

SEISMIC BEARING CAPACITY DEGRADATION AS A PART OF INTEGRATED EARTHQUAKE DAMAGE ASSESSMENT

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ABSTRACT

This paper describes a new approach developed to incorporate seismic bearing capacity degradation into integrated damage assessment methodologies. In accordance with the aim of the paper, a suitable method to define the possibility of ground damage due to seismic bearing capacity degradation is introduced based on the assumption that performance of a foundation should be measured in terms of the displacements it undergoes during earthquakes. In this context, seismic bearing capacity degradation and critical acceleration concepts are briefly discussed. A seismic settlement occurrence interpretation approach is developed and the calculated settlements with this approach are related with the expected damage levels based on the literature. The method has also been applied to Bakirkoy district of Istanbul, which is under the threat of a probable future earthquake. Expected seismic settlements of shallow foundations with different foundation geometries are calculated for the eighty-seven boreholes in the region. These settlement values are then evaluated within the frame of damage assessment approach and expected damage levels are displayed in GIS medium to show the spatial vulnerability of the area to bearing capacity failure.

Keywords: Seismic bearing capacity degradation, integrated damage assessment, soil-structure interaction, shallow foundation, seismic settlement

INTRODUCTION

Spatial earthquake hazard assessment studies play a key role in identifying and mitigating the potential consequences of an earthquake. As ground shaking affects the structures and also the foundation soils by changing their constitutive response, it is considered to be the primary damage-making phenomenon. On the other hand, events such as landslide, liquefaction, tsunami, fault rupture, etc. are the collateral causes of structural damage. At the present state of the art, existing spatial earthquake damage assessment studies fail to consider damage due to seismic bearing capacity degradation of shallow foundations. This consideration is important, because laboratory and theoretical investigations of the seismic behaviour of foundations indicate that, footing failure may not be solely due to liquefaction but to seismic bearing capacity reduction. While liquefaction is a threat for loose and granular soils, seismic bearing capacity degradation may occur in almost all types of foundation soils. Reported field evidence for seismic bearing capacity reduction is scarce as a result of tendency to attribute all types of ground failures to liquefaction (Cinicioglu et al, 2006). Seismic bearing capacity degradation concept attracts surging interest from the researchers working on the subject. These studies do not date back and hence have not found way to the codes and microzonation studies, but

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they do show the fact that this concept should be treated as an inherent part of earthquake assessment studies. Based on similar reasons, none of the existing integrated approaches for spatial damage analyses known to the authors include seismic bearing capacity degradation concept in the calculation of the total risk level.

This paper aims to focus on seismic bearing capacity degradation and consequent settlements and considers this concept within the context of integrated damage assessment studies. A comparative evaluation is proposed to find the expected damage due to liquefaction and/or bearing capacity degradation. If severe liquefaction case dominates bearing capacity degradation, there is no need to consider damage due to bearing capacity reduction. On the contrary, if bearing capacity degradation is more pronounced compared to liquefaction, damage assessment should be based on this incident.

SEISMIC BEARING CAPACITY DEGRADATION AND CONSEQUENT SETTLEMENTS

Effects of seismic forces on bearing capacity of the foundations have been examined by several different researchers using techniques such as; limit equilibrium, the upper bound limit analysis and the method of inclined slices (Sarma and Iossifelis, 1990, Richards et al., 1993, Budhu and Al-Karni, 1993, Shi and Richards, 1995, Pecker, 1996, Auvinet et al. 1996, Paolucci and Pecker, 1997, Kumar and Rao, 2002, Fishman et al, 2003, Merlos and Romo, 2006). These studies have shown that bearing capacity of shallow footing reduces during earthquakes due to the reduction of soil strength by the inertial forces within the soil mass and shear transfer at the soil-structure interface. Many of these studies attacked the problem from the point of seismic bearing capacity consideration. However, mere application of a factor of safety on bearing capacity without consideration of settlement may not properly assess the performance of a shallow foundation during strong earthquakes because even if the factor of safety drops below one at some time, it does not necessarily imply failure of the foundation (Cinicioglu et al., 2005a).

One of the few studies, which investigate seismic settlements due to reduction of bearing capacity, belongs to Sarma and Iossifelis (1990). The authors studied seismic bearing capacity factors of shallow strip foundations using the limit equilibrium technique of slope stability analysis with inclined slices. Their findings revealed that bearing capacity factors decreased with increasing horizontal coefficient of earthquake motion resulting in lower bearing capacity values. They stated that for earthquakes with an acceleration greater than a critical value, the deformation of the footing may be estimated by the sliding block technique, which implies that the soils did not lose strength with deformation. In this context, the authors provided graphs to determine the possible deformation of the footing based on maximum accelerations of the earthquake and critical acceleration, which initiates motion and period of the structure.

