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CASE HISTORIES OF FOUNDATIONS ON TOP OF A RUPTURING NORMAL FAULT DURING THE KOCAELI 1999 EARTHQUAKE

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ABSTRACT

Although the Kocaeli (Turkey) earthquake was triggered by reactivation of the North Anatolian strike-slip Fault, normal faulting also occurred within the pull-apart basin of Gölcük. Its effect on overlying structures is summarized in this paper. The normal rupture caused maximum vertical displacements of up to 2.5 m. Several structures were crossed by the rupture. As expected, many of them either collapsed or were severely damaged. Surprisingly, several structures survived the dislocation with no damage, while in some cases the rupture deviated, “avoiding” the structures. The foundations of the involved structures comprised a variety of types, ranging from simple separate footings to box foundation and 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).

INTRODUCTION

The disastrous $M_w = 7.5$ August 17 1999 earthquake was triggered 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 thousands of 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°) 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 numerically to reveal the main aspects of Fault Rupture – Soil Foundation Structure Interaction (FR-SFSI).

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 (2007(a)).

Figure 1 illustrates a plan sketch of the investigated area along with the surface trace of the dislocation. As depicted in the figure, the rupture emerged at the surface creating fault scarps of up to 2.4 m. In contrast, in some cases the dislocation could not be easily identified seeming to disappear, converted to widespread differential settlement of the ground surface rather than a distinct scarp. From east to west, a first impressive (even though perhaps merely fortuitous) success was that of a high-voltage electricity pylon : crossed by the fault rupture, the pylon did not collapse, sustaining only minor damage despite the “loss” of two of its four supports.

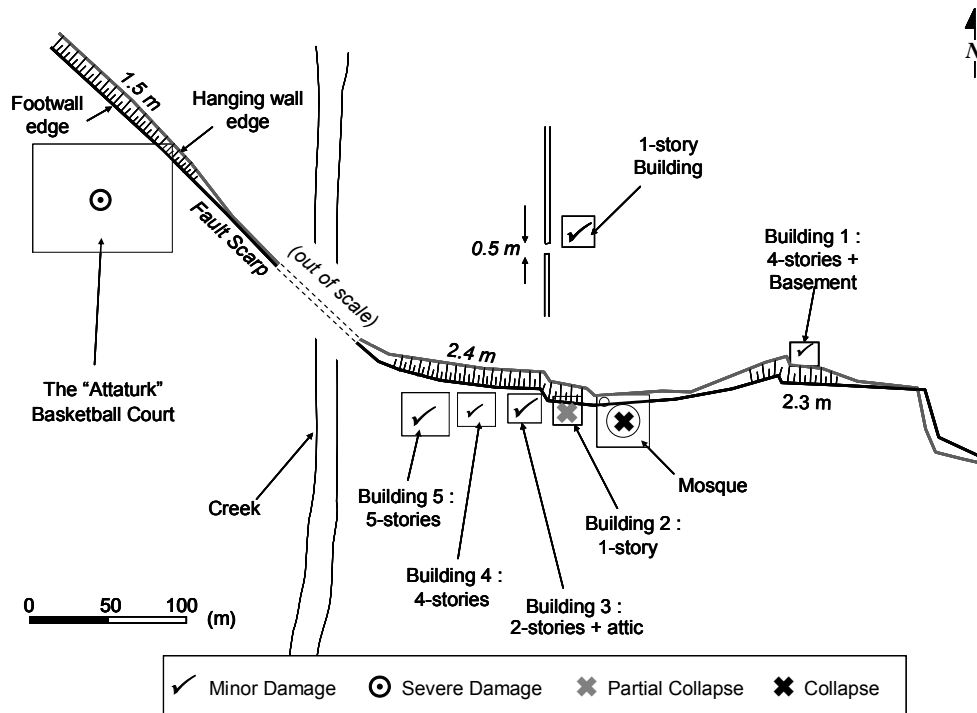


Figure 1. Plan sketch of Denizevler (east of Gölcük). The fault trace crossed a number of buildings, a mosque, and a basketball court. The extent of damage ranged from minor to full collapse. Values indicate the vertical component of the fault offset at the ground surface.

