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Correlation between ground failure and soil conditions in Adapazari, Turkey

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Abstract

Ground failure in Adapazari, Turkey during the 1999 Kocaeli earthquake ($M_w = 7.4$) was severe. In four central downtown districts, where more than 1200 buildings collapsed or were heavily damaged, hundreds of structures tilted and penetrated into the ground due in part to liquefaction and ground softening. Based on a multi-institutional subsurface investigation program, soil conditions along four lines in which ground failure was surveyed after the earthquake are classified into four generalized subsurface site categories. This classification is primarily based on the presence or absence of shallow and intermediate depth liquefiable soils. Observations of ground failure are found to correlate well with site categories that are susceptible to liquefaction according to current state-of-the-art methods without strict adherence to the Chinese criteria. Soils that liquefied were found to meet the liquid limit and liquidity index conditions of the Chinese criteria. However, soils that liquefied did not typically meet the clay-size condition for liquefiable soils by the Chinese criteria.

Keywords: Earthquake; Ground failure; Liquefaction; Cone penetration test; Standard penetration test; Building performance; Silt

1. Introduction

The August 17, 1999 Kocaeli, Turkey earthquake ($M_w = 7.4$) caused severe damage to hundreds of structures and lifelines in the City of Adapazari. According to data provided by the Turkish Federal Government, a total of 5078 buildings (27% of the building stock) were severely damaged or destroyed in Adapazari [1]. A large number of modern reinforced concrete buildings, generally of 3–5 storeys, penetrated the surrounding ground or tilted due in part to liquefaction and ground softening. Many of these buildings also had significant structural damage.

In a joint effort of the University of California at Berkeley, the University of California at Los Angeles, Brigham Young University, Sakarya University, ZETAS Corporation, Middle Eastern Technical University, and Boğaziçi University, an extensive field investigation program was conducted that included the documentation of structural and soil conditions at selected sites affected by ground failure throughout Adapazari [2,3]. The soil characterization included subsurface investigations by means of standard penetration tests (SPT) and cone penetration tests (CPT).

This paper focuses on preliminary findings from this ongoing study in which observed foundation penetration, as well as lack there of, is correlated with subsurface conditions. Subsurface conditions have been classified into four general subsurface site categories. The ground failure and liquefaction potential of each of the site categories and failure mechanisms that might have led to the observed building performance are discussed.

2. The city of Adapazari

Adapazari, the capital of the Sakarya Province, is home to approximately 180,000 people. The heart of the city lies in a fertile plain formed by recent fluvial activity of the Sakarya and Çark rivers, giving the city its name, i.e. 'island

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market'. The city is densely developed in most areas, primarily with 3–5 storey reinforced concrete frame buildings and older 1–2 storey timber/brick buildings. Reinforced concrete construction is primarily non-ductile, with shallow, reinforced concrete stiff mat foundations located at depths of typically 1.5 m due to shallow groundwater.

Most of the city is located over deep alluvial sediments. A deep boring recently performed in Yenigün District by the Federal Dam Agency (D.S.I.) did not reach bedrock at a depth of 200 m. The shallow soils (< 10 m) are recent deposits laid down by the Sakarya and Çark rivers, which frequently flooded the area until flood control dams were built recently. Sands accumulated along bends of the meandering rivers, and the rivers flooded periodically leaving behind predominantly non-plastic silts, silty sands, and clays throughout the city. Clay-rich sediments were deposited in lowland areas where floodwaters ponded [4].

3. Overview of damage from the 1999 Kocaeli earthquake

Buildings in Adapazari were strongly shaken by the Kocaeli earthquake. The Sakarya station recorded a peak horizontal (east–west) ground acceleration (PGA), velocity, and displacement of $0.41g$, 81 cm/s, and 220 cm, respectively. The Sakarya station is located in southwestern Adapazari at a distance of 3.3 km from the fault rupture. It is situated on the floor of a small one-storey building (with no basement) and is underlain by a shallow deposit of stiff soil overlying bedrock (average $V_s \sim 470$ m/s in upper 30 m of the soil and rock profile [5]. Downtown Adapazari is located at a distance of about 7 km from the fault rupture, and due to softer ground conditions, amplification of long-period components of the ground motion would be expected. Main shock ground motions recorded at similar site-source distances suggest that the PGA in Adapazari was on the order of $0.3–0.4g$.

Rapid damage surveys were performed along four lines across the city [1]. A total of 719 structures were mapped in Adapazari, which is about 4% of the building stock. The structural damage mapping was performed according to the system described by Coburn and Spence [6], where each building is assigned a structural damage index ranging from D0 (no observed damage) to D5 (complete collapse of the building or a storey within the building). Information on observed vertical building displacement or penetration relative to the adjacent ground, tilt, lateral movement, and eruption of sand boils was compiled by post-earthquake investigators, using the ground failure index described in Table 1. GF0 corresponds to no observable ground failure and GF3 to significant building penetration of more than 25 cm or 3° tilt [1].

Detailed surveys along several lines allowed general trends to be established regarding the relationship between ground failure and building damage. The density and height of construction was fairly consistent along the lines, so that variations in damage intensity are statistically meaningful. Some localities with severe ground failure also had significant structural damage, while others had only moderate structural damage. However, there were no broad areas with ground failure and only light structural damage. The compiled data indicate that the severity of structural damage generally increases with the level of ground failure [1]. Sand boils were observed within some of the ground failure zones, but were not widespread, and were absent from many areas.

