RESEARCH ARTICLE OPEN ACCESS

Evaluation of Post-liquefaction Reconsolidation Settlement based on Standard Penetration Tests (SPT)

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ABSTRACT
This paper aims to deal with the evaluation of post-liquefaction reconsolidation settlement of soils using Standard Penetration Tests data. Evaluation of the settlement is conducted at Semani site in Albania, according to the SPT method presented by Idriss and Boulanger 2008, 2010. The input data for the SPT method are SPT borings with depth, moment magnitude of the earthquake, maximum surface acceleration during earthquake, depth to ground water table, and the unit weights of the soils. The calculation procedure includes estimation of the cyclic stress ratio induced in the soil by the earthquake, cyclic resistance ratio that will cause liquefaction, factor of safety against the triggering of liquefaction, post-liquefaction strain and of the post-liquefaction reconsolidation settlement. The results of the calculations utilizing this procedure are shown in graphs and are compared to those based on CPT method. It is observed that the calculated post-liquefaction reconsolidation settlements based on SPT method are less than ones calculated based on CPT method.
***Keywords*:**Factor of safety,Liquefaction, Post-liquefaction reconsolidation strain, Standard penetration test, Settlement

I. INTRODUCTION

Liquefaction in saturated sand deposits is one of the most dramatic causes of damage to structures during earthquakes. Settlement of the soils induced by the earthquake is the vertical deformation of the ground surface caused by the reconsolidation of saturated sands after the shaking. This deformation is known as liquefaction-induced settlement or post-liquefaction reconsolidation settlement. Its evaluation is very important for the design of structures that can be constructed in areas where liquefaction is expected to occur.

Evaluation of post-liquefaction reconsolidation settlement requires evaluation of the liquefaction potential and post-liquefaction reconsolidation strain.

Potential of the liquefaction and post-liquefaction reconsolidation strain can be evaluated by different methods based on Standard Penetration Tests(here in after referred as SPT), Cone Penetration Tests(here in after referred as CPT) and Shear wave velocity (here in after referred as V_s) data.

Silver and Seed (1971), Tokimatsu and Seed (1987) were the first to propose the method for evaluating the post-liquefaction reconsolidation settlement in saturated sand based on the relation between cyclic stress ratio corrected SPT blow counts and post-liquefaction reconsolidation strain, ε_v. Ishihara and Yoshimine (1992), proposed the relations between thefactor of safety against the triggering of liquefaction, maximum shear strain γ_{max} and of the post-liquefaction reconsolidation strain ε_v, that were modified and improved by researchers such as Zhang et al., (2002), Yoshimine et al., (2006), Idriss and Boulanger (2008, 2010), Fred Yi (2010) for application to SPT, CPT and V_s data.

The aim of this paper is to evaluate the post-liquefaction reconsolidation settlement of soils at Semani site in Albania, based on SPT data according to the method presented by Idriss and Boulanger 2008, 2010 [1, 2].

In **Fig-1.,** presented below, is shownthe area of study that is a coastal zone of Albania where are performed 12 CPT soundings and 12 SPT borings up to 25 m.

According to the Geotechnical report, gravels, sands, silty sands, silty clays, and clays are presented in the zone and water table varies from 0.5 m to 1.5 m below the ground surface [3].