Then, a major building of an under-construction car factory of Ford also survived the faulting, but with damage. Further west, a 4-story building (denoted as Building 1) on the hanging wall, sustained no damage at all, with the fault rupture deviating around it. To the west, the Mosque was heavily damaged and demolished later. Next to it a 1-story lightly-founded building (Building 2) was literally cut by the fault and partially collapsed. Building 3 (2 stories + attic) remained on the un-moved “footwall” block and showed no damage at all, avoiding a direct “hit” thanks to diversion of the rupture path. The next two buildings (4 and 5), of 4 and 5 stories respectively, also did not suffer any visible damage. Then, further to the west, the rupture crossed a small creek heading to the “Ataturk” Basketball Gymnasium. This recently built facility, despite its “sophisticated” piled foundation, sustained substantial (but very local) damage.

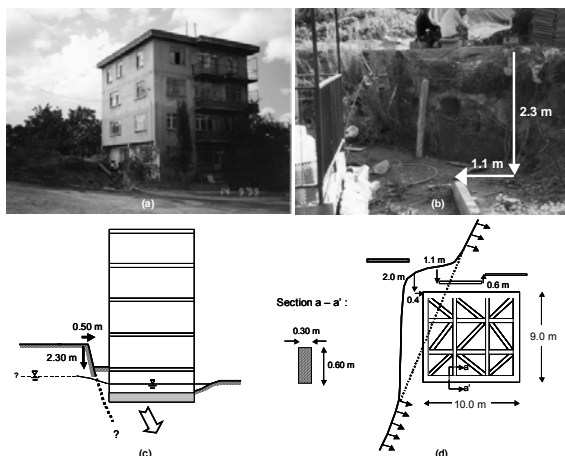


Figure 2. Building 1, four stories plus basement : Minor Damage

Building 1

As depicted in **Figure 2** 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 x ~0.3 m (height x width) transversely connected through tie beams of similar dimensions.

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 (**Figure 3**). However, it did not collapse completely, not causing fatalities. The rupture crossed its north-east corner tearing it apart from the rest.

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 (**Figure 4**). The vertical

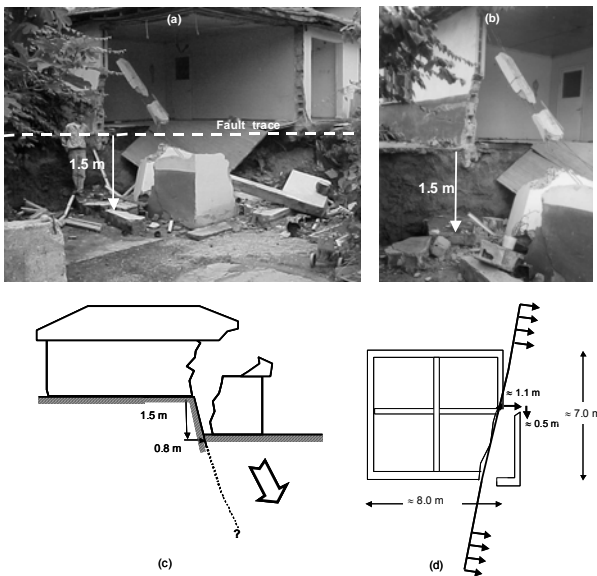


Figure 3. Building 2, one story cinder-block structure : Collapse

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.

