Design and evaluation of earthquake risk reduction measures in unplanned urban areas.

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ABSTRACT: Unplanned urbanization is an increasing trend worldwide and usually the built environment in such areas is exposed to various natural hazards, such as earthquakes, floods, landslides etc. Considering the severe earthquake-induced damages in unplanned urban areas, in the present paper, an earthquake risk reduction measure, which is applicable in shallow foundations on liquefiable soils is presented and applied for a typical structure. The predicted liquefaction-induced settlements of the foundation system, which affect the overall structural performance, are evaluated under a cost-benefit framework and useful conclusions about the applicability and associated cost of the method are drawn.

1 INTRODUCTION

In the past decades the speed and scale of urbanisation far exceeds any prior historic record. Urban dwellers are expanding at a very fast rate, especially in areas, which have never experienced similar development in the past (i.e. Asia and Africa). This trend is often combined with insufficient control of building activity, hence leading to unplanned city growth. Unplanned urban areas are characterized by (i) poor building quality, both in structural design and used materials, (ii) poor soil conditions, such as highly compressible or liquefiable soils, (iii) adverse terrain topography, such as hills and sloping ground.

The presence of one or all of the above features makes the specific areas particularly vulnerable to natural disasters, such as earthquakes. Extensive earthquake-induced damage (Nepal 2015, Haiti 2010) has led during the last decades to the adoption of a disaster risk reduction (DRR) philosophy, which is based on implementing upgrading policies in housing, infrastructure and amenities to mitigate the negative impacts of earthquakes. The financially viable policies are often appraised by means of Cost Benefit Analyses (CBA), which is an established tool for evaluating the benefits and costs of a specific project or activity (Shreve & Kelman, 2014).

Earthquake-induced liquefaction is amongst the most detrimental earthquake effects, and has been largely documented in urban areas, e.g. Kobe (1995), and Adapazari (1999). Excess pore pressure built up during shaking, leads to the severe degradation of shear strength of the foundation soil and the accumulation of excessive settlements. This in turn leads to widespread damage to buildings, which frequently exceeds the building capacity and leads to foundation failure.

The current design philosophy for building in liquefiable soils, dictates the installation of piles, which bypass the liquefiable layer and transfer the superstructure loads to deeper and non-liquefiable strata. Recent experimental and numerical studies [Liu & Dobry (1997), Naesgaard et al. (1998), Dashti et al. (2010), Sitar & Hausler (2012)], suggest that piles may be avoided, as long as a non-liquefiable layer, of adequate dimensions and shear strength exists on top of the liquefiable sand and its strength is taken appropriately into consideration. Dimitriadi et al. (2017 & 2018), propose a novel design methodology, which relies on the existence of a non-liquefiable layer on the soil surface, and allows the use of shallow foundations. The proposed method allows the evaluation of the liquefaction performance of the foundation, in terms of the accumulating seismic settlements (ρ_{dyn}) and the degraded post-shaking bearing capacity of the soil (q_{ult}).

In the present paper, a brief description of the DRR-CBA framework is initially provided. In the sequel, the proposed analytical methodology by Dimitriadi et al. (2017 & 2018) is briefly described and applied, in parallel with evaluating the obtained predictions from a cost-benefit perspective. Finally, a reflective narrative highlights the strengths and weaknesses of the proposed methodology and provides suggestions for future improvements.

2 DRR MEASURES AND COST BENEFIT ANALYSIS

Structural DRR measures need to combine, among others, information on construction materials, building configuration, structural outline, and engineering design quality (Reja et al., 2011). In economic terms, they have investment and maintenance costs, as well as potential benefits. The first are deterministic elements and are part of the risk reduction strategy, irrespectively of the probability of whether an earthquake occurs on not. The latter are probabilistic and arise only after the occurrence of an earthquake. Additionally, the benefits from reducing the anticipated earthquake losses are directly related to the magnitude of the seismic motion, in the sense that stronger earthquakes tend to cause more damages. The connection between the reduction of any anticipated losses and the earthquake magnitude is reversed when considering the probability of occurrence of a seismic event. Namely, stronger earthquakes (higher magnitude) cause more damage, but tend to occur less frequently, therefore have a lower probability of occurrence.