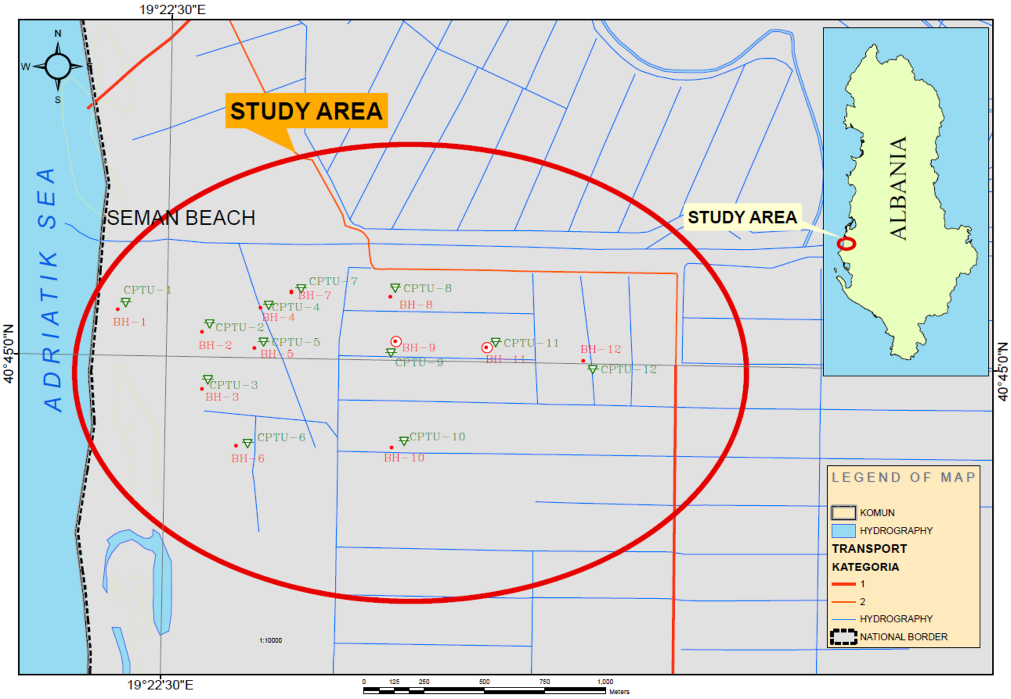

Figure1. Area of study

II. METHODOLOGY

Post-liquefaction reconsolidation settlement based on SPT data according to the method presented by Idriss and Boulanger (2008, 2010), is estimated in this paper.

The stress-based approach initiated by Seed and Idriss 1967 and presented by Idriss and Boulanger 2008, 2010, that compare the earthquake-induced cyclic stress ratio with the cyclic resistance ratio of the soil is used for evaluating the potential liquefaction. The relations proposed by Idriss and Boulanger 2008 are used for evaluating the post-liquefaction reconsolidation settlement of soils.

The results of the calculations utilizing this procedure are shown in graphs and are compared to those based on CPT method [4].

The calculation procedure includes estimation of the earthquake-induced cyclic stress ratio, cyclic resistance ratio, factor of safety against the triggering of liquefaction, post-liquefaction strain and of the post-liquefaction reconsolidation settlement. These parameters are presented below:

2.1 Earthquake-induced cyclic stress ratio $\left(CSR_{M,\sigma'_{vc}}\right)$

Earthquake-induced cyclic stress ratio, at a given depth, within the soil profile is estimated using the Seed-IdrissSimplified Liquefaction Procedure equation as follow:

$$CSR_{M,\sigma'_{vc}} = 0.65\frac{a_{max}}{g}\frac{\sigma'_v}{\sigma_v}r_d (1)$$

Where:

a_{max} = 0.26 is the peak gound acceleration for soil at Semani site according to Shkodrani et al. 2010 [5].

$r_d$is the shear stress reduction factor that account for dynamic response of the soil profile. Idriss (1999), in extending the work of Golesorkhi (1989), derived the following expression for this factor:

$$r_d = \exp(\alpha(z) + \beta(z)M) \quad (2)$$

$$\alpha(z) = -1.012 - 1.126\sin\big((z/11.73) + 5.133\big) \ (3)$$

$$\beta(z) = 0.106 + 0.118\sin\big((z/11.28) + 5.142\big) \ (4)$$

z = depth below the ground surface in meters; ≤ 20m;

M = 6.2 is the highest moment magnitude recorded to date, during the Fier earthquake of March 1962, according toSulstarova et al. 2010[6].