Table 1
Ground failure index classification system [1]

Index	Description	Interpretation
GF0	No observable ground failure	No vertical movement, tilt, lateral movement, or boils
GF1	Minor ground failure	Vertical movements, $\Delta < 10$ cm; tilt of > three-storey buildings < 1°; no lateral movements
GF2	Moderate ground failure	$10 < \Delta < 25$ cm; tilts of 1–3°; small lateral movements (< 10 cm)
GF3	Significant ground failure	$\Delta > 25$ cm; tilts of > 3°; lateral movements > 25 cm

A map of four central districts located in the downtown area of Adapazari is reproduced in Fig. 1. Also shown on the map are survey lines 1–4, along which building damage indices were recorded about 2 weeks after the August 17, 1999 Kocaeli earthquake. Subsurface soil conditions along these survey lines were developed through the analysis and interpretation of the 59 CPTs and 15 exploratory borings with closely spaced SPT performed along these lines for this study [3].

According to the damage statistics compiled by the Turkish Federal Government [1], the four districts mapped in Fig. 1 are among those most heavily damaged (see Table 2). A total of 1249 buildings either collapsed or were heavily damaged, accounting for 33% of the building stock. However, within these districts, areas of low structural damage and ground failure were also observed [1].

A total of 196 buildings were surveyed along the four lines shown in Fig. 1. Of these 196 structures, 134 (68%) are 3–5 storey buildings, 43 (22%) are 1–2 storey buildings and the remaining 19 (10%) are six storey buildings. A total of 48 buildings, or 25% of the total, were reported to have structural damage indices of either D4 (partial collapse) or D5 (collapse). Of the 48 buildings that suffered partial or total collapse, 34 structures (71%) are 3–5 storey buildings, 11 (23%) are 1–2 storey buildings, and the remaining 3 (6%) are six storey buildings. Hence, there is no apparent influence of the number of storeys

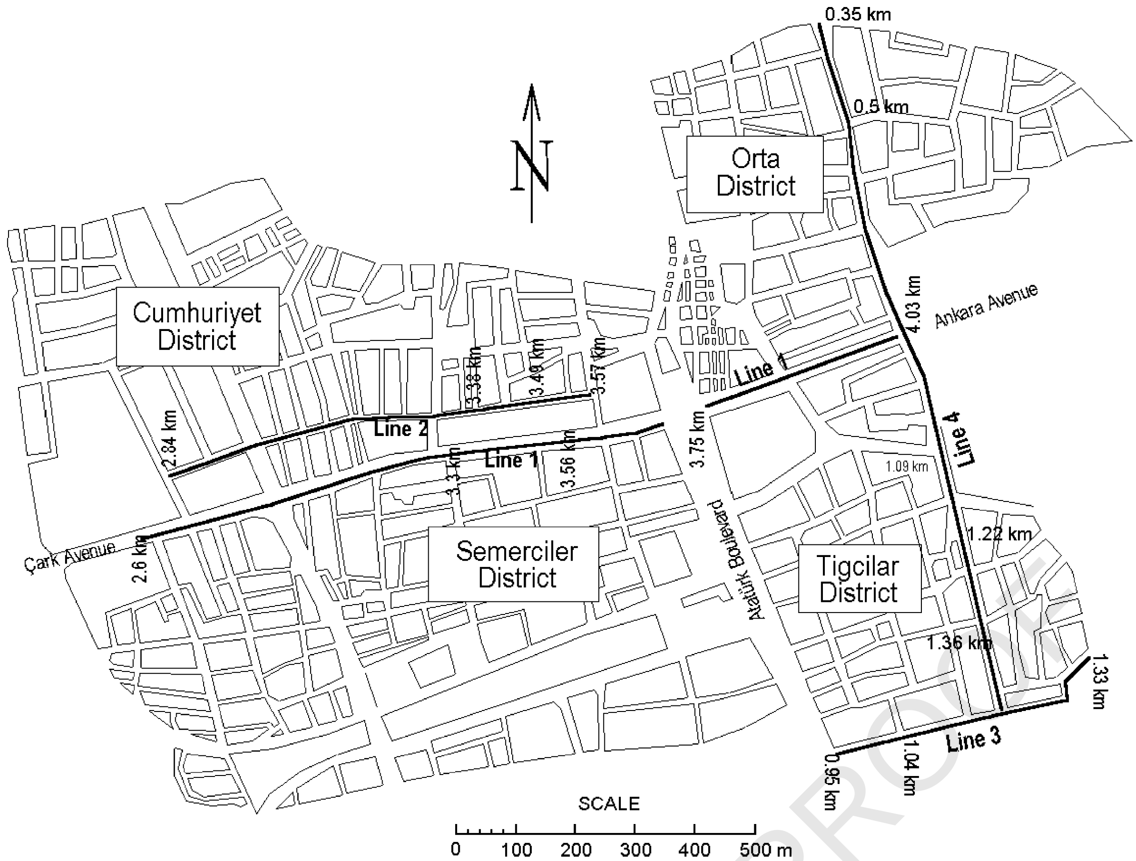


Fig. 1. Map of four downtown districts in Adapazari and location of survey lines.

(or building height) on the rate of collapse of the buildings surveyed in Adapazari.