The above correlation is illustrated in a Loss-Exceedance Curve (LEC), which shows the anticipated damage as a function of the probability of exceedance of an earthquake (see Figure 1). The area underneath the curve represents the expected annual losses, hence the risk associated with the occurrence of a hazard. Shifting the loss-exceedance curve downwards is the principal purpose of DRR measures and the relative difference in the expected annual losses is a means of assessing their benefits. Constructing a LEC both prior and after the adoption of any DRR measures can be an extremely intricate task, which requires data on the hazard itself, the vulnerability and exposure of the built environment and the anticipated impacts.

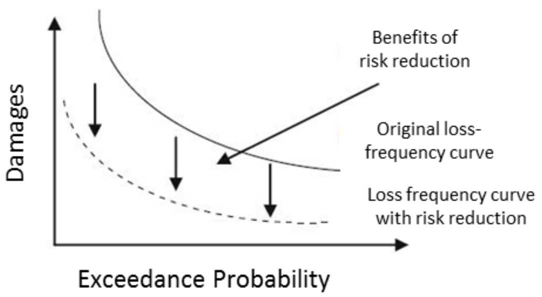

Figure 1: Benefits of DRR in shifting the loss-exceedance curve.

The assessment of DRR measures in monetary terms is typically performed using Cost Benefit Analyses (CBA). CBA are frequently used by policy-making agencies and governmental organizations, to compare costs and benefits of a project in monetary terms over a specific period. A set of three indices will be used here, i.e. the Net Present Value (NPV), the Benefit/Cost ratio (BCR) and the Present Value Ratio (PVR). These indices depend on the current interest rate; therefore, a sensitivity analysis will also be performed to evaluate the financial sustainability of the proposed DRR measure.

3 DESCRIPTION OF THE ANALYTICAL METHODOLOGY

In the analytical methodology of Dimitriadi et al. (2017, 2018), the non-liquefiable top layer is created artificially, through the installation of gravel drains, which is a technology frequently used for liquefaction mitigation. Gravel drains accelerate the excess pore pressure dissipation during seismic loading, and hence eliminate the possibility of liquefaction occurrence. Some excess pore pressures are still expected to develop, followed by shear strength degradation of the foundation soil, inevitably leading to the accumulation of seismic-induced settlements. However, those may be restricted to tolerable levels, ensuring that the superstructure will be far from collapse. The proposed methodology allows the specification of the dimensions of the improved zone around the foundation, as well as the accumulating seismic settlements (ρ_{dyn}) and the degraded post-shaking bearing capacity of the soil (q_{ult}). A typical configuration of the proposed foundation scheme is presented in Figure 2. The required input data for the application of the methodology and output information are outlined in Figure 3.

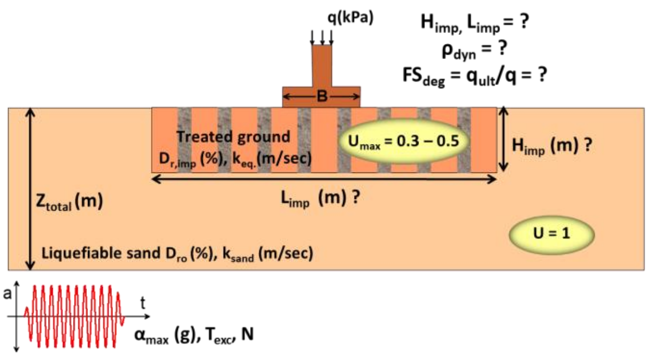

Figure 2: Typical configuration of the proposed foundation scheme of shallow foundations on top of liquefiable soils with prior ground improvement with the use of gravel drains.

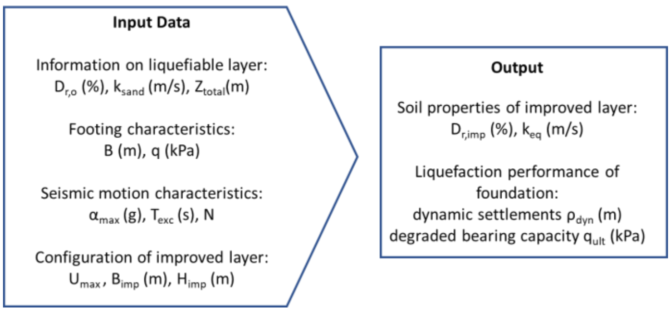

Figure 3: Flow chart with the required information for the application of the proposed methodology.