2.2 Cyclic Resistance Ratio $\left(CRR_{M,\sigma'_{vc}}\right)$

The correlation for Cyclic Resistance Ratio is developed for a reference $M = 7.5$, and $\sigma'_{vc} = 1$ and then adjusted to other values of M and σ'_{vc} as follow:

$$CRR_{M,\sigma'_{vc}} = CRR_{M=7.5,\sigma'_{vc}=1}.MSF.K_\sigma (5)$$

The following correlation between $CRR_{M=7.5,\sigma'_{vc}=1}$ and the equivalent clean sand $(N_1)_{60cs}$ value for cohesionless soils is developed by Idriss and Boulanger 2004, 2008:

$$CRR_{M=7.5,\sigma'_{vc}=1} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) (6)$$

$(N_1)_{60cs}$ is the equivalent clean-sand SPT penetration resistance.

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \quad (7)$$

$\Delta(N_1)_{60}$ is the equivalent clean-sand adjustment empirically derived by Idriss and Boulanger 2004, 2008. It is used to account for the effects of fine content on CRR.

$$\Delta(N_1)_{60} = \exp(1.63 + (9.7/(FC+0.01)) - (15.7/(FC+0.01))^2) \quad (8)$$

FC= fines content;
$(N_1)_{60}$ =the overburden corrected penetrations resistance $(N_1)_{60} = C_N N_{60}$ (9)
Idriss and Boulanger (2003, 2008) recommended the following relation for overburden correction factor C_N [7, 1]

$$C_N = (P_a/\sigma'_{vc})^m \leq 1.7 \quad (10)$$

$$m = 0.784 - 0.0768\sqrt{(N_1)_{60}};\ (N_1)_{60} \leq 46 \quad (11)$$

CRR of soils is affected by the magnitude scaling factor, MSF and overburden effective stress expressed by an overburden correction K_σ factor.
MSF is used to account for number of loading cycles on CRR. It is calculated based on the relation recommended by Idriss (1999). [8]

$$MSF = 6.9\exp(-M/4) - 0.058 \leq 1.8 \quad (12)$$

The overburden correction factor K_σ, is introduced by Seed 1983 to adjust the CRR value to a value of effective overburden stress. The following relation recommended by Idriss and Boulanger 2008 is used in this paper.

$$K_\sigma = 1 - C_\sigma \ln(\sigma'_{vc}/P_a) \leq 1.1 \quad (13)$$
$$C_\sigma = 1/\left(18.9 - 2.55\sqrt{(N_1)_{60}}\right) \leq 0.3;\ (N_1)_{60} \leq 37 \quad (14)$$

2.3 Factor of Safety against the triggering of liquefaction
The factor of safety against the triggering of liquefaction FS_{liq} is calculated as the ratio of the earthquake-inducedcyclic resistance ratio $\left(CRR_{M,\sigma'_{vc}}\right)$ to the cyclic stress ratio $\left(CSR_{M,\sigma'_{vc}}\right)$.

$$FS_{liq} = CRR_{M,\sigma'_{vc}}/CSR_{M,\sigma'_{vc}} \quad (15)$$

2.4Post-Liquefaction Reconsolidation Strain

Post-liquefaction reconsolidation strain ε_v is estimated based on the approach developed by Ishihara and Yoshimine (1992) expressed in terms of SPT penetration resistance as follow:

$$\varepsilon_v = 1.5\exp\left(-0.369\sqrt{(N_1)_{60cs}}\right).\min(0.08, \gamma_{max}) \quad (16)$$

Where:
γ_{max} = the maximum shear strain, as a decimal, calculated following the relationsderived from Yoshimine et al. (2006).

$$\gamma_{max} = 0 \ \ if\ \ FS_{liq} \geq 2 \quad (17)$$

$$\gamma_{max} = \min(\gamma_{lim}, 0.035(2 - FS_{liq})(1 - F_\alpha)/(FS_{liq} - F_\alpha)) \quad if\ 2 > FS_{liq} > F_\alpha \quad (18)$$

$$\gamma_{max} = \gamma_{lim} \ \ if\ \ FS_{liq} \leq F_\alpha \quad (19)$$

γ_{lim} = the limit of the maximum shear strain:

$$\gamma_{lim} = 1.859\left(1.1 - \sqrt{(N_1)_{60cs}/46}\right)^3 \geq 0 \quad (20)$$