Conversely, of the 48 buildings that suffered moderate to significant ground failure (GF2 and GF3; see Table 1), 37 structures (77%) are 3–5 storey buildings, 3 (6%) are 1–2 storey buildings, and the remaining 8 (17%) are six storey buildings. Hence, relative to the number of buildings of various storeys, the buildings with 3–6 storeys exhibited a higher incident of ground failure than the 1–2 storey buildings. Moreover, post-earthquake investigators noted that ground failure was less prevalent in the open fields away from buildings [1]. Thus, observations of ground failure appear to be more prevalent for taller buildings, suggesting that building height or weight contributed to the severity of ground failure in Adapazari.

Table 2
Damage statistics in central districts of Adapazari (source: Turkish Federal Government [1])

District	Collapse/heavy damage (%)	Medium damage (%)	Light damage (%)	No of damage (%)	Total buildings
Cumhuriyet	22	13	26	39	837
Semerciler	33	9	23	35	1855
Orta	35	29	27	9	774
Tiğcılar	54	17	12	17	348

4. Soil characterization in Adapazari

During the Summer of 2000, 135 CPT soundings were installed (of which 19 were seismic CPTs) and 46 soil borings with closely spaced SPT were drilled in Adapazari to investigate the subsurface conditions at sites where ground failure was or was not observed. Details of this site investigation program, including downloadable CPT profiles and borehole logs, are available at the Pacific Earthquake Engineering Research Center (PEER) web site: http://peer.berkeley.edu/turkey/adapazari [3]. The procedures outlined by ASTM standards D5778-95, D1586 and D6066-98 were carefully followed to ensure the quality of the data. Additionally, the energy delivered by the driving system during the SPT was measured to obtain accurate values of N_{60}. More detail regarding the equipment and procedures used in this study are reported by Bray et al. [2].

After carefully reviewing the soil profiles obtained from analysis and interpretation of the CPT data and soil index test data from SPT samples, four general subsurface site categories were developed as shown in Fig. 2. These soil profile types are described as follows.

Soil Type 1. This site category is characterized by the presence of brown to reddish brown, loose non-plastic silt and sandy silt in the upper 4 m of the soil column. The thickness of this stratum across the area explored ranges

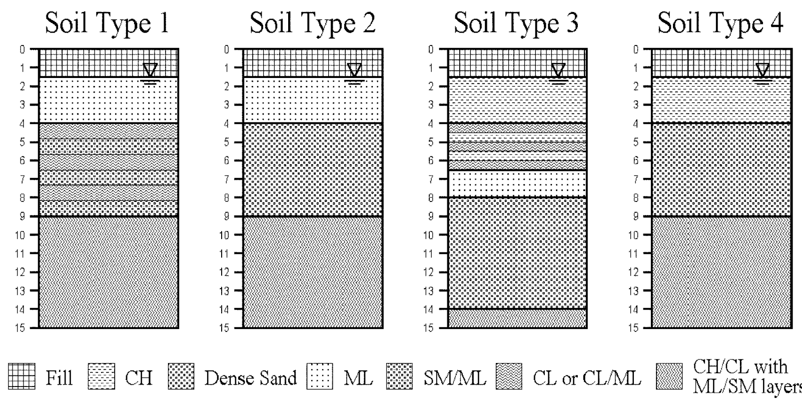


Fig. 2. Generalized subsurface soil profiles in downtown Adapazari.

from 0.5 to 2.5 m. Liquid limit (LL) indices for the silt range from 25 to 35% and its natural water content is generally greater than 0.9LL. The fines content (FC) of the soil samples recovered in this stratum ranges from 52 to 97%, and is generally greater than 75%. The percentage of particles smaller than 5 μm ranges from 10 to 35%, and is normally between 20 and 30%. The corrected penetration resistance of this stratum, $(N_1)_{60}$, ranges from 3 to 15 (blows/30 cm), and is generally between 7 and 10. Organic matter within this material at a depth of 4 m was dated to be approximately 1000 years old, indicating that the upper brown silty materials are recent flood plain deposits that have a high susceptibility to liquefaction [7].

Interspersed strata of low plasticity clays and medium dense to dense silt to sandy silt underlie the upper brown silt. The color of these lower strata transitions from brown to gray at approximately 5 m. At depths greater than about 9 m the soils consist of interbedded clays, silts and sands.

Soil Type 2. This site category is similar to Soil Type 1, however, this category differs from Type 1 in that the soil directly beneath the brown loose silt is replaced by dense ($q_{c1N} > 160$ and $(N_1)_{60} > 30$), gray sand to a depth of approximately 9 m. Samples from this sand layer generally contained less than 5% fines and also contained less than 5% gravel. However, at a few locations retrieved samples had fine gravel contents as high as 27%. The mean grain size (D_{50}) of this stratum ranges from 0.4 to 1.7 mm.

Soil Type 3. The soil between approximately 1.5 and 4 m depth in this type profile is brown highly plastic silty clay underlain by interspersed lenses and layers of low to highly plastic silty clay and clayey silt. These plastic sediments continue to depths of approximately 7 m. The soil's color transitions from brown to gray at approximately 5 m. The values of LL range from 29 to 65%, but most are greater than 35%. Some of these soils have natural water contents that are close to or in excess of the LL. The thickness of each silt layer is generally less than 25 cm, but some were as thick as 50 cm. The combined thickness of these strata range from 1 to 3 m. The penetration resistances of the silts and clay-silt mixtures are low ($q_c < 2$ MPa, $q_{c1N} < 15$ and $(N_1)_{60} < 10$). From approximately 7 m to a depth of 8–9 m the soil is generally gray medium dense ($40 < q_{c1N} < 90$, $12 < (N_1)_{60} < 22$) sandy silt to silty sand interbedded with seams of silty clay. These lower non-plastic silts overlie dense ($q_{c1N} > 160$ and $(N_1)_{60} > 30$) gray sand to sand with silt to a depth of approximately 14 m. Soil samples from this layer contained between 5 and 25% fines. D_{50} from these samples ranged between 0.1 and 0.6 mm, but was predominantly between 0.2 and 0.3 mm. At greater depths the soils consist of interbedded clays, silts and sands.

