies. In contrast, Bray et al. (1994) utilising a FE code with a hyperbolic non-linear elastic constitutive law achieved good agreement between analysis and experiments. Equally successful were the analyses performed by Roth et al. (1982) who made use of the finite difference method (FDM) with an elastoplastic constitutive model, Mohr-Coulomb failure criterion, and strain softening.

Following a thorough review of the literature, we adopted an elastoplastic constitutive model: Mohr-Coulomb failure criterion, with an isotropic strain softening rule for the cohesion c, the friction angle φ, and the dilation ψ. Denoting γ_f the plastic shear strain at which soil reaches its residual strength, we consider c, φ and ψ as linearly decreasing with the total plastic strain to their residual values c_{res}, φ_{res}, and ψ_{res}. Equally important is the "yield" strain γ_y, which depends on the strength parameters as well as on the shear stiffness. Both γ_y and γ_f are calibrated through numerical simulation of the direct shear test. A parametric study of fault rupture propagation in the free field has been conducted (Anastasopoulos & Gazetas, 2004b), and the results were compared with case-histories, experimental results, and earlier numerical studies. Additionally, a Class "A" prediction was conducted before performing centrifuge experiments at the University of Dundee, as part of the "QUAKER" research project (Davies & Bransby, 2004). This verification gives the necessary confidence for using our numerical modelling methodology.

As already discussed, soil conditions in Denizevler did not differ significantly from point-to-point, while the stiffnesses of the structural systems of the 3 buildings can also be considered roughly similar. With the exception of Bldg. 2, which is made of cinder-block walls, the buildings are similar in terms of superstructure: they are of reinforced concrete with typical column grid in the order of 5 x 5 m having strong infill brick walls. They mainly differ in the number of stories and in the foundation system. Without underestimating the importance of the details of each superstructure, we treat all structures "equivalently" in this respect, changing only the number of stories. This way it is easier to develop insights on the influence of the type and stiffness of the foundation, and on the effect of the structural load on FR-SFSI. Therefore, a typical building width of 10 m and a column grid of 5 x 5 m is utilised. Columns and beams are of 50 cm square cross-section.

5 RESULTS

The results of our FR-SFSI analyses are discussed in terms of the deformed mesh and the distribution of plastic strains. The differential settlement Δy of the foundation and the maximum bending moment M_{max} in the superstructure (beams or columns) are also reported to provide an estimate of the relative distress of each structure.

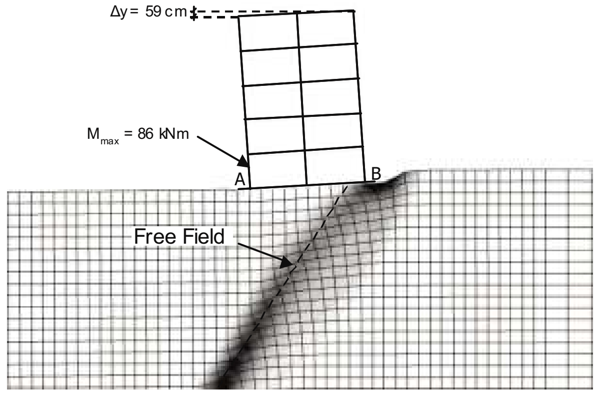

Figure 5. FE analysis of Building 1 : Deformed mesh and plastic strain

5.1 *Building 1*

As clearly seen in Fig. 5 the rupture path is diverted away from the building (towards the footwall), as it approaches the ground surface (topmost 10 m of the propagation path). As it deviates to the right of the building, the plastic strain does not remain as concentrated as along the free-field rupture path, but is diffused over a wider area. The building tilts towards the hanging wall and the differential settlement reaches 59 cm. Despite this significant differential settlement the maximum bending moment M_{max} in the superstructure does not exceed 86 kNm. The rigid foundation not only managed to divert the rupture, but also allowed the building to rotate essentially as a rigid body, without stressing its superstructure. Although the differential settlement is significant (6 % is much higher than the usually accepted maximum of 1/300), the analysis does not indicate significant distress of the building's superstructure. This agrees fairly well with the observed performance: the building sustained no structural damage. However, in reality, the tilting of the building was not as large as the predicted. We identify two possible explanations: (i) post-seismic consolidation near-the-edge of the building due to the increased contact stresses under that part, (ii) the rupture did not cross the structure perpendicularly as assumed in our analysis: it intersected only at the corner of the building, which is more favorable than our plane strain assumption.

