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# Evaluation of system response and liquefaction damage assessment tools applied to Adapazari cases in Kocaeli 1999 earthquake

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ABSTRACT: Ground movements in Adapazari (Turkey) during the 1999 Kocaeli earthquake caused large devastations in the city, which were largely attributed to liquefaction of low plasticity silty soils underneath buildings with shallow foundations. In this study, CPT and laboratory test data were compiled adjacent to the liquefaction-induced damaged buildings. To understand the capability of system response approach and damage assessment tools such as LSN and LPI, fifty five cases were selected. Results revealed that the system response can be applied to shallow silty soils in Adapazari. It was evaluated that soils with soil behavior index ($I_c$) greater than 2.6 may liquefy (even if they are located over the ground water level), if they have Plasticity Indices lower than 15%. Another important finding was that LSN values calculated from the foundation level were better indicators than LPI values for estimating the liquefaction induced damage level in Adapazari cases.

## 1 INTRODUCTION

Adapazari, which is the capital of Sakarya City in Turkey, was affected by a large earthquake in 17 August 1999 and widespread damage to buildings occurred throughout the city. Some of the damages were attributed to liquefaction, which caused excessive settlements, tilt and bearing capacity failures for several buildings on shallow foundations on saturated low plasticity silty soils and silt mixtures. Liquefaction was pervasive in some parts of the city, producing several phenomena ranging from minor settlement of buildings to complete toppling of structures (Bardet et al., 2000). Figure 1 shows photographs of buildings which suffered liquefaction induced damage in Adapazari. Following the earthquake, an extensive field investigation was carried out in the city by different parties (Bay & Cox, 2001,Yoshida et.al, 2001, Sancio, 2003, U.C Berkeley et al., 2003). These investigations consisted of assessment of liquefaction-induced damage levels and conducting soil investigations in the area including CPT's, boreholes, in situ tests and laboratory tests in order to understand the underlying mechanisms for liquefaction. The results of these studies were published by several researchers (Sancio et al., 2004, Bray et al.,2001, Bol et al., 2010, Quintero et al. 2018). Ground failure in Adapazari was primarily observed adjacent to buildings (Bray et al, 2001) and Sancio et al. (2003) attributed this, among other factors to the detrimental effect of an increase in confining stress on the cyclic strength in terms of cyclic stress ratio. This may mean that effects of the structures on the liquefaction susceptibility of soils should be taken into account in liquefaction triggering analyses.

This study aims to revisit the liquefaction induced damage in Adapazari based on the latest state of the art on the subject and in this context, 55 buildings with well documented

Figure 1. Typical photographs for buildings with Liquefaction Induced Damage (LID) (Provided by Adapazari and Sakarya Municipalities)

liquefaction induced damage performances were investigated in detail. The liquefaction induced ground damage data which was utilized, was compiled based on the reports of Bay & Cox, 2001, Yoshida et. al, 2001, Sancio, 2003, Sancio et al., 2004, Bray et al., 2001.

Following the Christchurch earthquakes in 2010 and 2011, Cubrinovski et al. (2018) recommended "soil system response" to evaluate the liquefaction-induced damage. In this concept following main considerations are as below:

a. Liquefaction occurs in the first ten meters.
b. The liquefaction is dominated by the critical layers ($L_{crit}$) and in this context, the shallowest critical layer is of outmost importance. In this approach critical layer is defined as the layer which is most likely to trigger and manifest liquefaction at the ground surface of a given site. The critical layer and the layers of low liquefaction resistance that are vertically continuous form the critical zone.
c. The critical layer is characterized by $q_{c1ncs}$<85.
d. The liquefiable layers between the critical layers and even the thin layers which are not liquefiable may contribute to the liquefaction. However, interbedded deposits of liquefiable and non-liquefiable soils were accepted to result in vertical discontinuity in soils that did not liquefy.
e. The first 2.5 m layer from the ground surface which does not liquefy is called the crust layer and the presence of a crust layer prevents liquefaction.
f. The soils above the groundwater table can liquefy due to seepage induced liquefaction. When liquefaction occurs in the critical zone, due to vertical communication of excess pore water pressures, the soil above the water table at shallow depths liquefies due to an upward flow from the critical zone.

Within the context of this paper, the applicability of these considerations for Adapazari cases was investigated through the studied database. Well known liquefaction damage assessment tools; Liquefaction Potential Index (LPI) and Liquefaction Severity Index (LSN) were applied on 55 cases to evaluate their capability of capturing the real damage. These approaches were implemented through CPTU test data and it was also aimed to see the applicability of these methodologies to saturated silts and silty sands which are dominant in the studied area.