Soil Type 4. This site category is similar to Soil Type 2, but differs in that the shallow loose brown silt characteristic of Type 2 is replaced by 1.5–4 m of medium to high plasticity silty clay. Thin layers and lenses (thickness < 0.5 m) of low plasticity clayey silt may lie within the upper 4–5 m of the soil column.

5. Liquefaction evaluation

Ground shaking in Adapazari was sufficiently intense to induce free-field cyclic stress ratios (CSR) between 0.3 and 0.4 within the upper 5 m of the soil profiles. Over this depth, SPT and CPT-based cyclic resistance ratio (CRR) curves are fairly straight and nearly vertical [8]. Hence, any uncertainty in estimating the CSR or demand from the 1999 earthquake does not significantly affect the liquefaction triggering assessment, because the evaluation at this level of CSR is not very sensitive to reasonable variations in calculated CSR [2].

Most of the soils in Adapazari contain significant amounts of fines (i.e. $> 35\%$ passing the #200 sieve). The liquefaction-triggering database of the widely used correlations by Seed et al. [9] is dominated by cases for clean sands and silty sands with less than 35% fines. Only 13 cases with more than 35% fines were available when this CRR line was developed in the mid-1980s.

The liquefaction susceptibility of the Adapazari soils was initially evaluated using criteria recommended by Youd et al. [8]. Accordingly, a soil may be susceptible to liquefaction if the CPT-based soil behavior index type, I_c, [10] is < 2.6, or if the Chinese criteria is met. The Chinese criteria, as originally defined by Seed and Idriss [11] and restated in the state-of-the-art liquefaction triggering recommendations of Ref. [8], specify that liquefaction can only occur if all three of the following conditions are met: (1) Percent of soil particles less than 5 μm diameter is less than 15%, (2) the LL is less than 35%, and (3) water content (w_n) is greater than 0.9LL.

Fig. 3(a)–(d) show the values of I_c for representative CPT profiles of subsurface site category types 1–4, respectively, as well as the range of depth for which $I_c < 2.6$. In Fig. 3(a) (Soil Type 1 profile), three zones were identified that predominantly have I_c values below 2.6; these are from 1.8 to 3.2 m (Zone A), from 5.0 to 9.1 m (Zone B), and from 11.6 to 14.3 m (Zone C). Samples recovered in the vicinity of this CPT show that the soil in Zone A is brown, low plasticity silt. Although the characteristics of the samples comply with the LL and w_n/LL criteria of the Chinese criteria (i.e. LL < 35% and LL > 0.9LL), this soil would not be considered liquefiable if the criteria are strictly applied as recommended in Ref. [8], because the amount of particles smaller than 5 μm is generally greater than 15%.

For this study, the clay-size condition of the Chinese criteria will not be considered to preclude a soil from being susceptible to liquefaction, and an abbreviated Chinese criteria based solely on the LL and liquidity index conditions will be employed. [2] found that the use of these two conditions to identify soils susceptible to liquefaction was consistent with field observations, and that the use of the <5 μm condition of the Chinese criteria erroneously classified liquefiable soils as non-liquefiable. This is shown in Fig. 4, which shows soil index properties at Site C, a location where two buildings settled and slid more than 25 cm and sand ejecta was observed (i.e. ground failure index 3). Although the shallow soils that liquefied had LLs generally below 35 and high liquidity indexes, the soils contained more than 15% finer than 5 μm.