Merlos and Romo (2006) studied the fluctuant bearing capacity of shallow foundations during earthquakes. They argued that bearing capacity failures were documented after the September 1985 Mexico City Earthquake and presented examples of buildings, which suffered bearing capacity loss and consequent deformations up to 100 cm's. They mentioned that the bearing capacity failures could be tracked from the observed failure surfaces and bulging of the neighboring soil. They also presented a limit equilibrium method to compute both dynamic bearing capacity and earthquake-induced settlements of foundations on soils and showed that foundations could experience reductions in its bearing capacity during earthquakes. The reliability of their method was verified by comparing the calculated and observed permanent movements of foundations for the September 1985 Earthquake in the soft ground area of Mexico City. Their results revealed that the characteristics of the failure surface and corresponding seismic safety of a foundation was dependent on acceleration magnitudes developed through the seismic event, and the values of safety factor for the bearing capacity of the studied buildings were acceleration time history dependent.

Richards et al. (1993) presented a comprehensive approach to the problem of reduction of foundation bearing capacity and consequent settlements under seismic loading. They based their approach on Coulomb mechanism with two triangular wedges and developed a dynamic limit analysis model. Based on this model, dynamic bearing capacity factors were calculated and patterns of slip surface changes with increasing acceleration were depicted. The model also included a sliding block approach to calculate the settlements due to earthquake loading. In this study, seismic bearing capacities and permanent footing settlements due to seismic loading were calculated by sliding block approach used by Richards et al. (1993) since their method can easily be extended for regional scale analysis. According to Richards et al., static and seismic bearing capacities; $q_{u,S}$ and $q_{u,E}$, respectively, can be calculated as shown;

$$q_{u,S} = cN_c + \gamma D_f N_q + \frac{1}{2} \gamma B N_\gamma \quad (1)$$

$$q_{u,E} = cN_{cE} + \gamma d N_{qE} + \frac{1}{2} \gamma B N_{\gamma E} \quad (2)$$

where N_{cE} , N_{qE} , $N_{\gamma E}$ are the dynamic counterparts of the static bearing capacity factors, N_c , N_q , N_γ . c and γ are cohesion and unit weight of foundation soil, B and d are the width and depth of the foundation, respectively. Their model is based on the simplified static slip field surfaces shown in Figure 1. Contrary to original Prandtl model which has three regions; active wedge (*I*), passive wedge (*III*) and a logarithmic radial-fan transition region, the simplified model consists of two main wedges, triangles ABC and ACD. The considerations behind this assumption are given in Richards et al (1993) work.

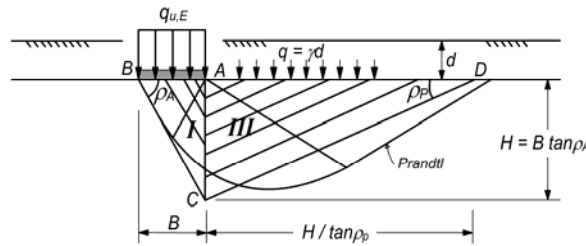


Figure 1. Failure mechanism assumed by Richards et al. (1993) (ρ_A and ρ_P are active and passive wedge angles for static and passive wedges respectively)

The Richards et al. (1993) work based on the above mechanism showed that seismic bearing capacity factors were dependent on earthquake acceleration intensity in horizontal and vertical directions; however, the effect of horizontal acceleration was more pronounced. Corresponding geometry of the failure mechanism changed for a particular earthquake acceleration intensity and as the acceleration increased, the passive thrust decreased and the wedge angles became smaller. Seismic bearing capacity factors derived by Richards et al. (1993) are presented below.

$$N_{qE} = \frac{K_{pE}}{K_{aE}} \quad (3a)$$

$$N_{cE} = (N_{qE} - 1) \cdot \cot \phi \quad (3b)$$

$$N_{\gamma E} = \tan \rho_{AE} \cdot \left(\frac{K_{pE}}{K_{aE}} - 1 \right) \quad (3c)$$

where K_{AE} , K_{pE} are formularized to be functions of internal friction angle, ϕ , horizontal and vertical earthquake coefficients k_h , k_v , and friction between the wedges, δ . ρ_{AE} is the active wedge angle during earthquake motion. The formulations are given in detail by Richards et al. (1993).

According to Richards et al. (1993), analogous to the lateral seismic movement of walls, foundations would settle in a similar fashion in an earthquake, whenever the critical acceleration was exceeded. They assumed that at this critical acceleration, seismic bearing capacity ($q_{U,E}$) was equal to the allowable bearing capacity (q_{all}) and equilibrium of the wedges would be violated and active wedge ABC shown in Figure 1 would move due to the earthquake motion. This assumption is based on Newmark's displacement method originally developed in 1965. Newmark's method assumes that permanent downhill displacement, D , occurs each instance when some critical acceleration is exceeded by earthquake ground shaking. In the approach proposed by Richards et al. (1993), a further assumption was made that movement would take place when the critical acceleration is exceeded in either direction. Additionally, once the sliding is initiated, the active wedge ABC would move downward and sideways at an angle ρ_{ae} , pushing the passive wedge sideways. This way, the footing worked its way downward as it also moves back and forth in a series of small slip movements that were calculable for a given record. Richards et al. (1993) expressed that since lateral displacements in both directions contributed to seismic foundation settlement due to bearing capacity degradation, w , could be expressed as;

$$w = 2D \tan(\rho_{AE}) \quad (4)$$

where D is the unidirectional sliding block displacement of the wedge. D can be found based on block on plane models to obtain the earthquake induced displacements of embankment slopes and retaining structures. Some of these models can be listed as Richards and Elms (1979), Nadim and Whitman (1983) and Yegian et al. (1991). These models generally require a critical acceleration value, which initiates motion, and parameters related to the frequency content of the earthquake.

