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PERFORMANCE OF BUILDINGS OVER LIQUEFIABLE GROUND IN ADAPAZARI, TURKEY

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SUMMARY

A large number of structures collapsed and were damaged due to strong ground shaking during the August 17, 1999 Kocaeli earthquake. Many buildings were also affected by ground failure due to liquefaction of shallow silt deposits. Ground failure was indicated by relative vertical displacement of the building into the ground, tilt, and lateral translation over the ground. The occurrence of structural damage was found to be related to the occurrence of ground failure. This paper describes the different types of foundation failures observed in Adapazari, and through the presentation of available building survey data, shows the interdependence of structural damage and ground failure. Measurements of vertical displacement relative to the surrounding ground are correlated to variables that are commonly known to affect foundation settlement, such as the applied building contact pressure. Additionally, the mechanisms that might have led to the observed building performance are described.

INTRODUCTION

The City of Adapazari suffered the largest level of gross building damage and life loss of any city affected by the August 17, 1999 Kocaeli earthquake [1]. According to Turkish Federal Government data, 5,078 buildings (27% of the total stock) were either severely damaged or destroyed [1]. The official loss of life in Adapazari was 2,627, although the actual number was probably much higher.

Ground failure under and adjacent to buildings in Adapazari was pervasive. Hundreds of buildings settled and tilted into the ground; others also translated horizontally over the ground. Additionally, many of these buildings had structural damage. Rapid damage surveys were performed along four lines across the city [1]. A total of 719 structures were surveyed in Adapazari, which is about 4% of the building stock. The degree of structural damage to a building was described using a system proposed by Coburn and Spence [2], where each building is assigned a Structural Damage Index ranging from D0 (no observed damage) to D5 (complete collapse of the building or a story within the building). Information on observed vertical building displacement or penetration relative to the adjacent ground, tilt, lateral movement, and eruption

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of sand boils was compiled by post-earthquake investigators, using the Ground Failure Index described by Bray and Stewart [1]. GF0 corresponds to no observable ground failure and GF3 to significant building penetration of more than 25 cm, building translation of more than 10 cm, or 3 degrees tilt [1]. Detailed description of the structural damage index and ground failure index is also provided in these conference proceedings in Bird et al. [3].