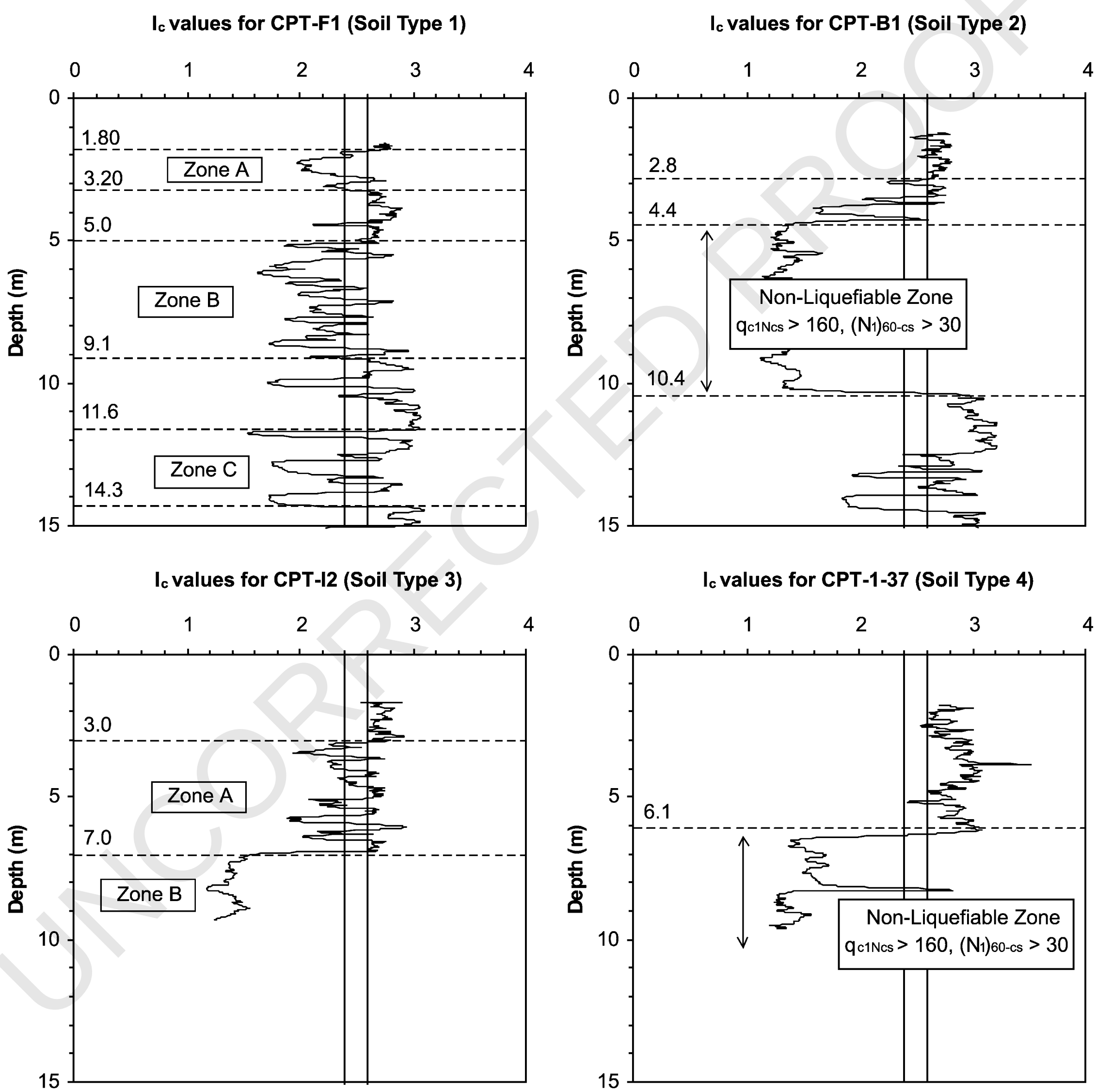


Fig. 3. Representative soil behavior index (I_c) values for (a) Soil Type 1 profile, (b) Soil Type 2 profile, (c) Soil Type 3 profile, and (d) Soil Type 4 profile.

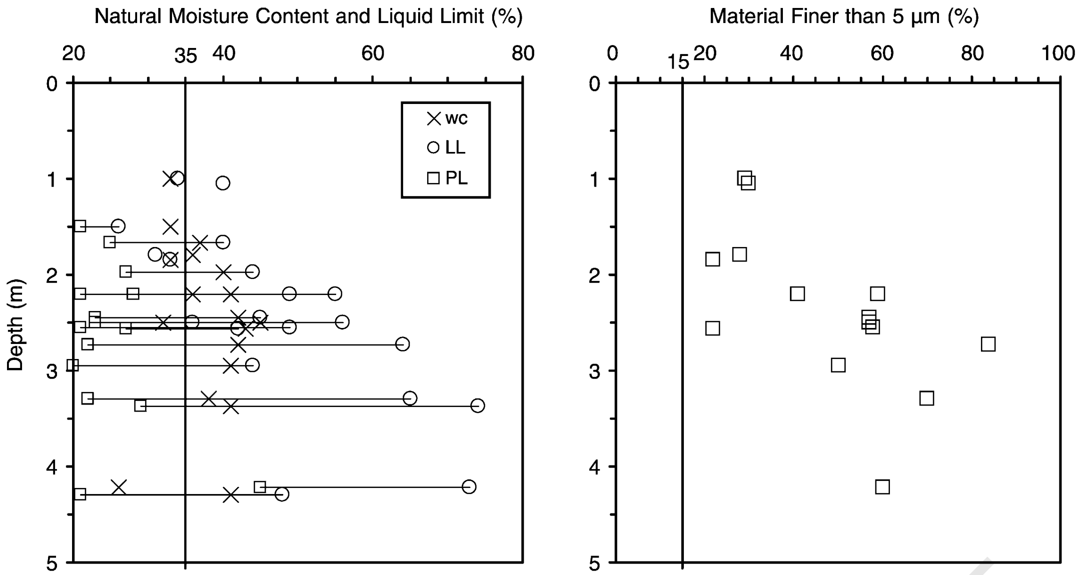


Fig. 4. Soil characteristics based on soil index tests of samples retrieved at Site C in the range of 1.0 and 5.0 m (from data presented in Refs. [2,3].

As mentioned in Section 4, the stratum identified as Zone A has low penetration resistance, hence exhibits low CRR and can liquefy in the free-field at the PGA values that are estimated to have occurred in Adapazari (0.3–0.4g).

The soil strata in Zone B with $I_c < 2.6$ is silty sand to low plasticity silt. Silt samples retrieved from this range of depth have LL = 28–31%, water content near the LL, and clay content (% < 5 μm) close to or smaller than 15%. The penetration resistance of this material is highly variable, hence some of these layers may have liquefied in the free-field during the earthquake perhaps contributing together with the upper silt to ground failure. Finally, the deep sands found in Zone C have high penetration resistance ($q_{c1N} > 160$ and $(N_1)_{60} > 30$) and are too dense to liquefy.