SEISMIC SETTLEMENT OCCURRENCE INTERPRETATION APPROACH

This section describes the seismic settlement occurrence interpretation approach developed in this study for shallow foundations. The method investigates both seismic bearing capacity degradation and the resulting settlements and aims to present a methodology, which can be easily applied for spatial analysis.

Defining the parameters that affect critical acceleration coefficient

At critical acceleration, k_c , seismic bearing capacity drops to the allowable bearing capacity, which can be expressed as;

$$q_{U,E} = q_{all} \quad (5)$$

Allowable bearing capacity for a specific foundation is found by applying a specified value of factor of safety on the ultimate bearing capacity value; as depicted in Equation 6.

$$q_{all} = \frac{q_{U,S}}{FS} \quad (6)$$

In this equation, FS is the static safety factor, which generally ranges between two and four and is selected as three in most of the cases. However, some codes recommend maximum bearing capacity values corresponding to different soil types (Day, 2002). This implies that, there should be a limit for allowable bearing capacity values, regardless of the value calculated from Equation 6. In this paper, based on this argument, a maximum allowable bearing capacity for the studied region is defined and is named as q_{region} . In this context, earthquake coefficient value corresponding to that which degrades the ultimate bearing capacity to minimum of these two values (q_{all} in Equation 6 and q_{region}) should be used

to assign the critical acceleration value. Based on the above arguments, critical acceleration is dependent on the following parameters;

- selected q_{region} ,
- foundation geometry (d, B),
- static factor of safety (FS),
- unit weight of foundation soil (γ),
- static bearing capacity factors, which are based solely on internal friction angle of foundation soil (ϕ) and
- seismic bearing capacity factors, which are functions of both internal friction angle of the soil and earthquake acceleration (ϕ, k_h).

Defining the conditions, which initiate seismic settlement and finding critical accelerations

Figure 2 illustrates the seismic settlement occurrence interpretation and critical acceleration determination approach developed in this study. First step in this stage, is to find the coefficient of critical acceleration, k_c and the second step is to compare k_c with the coefficient of earthquake motion, k_{site} , which represents maximum earthquake motion expected in the studied location. k_{site} can either be taken from the codes or calculated from site-specific analysis. The basis of the methodology for seismic settlement is that, if the foundation soil beneath a structure experiences a greater earthquake motion (k_{site}) than the critical acceleration value (k_c), then seismic settlements should occur. Lesser of q_{all} and q_{region} is used to assign critical acceleration coefficient, k_c . The three different cases defined within this context are described and schematized below.

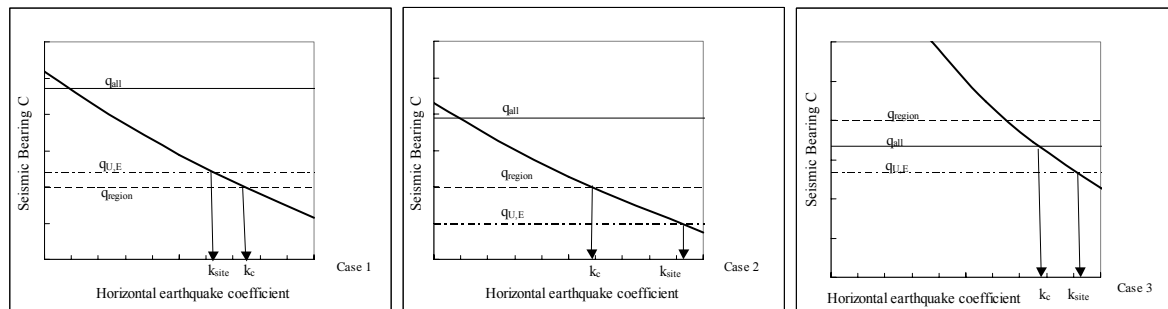


Figure 2. Seismic settlement occurrence and critical acceleration coefficient interpretation approach used in this study

Case 1: If $q_{all} > q_{region}$, k_c should be found as the value corresponding to q_{region} . As shown in Figure 2a, $k_{site} < k_c$ so no seismic settlements should be expected.

Case 2: If $q_{all} > q_{region}$, k_c should be found as the value corresponding to q_{region} . As shown in Figure 2b, for this case, $k_{site} > k_c$, and seismic settlements should be expected.