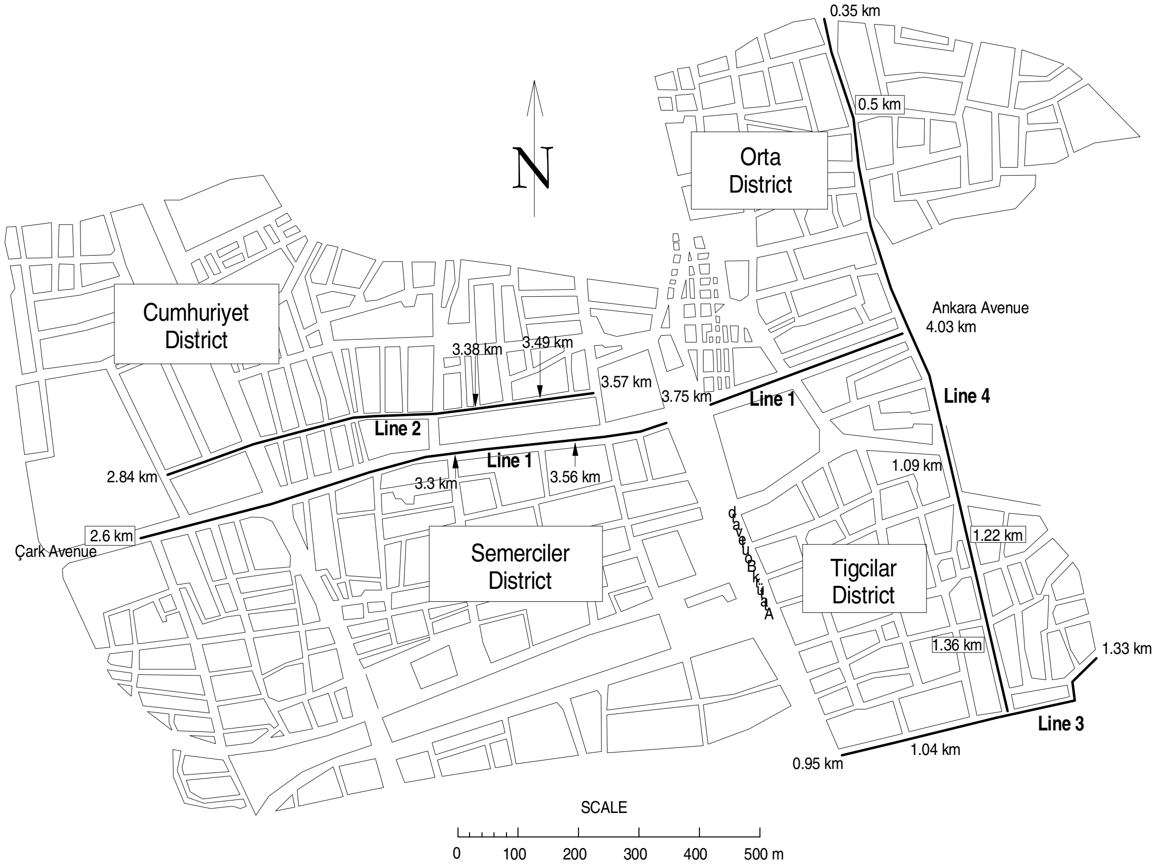

Figure 1: Map of central Adapazari, Turkey showing the locations of the building damage and ground failure survey lines provided in Bray and Stewart [1].

An extensive field investigation program was carried out at selected building sites and along streets surveyed previously in Adapazari to document the subsurface conditions and to identify the soil deposits that might have had a detrimental effect on building performance during the earthquake. The site investigation program included 135 Cone Penetration Test (CPT) profiles and 46 exploratory borings with closely spaced Standard Penetration Tests (SPT) with energy measurements. The details of this investigation have been described by Bray et al. [4].

Sancio et al. [5] developed four general subsurface profiles, Type 1 through Type 4, for four central districts of downtown Adapazari (see Figure 1), and subsequently classified the soil conditions along the lines into one of these four generalized soil profiles. Soil profiles Type 1 through Type 3 contain soil deposits that are susceptible to liquefaction. Soil profile Type 4 does not contain soils susceptible to liquefaction. The degree of ground failure, as expressed by the ground failure index (GF), that was documented along four lines that traverse these four central districts appears to be principally controlled by shallow soil deposits. Ground failure principally occurred in zones that contained shallow, saturated, loose silts that were susceptible to liquefaction based on newly proposed criteria [6] that supercedes the Chinese Criteria by Seed and Idriss [7] which has been shown to be unconservative due primarily to the clay-size criterion.

The purpose of this paper is to describe the performance of buildings situated atop liquefiable ground in Adapazari during the Kocaeli earthquake and develop possible mechanisms to explain foundation failure. The data of settlement of foundations over soils of similar characteristics (Type 1 and Type 2) that was used by Sancio et al. [5] is reexamined, and an attempt is made to correlate the magnitude of vertical settlement to variables such as building weight and the width of its foundation.

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. The city is densely developed in most areas, primarily with 3 to 6 story reinforced concrete frame buildings and older 1 to 2 story 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 sediments, e.g. Komazawa et al. [8], Kudo et al. [9], Endes et al. [10], and Rathje et al. [11]. The Adapazari basin is a former Plio-Pleistocene lake. The lake sediments are overlain by Pleistocene and early-Holocene alluvium transported from the mountains north and south of the basin. The shallow soils (depth < 10 m) are recent Holocene 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 nonplastic silts, silty sands, and clays throughout the city. Clay-rich sediments were deposited in lowland areas where floodwaters created ponds (Onalp et al. [12]).

Construction in the area primarily consists of 3 to 6 story reinforced concrete buildings designed with a beam-column system. Shear walls are uncommon. Interior walls are built with hollow clay bricks covered with stucco, and exterior walls generally consist of lighter, porous, solid blocks to provide thermal insulation. The roof of the concrete buildings are inclined and covered with clay tiles. Older buildings of 1 to 2 stories that were built with timber and clay bricks are also found, but are less prevalent.