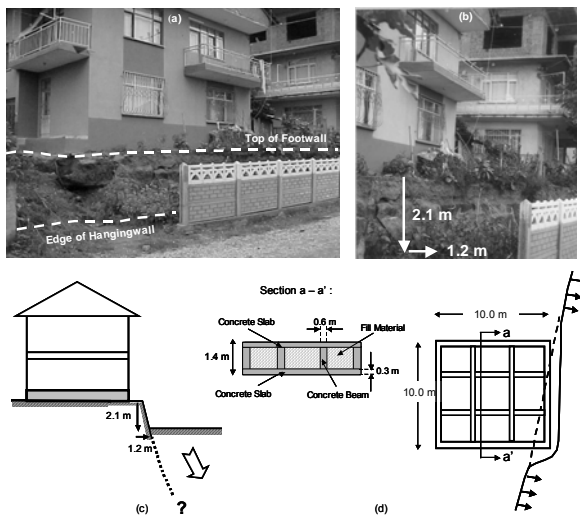


Figure 4. Building 3, two stories plus attic : No Damage

The Mosque

As shown in **Figure 5**, in the area of the Mosque (less than 150 m west of Building 1) the rupture did not create a visible fault scarp. On the contrary, the dislocation appeared at the surface as a widespread differential settlement, not easily observable. Despite this seemingly “favorable” situation, the Mosque partially collapsed, and was fully demolished later. The

superstructure of the mosque was also of reinforced concrete, but its foundation comprised several isolated footings, apparently without any connection between them. No shear walls or stiff tie beams existed between the columns. In conjunction with its rather “heavy” arched roof, its structural system was less stiff than that of Building 1. Its foundation is, obviously, discontinuous and thus quite flexible. Hence, the differential settlements were transmitted to the superstructure practically unaltered. We have to exclude the intensity of ground shaking from being a principal cause of the collapse, since the observed damage and cracking did not indicate horizontal shear failure. The minaret of the mosque confirms this hypothesis: in most of the regions where ground shaking was the main cause of damage, the minarets of the Mosques were quite susceptible to collapse. In this case, the minaret did not collapse.

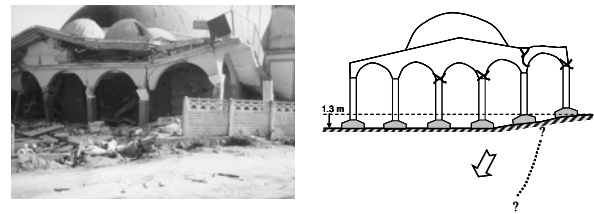


Figure 5. Mosque : Collapse

In fact, no indication of damage due to intense ground shaking was observed near the fault rupture. Such damage was observed further away and to the south of the dislocation, but not so much next or to the north of it. This constitutes an indirect indication that ground shaking may have been stronger on the footwall—in contrast to a rather prevailing opinion. The residents of the area tend to agree with such an allegation: the ones residing on the hanging wall were not as terrified as the ones living to the south of it, on the footwall. However, the validity of such behaviour cannot be confirmed at present. Reliable data would be required, such as several ground motion records on both sides of the fault.

The “Ataturk” Basketball Court

This Basketball Court, had just been constructed when the 1999 earthquake struck. As shown in **Figure 6a** the rupture crossed its northeastern corner causing significant local damage to its reinforced concrete superstructure. **Figure 6b** shows the southern part of the building, which sustained practically no damage. Again, the damage can be attributed solely to differential tectonic displacement and not strong seismic shaking. **Figure 6c** depicts the extent of damage suffered by the northeastern part of the building, near the corner struck by the dislocation. Several of its concrete shear walls failed, while its non-bearing brick walls were diagonally cracked, indicating tensile failure at 45° due to differential settlement. **Figure 6d** depicts the damage to the piled foundation. The pile at the photo had been pulled downward and outward, and tensile cracking was easily observable. Its adjacent pile (not seen in the photo) had failed in tension completely, and was totally detached from the pile cap.