The soil between 2.8 and 4.4 m ($I_c < 2.6$) in Fig. 3(b) (Soil Type 2 profile) has similar index properties and penetration resistance as the soil in Zone A of a Type 1 profile. Samples of potentially liquefiable brown silt from this layer generally met the LL and w_n/LL conditions of the Chinese criteria as mentioned above, whereas the amount of particles <5 μm is mostly greater than 15%. This analysis indicates that the sand strata between 4.4 and 10.4 m is susceptible to liquefaction as I_c is less than 2.6. However, given the high penetration resistance of the soil, this layer should not have liquefied during the earthquake.

The stratified silts and clays between 3 and 7 m, Zone A in Fig. 3(c) (Soil Type 3 profile), are the only sediment in the upper part of this subsurface profile that is susceptible to liquefaction ($I_c < 2.6$). Additionally, this section of the soil column has a penetration resistance for which the CRR is smaller than the expected CSR that was imposed by the earthquake. Some soil samples within this stratum have LL < 35% and w_n/LL > 0.9. Although the amount of particles smaller the 5 μm is consistently greater than 15%, this soil may have liquefied during the earthquake. The stratum of gray poorly graded sand, Zone B in Fig. 3(c), has high penetration resistance characterized by $q_{c1N} > 160$ and $(N_1)_{60} > 30$ and is too dense to liquefy.

With the exception of thin silt lenses at approximately 2.5 and 5 m, the I_c value for the soil profile shown in Fig. 3(d) (Soil Type 4 profile) is greater than 2.6 which indicates a sediment that is too clayey-rich to liquefy. Soil borings near CPTs at similar sites indicate that the upper soils were of too high plasticity to have liquefied. However, weak clayey soils not prone to liquefaction could still be susceptible to significant strength loss upon cyclic loading, so ground failure at these sites cannot be absolutely precluded.

6. Correlation between ground failure and subsurface soil conditions

The distribution of ground failure index (Table 1) along the portions of the survey lines 1–4 shown in Fig. 1 is shown in Fig. 5(a)–(d), respectively. As can be seen, ground failure was predominantly observed where the subsurface soil conditions are Soil Types 1, 2, and 3. Where Soil Type 4 soil conditions exist, ground failure was minor or absent. A slash (/) indicates transition between soil types.

Ground failure appears to be most prevalent where Soil Type 1 and 2 conditions exist, i.e. between approximately 3.75 and 4.2 km along Line 1 and 0.4 and 1.1 km along Line 4. The presence of the shallow, loose reddish brown, low plasticity silt in Soil Type 1 and 2 profiles is believed to have a significant influence on the observed ground failure. The absence of ground failure in some of the areas where the soil conditions are Type 1 may be attributed to variations found in the thickness and plasticity index of the silt layer. As previously mentioned, the LL of this material was found to vary from 25 to 35% and the thickness from approximately 0.5 to 2.5 m. Greater damage may have occurred in areas where the silt exhibits lower plasticity and greater thickness. A second contributing factor may be the variability of thickness and penetration resistance of the lower silts and sands between 5 and 9 m.

The relatively few buildings with reported ground failure observed along the section of Line 3 shown in Fig. 1 (see Fig. 5(c)) is due to the high number of buildings with high index of structural damage (total collapse). Foundation performance could not be assessed from fully collapsed buildings [1]. Along this portion of Line 3, 11 buildings of a total of 24 surveyed, suffered structural damage classified as either D4 (partial collapse) or D5 (collapse), which represents 46% of the surveyed building stock.

The distribution of ground failure index presented in Fig. 5 has also been classified according to the number of storeys of the building surveyed. In general, it can be noted that the severity of ground failure appears to be most strongly influenced by the type of soil profile over which the structure is founded. At sites where soils susceptible to liquefaction were present (e.g. predominantly Soil Type 1