The foundations of the reinforced concrete buildings in Adapazari are atypically very robust compared to foundation systems commonly employed for buildings of these heights. These foundations generally consist of a 30 to 40 cm thick reinforced concrete mat that is stiffened with 30 cm wide and 100 cm to 120 cm deep reinforced concrete grade beams that are typically spaced between 4 m and 6 m in both directions. The open cells between adjacent grade beams are filled with compacted soil and then covered with a thin concrete floor slab. The resulting foundation is effectively a very stiff and strong mat foundation that is about 1.5 m thick. Tilting of structures after the earthquake without significant structural damage is generally attributed to the exceptional robustness of these foundations, which allows the building to respond more as a rigid body (if the overlying structural system does not fail) while it undergoes significant differential downward movement, tilt, or lateral translation.

GROUND FAILURE AND BUILDING PERFORMANCE IN ADAPAZARI

The widespread occurrence of ground failure in Adapazari made it conducive for the investigation of the response of buildings on shallow foundations. Excessive settlement and tilt of buildings on shallow foundations overlying sandy deposits have been previously documented in other earthquakes, for example in Niigata after the 1964 Niigata earthquake by Seed and Idriss [13] and in Dagupan City after the 1990 Luzon, Philippines earthquake by Tokimatsu et al. [14]. However, the phenomena observed in Adapazari are of particular interest because of the fine-grained nature of the soil deposits that underwent ground failure and because of the predominance of moderate deformations that sometimes allowed for the building to be inhabitable after limited repairs, if the corresponding structural damage was light.

Different forms of ground failure-related building damage were identified during the reconnaissance missions that followed the earthquake, some of which are shown in Figures 2 and 3 to aid the descriptions that follow:

> <u>Uniform vertical displacement</u>: Many buildings in Adapazari sunk into the ground, many times without noticeable tilt as is shown for the case of Figure 2a. At times, heave of the surrounding ground was observed as in the case of Figure 2b.
>
> <u>Vertical displacement with tilt</u>: Some buildings experienced non-uniform vertical deformation, causing the building to be condemned albeit devoid of structural damage as for the example shown in Figure 2c. Toppling of buildings, depicted in Figure 2d, was typically observed in laterally unconstrained slender buildings, i.e. large ratio of building height (H) to its width (B).
>
> <u>Lateral translation</u>: A previously undocumented failure mode was also observed in Adapazari. Some buildings translated laterally over liquefied soil directly beneath their foundation. Two such cases are depicted in Figure 3. In the first case, the structure displaced 31 cm away from the previously adjacent sidewalk. In the second case, the structure displaced approximately 110 cm in the direction of an open alley mobilizing a wedge of soil. This building also translated 50 cm to 55 cm in the perpendicular direction.

Structural Damage and Ground Failure

The building data collected in the detailed surveys along the lines allowed general trends to be established regarding the relationship between ground failure and building damage [1]. 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, whereas others had only moderate structural damage. Broad areas with ground failure and only light structural damage were not prevalent, but they did exist. However, overall, the compiled data indicate that the severity of structural damage generally increases with increasing levels of ground failure [1]. Nevertheless, areas with a high degree of ground failure and low degree of structural damage were also identified. Sand boils were observed within some of the ground failure zones, but were not widespread, and were absent from many areas.

Figure 2: Examples of ground failure-induced building damage in Adapazari after the August 17, 1999 Kocaeli earthquake: a) vertical displacement, b) vertical displacement with ground heave, c) vertical displacement with significant tilt, and d) bearing capacity failure (Sancio [12]).

Figure 3: Examples of lateral displacement of structures on mat foundations in Adapazari after the 1999 Kocaeli earthquake. Note the development of a passive resistance wedge in the photograph on the right.

Figure 4 presents the surveyed data for 60 buildings along the portion of Line 1 that traverses the four central districts shown in Figure 1 for which the structural damage index and the ground failure index were obtained. Overall, 81 buildings were surveyed, however, the ground failure index could not be obtained for 18 buildings that underwent total collapse (D5), and 3 buildings with none or minor damage (D0 and D1). Structural damage was pervasive along this portion of the city. Thirty-three percent (33%) of the buildings surveyed exhibited complete or partial collapse (D4 and D5), and only 19% of the buildings did not suffer any structural damage. Similarly, ground failure was also widespread, although not as widespread as structural damage. Thirty-six percent (36%) of the building sites surveyed exhibited moderate to high ground failure (GF2 of GF3), but 47% of the building sites surveyed (28 buildings) did not exhibit any type of noticeable ground failure. As shown in Figure 4, these buildings appear to be concentrated in that part of the survey line between 3.3 km and 3.7 km, where the subsurface soils are not susceptible to liquefaction (i.e. soil type 4) [5]. In general, along this line, greater structural damage tended to coincide with manifestations of ground failure.