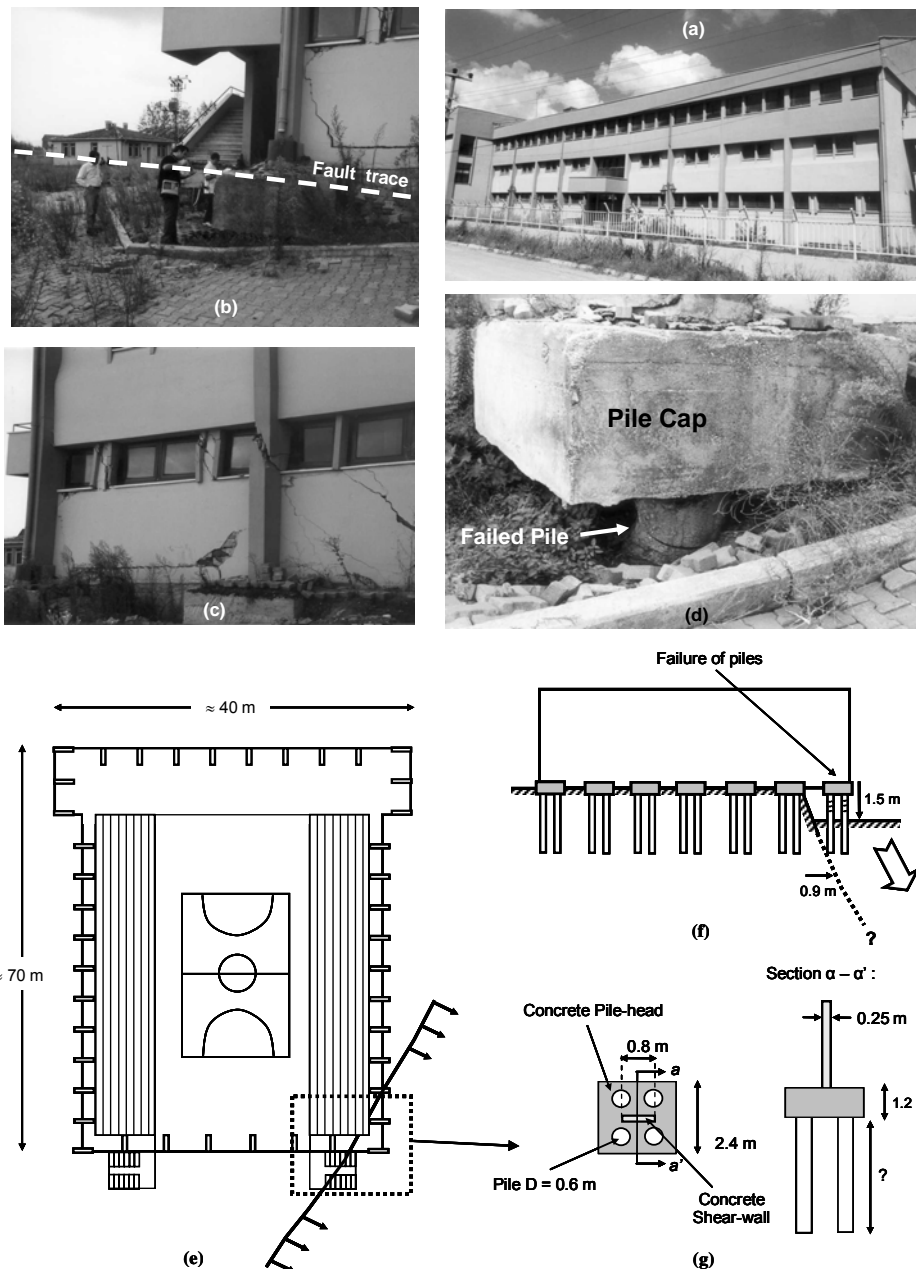


Figure 6. Basketball Court : Severe Local Damage

The plan view of the Basketball Court is sketched in **Figure 6e**. Its structural system comprised shear-wall type columns 0.25 m x 0.80 m in plan, positioned along the perimeter. As depicted in **Figure 6f**, each column is founded through a 2 x 2 pile group. The piles are 0.6 m in diameter, connected together through a 2.4 m-square pile cap, 1.2 m in thickness. Although the building survived the induced differential displacement, the extent of damage was quite significant; the structure was deemed as “beyond the limit of repair”. It can be argued that this constitutes a case where the piled foundation possibly contributed to the damage, by forcing the superstructure to follow the imposed displacement. In fact, if the piles had not failed (in tension), the situation might have been even worse!