The boundary value for soil behavior index ($I_c$) and the occurrence of seepage induced liquefaction in Adapazari silty soils above the groundwater level were questioned based on the studied case studies. Classical and modified approaches for liquefaction vulnerability assessment were compared based on their capability to assess the liquefaction-induced building damage level. Some recommendations were given.

The results were then mapped in GIS platform and spatial distribution of damage levels were compared with the actual damage levels. Probable underlying mechanisms for the liquefaction induced damages in Adapazari were also briefly presented.

## 2 METHODOLOGY

The study carried out for this research consisted of several steps. The first step was gathering the liquefaction induced ground damage database using the reports and documentation that were created after the 1999 Adapazari earthquake (Bay & Cox, 2001,Yoshida et. al, 2001, Sancio, 2003, Sancio et al., 2004, Bray et al., 2001). Several site visits were carried out by the authors to the city in years 2017 and 2018 to understand the locations of the buildings and see their current conditions. All the data was gathered and a database of approximately 1200 buildings which suffered from liquefaction induced damage was created. In the next stage, the soil data was gathered based on a very detailed literature study. The geotechnical investigations that were used in the analyses included the CPTu's that had been carried out in the studies following the earthquake (Bay & Cox, 2001,Yoshida et. al, 2001, Sancio, 2003, Sancio et al., 2004, Bray et al., 2001). Some additional data (50 boreholes and 20 CPTUs) was provided by the Adapazari and Sakarya Municipalities and for the locations where the data was lacking, new CPTU's and boreholes (thirteen CPTu's and thirteen boreholes) were performed within the context of this study. This made a total of about 98 CPTU soundings and relevant soil data for the whole region. 55 of these CPTU data that were adjacent to the buildings where liquefaction induced damage was observed were coupled with these buildings.

In Adapazari, the building stock consists mainly of two to six story concrete frame buildings with a rigid raft foundations which are located at depths of typically 1.0 to 1.5 m.. The number of structures that were investigated within the context of this paper is 55 and are all the same typology described above. The following data were compiled for each structure. The coordinates and photographs of the building, amount of settlement, tilt and lateral movement if there are any, presence of sand boils in the vicinity of the building, adjacent CPT data and other field and laboratory data if available. It should be recalled that the analyses could be carried out for the buildings which did not collapse, since otherwise it would be impossible to observe ground failure.

For these 55 cases, the liquefaction damage assessment tools were applied to the most rigorous level and the expected damage levels were determined. Ground Failure Indices (GFI) developed by Bray et al. (2001) were also used as a guide. This classification system is given in Table 1. This damage evaluation system uses an index for observed damage ranging from "no observable ground failure" to "significant ground failure" level. For each building, "Observed" ground damage level was also evaluated based on the LPI and LSN classifications given in Tables 2 and 3. The relevant grades and damage levels were assigned based on all these classifications. It should be recalled that the majority (50) of the buildings in the 55 building database suffered from GF2 and GF3 level ground damage. The whole CPTu data that was compiled in this study was used in creating the spatial distribution of the damage

Table 1. Ground Failure Index (GFI) classification system (Bray and Stewart, 2000)

| Index | Description | Interpretation |
|---|---|---|
| GF0 | No observable ground failure | No vertical movement, tilt, lateral movement or boils |
| GF1 | Minor ground failure | $\Delta$<10 cm; tilt of >3-story buildings <1°; no lateral movements |
| GF2 | Moderate ground failure | 10<$\Delta$<25 cm; tilts of 1°-3°; small lateral movements (<10cm) |
| GF3 | Significant ground failure | $\Delta$>15 cm; tilts of>3°; lateral movements>25 cm |

Note: $\Delta$=Vertical movements

Table 2. Liquefaction Potential Index (LPI) – Iwasaki et al (1982) & Sonmez (2003)

| LPI | Expected Damage Level |
|---|---|
| 0 | No liquefaction |
| 0 - 2 | Low |
| 2 - 5 | Moderate |
| 5 - 15 | High |
| ≥ 15 | Very high |

Table 3. Liquefaction Severity Number (LSN) by Tonkin and Taylor Ltd. (2013)

| LSN | Expected Damage Level |
|---|---|
| < 10 | None to Little |
| 10 - 20 | Minor |
| 20 - 30 | Moderate to Severe |
| 30 - 40 | Severe |
| 40 - 50 | Major |
| >50 | Extensive |

assessment indicators and GIS mapping. The spatial damage indicator distribution was compared with the near 1200 building database that was compiled in the last part of the paper.