Examining the survey data of Line 1 further, of the 60 buildings where both structural damage and ground failure indices were available, 28 buildings have either four or five stories. Of these 28 buildings, 15 buildings exhibited no damage or cosmetic cracking (i.e. DO or D1). Of these 15 buildings with no to little damage, 10 buildings (i.e. 67%) had no or minor ground failure (i.e. GF0 or GF1). Of the 28 buildings with either four or five stories, 9 buildings exhibited partial or total collapse (i.e. D4 or D5) and of these 9 buildings, 6 buildings (i.e. 67%) also underwent moderate to significant ground failure (i.e. GF2

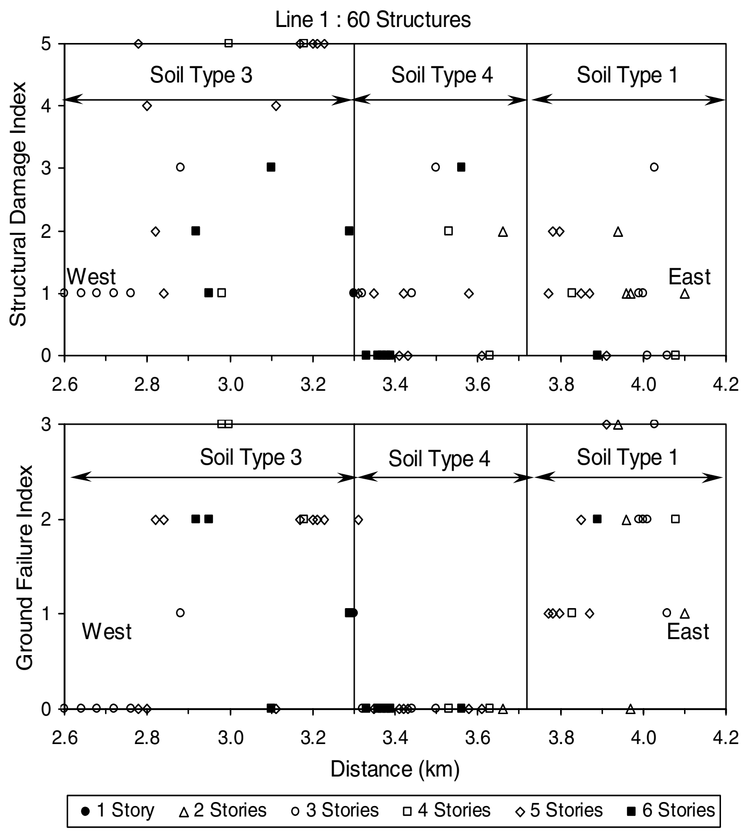

Figure 4: Plot of structural damage index (D) and ground failure index (GF) along the portion of Line 1 of Bray and Stewart's [1] survey data shown in Figure 1.

or GF3). Thus, all else being equal along this line, most of the buildings with four or five stories that suffered significant structural damage also underwent significant ground failure. Conversely, most buildings with none to minor structural damage were located at sites that did not experience significant ground failure.

Figure 5 shows the plots of structural damage and ground failure index for 55 buildings along a portion of Line 4 (see Figure 1). As opposed to the example described previously, ground failure along this line was pervasive. Sixty four percent of the buildings surveyed (35 buildings) exhibited minor to significant ground failure (GF1 to GF3). Buildings towards the South appear to exhibit a higher degree of ground failure. Conversely, the same degree of structural damage was not observed throughout the line. Structural damage appears to decrease between 0.5 km and 0.9 km. Buildings between 0.9 km and 1.5 km only exhibited none to low structural damage (i.e. D0 and D1), while most underwent minor to moderate ground failure.