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 (Pavlidis, 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.

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. The analysis is conducted in two steps as illustrated in **Figure 7**. First, fault rupture propagation is analysed in the free field, ignoring the structure.

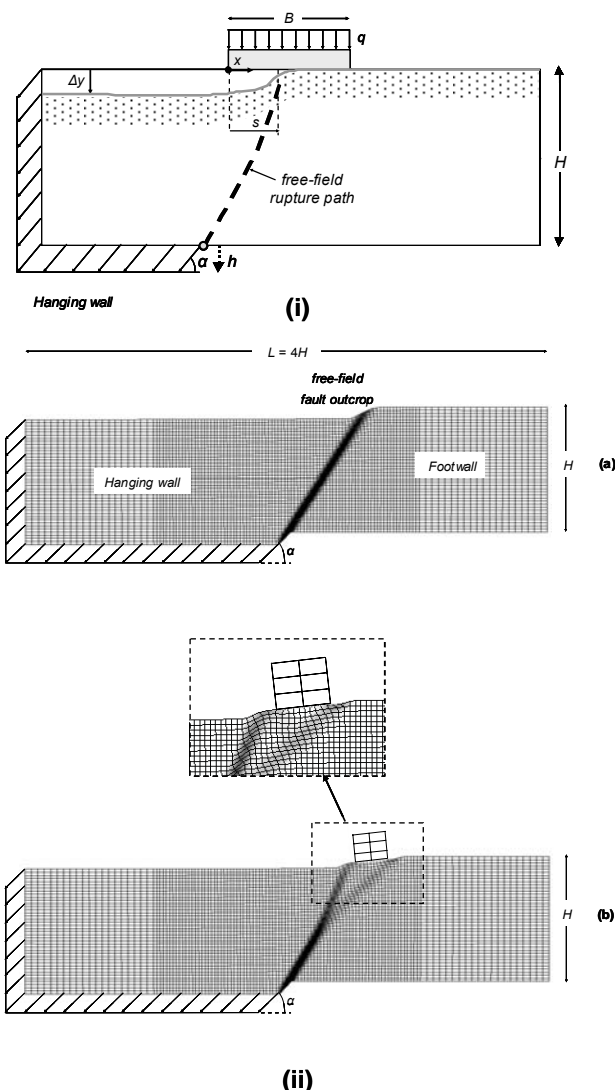


Figure 7. (i) Configuration of the soil–foundation system subjected ; (ii) Finite element discretisation and the two steps of the analysis: (a) fault rupture propagation in the free-field, and (b) interplay between the outcropping fault rupture and the structure.

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 **Figure 7(b)**, 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.

Several experimental and numerical studies have shown that soil behaviour after failure is decisive in rupture propagation. Early attempts 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 analyses making 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 et al, 2007c), 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.

A typical result elucidating the interplay between loose ($D_r = 45\%$) soil, rupture path, and a perfectly rigid foundation carrying a 4-storey structure is given in **Figure 7(d)**. A base rock dislocation of 2 m (5% of the soil thickness) is imposed. The structure is placed symmetrically–straddling the free-field fault breakout (i.e. the foundation is placed with its middle coinciding with the location where the fault would outcrop in the free field). Yet, a distinct rupture path (with high concentration of plastic shearing deformation and a resulting conspicuous surface scarp) is observed only in the free–field. The presence of the

structure with its rigid foundation causes the rupture path to bifurcate at about the middle of the soil layer. The resulting two branches outcrop outside the left and the right corner of the foundation, respectively. The soil deformations around these branches are far smaller and diffuse than in the free-field, and the respective surface scarps are much milder. Thanks to the substantial weight of the structure and the flexibility of the ground, the structure settles and rotates as a rigid body. The foundation does not experience any loss of contact with the ground ; apparently, the foundation pressure is large enough to eliminate any likely asperities of the ground surface.