F_α = the limiting values of FS_{liq}:

$$F_\alpha = 0.032 + 0.69\sqrt{(N_1)_{60cs}} - 0.13(N_1)_{60cs};\ (N_1)_{60cs} \geq 7 \quad (21)$$

2.5 Post-Liquefaction Reconsolidation Settlement
Post-liquefaction reconsolidation settlement is estimated according to Idriss and Boulanger (2008) as a function of the post-liquefaction reconsolidation strain:

$$S_{v-1D} = \int_0^{z_{max}} \varepsilon_v d_z \quad (22)$$

III. RESULTS

The results of the calculations are presented below in graphs and tables. Post-Liquefaction reconsolidation settlements are calculated based on SPT data following the procedure presented in the previous sections. The results of calculation based on CPT method presented by Idriss and Boulanger 2008 are also shown in these graphs. In the SPT method are primarily used the N measured values of SPT (**Fig. 2**) and then N values of SPT derived from CPT correlations (**Fig. 3** to **Fig. 5**). The calculated settlement based on CPT and SPT data are shown below in Table 1, 2 and 3.

IV. DISCUSSIONS AND CONCLUSIONS

Factor of safety against liquefaction, post-liquefaction reconsolidation strain and post-liquefaction reconsolidationsettlement were calculated based on SPT data following the procedure presented in the previous sections.
The graphs of the factors of safety in the SPT method based on measured N values of SPT show that the liquefaction phenomena is not expected to occur in this site. We think, this result is related to the accuracy of the test performance of SPT.
The graphs of the safety factors calculated using the SPT method based on N values of SPT derived from correlations CPT indicate that the liquefaction phenomena is expected to occur in this area.

The calculated post-liquefaction reconsolidationsettlements based on SPT method are less than ones calculated based on CPT method.
As it can be seen from Table 1, 2 and 3 the calculated post-liquefaction reconsolidationsettlements in this site are0.03m up to 0.07m based on SPT method and 0.15m up to 0.27m based on CPT method.

Table 1 The calculated settlement based on SPT andCPT method

Settlement	SPT-1	SPT-2	SPT-3	SPT-4
S(m)	0.07	0.06	0.04	0.05
Settlement	CPT-1	CPT-2	CPT-3	CPT-4
S(m)	0.26	0.23	0.23	0.18

Table 2 The calculated settlement based on SPT andCPT method

Settlement	SPT-5	SPT-6	SPT-7	SPT-8
S(m)	0.03	0.05	0.05	0.05
Settlement	CPT-5	CPT-6	CPT-7	CPT-8
S(m)	0.20	0.23	0.15	0.26

Table 3 The calculated settlement based on SPT andCPT method

Settlement	SPT-9	SPT-10	SPT-11	SPT-12
S(m)	0.04	0.05	0.03	0.03
Settlement	CPT-9	CPT-10	CPT-11	CPT-12
S(m)	0.24	0.27	0.22	0.18

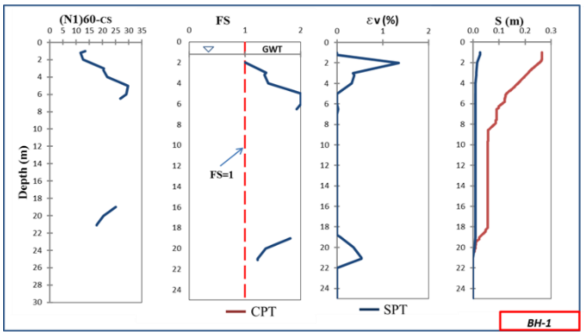

Figure 2 Evaluation of the post-liquefaction reconsolidationsettlement in SPT-1 with N measured values of SPT