Thus, structural damage in Adapazari was observed in areas with and without ground failure. As may be observed from these two line surveys, extensive ground failure does not always produce heavy structural damage. However, this may be largely due to the robust foundation mats with intersecting grade beams that are built in Adapazari. Additionally, heavy structural damage may occur in combination with extensive ground failure or in the absence of ground failure due to just strong ground shaking and poor structural performance due to inadequate design or construction.

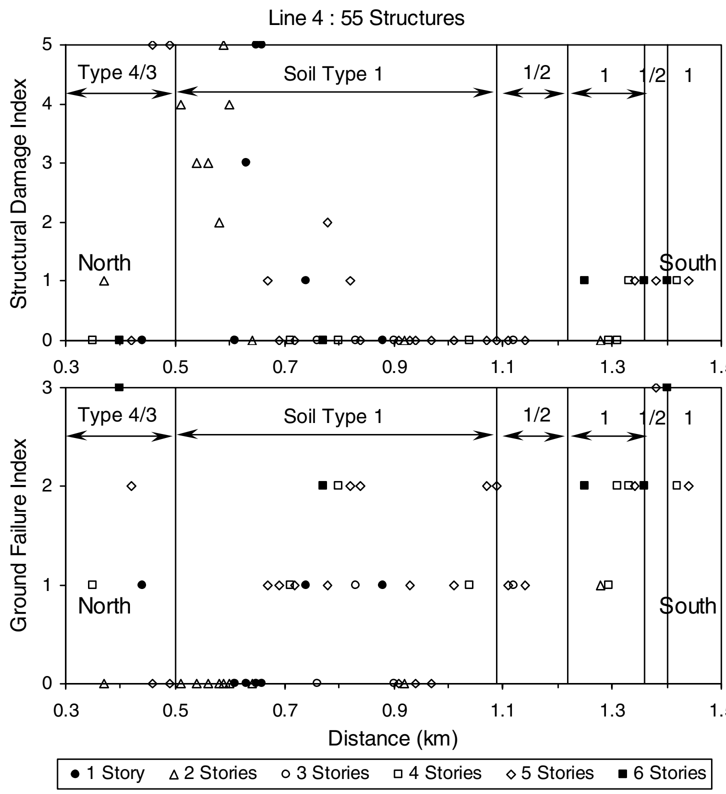

Figure 5: Plot of structural damage index (D) and ground failure index (GF) along the portion of Line 4 of Bray and Stewart's [1] survey data shown in Figure 1.

Settlement of buildings founded on soil profiles with shallow liquefiable silts

As found by Sancio et al. [5] (and illustrated with the survey data shown in Figures 4 and 5), the occurrence of ground failure was more prevalent when buildings were founded over soil profile Type 1 or Type 2. Sancio et al. [5] related this observation to the liquefaction susceptibility and potential of the shallow silty soils in these soil profiles. A brief description of the characteristics of the Type 1 and Type 2 soil profiles and their liquefaction susceptibility is included for the sake of completeness. More extensive descriptions are found in Sancio et al. [5].

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 from 0.5 m to 2.5 m. The index properties are in the range that has been identified for soils susceptible to liquefaction by Bray et al. [6], and given that the corrected penetration resistance of this stratum ($(N_1)_{60}$ or q_{c1N}) was low, this stratum likely liquefied during the Kocaeli earthquake. 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 [15].

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. These soils are also susceptible to liquefaction, but are not the critically weakest of the soil profile.

<u>Soil Type 2:</u> 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 dense (q_{c1N} > 160 and $(N_1)_{60}$ > 30), gray sand to a depth of approximately 9 m. The only layer considered to be liquefiable is the same as the shallow silt described above for soil profile Type 1.

Table 1 lists the dimensions of 27 buildings located along the lines shown in Figure 1 and located on soil profiles Type 1 and Type 2 [5]. The dimensions of these buildings were measured by C. Christensen and are provided in Bray et al. [4]. In lieu of height measurements, the number of stories was multiplied by 2.9 m, which is the typical story height for buildings in Adapazari. Most of the buildings were located along Line 4 (Figure 5), and a few others were located along Lines 1 and 3.