As a result of such behaviour, the structure and its foundation do not experience any substantial distress, while their rotation and settlement could perhaps be acceptable. The main factors influencing FR-SFSI are :

- The style of faulting (normal, thrust, strike-slip), the angle of dip and the offset (dislocation) at the basement rock.
- The total thickness (H) of the overlying soil deposit, and the stiffness (G), strength (ϕ , c) and kinematic (ψ) characteristics of the soil along the depth.
- The type of the foundation system (for example, isolated footings, mat foundation, box-type foundation, piles, caissons).
- The flexural and axial rigidity of the foundation system (thickness of mat foundation cross-section and length of tie beams, etc.)
- The load of the superstructure and the foundation.
- The stiffness of the superstructure (cross section of structural members, spacing of columns, presence or not of shear walls).
- The location S from the foundation corner to the free-field outcrop.

However, a detailed investigation of the role of all the above parameters is beyond the scope of this chapter. Reference is made to Anastasopoulos (2005) and Anastasopoulos & Gazetas (2007, b) for such a parameter study. Here we only outline a few characteristic results pertaining to a 20 m wide rigid mat foundation, supporting a 2-storey building frame. The soil layer is either loose ($D_r \approx 45\%$) or dense ($D_r \approx 80\%$) sand of total thickness $H = 40$ m. Three locations of the foundation with respect to the free-field outcrop are considered : $S = 4$ m, 10 m, and 16 m, i.e. near the left edge, in the middle, and near the right edge of the foundation, respectively.

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.

SUMMARY OF NUMERICAL 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.

Building 1

As clearly seen in **Figure 8**, 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.

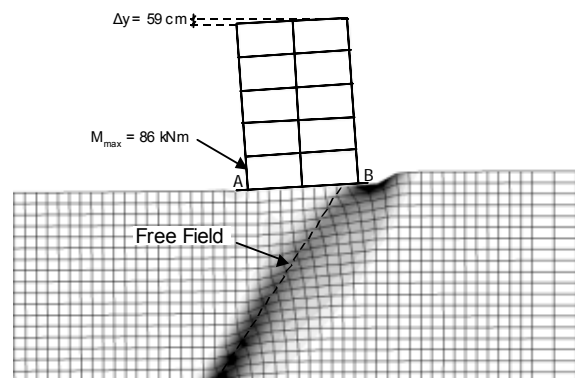


Figure 8. FE analysis of Building 1 : Deformed mesh and plastic strain

The rigid foundation not only diverted 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.

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 (**Figure 9**). 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 loses 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.

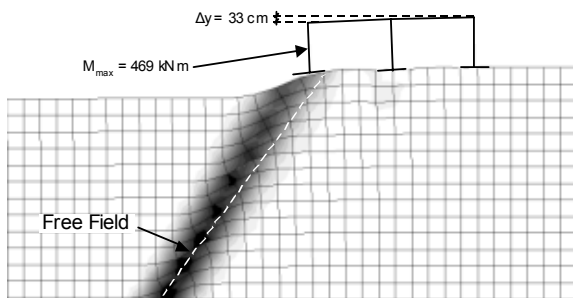


Figure 9. FE analysis of Building 2 : Deformed mesh and plastic strain

Building 3

Until the rupture reaches a depth of about 12 m it follows the same propagation path as in the free field (**Figure 10**). 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 hanging wall 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° (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.

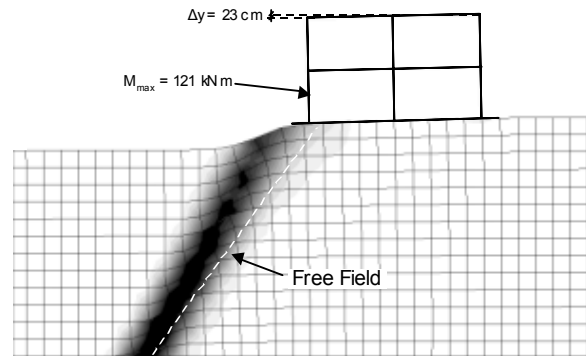


Figure 10. FE analysis of Building 3 : Deformed mesh and plastic strain

The Mosque

Our analysis results for the Mosque are presented in **Figure 11**. Admittedly, this is not a faithful representation of the structure, but one that roughly captures the stiffness characteristics of the superstructure and its foundation. The deformed mesh reveals that the rupture follows its original (free-field) path, almost unaltered by the presence of the structure. In contrast to Building 1, where a fault scarp can be clearly identified to the right of the structure, one can now see most of the deformation taking place between the isolated footings of the Mosque, with a diffuse failure zone.