### 2.1 *Study area and database*

The geology of the studied region, the locations of the CPTu's, boreholes, SPT's together with the estimated peak ground accelerations are presented in Figure 2a. The location of the North Anatolian Fault is also located in the map, and it can be seen in this figure that the distance of the fault to the studied area ranges from 5 km to 10 km. The figure shows that the studied area lies between two rivers and the main geological unit in the area is Alluvium. Figure 2b shows the locations of buildings which were reported to suffer from liquefaction-induced damage. Figure 2c shows the locations of the all damaged buildings, both due to liquefaction and ground shaking. There was no or limited development in some parts which are shown in Figure 2c.

### 2.2 *Soil conditions in the studied region and liquefaction triggering analyses*

Holocene alluvial deposits of the Sakarya River overlie older lake bed sediments in Adapazari. Bardet et al. (2000) emphasize that due to active sedimentation and fluvial action, the subsurface conditions in Adapazari are such that large variations of soil type and state are to be expected in both the vertical and horizontal directions. The subsoil is heterogeneous and consists of fine sands, silty sands, silty clays and gravels. The typical soil profile consists of alternating layers of silty sand and silty clay and non-plastic silts. The groundwater table lies generally at 1 m to 2 m depth from the ground surface. In this context, the soil profile consists of potentially liquefiable soil layers.

Figure 3 shows a typical profile from the area. The figure depicts that the soil profiles consist of silty layers with different thicknesses ranging from 0.7 m to 7.8 m. The depths of the upper silty layer from the ground surface varies from 0.5 m to 4.4 m. The normalized clean sand equivalent cone tip resistance ($q_{C1Ncs}$) values in these silty layers are small, in the range

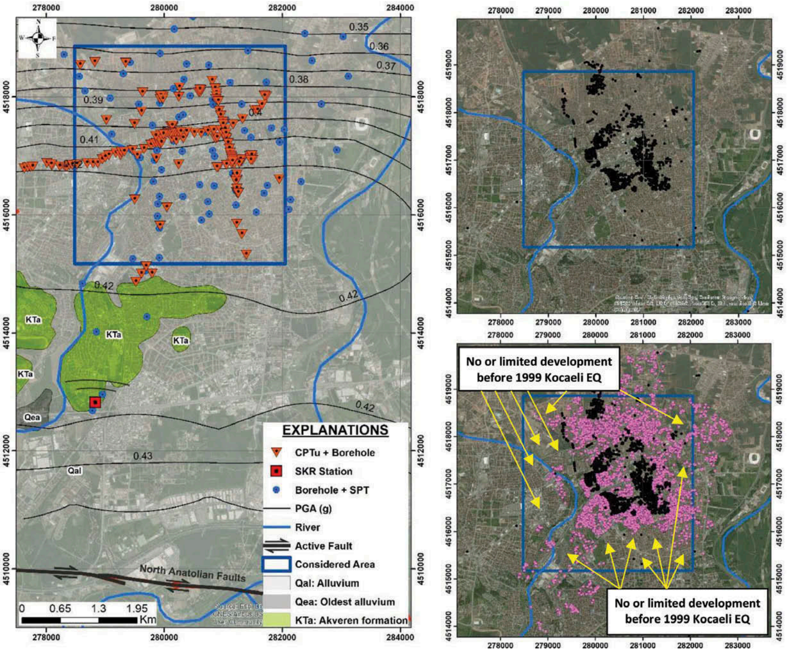


Figure 2. (a) Geology and seismic demand map including field investigations, (b) Liquefaction Induced Damage (LID) buildings, (c) All damaged buildings in Adapazari following the 1999 Kocaeli Earthquake

of 40-80, generally less than 70 and the soil behavior type index ($I_c$) values ranges in the 1.2 – 3.2 spectrum. The soil profiles were obtained by idealizing $q_c$ and fs profiles (Figures 3a and 3b) and they were also evaluated from a system response point of view. These typical profiles were evaluated in terms of critical layers ($L_{crit}$) which has been defined by Cubrinovki et al. (2018), as the layer that is most likely to trigger and manifest liquefaction at the ground surface of a given site. The detailed evaluations showed that the candidate critical layers laid