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, the thickness of the liquefied layer, and the foundation contact pressure [e.g. 16, 17 and 18]. A similar relationship is found for static settlements on cohesionless soils, e.g. Meyerhof [19]. This relationship has also been studied with model tests [e.g. 16 and 20], which found vertical foundation movement to be inversely proportional to foundation width.

As can be noted in Table 1, most of the structures' foundation width is in the range of 5 m to 20 m. Within this range, most buildings experienced vertical displacement between 0 cm and 30 cm, which indicates ground failure indices between GF1 and GF3. All but one building listed in Table 1 experienced none to minor damage (i.e. D0 or D1).

The average measured relative vertical displacement (Δ) divided by the width of the building (B), is plotted in Figure 6 as a function of the height of the building also divided by the width of the building (H/B) for structures founded on soil profiles that classify as Type 1 and 2. The height of the building divided by the width (H/B) is known as the aspect ratio, but it is also related to the contact pressure (q). As was described previously, excessive building tilt or toppling was sometimes observed in laterally unconstrained buildings with high aspect ratios. Buildings that experienced excessive tilt or toppling have been excluded from Table 1 and Figure 6.

Examining Figure 6, the amount of vertical displacement of the building relative to the surrounding ground is found to be roughly proportional to the aspect ratio of the building (i.e. H/B), which is relatively equivalent to the applied contact pressure. All else being equal, buildings of higher contact pressure (and also higher aspect ratio) experienced more vertical displacement. Regardless of the width of the foundation, on average, the taller, heavier buildings experienced greater vertical movement than the smaller, lighter buildings.

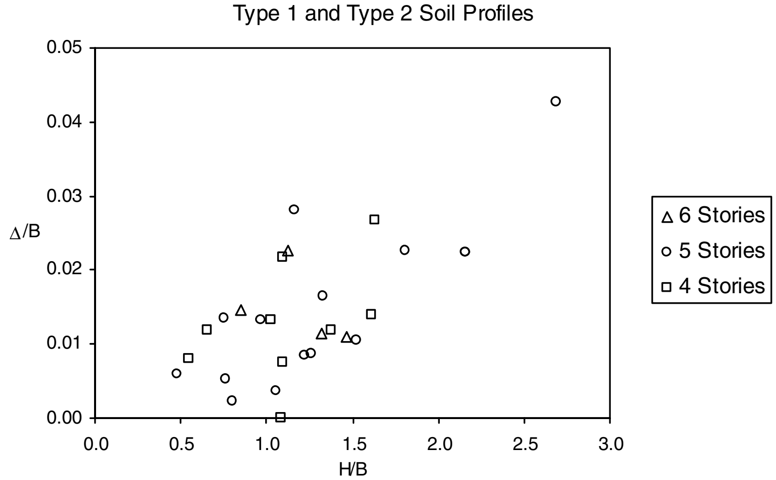

Figure 6: Relationship between building settlement (Δ) and the building height (H) normalized by the foundation width (B).

Table 1: Characteristics of the buildings surveyed that are founded over soil profiles Type 1 or Type 2.

Building	No. Stories	Width, B (m)	Length, L (m)	Soil Type	Δ (cm)	D	GF
4-b-8	6	13.2	22	1	15	1	2
4-k-4	6	11.9		1	13	0	2
4-o-9	6	20.5	25.3	1	30	0	3
4-b-6	6	15.5	17	1-2	35	1	3
4-b-4	5	15	33	1	20	1	2
4-e-4	5	6.7	19.8	1	15	0	2
4-e-5	5	30.5	39.5	1	18	0	2
4-f-2	5	18.2	23.4	1	4	0	1
4-k-1	5	10.9	12.3	1	18	1	2
4-k-3	5	11.5		1	10	2	1
4-m-1	5	13.7	21.8	1	5	1	1
4-o-8	5	9.5	18.1	1	10	0	2
1-n-7	5	5.4	16.8	1	23	1	2
1-o-1	5	19.2	22	1	26	0	3
4-b-7	5	12.5	18.5	1-2	35	1	3
4-e-1	5	19	19.8	1-2	10	0	1
4-e-3	5	11.9	19.8	1-2	10	0	1
4-c-1	5	8	16.3	1	18	1	2
3-h-4	5	6.7	15	2	15	1	2
4-b-5	4	11.3	13.5	1	15	1	2
4-c-5	4	7.2	14.9	1	10	0	1
4-k-2	4	21.2	22.5	1	17	0	2
4-p-2	4	8.4	13.3	1	10	0	1
1-o-12	4	17.8	17.9	1	21	0	2
3-e-1	4	10.6	24.2	1	23	1	2
3-e-3	4	10.6	11.3	1	8	1	1
4-c-2	4	7.1	13.6	1	19	1	2