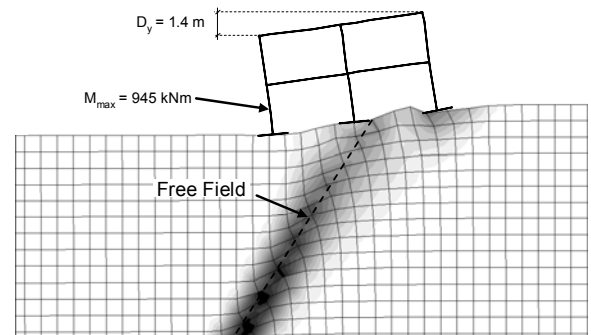


Figure 11. FE analysis of the Mosque : Deformed mesh and plastic strain

The footings only barely divert the rupture from emerging directly beneath them, but not beyond the limits of the structure. The Mosque is tilting towards the hanging wall with the differential settlement D_y reaching 1.4 m. Unlike the previous case, D_y is not “absorbed” by the rigidity of the foundation. The Mosque not only rotates as a rigid body, but is also substantially distressed (tilting with significant distortion). The maximum bending moment M_{max} in its structural elements reaches 945 kNm. Such stressing would certainly cause collapse, given the dimensions and reinforcement of its structural members. The vertical displacement Δy , the distortion β , and the hori-

zontal strain ε_x , all clearly indicate that very little interaction takes place between the rupturing plane and the structure. In other words, FR-SFSI is hardly affecting the emergence of the rupture on the ground surface. Compared to the free field, the maximum distortion β remains almost unaltered, and occurs at about the same location. The horizontal tensile strain is spread over a wider area, but its peak is almost half of the free-field. In conclusion, the FR-SFSI analysis agrees quite well with the actual performance of the Mosque.

Basketball Court

Our FR-SFSI analysis results for a small part of the Basketball Court are summarized in **Figure 12**. Note that the dislocation follows its free-field propagation path up to the vicinity of the corner pile, at a depth of about 10 m. It is then strongly diverted towards the hanging wall (to the left of the building). Plastic strain is localized in a very narrow band and a distinct fault scarp develops right next to the pile. The building tilts slightly towards the hanging wall with the differential settlement D_y not exceeding 7 cm, while at the same time the left pile cap loses contact with the ground — in accord with our field observations. Surprisingly, despite the relatively minor D_y , the distress of the superstructure is quite substantial: M_{max} reaches almost 400 kNm. Although the piles divert the dislocation, some differential settlement and, especially, differential extension takes place between the columns of the structure. This small but non-negligible deformation is imposed on the superstructure by the piles. The latter are being pulled down and out (even if slightly) by the downward moving hanging wall, thereby forcing the superstructure to follow. In contrast to the *continuous and rigid* box foundation of Buildings 1 and 3, the *discontinuous* piled foundation does not allow the superstructure to rotate as a rigid body without being distorted.

In conclusion, our analysis predicts significant distress at the corner of the Basketball Court, agreeing well with its actual performance. However, the limitations of our model for the piled foundation must be clearly spelled out. The plane strain assumption implies that our “piles” are in (the computational) reality continuous “walls” (diaphragm type). Such walls are subjected to higher normal actions (per unit length) from the downward and outward moving soil than individual piles. This is because : (i) soil can “flow” around the piles, but not around the plane “wall” ; (ii) the frictional capacity of the pile-soil interface is not unlimited, as implicitly assumed in our “bonded” model, thus making the downward “flow” of the soil even easier ; and (iii) in reality the corner piles failed in tension, thus reducing their pull-down of the superstructure (allowing it not to follow ground deformation completely). Nevertheless, in a qualitative sense the results of our (admittedly imperfect) analysis reveal the trends that were observed in the field.