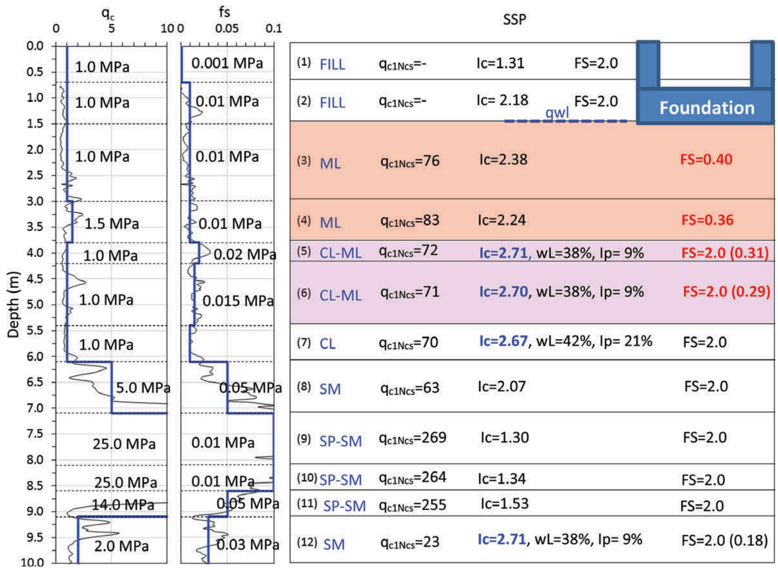


Figure 3. Soil profiles in Adapazari

between 0.3 m and 4.2 m. The depth of the crust layer which is defined as the soil layers that do not liquefy ranged between 0.5 m and 4.4 m.

In order to perform liquefaction damage assessment indicators, liquefaction triggering analyses have to be performed in the first place. In this context, Cyclic Stress Ratio (CSR) and normalized clean sand equivalent cone tip resistances ($q_{C1Ncs}$) were used to calculate the factor of safety values through the depth using the procedures given by Bounlanger and Idriss (2016) and Robertson and Wride (1998). The method developed by Robertson and Wride (1998) uses soil behavior index ($I_c$) which is based on cone tip resistance ($q_c$) and sleeve friction ($f_s$). Soils with higher $I_c$ values behave more like fine-grained materials and those with lower values like granular materials. In this context, this methodology accepts that soils with $I_c$ values greater than 2.6 are too clay rich to liquefy and therefore these soil layers are accepted to not to liquefy. This $I_c$ value of 2.6 has also been accepted by Cubrinovski et al. (2018) as the upper boundary of liquefiable soils. The presence of $I_c$ values greater than 2.6 in the shallow depths in typical Adapazari profiles where significant liquefaction-induced damage makes this boundary values open to discussion and this is one of the points considered in this paper.

### 2.3 *The earthquake and maximum horizontal acceleration*

The city of Adapazari is located 7 km north of the fault rupture which caused a 7.4 magnitude earthquake. As detailed in Rathje et al. (2000), the Sakarya strong motion in Adapazari is located on stiff soil in the southern part of city. The largest recorded maximum horizontal acceleration was 0.41g, however the softer sediments underlying the severely damaged sections in Adapazari is believed to amplify the intensity and increase the long period content of ground motions. In this paper, an attempt was made to determine the maximum horizontal acceleration for each studied location. For this purpose, EERA (2000) software was used. Based on these calculations, the $a_{max}$ values at the ground surface were found to range between 0.32g and 0.44g in the studied area and for each location, the corresponding $a_{max}$ values were used in the liquefaction triggering analyses.

### 2.4 *Liquefaction damage assessment indicators*

Different liquefaction damage assessment indicators were evaluated in this study in order to compare with the observed ground damage. All the vulnerabilities were calculated both from ground surface and from the foundation base. However, all the liquefaction factor of safety values were calculated for the free-field conditions. These indicators are Liquefaction Potential Index and Liquefaction Severity Index; applied in their original form and also from a system response point of view. Some modifications were also made regarding the soil behavior index, in order to include the soils with soil behavior index greater than 2.6 as being liquefiable depending on the Plasticity Index value. In this context, soils with soil behavior index greater than 2.6 and lower than 2.8 were accepted to liquefy in case their Plasticity Index (PI) values were less than 15. The PI criteria was based partially on the literature on the subject. Chinese criteria (Wang, 1979) classifies the soils with PI <12 and $w_c$/LL>0.85 (where $w_c$ is natural water content) as liquefiable soils. Sancio et al. (2003) classifies the soils with PI <12 and $w_c$/LL>0.85 as liquefiable soils. Sancio et al. (2003) determined that soils with PI>20 did not generate significant cyclic strains after a large number of cycles at low confining stresses representing the mean effective stress for soils under the corner of the mat foundation of typical 4 to 5 story structures in Adapazari. In this context, PI of 15 can be a reasonable boundary.