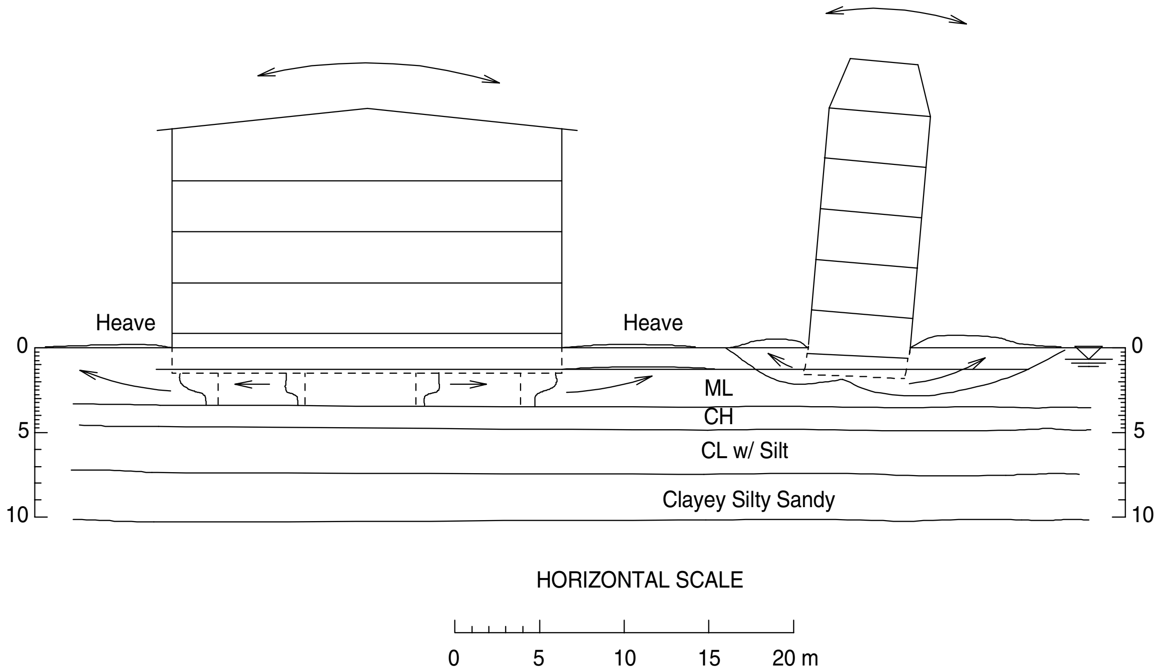

Figure 7: Modes of failure of stout and slender buildings in Adapazari.

MECHANISM OF FOUNDATION FAILURE IN ADAPAZARI

Figure 7 depicts two common modes of building performance in Adapazari after the Kocaeli earthquake. The drawing to the left shows a stout building with a large mat foundation where its width is much greater than the thickness of the underlying liquefiable silt deposit. The drawing to the right depicts a slender building with a narrow foundation width.

Based on the interpretation of the results of the in situ tests by Sancio [21], the shallow silt deposit was identified as the critical layer under most of the buildings studied in Adapazari. In general, deeper deposits (5 m < depth < 10 m) of silt and sand were often too dense to have contributed significantly to the observed building performance [21]. Soil specimens obtained from the deeper silt strata that were potentially liquefiable and tested in a cyclic triaxial system exhibited significantly greater cyclic strength than the shallow silt [21]. Although at some sites the deeper layers might have contributed to the overall building performance, this contribution will be neglected for the sake of this discussion, because it appears that in many cases the response of the upper silt dominated the building response. It can therefore be assumed, without considerable error, that only the silt layer (ML) shown in the drawing lost significant strength during the earthquake. Additionally, it will be assumed that most of the deformations occurred over a short period of shaking, or equivalently, one to two intense shear stress cycles. Subsequent shaking and lower intensity stress cycles are not considered to have significantly affected the overall building performance.