Case 3: $q_{region} > q_{all}$, so k_c should be found as the value corresponding to q_{all} . In this case, $k_{site} > k_c$ so the seismic settlements should be calculated.

For Cases 1 and 2, critical acceleration value is a function of the selected q_{region} , whereas for Case 3, it is dependent on q_{all} . At this stage, a further attempt is made for Case 3 to obtain dimensionless parameters, which will make the method easier to apply in regional basis. In this context, a new

dimensionless parameter, Reduction Factor, (RF) is defined as the ratio of ultimate static bearing capacity to seismic bearing capacity for any earthquake acceleration;

$$RF = \frac{q_{U,S}}{q_{U,E}} \quad (7)$$

Since seismic bearing capacity is a function of horizontal earthquake acceleration, RF is time dependent and varies during the earthquake. If RF is equal to or less than the static design safety factor (FS), this implies that equilibrium of the foundation wedges is satisfied, otherwise it is violated. In other words; if an RF value is equal to the static safety factor (FS), the corresponding earthquake motion is the critical acceleration value for the selected foundation.

In an effort to search for a simplified approach for defining the critical acceleration values, seismic bearing capacity values were calculated for different foundation geometries, soil properties and different horizontal acceleration coefficients. These values were then used to calculate the reduction factors, which were then plotted against earthquake acceleration coefficients for RF values for different d/B ratios and internal soil friction angle values. These graphs are presented in Figure 3. Horizontal earthquake acceleration values corresponding to RF values of 2, 3, and 4 were considered to be of greater importance, since these are typical static design safety factors used by the engineers and therefore define the critical acceleration values. The results revealed that, for any d/B ratio, there was nearly a unique critical acceleration coefficient regardless of the internal soil friction angles greater than 30°. For ϕ values of 25°, the critical acceleration values were only slightly smaller. Based on this information, a new relationship was obtained between d/B ratios and k_h for different static factor of safety values. This figure is presented in Figure 4.

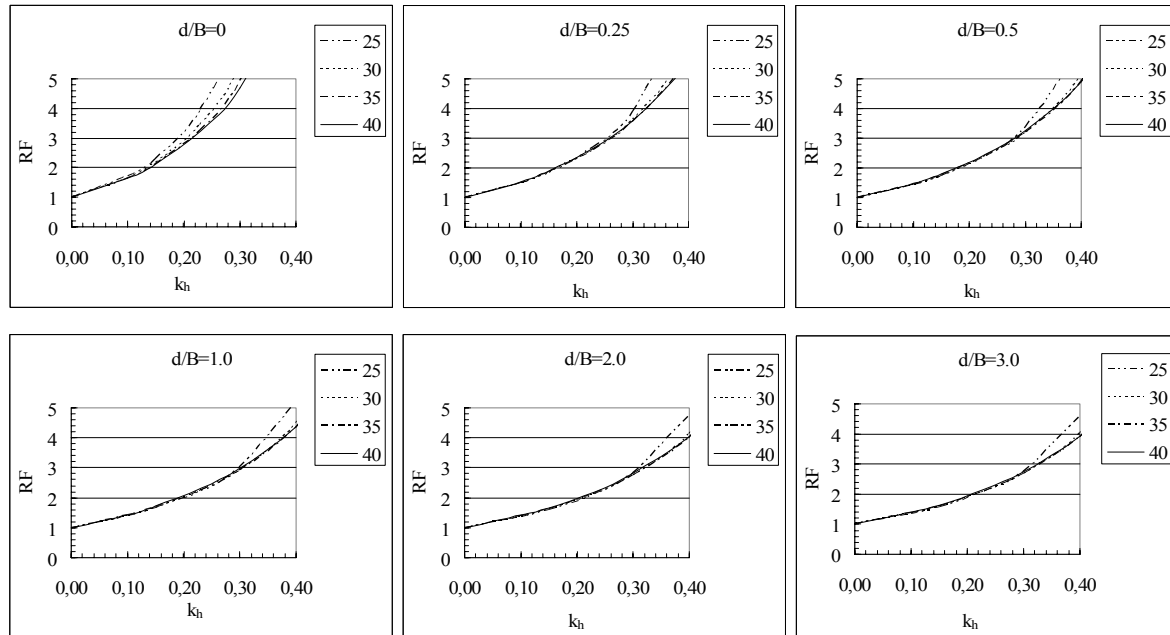


Figure 3. Variation of Reduction Factor (RF) values for different foundation geometries, soil properties and horizontal earthquake coefficients