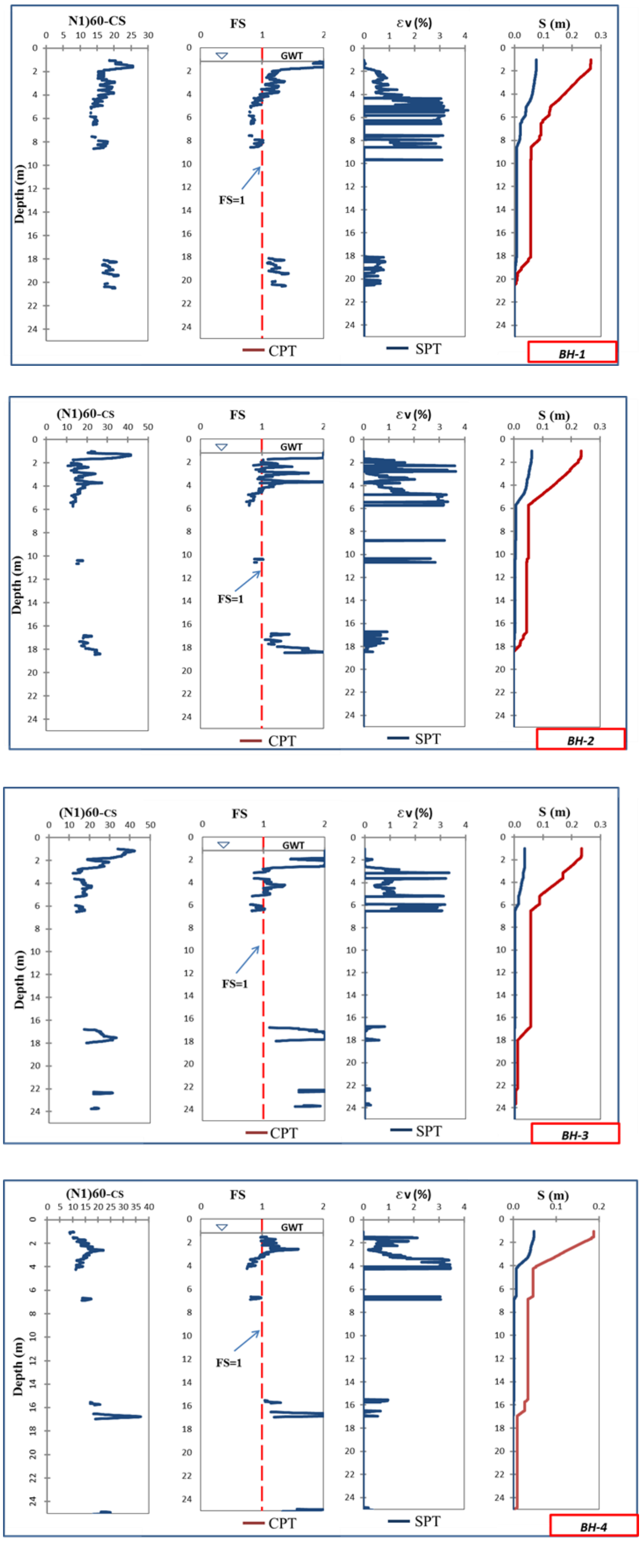

Figure 3 Evaluation of the post-liquefaction reconsolidationsettlement in SPT-1, SPT-2, SPT-3andSPT-4.