The earthquake-induced shear stresses under the stout building that are imposed on the soil elements cause an immediate generation of positive pore water pressures and subsequent loss of strength and stiffness. Additionally, the soil in the free-field has also developed significant pore water pressure and perhaps is undergoing liquefaction. Under these conditions, the soil under the building can no longer withstand the weight of the structure, and thus, it is squeezed laterally while maintaining constant volume given the short period of time over which this occurs and the low hydraulic conductivity of the silt. These imposed deformations are large enough for the soil to dilate eventually and recover its shear strength and once again withstand the weight of the structure.

Subsequent cycles may also cause minor build-up of additional excess pore water pressures, which may produce additional vertical deformation. These smaller stress cycles may sufficiently soften the soil such that it undergoes a few significant cycles of strength loss with limited strain potential, i.e. cyclic mobility, and this may contribute to the building movement. However, this "ratcheting" effect is believed to be of secondary importance relative to the initial response of the soil undergoing the primary loading cycles during this near fault forward-directivity earthquake ground motion.

Given that the failure is shallow, the initial squeezing causes some heave at the surface as was observed at some of these sites (Figure 2b). Conventional methods used to calculate deformations along potential sliding surfaces (i.e. Newmark-type analysis) are not directly applicable, because the soil in this case is not sliding along a surface but deforming over it. The implementation of the finite element method to model the problem is perhaps a better way to analyze the deformations to which soil elements are subjected. These analyses should be the work of continuing research into the performance of buildings over liquefiable ground to develop methodologies that allow design engineers to estimate seismically induced settlement of foundations over potentially liquefiable ground.

The second failure mechanism shown is more representative of a typical bearing-type failure where the soil slides along a circular or semi-circular surface. In this case, as in the one previously described, the generation of positive pore water pressures causes the soil to temporarily lose strength. Additionally, horizontal shaking causes the building to apply an overturning moment at the foundation level, or equivalently, an eccentricity of the vertical load. The magnitude of the overturning moment and thus the eccentricity is a function of the seismic response of the building and the height of the building.

If the mat foundation is narrow, the effect of the eccentric load is greater because it causes stress concentrations over a smaller area of the mat foundation. When this stress approaches or exceeds the seismic bearing capacity of the soil (i.e. considering the reduction of strength due to excess pore water pressure), the building begins to tilt. As tilting is initiated, the area over which the stresses are applied is reduced, thus the magnitude of the stress increases. Under these conditions, a progressive failure is possible. Continuing tilt will cause toppling unless the bearing capacity of the soil increases sufficiently due to dilation of the soil or due to an increase of effective stress due to dissipation of excess pore water pressure, or cessation of shaking which causes the overturning moment to reduce significantly.

CONCLUSIONS

Although structural damage in Adapazari was primarily due to strong ground shaking and poor structural design and construction, structural damage also appears to be somewhat related to ground failure, with significant building damage being more likely to occur in areas of significant ground failure. A large number of buildings in Adapazari experienced liquefaction induced ground failure as well as significant structural damage. However, there were also many cases in which buildings that did not experience significant ground failure were heavily damaged, and sometimes buildings undergoing significant ground failure were not heavily damaged.

In Adapazari, vertical displacement of buildings into the ground during the Kocaeli earthquake appears to be related to the applied contact pressure at the foundation of these buildings. Buildings with higher aspect ratios (i.e. large height to width ratios and consequently higher foundation contact pressures) underwent correspondingly higher amounts of settlement. However, building settlement is also affected by a large number of other variables that cannot be independently assessed. Hence, the development of engineering tools for evaluating the consequences of liquefaction on building performance warrants more attention.

The most common mechanism of building settlement in Adapazari after the Kocaeli earthquake is believed to be caused by spreading of the soil directly under the building towards the sides due to a temporary loss of bearing capacity caused by seismically induced pore water pressures. These deformations occur rapidly and are followed by additional vertical displacement caused by the dissipation of remaining excess pore water pressures after the cessation of ground shaking. In cases where the building is slender (high H/B) and unconstrained laterally, it may fail dramatically in a bearing capacity-type failure that leads to excessive tilt or even toppling of the building.

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