4 APPLICATION OF THE ANALYTICAL METHODOLOGY

For the application of the proposed methodology, it is assumed that the liquefiable sand layer has an initial relative density of $D_{r,o}$ = 45% permeability of $k_{sand} = 6.6*10^{-5}$ m/s, and total thickness of Z_{total} = 14 m. The strip foundation has a width of B = 5 m and bears a uniform pressure q (kPa). It is also conservatively assumed that the embedment depth D is equal to D = 0 m. The allowable maximum excess pore pressure ratio within the improved layer U_{max}^{i} is set equal to 0.40. The improved zone around the shallow foundation, has a width equal to B_{imp} = 6 m (hence extending 0.5 m from each side of the 5 m wide footing).

The analytical methodology is going to be applied for: (i) three different applied bearing pressure values q, i.e. q = 50, 75, 100 kPa, (ii) six earthquake motions of different magnitude, expressed through the maximum acceleration magnitude α_{max} (g) and the number of significant cycles N, as summarized in Table 1 (considering a constant excitation period T_{exc} (s) equal to T_{exc} = 0.35s), and (iii) two ground improvement depths, H_{imp} = 4 & 6 m. Note that the return period values (and annual probabilities) selected here, are estimates for demonstration purposes. A site-specific study considering local faulting and historical seismicity would be required for actual applications.

Table 1. Correlation between max. acceleration α_{max}, significant number of cycles N and return period.

α_{max} (g)	N	Return Period	Annual Probability
0.05	3	10	0.10
0.10	5	25	0.04
0.15	7	50	0.02
0.20	9	100	0.01
0.25	11	250	0.004
0.35	15	500	0.002

In risk assessment, the physical vulnerability refers to the degree of loss to a given element at risk or set of elements at risk, which result from the occurrence of a natural phenomenon of a given magnitude. It is expressed on a scale from 0 (no damage) to 1 (total damage). Following the above rationale, the obtained values of seismic settlements (ρ_{dyn}) will be normalized against a threshold settlement value, whose exceedance will indicate total damage (i.e. vulnerability value equal to 1).

This threshold is twofold and structured upon two performance criteria. The first refers to the post-shaking soil's bearing capacity. It is expressed through the degraded bearing capacity or the corresponding degraded factor of safety FS_{deg}. The degraded factor of safety essentially serves as an indicator showing how far from failure the improved soil – foundation system is, at the end of shaking, and is an indispensable part of the design process. This implies that cases in which the degraded factor of safety is less than unity cannot be acceptable in design and indicate soil foundation failure. Hence, the vulnerability should be set to unity.

The second relates to the foundation settlements, which the structure is able to withstand. Structural damage, such as wall cracking, is not caused by uniform settlements, but rather by the differential settlements developing between two footings and the subsequent angular distortion. Based on Eurocode 7, which provides a complete design approach regarding performance limit states, the maximum angular distortion β (=$\delta\rho_{max}$/L) of a framed building on isolated foundations located at a distance equal to L = 12 m, should not exceed the value of 1/150 in the Ultimate Limit State[ii]. Based on this criterion, the maximum differential settlement $\delta\rho_{max}$ will be equal to $\delta\rho_{max}$ = 8 cm. Based on the correlation proposed by Burland et al. (1977) and Bjerrum (1963) between differential and total maximum settlements, the estimated maximum settlement for the Ultimate Limit State is going to be in the order of ρ_{max} = 12 - 15 cm for sand foundation layers.