5.2 *Building 2*

The model is only an approximation of the actual cinder-wall superstructure. The rupture is only locally diverted towards the hanging wall to avoid the far-left "footing" of the building (Fig. 6). The dislocation follows the same propagation path as in the free field, with the exception of the top 4 m. The building tilts towards the hanging wall, with the differential settlement reaching 33 cm. Part of the edge footing looses its support from the ground. Despite the smaller differential settlement, M_{max} reaches 469 kNm. Evidently, such a distress could not be accommodated by the cinder walls of this structure. Again, FR-SFSI does not affect either the path of dislocation, or the deformations along the surface. We can safely argue that the analysis agrees quite well with the observed performance, despite the crude modeling of the superstructure.

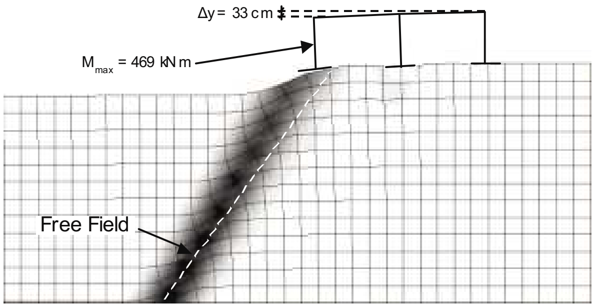

Figure 6. FE analysis of Building 2 : Deformed mesh and plastic strain

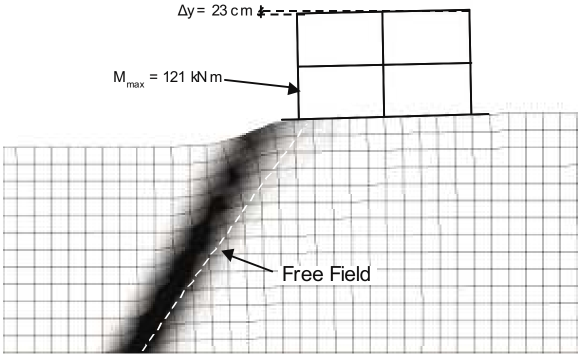

Figure 7. FE analysis of Building 3 : Deformed mesh and plastic strain

5.3 *Building 3*

Until the rupture reaches a depth of about 12 m it follows the same propagation path as in the free field (Fig. 7). Then it is diverted to the left of the building, towards the hanging wall. The plastic strain seems to be quite localized and a distinct fault scarp is predicted numerically. The building tilts slightly towards the hangingwall and the differential settlement does not exceed 23 cm. Despite the considerable differential settlement the maximum M_{max} only reaches 121 kNm. Again, as in the case of Bldg. 1, the rigid box-type foundation not only succeeds in diverting the dislocation, but it also "converts" the differential displacement to a rigid body rotation. Although the differential settlement is an appreciable 2 %, no sign of distress is predicted for the building. One must realize that despite the commonly accepted 1/300 rule of desired maximum tilting, a 2% tilting is not easily observable and as seen in the article of Charles & Skinner (2004) would not cause any structural distress in buildings on stiff rafts. Of interest are some additional examples from the Kocaeli (Turkey) earthquake. For instance, there were many buildings in Adapazari with post-seismic tilting of about 3^{o} (tilting ≈ 5%), or more, that exhibited absolutely no structural damage. This is always the case when the foundation is rigid enough to keep the differential settlement only in the form of rigid-body rotation. As a conclusion, our FR-SFSI analysis agrees well (at least qualitatively) with the observed performance of Building 3.

6 CONCLUSIONS

The main conclusions of our study are as follows :

1. Several buildings with different foundations were subjected to the real-scale natural "experiment" of Denizevler. The diversity of their foundations, as well as the crossing geometry being different in each case, provides a unique case history of FR-SFSI.
2. Buildings on rigid box-type foundations may divert the surface rupture from emerging underneath them. Even if the diversion is partial, the rigidity of such foundations "spreads" the deformation and allows the structure to rotate as a rigid body, without experiencing significant distress. The structure may locally separate from the supporting soil, and may thus be relieved from the imposed displacements.
3. Buildings on isolated footings can only very locally divert the rupture (to avoid emerging right beneath the footing). The rupture outcrops within the limits of the structure, imposing substantial differential displacements and disastrous structural distress. Tie beams can partially ameliorate the performance of buildings founded on separate footings.
4. Even moderately reinforced buildings are proven capable of performing as cantilevers bridging locally-generated gaps, provided that they are founded on rigid foundation systems. Buildings 1 and 3 are real examples of this encouraging performance.

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