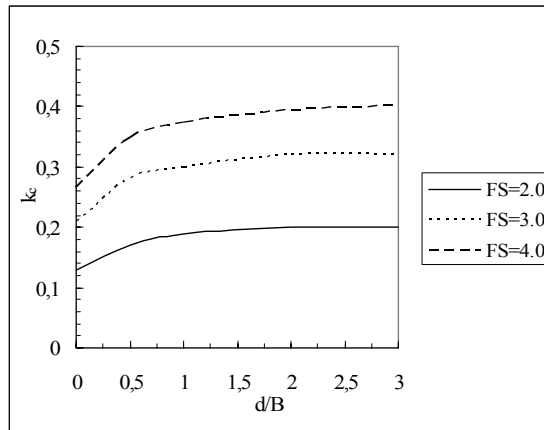


Figure 4. Variation of critical earthquake coefficient value for different d/B ratios and static factor of safety, FS

Seismic settlement calculation

The last step of the analysis includes the calculation the permanent settlements due to seismic degradation of bearing capacity. In this context, the equation by Yegian et al. (1991) was used to estimate the sliding block displacement, D . Yegian et al.'s approach is based on the results of the application of the Newmark's approach to eighty-six actual recorded acceleration time histories. This empirical equation is given by Equation 8.

$$\log \frac{D}{k_{site} N_{eq} T^2} = 0.22 - 10.12 \left(\frac{k_c}{k_{site}} \right) + 16.38 \left(\frac{k_c}{k_{site}} \right)^2 - 11.48 \left(\frac{k_c}{k_{site}} \right)^3 \quad (8)$$

where, N_{eq} is the number of equivalent uniform cycles and T is the period of a motion at a critical sliding mass, k_c and k_{site} are the critical and maximum ground accelerations, respectively. N_{eq} can either be determined from site-specific response analysis (e.g. SHAKE91) or can be defined based on the expected earthquake magnitude as given by Kramer (1996). T can also be found from site-specific response analysis.

Use of seismic settlement values in Integrated Damage Assessment Studies

If application of the approach defined above indicates a possibility of seismic settlements in a studied borehole for the selected type of foundation, the amount of settlement should be calculated. Table 1 presents the categorization of settlement calculated in this study. It should be noted here that, Bray et al. (2000) states that seismic foundation settlements greater than 10 cm correspond to moderate ground failures, whereas settlements greater than 25 cm lead to significant ground damage.

Table 1. Expected settlement and description of expected damage

Expected settlements (w) due to seismic bearing capacity degradation (cm)	Class
$0.0 < w < 2.5$	A
$2.5 < w < 5.0$	B
$5 < w < 10.0$	C
$w > 10$	D

APPLICATION OF THE METHODOLOGY TO BAKIRKOY DISTRICT

Bakirkoy is the most intensely populated district of Istanbul, which is under the threat of a serious earthquake in the near future. The district is underlain by two Tertiary (upper Miocene) formations; Bakirkoy formation (Tb) and Gungoren formation (Tg). Bakirkoy formation consists of limestone, marl, silty clay and clayey silt stratification with varying thicknesses. Gungoren formation (Tg) generally underlies the Bakirkoy formation. However, on the eastern part of the study areas, the formation outcrops. Dominant lithology is silty clay and clayey silts in this formation. Clays are of high plasticity and overconsolidated. As depth increases, overconsolidated clays turn into claystone. Quarternary deposits are encountered in a minor region as shown in Figure 5. This alluvial sedimentation is mainly formed by soft and organic clays. Extensive geological and geotechnical investigations were carried out by Istanbul University in Bakirkoy district for site safety evaluation and microzonation studies (Istanbul University, 2000). Cinicioglu et al. (2001, 2003a, 2003b) conducted site-specific response analysis for the region using SHAKE91 software and calculated horizontal acceleration values ranging from 0.41g to 0.54g in the studied region.

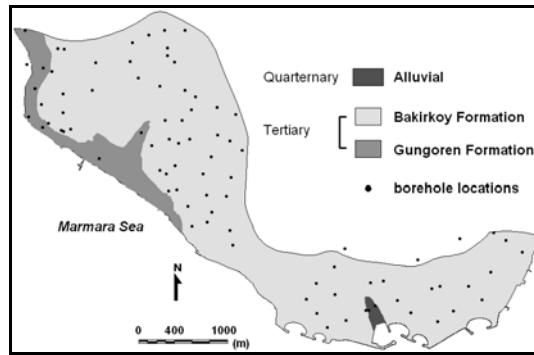


Figure 5. Surface geology of Bakirkoy district with boring locations.

The methodology developed in this paper was applied to Bakirkoy district for the probable future earthquake. Based on a comprehensive study conducted for the district (Cinicioglu et al, 2003a, Cinicioglu et al., 2005b), there are about 11 000 residential buildings with different number of storeys, foundation properties and construction qualities in Bakirkoy district. Cinicioglu et al. (2003a, 2005b) reported that the percentage of buildings with one or more basements is about 81%, whereas only 19% of the buildings are without basements. The survey also showed that majority of buildings had shallow foundations with strip footings.

In applications, q_{region} was selected as 200 kN/m² based on the local civil engineering experience and applications in the district. In two of the analyses, factor of safety values were taken as 3.0. Two more analyses were conducted for a factor of safety of 2.0 to determine the effects of using lower safety factors on the amount of seismic settlements. Soil properties such as unit weight, internal friction angles and site-specific response parameters were taken from the reports prepared by Istanbul University, (2000) for the region and from Cinicioglu et al. (2001) and (2003b), who conducted detailed site-specific response analysis in the region.