Following Cubrinovski et al. (2018), seepage induced liquefaction may occur in a soil layer above groundwater level, therefore in this study, seepage induced liquefaction was considered as one of the factors within the damage assessment indicators. The definition of the liquefaction vulnerability indicators and expected damage levels associated with these indicators are given in the following sections. The indicators are classified as classical approaches, and modified approaches.

### 2.4.1 *Classical approaches*

The approaches in this category are named as LPI-1 and LSN-1. Liquefaction Potential Index (LPI) developed by Iwasaki (1978), indicates vulnerability to liquefaction effects and evaluates the liquefaction potential of the soil using the factor of safety, the thickness of the layer and the depth of the relevant layer. Liquefaction Potential Index is estimated as;

$$LPI = \int F_1 W(z) dz \tag{1}$$

where $F_1$=1-FS for FS ≤1.0, $F_1$=0 for FS>1.0 and W(z)=10-0,5z. The calculations are carried out for the top 20 m as it is accepted that liquefaction effect on the building is negligible at depths greater than 20 m. Liquefaction potential categories related to LPI are given in Table 2.

Liquefaction Severity Index (LSN), is a recent parameter which defines the liquefaction related vulnerability of structures. It was developed by Tonkin and Taylor Ltd. (2013) based on the liquefaction damage observations resulting from 2010 and 2011 New Zealand earthquakes. This value depends on the volumetric densification values and the depth weighted factor. The volumetric strains are calculated for layers with FS less than 2.0 and these values are then used to calculate the LSN values as given in Equation 2.

$$LSN = 1000 \int \frac{\varepsilon_v}{z} dz \tag{2}$$

In this equation, $\varepsilon_v$ is the volumetric densification or strain for 1D post-liquefaction reconsolidation and is calculated using Zhang et al. (2002) or Idriss and Boulanger (2008). z is the depth to the layer of interest in meters below the ground surface. With 1/z depth weighing factor, the effect of the depth is much more influenced compared to LPI. The liquefaction potential categories based on LSN are given in Table 3.

### 2.4.2 *LPI and LSN for the first ten meters*

The approaches in this category are LPI-2 and LSN-2. In this approach, the analyses were carried out for the first 10 meters as suggested by Cubrinovski et al. (2018).

### 2.4.3 *Modified approaches*

The last group of analyses were based on some modifications for Ic value boundary and seepage induced liquefaction concept recommended by Cubrinovski et al. (2018). These approaches were developed by Istanbul University-Cerrahpasa team within the context of the Liquefact project. The approaches in this category are named LPI-3, LPI-4, LSN-3 and LSN-4.

LPI-3 and LSN-3 values consider an upper boundary of 2.8 for $I_c$ values coupled with a Plasticity Index of 15%. These modification was made in order to capture the actual observed damage levels. This meant that the boundary for liquefiable soils was elevated in order to include silty soils with low plasticity indices. These indicators considered the top ten meters.

For LPI-4 and LSN-4 values, seepage induced liquefaction was accepted to occur in soil layers above the ground water table. However, seepage induced liquefaction was accepted to occur in a soil layer only in cases where it satisfied the following criteria; the soil behavior type index ($I_c$) causing liquefaction is less than 2.8 coupled with a Plasticity Index of 15 and normalized clean sand equivalent cone tip resistance value is less than 85. It is clear that the depth of the GWL (with respect to the layer bottom) should affect this seepage induced liquefaction mechanism. In the studied cases, this depth ranged from 50 cm to 150 cm.

## 2.5 *Evaluation of system response for the studied cases*

The evaluation of the results from a system response point of view revealed that almost all of the findings by Cubrinovski et al. (2018) were valid for Adapazari liquefaction cases in this study. The critical layers in Adapazari dominated the liquefaction damage and $q_{c1ncs}$<85.

While the system response evaluations by Cubrinosvski et al. (2018) defines that a top 2.5 m crust prevents the liquefaction-induced ground damage in Christchurch cases, in Adapazari cases, it was observed that a 3.0 m crust layer below the foundation base prevented liquefaction-induced damage. This was observed in five cases, where the liquefaction damage was not observed. Considering the crust layer depth from the foundation base level is consistent with the shear induced type of liquefaction that occurred in Adapazari.