The normalised dynamic settlements, mentioned hereafter as losses, against the annual probability of each earthquake motion are presented in Figure 4. In the same figure, the losses prior to any liquefaction mitigation are represented as a horizontal, light grey coloured line. This representation assumes that any structure founded on shallow foundations directly on liquefiable soil faces an extremely high collapse potential. Therefore, the associated losses index will be equal to unity for any return period. Based on Figure 4, it is concluded that for H_{imp} = 4 m (upped graph), the anticipated losses are not affected considerably for applied foundation pressures up of to 75 kPa but are considerably greater for q = 100 kPa. Indeed, it appears that the selection of a 4 m thick ground improvement scheme for foundation pressure of 100 kPa fails to protect the structure even against a low to medium magnitude seismic event with annual probability equal to 0.04 (25-yr return period). This is not observed for H_{imp} = 6 m, where the anticipated losses are considerably reduced even for the maximum applied pressure q (kPa). Moreover, the beneficial effect from increasing H_{imp} (m) becomes more prominent for less frequent (i.e stronger) seismic events. This is better appraised in Figure 5, demonstrating the percent decrease in the anticipated losses from increasing H_{imp} from 4 m to 6 m against the return period (RT). It is observed that this decrease reaches a maximum for a RT from 50 to 100 years. It is also concluded that in the case of strong seismic motions (RT = 500 yrs) this effect is essentially eliminated. This implies that the anticipated damages greatly overcome the building capacity, and the collapse probability of the structure is extremely high.

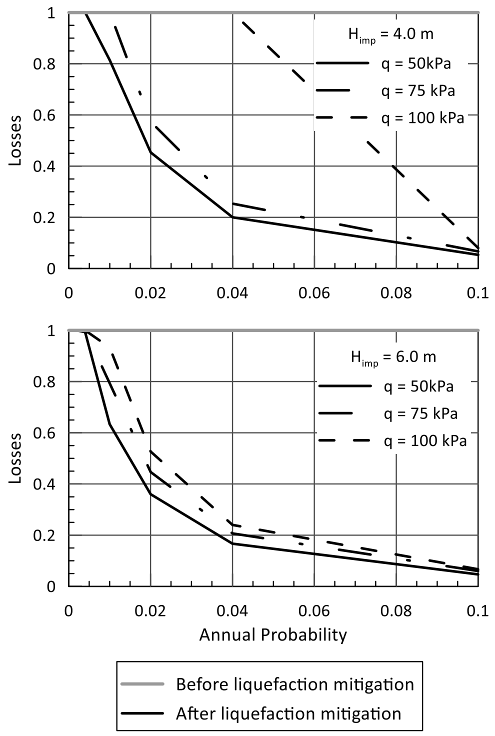

Figure 4: Losses before and after ground improvement for two ground improvement depths H_{imp} = 4 & 6 m and three applied foundation pressures q (kPa).

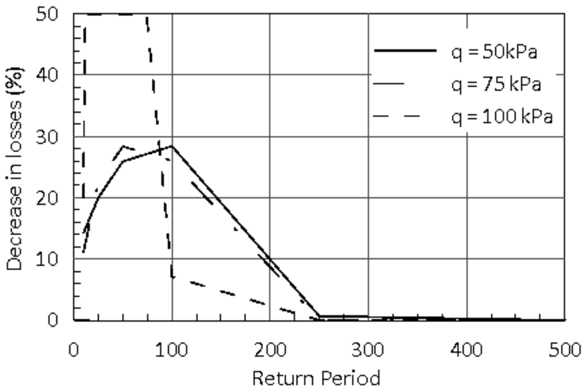

Figure 5: Percent decrease in anticipated losses from increasing the ground improvement depth H_{imp} from 4 to 6 m.

5 COST BENEFIT ANALYSIS

For the cost-benefit analysis, it is assumed that the foundation scheme is applied for a building with plan dimensions 48 m × 30 m, which is supported by 4 strip footings, of width equal to 5 m each, at a centre-to-centre distance equal to 12 m (as assumed for the evaluation of differential settlements) and length equal to L = 30 m (rendering a ratio of L/B = 6, classifying the footing into strip footings). Based on the required replacement ratio α_s[iii] (=$0.91 * D/s^2$ for a triangular arrangement), and a diameter of the drains D = 0.60 m, the centre-to-centre spacing will be equal to s = 2.40 and 2.20 m for H_{imp} = 4 and 6 m respectively. This is translated in total, to approximately 156 and 168 gravel drains for H_{imp} = 4 & 6 m. The total amount of running meters of gravel drains will be equal to 156 × 4 = 624 m and 168× 6 = 1008 m for H_{imp} = 4 and 6 m respectively.