Application 1: The first application for expected seismic settlement belongs to buildings with $B=1$, $d=3$, $d/B=3$ and factor of safety of 3.0. This analysis considers the buildings with shallow foundations and one basement. As depicted in Figure 6, no seismic settlements are expected in the whole region, due to the positive effect of basement on seismic bearing capacity values.

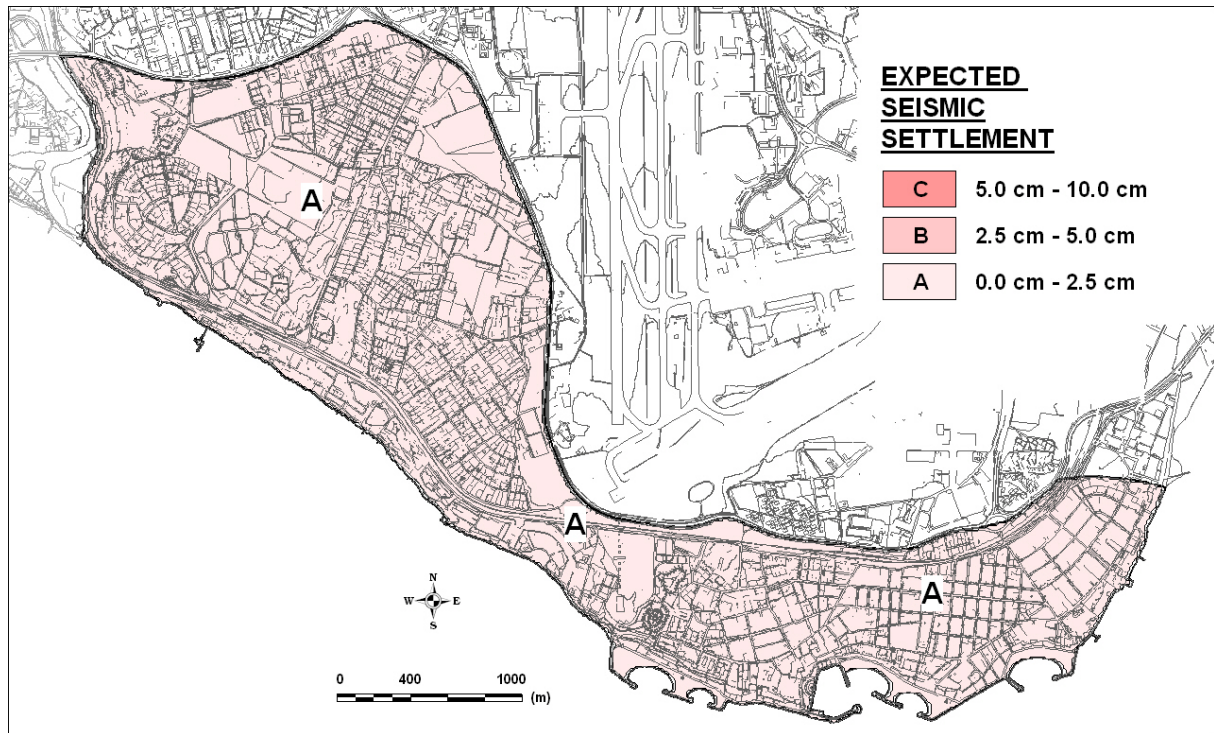


Figure 6. Expected seismic settlements in Bakirkoy District for buildings with one basement

Application 2: The second application analyses the expected seismic settlement for the buildings with $B=1$, $d=1.5$, $d/B=1.5$ and factor of safety of 3.0. This analysis covers the buildings with shallow foundations and no basement. As seen from Figure 7, some regions in the district are prone to seismic settlements due to seismic bearing capacity reductions. The expected settlements are less than 5 cm in the whole region.