### 2.6 *Statistical evaluation of liquefaction vulnerability assessment for Adapazari database*

The liquefaction damage assessment indicators were applied to 55 cases in Adapazari and the results were compared with the actual damage levels for the buildings. The estimation capability of each approach was evaluated in three categories. The estimation can either be a successful estimation of the damage level, an overestimation damage level or an underestimation of the damage level. After this evaluation was performed for each building, the percentages of these estimations in the whole database were calculated for each category, as a percentage. The findings are given below in Table 4. It should be recalled that the calculation were made both from the ground surface and from the foundation base level.

As seen in Table 4, when the estimations for LPI values calculated from the ground surface and foundation base are compared, the estimation percentages are found to be very close. The LPI values calculated from the ground surface and from the foundation level can predict the correct damage level as high as 50-70%. The "overestimation" percentage is about 18% for all approaches. LPI-1 and LPI-2 give similar performances in "successful estimation" percentages. The results reveal that there is not much difference if the LPI values are calculated for the first 10 m or 20 m, since the soils are generally not liquefiable for depths under 10 m in the studied area. There is slight increase in "successful" prediction percentages with LPI-4, with an estimation rate of 67%. It seems that extending the liquefiable criteria with modified $I_c$ value coupled with a PI value increased the estimation rate. In this group, LPI-4 shows the best performance with the lowest "underestimation" percentage and highest "successful estimation" percentage. It should be recalled that for LPI-4, seepage-induced liquefaction phenomenon together with the modified $I_c$ values coupled with a PI value is taken into account.

When the LSN results obtained from Table 5 are studied, it is seen that no overestimation is obtained in all cases. The main difference in the values obtained for "underestimation" and "successful estimation" rates is due to the considered level of the analyses; whether from the

Table 4. LPI results for 55 buildings in Adapazari

| | From ground surface | | | | From foundation base | | | |
|---|---|---|---|---|---|---|---|---|
| | LPI-1 | LPI-2 | LPI-3 | LPI-4 | LPI-1 | LPI-2 | LPI-3 | LPI-4 |
| Successful estimation (%) | 52.7 | 50.9 | 58.2 | **67.3** | 54.5 | 54.5 | 61.8 | **67.3** |
| Underestimation (%) | 29.1 | 30.9 | 23.6 | **14.5** | 27.3 | 27.3 | 20.0 | **14.5** |
| Overestimation (%) | 18.2 | 18.2 | 18.2 | **18.2** | 18.2 | 18.2 | 18.2 | **18.2** |

Table 5. LSN results for 55 buildings in Adapazari

| | From ground surface | | | | From foundation base | | | |
|---|---|---|---|---|---|---|---|---|
| | LSN-1 | LSN-2 | LSN-3 | LSN-4 | LSN-1 | LSN-2 | LSN-3 | LSN-4 |
| Successful estimation (%) | 16.4 | 16.4 | 20.0 | **40.0** | 63.6 | 63.6 | 87.3 | **96.4** |
| Underestimation (%) | 83.6 | 83.6 | 80.0 | **60.0** | 36.4 | 36.4 | 12.7 | **3.6** |
| Overestimation (%) | 0.0 | 0.0 | 0.0 | **0.0** | 0.0 | 0.0 | 0.0 | **0.0** |

ground surface or from foundation base. Foundation base analyses have significantly higher "successful estimation" rates, compared to ground surface estimation rates. With LSN values from ground surface calculations, the "successful estimation" are only as high as 16%, while with the foundation base, the "successful estimation" rates increase to about 64% with LSN-1 and LSN-2 approaches. The indicators calculated with LSN-3 and LSN-4 from the foundation base, have very high successful estimation rates, 87% and 96% respectively. It is clear that when the estimations are made based on the foundation level, the LSN becomes a much better indicator as compared to ground surface calculations. The superiority of the LSN approach when the calculations are performed for the foundation level is indisputable. This may be a proof that, with the foundation level closer to the shallow liquefiable layers, the effect of "z", in Equation 2 dominates the results and therefore the correct depth to the shallowest layer of any thickness is of outmost importance. The superiority of LSN-3 approach over LSN-1 and LSN-2 may be attributed to the modified Ic and PI criteria. However, consideration of seepage induced liquefaction together with the relevant Ic and PI criteria defined in this paper also seems a proper approach to estimate damage level.