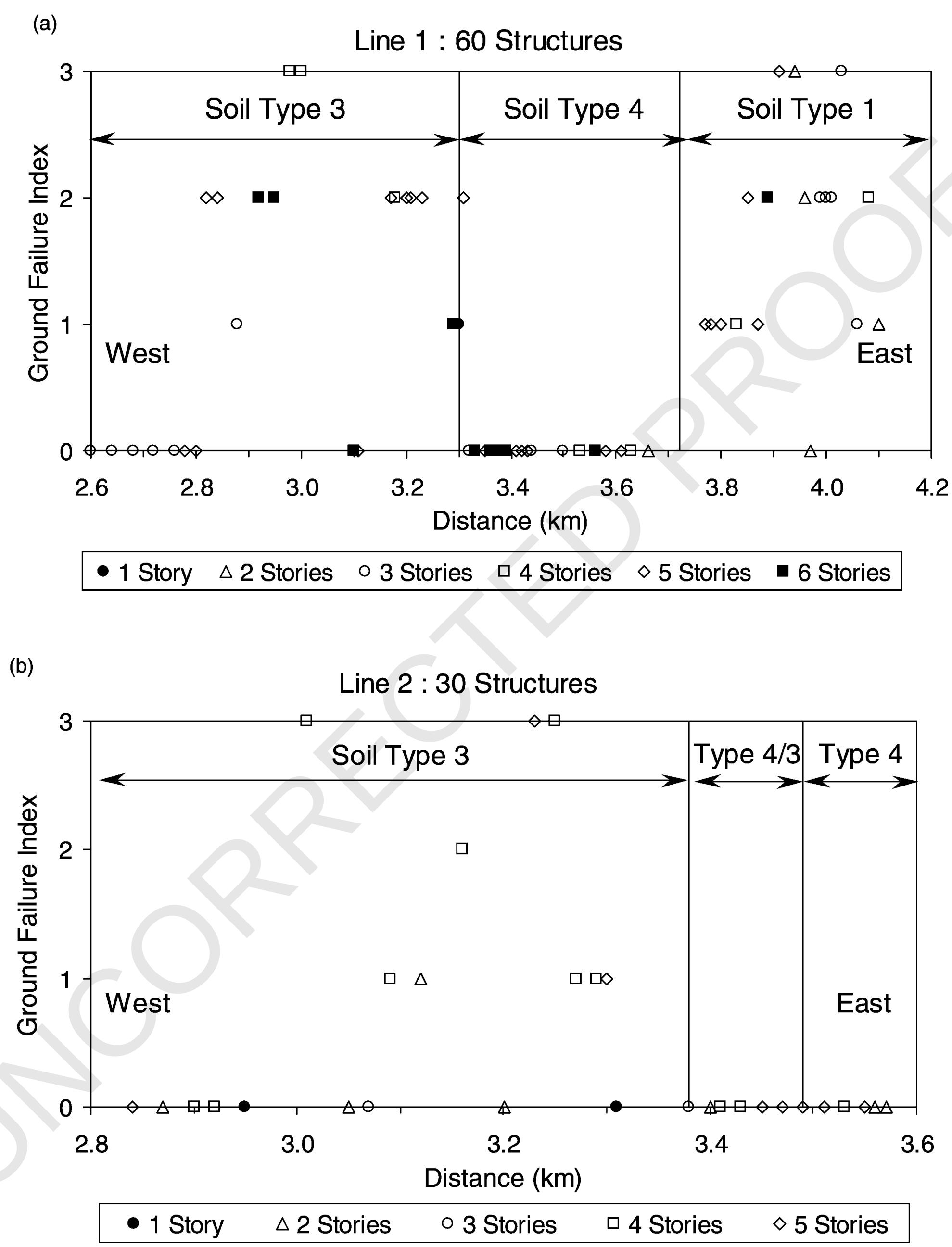
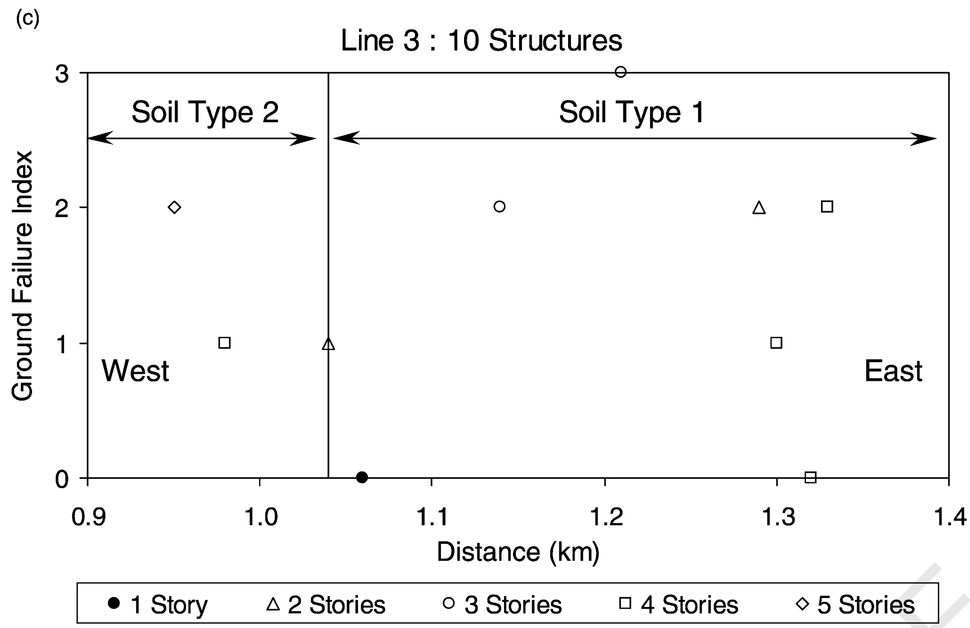


Fig. 5. (a) Ground failure index and soil profile type distribution along Line 1; (b) ground failure index and soil profile type distribution along Line 2; (c) ground failure index and soil profile type distribution along Line 3; (d) ground failure index and soil profile type distribution along Line 4.