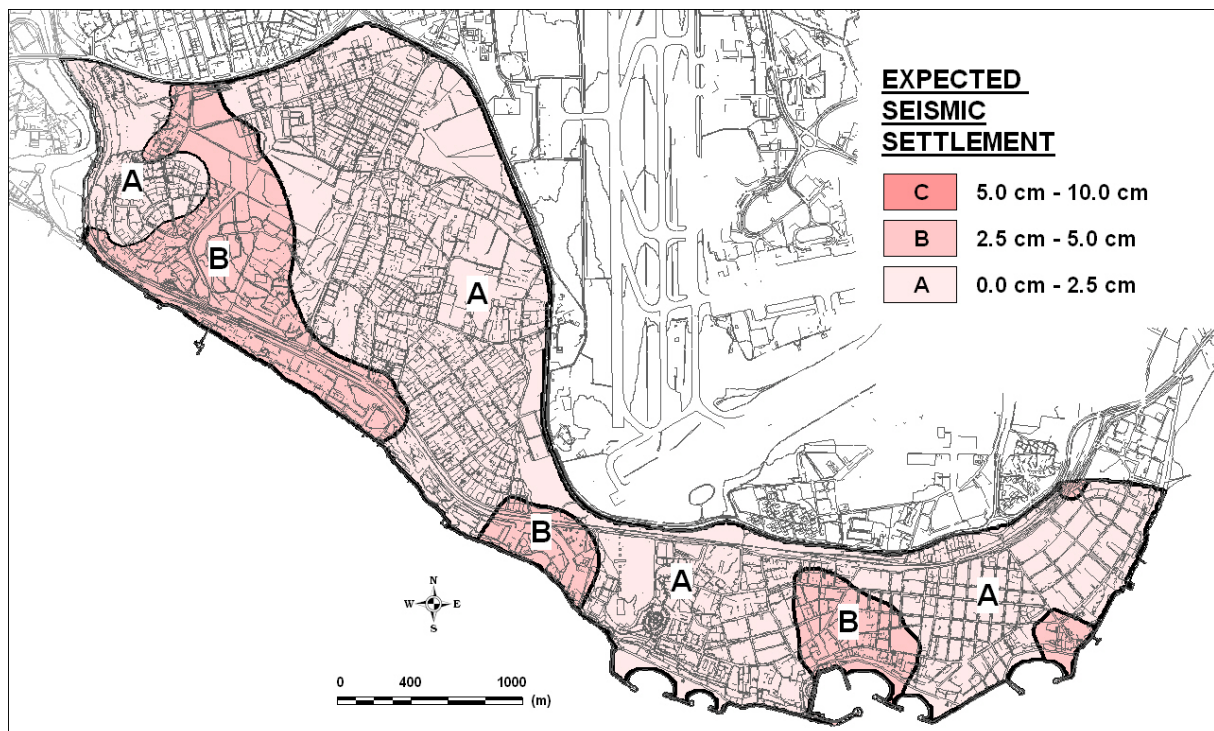


Figure 7. Expected seismic settlements in Bakirkoy District for buildings with no basement

Application 3: The third application has been made for the expected seismic settlement for buildings with $B=1$, $d=1.5$, $d/B=1.5$ and factor of safety of 2.0. This application aims to find out the effect of factor of safety on expected seismic settlement. Using lower factors of safety in the static design increased the seismic settlements up to 10 cm in some of the regions.

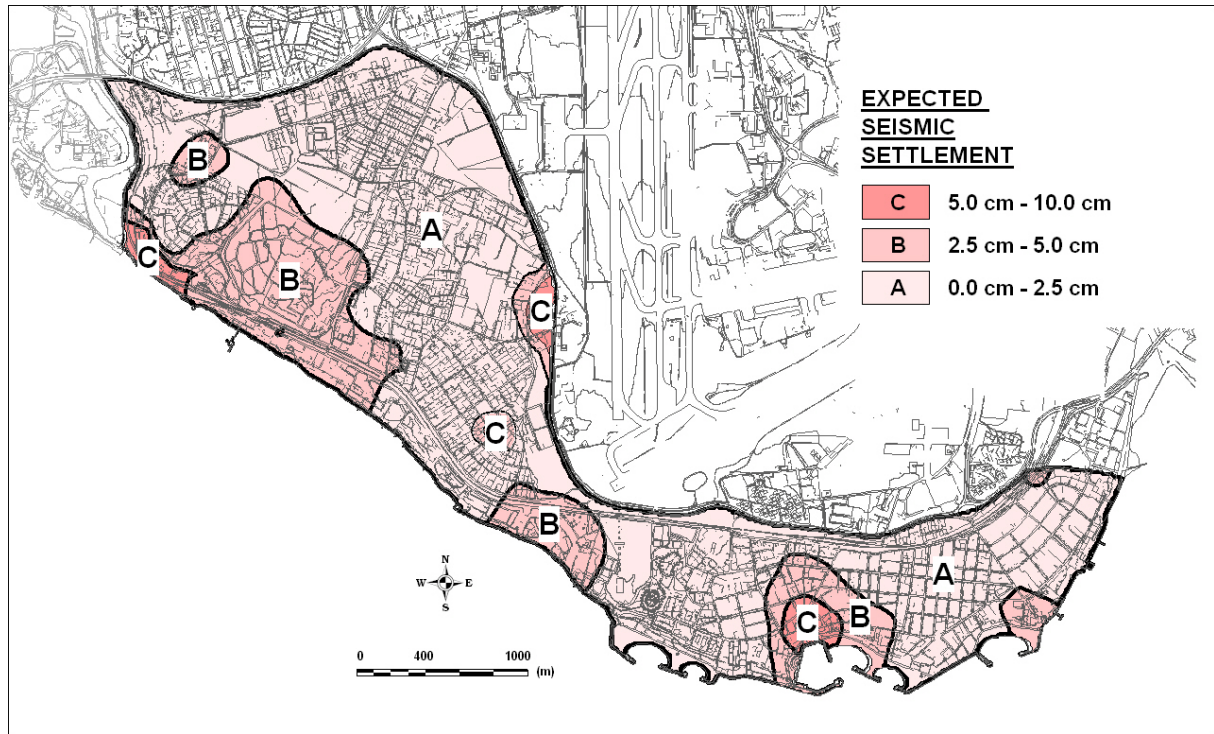


Figure 8. Expected seismic settlements in Bakirkoy District for buildings with no basement and lower static factor of safety