### 2.7 *Spatial distribution of the indicators*

The spatial distribution of liquefaction assessment indicators are given in Figure 4 for LPI indicators and in Figure 5 for LSN indicators. The distribution is based on all CPT soundings in Adapazari. For all these soundings, the LPI and LSN indicators were calculated and they were mapped in a GIS platform. The approximately 1200 buildings in the database for having liquefaction induced damage were also located in the map and the level of consistency was evaluated.

For LPI indicators, the damage levels ranges from "Low to very high", while for LSN indicators, the range was between "None to little" to "Extensive". For LPI values from the ground surface and from the foundation base, the maps were generally similar, with the "foundation base" maps being slightly superior in capturing the liquefaction damage data. LPI-4 gives the most successful estimations spatially. For LSN values, the spatial distributions give significant differences for ground surface and foundation level calculations, with the superiority being with the foundation level calculations. LSN-3 and LSN-4 maps give the most successful predictions. In all cases, LSN seems to be a better indicator for liquefaction damage assessment for Adapazari soils provided that the calculations are made from the foundation base.

## 3 CONCLUSIONS

An extensive study regarding the liquefaction induced damage in Adapazari due to 1999 earthquake was carried out within the context of Horizon 2020, LIQUEFACT Project by the Istanbul University-Cerrahpasa team. The study included the compilation of a database for the buildings which suffered liquefaction induced damage. A very large database of damaged buildings was compiled and 55 of these buildings were studied in detail together with the relevant soil data. The soil profile included liquefiable saturated silts and silty sands. The liquefaction damage assessment indicators were calculated in their original form and with some modifications. The results were evaluated statistically and spatially. The results showed that LSN was a better indicator for estimating the liquefaction induced damage levels in Adapazari, in case the values are calculated from the foundation base. Among the LSN approach, the best estimation rates were obtained with LSN-3 and LSN-4 approaches, which took into account the modified $I_c$ and PI values and seepage induced liquefaction in weak layers overlying the groundwater level. This may be an evidence that, the unsaturated, shallow, weak silty layers (with normalized clean sand equivalent cone tip resistance less than 85) coupled with modified soil behavior index ($I_c$<2.8 and PI <15) contributed to liquefaction in 1999 Adapazari earthquake.