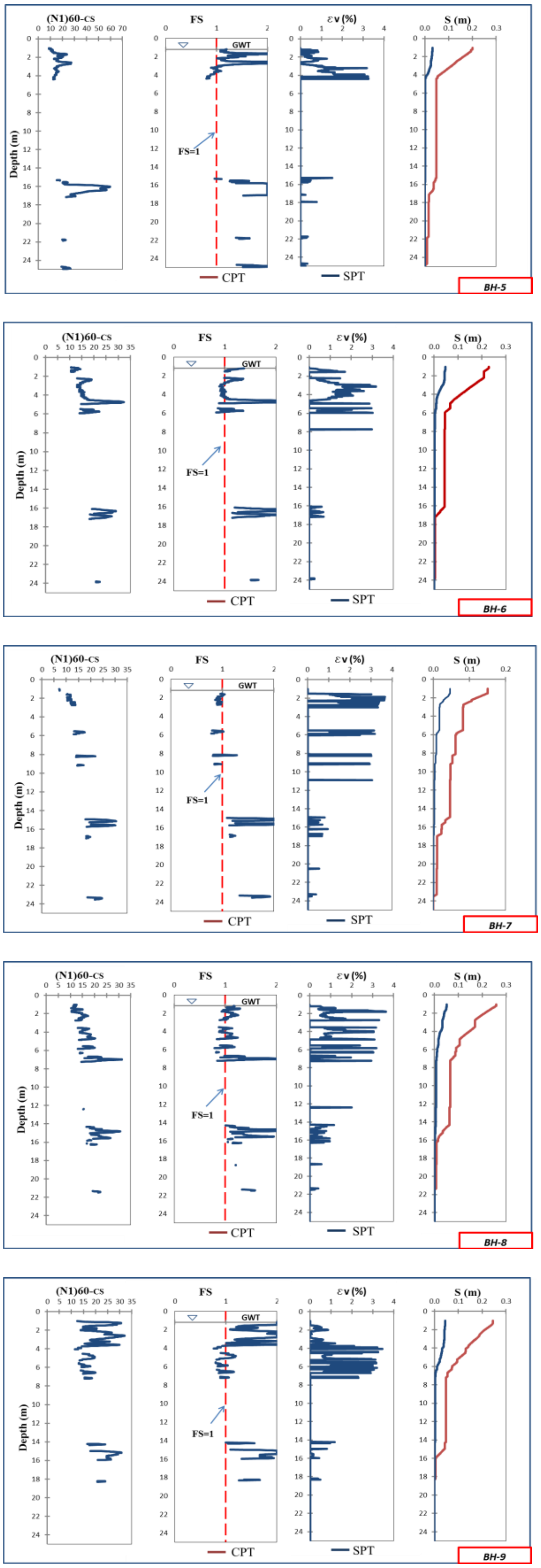

Figure 4 Evaluation of the post-liquefaction reconsolidationsettlement in, SPT-5, SPT-6, SPT-7 SPT-8 and SPT-9.

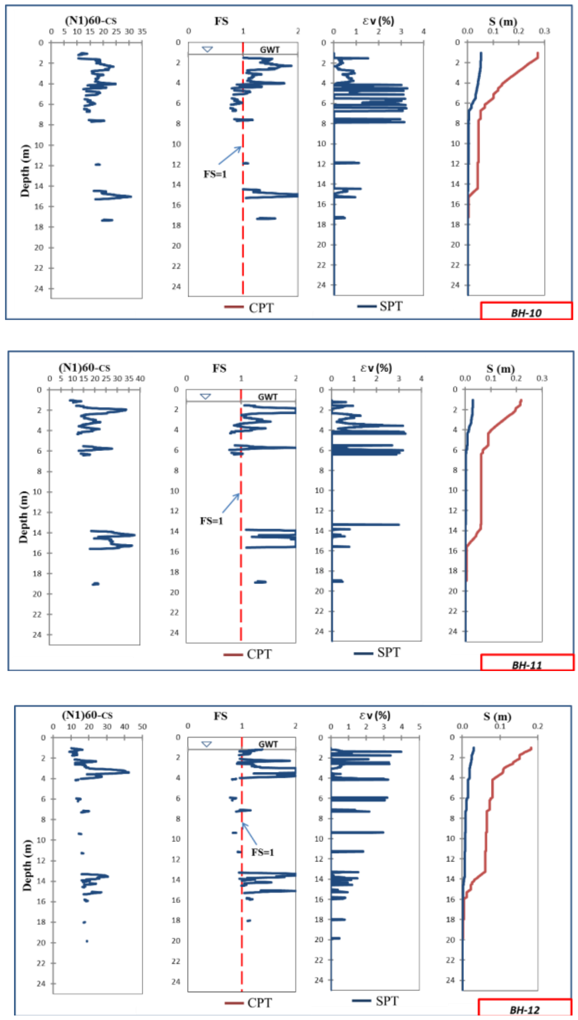

Figure5 Evaluation of the post-liquefaction reconsolidationsettlement in SPT-10, SPT-11 and SPT-12.

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