Application 4: The fourth application is for the expected seismic settlements for buildings with $B=2$, $d=0.5$, $d/B=0.25$ and factor of safety of 2.0. This analysis aims to investigate the buildings with very shallow footings and lower factors of safety. As expected, very shallow footings and lower factor of safeties resulted in higher expected seismic settlements. In some regions, settlements were greater than 10 cm.

CONCLUSIONS

It is essential to understand the basic mechanisms of soil behavior and soil structure interaction in order to manage with the earthquake caused ground damages. In this context, this paper aims to emphasize that seismic bearing capacity degradation can cause seismic settlements, and hence depending on the amount of these settlements, structural damage. A new approach is developed in this study to consider seismic bearing capacity degradation and consequent settlements as a part of spatial damage assessment studies. The developed methodology was applied to Bakirkoy district of Istanbul, which is under the threat of a future earthquake. The application of the methodology revealed that bearing capacity reduction during earthquakes may cause considerable settlements if not local or general ground failures. The effects of basements and higher factor of safety values were pronounced in achieving lower seismic settlements beneath the structures.

The authors believe that degradation of seismic bearing capacity has been responsible for several ground failures in the past earthquakes. However, the main focus of recent research seems to be diverted solely to the liquefaction phenomenon and this unfortunately casts shadow on seismic bearing

capacity degradation mechanism, which may well be the more probable cause of some of the observed performances.

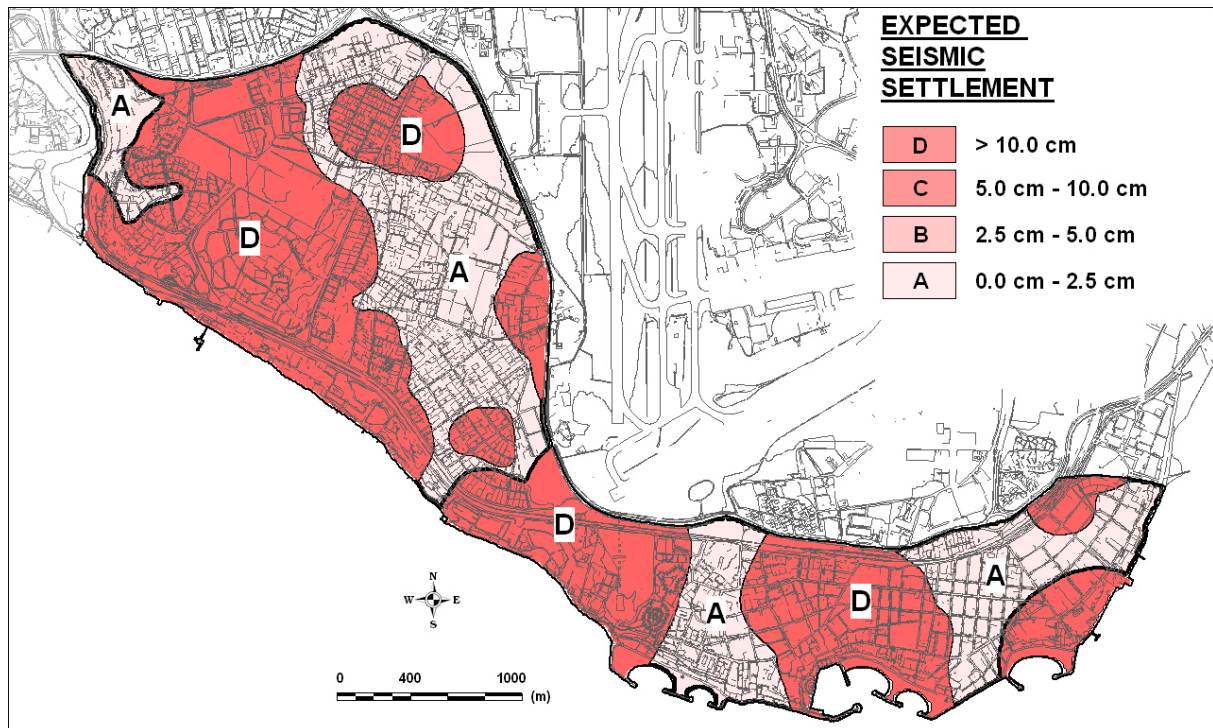


Figure 9. Expected seismic settlements in Bakirkoy District for buildings with no basement, very shallow footings and lower static factor of safety

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