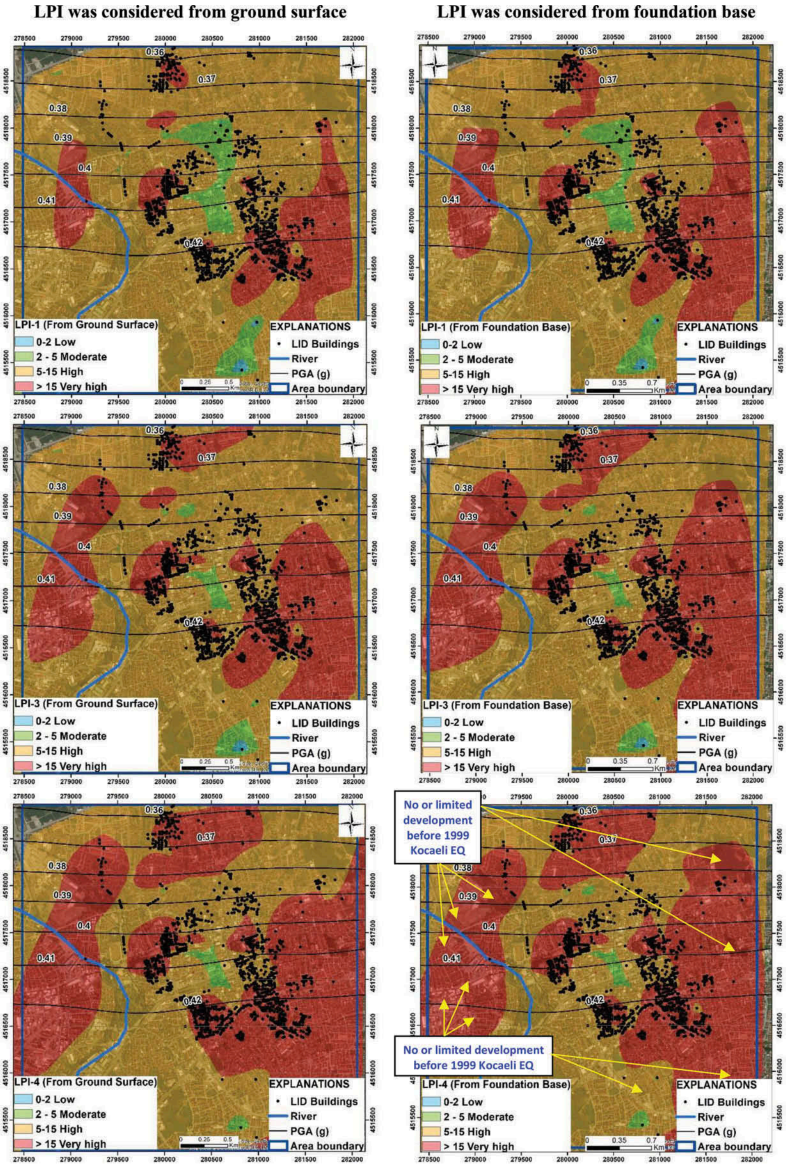


Figure 4. Comparison of LID buildings and Liquefaction Potential Index (LPI) maps for Adapazari considered from ground surface and foundation base

The study carried out in this paper showed that system response approach developed by Cubrinovski et al. (2018) could be applied with great success to Adapazari silty soils. In Adapazari, liquefaction occurred in the first 10 m as described in the system response approach, however with the effect of foundation and shear stresses. The critical layers dominated the liquefaction occurrence and the shallowest critical layer was of critical importance. The saturated silty layers below the foundation levels affected to the liquefaction damage considerably no matter how thin they were. The critical layers in Adapazari were characterized by $q_{c1ncs}$ values between 40-85 in all 55 cases. Seepage induced liquefaction was shown to occur in Adapazari cases, in cases where tip resistance, soil behavior index and Plasticity Index criteria were met. While the system response approach defines that a top 2.5 m crust prevents the

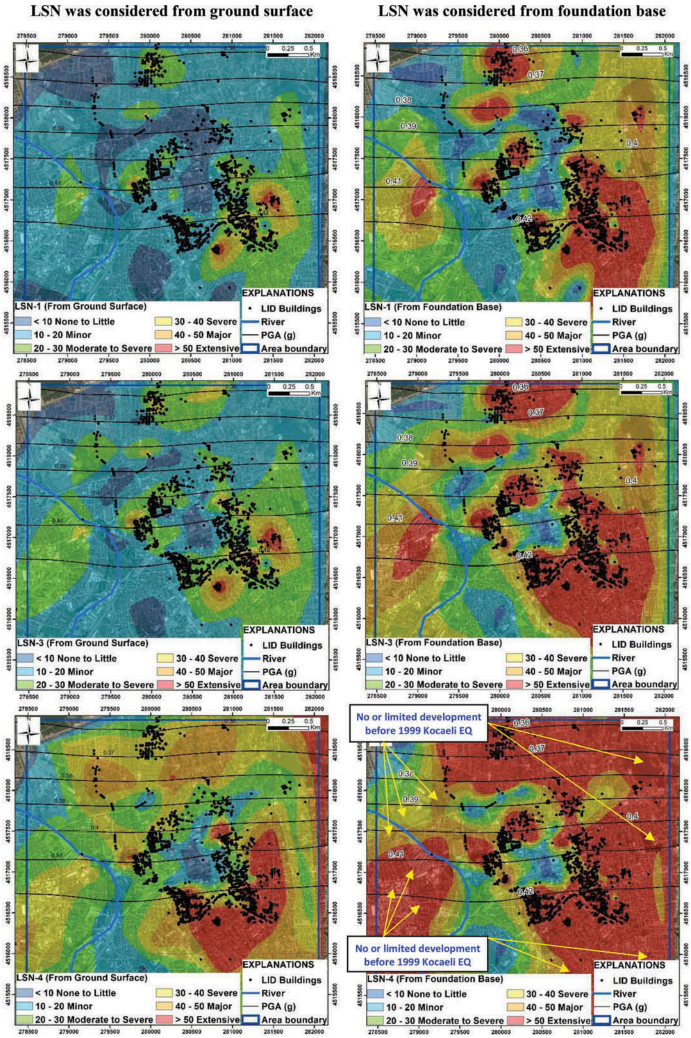


Figure 5. Comparison of LID buildings and Liquefaction Severity Number (LSN) maps for Adapazari considered from ground surface and foundation base conclusıons

liquefaction-induced ground damage in Christchurch cases, in Adapazari it was observed that a 3.0 m crust layer below foundation base prevented liquefaction-induced damage.

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