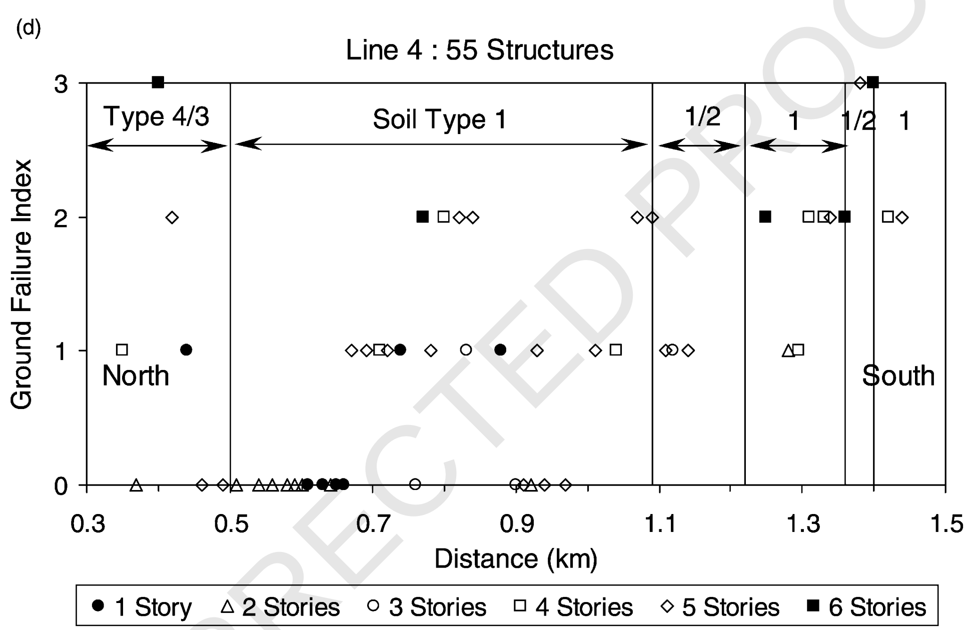


Fig. 5. (*continued*)

and 2 profiles), the height (or weight) of the building was found to contribute to the severity of ground failure. This issue will be studied and addressed in more detail in the following paragraphs.

As described earlier, the reported ground failure index is principally a function of measured relative vertical displacement and tilt of the building. The average measured relative vertical displacement is shown in Fig. 6 as a function of building width and number of storeys (which, in turn, is related to contact pressure) for buildings founded on soil profiles that classify as Type 1 and 2. Buildings that experienced excessive tilt are not included. Previous empirical studies have found that earthquake induced vertical displacements of foundations on granular soils are related to, among other factors, the width of the foundation and the foundation contact pressure [12–14]. This relationship has also been studied with model tests [12,15], which found vertical foundation movement to be inversely proportional to foundation width.

As can be noted in Fig. 6(a), most of the structures' foundation width is in the range of 5–20 m. Within this range, most structures experienced vertical displacement

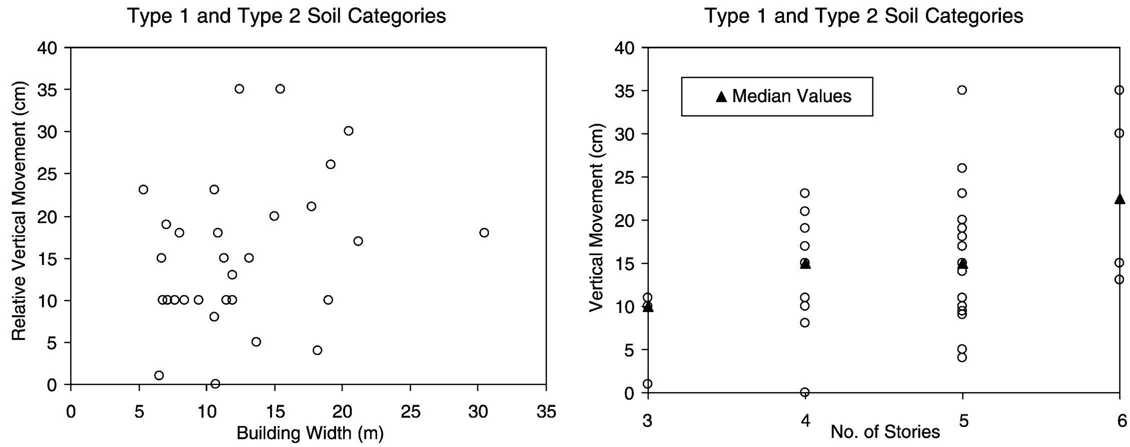


Fig. 6. Relationship between observed vertical building movement relative to the adjacent ground and (a) building width, and (b) number of storeys of the structure.

between 0 and 30 cm, and a relationship between these two factors is not apparent. However, the amount of vertical displacement of the building relative to the surrounding ground is found to be dependent on the building's number of storeys as shown in Fig. 6(b). There is a trend for buildings with more storeys (or analogously, greater contact pressure) to undergo more relative vertical displacement than shorter buildings. Regardless of the width of the foundation, on average, the taller, heavier buildings experienced greater vertical movement than the smaller, lighter buildings.

7. Conclusions

Local variations in the characteristics of alluvial sediments in Adapazari appear to have played an integral role in the occurrence and non-occurrence of ground failure and associated building damage. The degree of ground failure observed along four lines that traverse four downtown districts appears to be principally controlled by soil condition, with ground failure occurring in zones that are susceptible to liquefaction based on conventional criteria, with the exception of using an abbreviated Chinese criteria that does not include the percent clay criterion. It is not the percent 'clay-size' particles that is important; rather it is the percentage of clay minerals present in the soil. In this case, a significant amount of the clay-size soil particles are non-clay minerals such as quartz and feldspar, and therefore, liquefaction criteria based on percent clay-size, such as the Chinese criteria, were found to be unreliable. Ground failure was largely absent from areas having soils that are too clay-rich to be considered liquefiable.

Our preliminary interpretation of the field data suggests that the type and width of structures do not significantly influence the degree of ground failure. However, the localization of observed settlements around buildings, the relative infrequency of observations of liquefaction in open fields, and the higher rate of severe ground failure for taller buildings suggests that ground strains associated with soil–structure interaction may have contributed to the triggering and severity of ground failure. Ongoing studies are evaluating soil–structure interaction effects, and the role building response had on ground failure.

Acknowledgments

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