According to Geotechtools.org, gravel drain installation incurs the following costs: (i) the cost of materials, which is estimated to range between 20 – 60 US$ per linear foot, that being translated into 60 – 180 US$ per linear meter, and (ii) the mobilization costs, which may range between 20,000 – 40,000 US$, with an average cost taken equal to 30,000 US$. Hence, the total estimated costs for the foundation system will be equal to 105,000 and 120,000 US$ approximately for ground improvement depths equal to H_{imp} = 4 and 6 m respectively. Due to space limitations, the CBA results for H_{imp} = 6 m will be presented in the sequel.

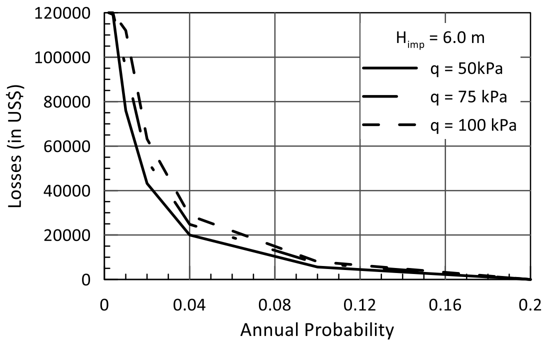

Figure 6: Losses in monetary values (US$) for ground improvement depth H_{imp} = 6 m and three applied foundation pressures q (kPa).

The Loss Exceedance Curve for H_{imp} = 6 m is presented in **Figure 6**. Based on the area underneath each of the three curves, the annual expected losses are calculated and are equal to 4,246, 4,794 and 5,220 for q = 50, 75 and 100 kPa. The monetary losses in case of no ground improvement will be equal to 12,000, assuming a horizontal initial LEC.

In the sequel, in **Figure 7**, the Net Present Value (NPV) is plotted against the applied Interest Rate (IR) (%), which ranges from 1% to 20%. It is observed, that NPV presents a wide variation with interest rate, which receives negative values for IR greater than about 5%, which implies that the project is not financially viable. Additionally, irrespectively of the applied foundation pressure q (kPa), all NPVs are positive for interest rate values up to 4% – 5%.

For the range of the interest rate values that NPV is positive, the Benefit-Cost ratio (B/C ratio) and Present Value Ratio (PVR) are calculated, and the obtained results are illustrated in **Figures 8** and **9**. From **Figure 8**, it is observed that the B/C ratio is greater than unity, i.e. the proposed foundation design is acceptable, for the greater part of the examined interest rates and decreases with increasing applied foundation pressure q (kPa). Similar conclusions are drawn from **Figure 9**. PVR is positive for interest rates up to 4% - 5%, depending on the applied foundation pressure q (kPa), indicating a financially sustainable foundation solution.

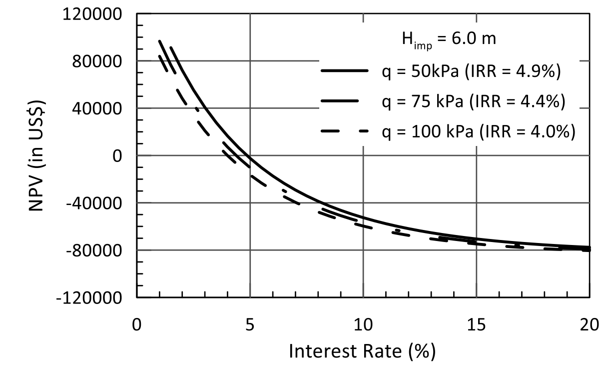

Figure 7: Net Present Value (NPV) against Interest Rate (%) for ground improvement depth H_{imp} = 6 m, and three different applied foundation pressures q (kPa).

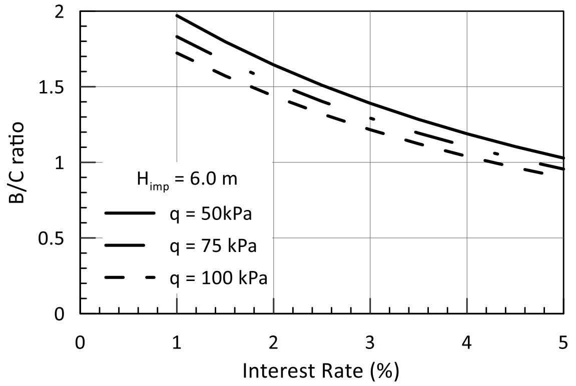

Figure 8: Benefit/Cost (B/C) ratio against Interest Rate (%) for ground improvement depth H_{imp} = 6 m and three different applied foundation pressures q (kPa).