To further investigate the role of the piled foundation, we analysed the same building but with a *continuous and rigid* box-type foundation instead of piles. The dislocation was again diverted towards the

hanging wall, but was more diffuse and created a smoother surface settlement profile. The rupture followed its free field propagation path from the base rock up to a depth of about 12 m. It then diverted towards the hanging wall (to the left of the building). The distortion angle was much less localized compared to the piled alternative, maintaining essentially the same value as in the free field. The building now tilted more, with the differential settlement D_y reaching 57 cm. Nevertheless, as expected, the superstructure was not distressed : $M_{max} = 121$ kNm, only. The Basketball Court could most likely have behaved better had it been founded on a continuous rigid box-type or raft foundation, rather than on piles, in this specific case of the fault rupturing near the corner.

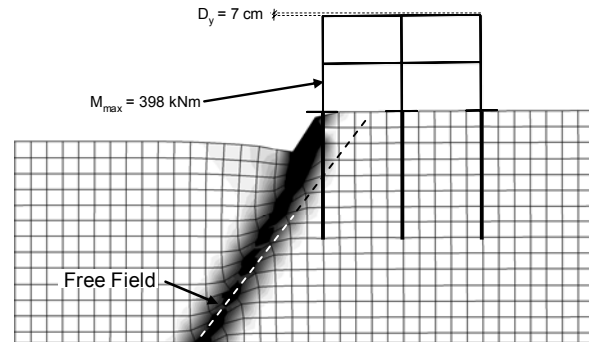


Figure 12. FE analysis of the Basketball Court : Deformed mesh and plastic strain

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.

On the basis of additional parametric analyses and field observations from several earthquakes the following recommendations are made for future seismic codes for structures on active faults:

- (a) Building in the vicinity of active seismic faults could be allowed only after a special seismotectonic–geotechnical–structural study is performed. In this study the effects of all faults in the vicinity of the structure shall be investigated, and measures shall be taken to face their rupturing effectively.
- (b) The exact location of surface outcropping of a seismically active fault cannot always be predicted with accuracy. Therefore, its relative location to the structure shall be analysed parametrically. The uncertainty on the size of fault displacement should also be considered.
- (c) The presence of a structure may lead to diversion of the rupture path, as well as to modification of the surface displacement profile caused by the emerging fault rupture. Depending on the rigidity, continuity, and weight of the foundation–structure system, even a complete diversion of the fault path may take place. Additionally, depending on how soft/loose the soil is a distinct (and steep) fault scarp may be diffused by the structure to a widespread differential settlement. Hence, soil–foundation interaction should be taken into account in the design of structures in the vicinity of active faults.
- (d) The foundation type plays a crucial role in the response of a structure to fault-induced displacement. Continuous and rigid foundation systems, such as rigid mat or box-type foundations, are advantageous and should be preferred. Isolated footings should in general be avoided. Even if the weight of the structure is enough to cause diversion, the lack of continuity may lead to fault outcropping within the limits of a structure. If used, isolated footings should always be connected with rigid tie-beams.
- (e) Piled foundations, if required, should be designed with special care. They tend to “force” the structure to follow the fault-induced displacement. They should be combined with rigid and continuous pile cap. “Isolating” the pile from the potentially downwardly moving soil should be explored.
- (f) For bridge structures, where foundation continuity is not possible (each pier is founded on a separate foundation), continuous superstructure systems are disadvantageous and simply supported superstructures are preferable. Special care should be taken to avoid deck collapse due to excessive relative displacement.
- (g) In the case of underground structures, such as bored and cut-and-cover tunnels, “open” cross-sections should be avoided. In cut-and-cover tunnels, the weight of the fill (cover) plays a

significant role and should be taken into account.

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