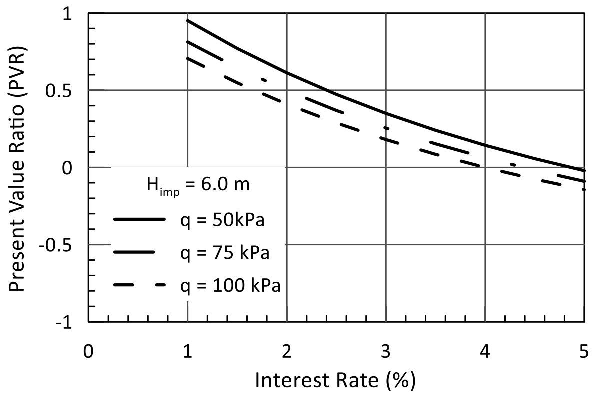

Figure 9: Present Value Ratio (PVR) against Interest Rate (%) for ground improvement depth H_{imp} = 6 m and three different applied foundation pressures q (kPa).

6 CONCLUSIONS

Earthquake-induced damages in unplanned urban areas with poor soil conditions, pose a significant threat to the exposed population and may result in extensive structural damage and casualties. Poor soil conditions may frequently refer to soils susceptible to liquefaction and recent history has shown that liquefaction-induced failure is not to be overlooked.

In the present paper, a novel foundation design methodology is proposed as a potential risk reduction measure against earthquake-induced liquefaction. Its main concept refers to the use of shallow foundations, following the appropriate ground improvement of the foundation soil with the use of gravel drains. The proposed set of equations is applied parametrically to appraise the seismic performance of a shallow foundation under a liquefaction regime. In the sequel, the analytical predictions are assessed my means of a Cost Benefit Analysis (CBA), to explore the method's optimum applicability, from a financial perspective. The main observations are outlined below:

The optimum configuration of the ground improvement depends on: (i) the performance objective of the shallow foundation (i.e. tolerable seismic-induced settlements), as well as on (ii) the probabilistic framework adopted regarding the seismic motion. The former greatly depends on the structural system of the building. In the present paper, a simple framed structure is examined, but soil-structure interaction effects are not considered. With regards to the latter, it greatly depends on the local fault system and historical seismicity, which may be the subject of a separate study. For the purposes of the present application, an approximation based on the literature and the Author's judgement is made.

The reduction in the anticipated losses is higher as the ground improvement depth H_{imp} (m) increases. The opposite trend is observed with increasing applied foundation pressures q (kPa).

The CBA reveals that the method is financially viable for interest rates up to 4% – 5%. Additionally, based on all the three indices calculated herein, for a ground improvement depth of H_{imp} = 6 m, there is an inversely analogous relation between the applied foundation pressure q (kPa) and the respective index of financial sustainability.

The analytical methodology refers to strip footings and is currently being extended in the case of square and rectangular footings. Additionally, soil-structure interaction effects are not considered, which could potentially increase the predicted the seismic settlements.

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[i] The excess pore pressure ratio U in generally defined as the ratio of the excess pore pressures (Δu) over the initial effective vertical stress (σ'_{vo}) and is an indication of the loss of shear strength of a soil layer at a specific depth. It ranges between 0 (no shear strength degradation) and 1 (onset of liquefaction and complete loss of shear strength). In most ground improvement methods, U_{max} typically ranges between 0.3 and 0.50.

[ii] Ultimate Limit State (ULS): A state beyond which the structure no longer fulfils the relevant design (performance criteria), expressed by loss of equilibrium of the structure as a rigid body, failure, collapse and loss of stability, failure caused by fatigue or other time-dependent causes.

[iii] Replacement ratio α_s is defined as the ratio of the plan view area of the gravel drain, over the area of the influence zone around the drain.