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Per Hafström
Royal Institute of Technology, Stockholm, Sweden

Jan Skogsberg
Royal Institute of Technology, Stockholm, Sweden

Anders Bodare
Royal Institute of Technology, Stockholm, Sweden

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Per Hafström & Jan Skogsberg
Graduate Students, Royal Institute of Technology, Stockholm, Sweden

Anders Bodare
Asst. Professor, Royal Institute of Technology, Stockholm, Sweden

SYNOPSIS Sites within the area affected by liquefaction due to the 22 April 1991 Limón–Telire earthquake, have been investigated in order to compare the results of some empirical methods with the incidents observed during the earthquake. The purpose of this comparison was to suggest a suitable method to be used when assessing the risk for liquefaction in Costa Rica in the future.

INTRODUCTION

During the last years Costa Rica has suffered a period of strong seismic activity. It was initiated by the M=6.9 Cobano earthquake of March 25, 1990. This event might have acted as the detonator that activated local faults near the town of Puriscal, producing a seismic swarm that lasted from May to July 1990. It was the rupture of a fault in the same area that caused the M=5.7 Alajuela earthquake on December 22, 1990. The activity culminated with the event of interest to this work: the Limón-Telire earthquake.

On April 22, 1991, 15:56 local time, the Caribbean region of Costa Rica and the western part of Panamá were struck by a major earthquake. The earthquake reached a magnitude of Ms=7.6 and had a depth of 17 km. Its epicentre was in Valle de Talamanca. It is believed to have been the largest intraplate event in this century to occur inland Costa Rica within the upper crust of the continental platform (Sauter,1991) and was caused by the rupture of a reverse fault.

Also a zonation based on the surface geology of the region and historic intensities was made (Hafström et al, 1994)

INCIDENTS AND DAMAGES

The area affected by the earthquake within Costa Rica is made up by lowlying coastal plains of soft alluvial soils. These soils are likely to amplify the seismic waves and are in addition prone to liquefaction.

Except for the cities of Limón and Moín, with a surrounding population of 100 000 people, the region of high intensity ground motions is sparsely populated. During the earthquake 48 people lost their lives, while nearly 400 people were injured. About 2 500 building structures were damaged beyond repair, mostly small wood-frame homes and comercial buildings. The most severe impact on engineered structures due to the earthquake was to lifelines. Most damage suffered by roads and railroads was due to soil failures and liquefaction, which caused lateral spreading and settlement of road embankments, deformation of railway lines and loss of support to bridges.

LIQUEFACTION DUE TO THE EARTHQUAKE

Soil liquefaction due to the earthquake occured over a large area in the low lying areas of eastern Costa Rica and the northeast part of Panamá. See fig 1.

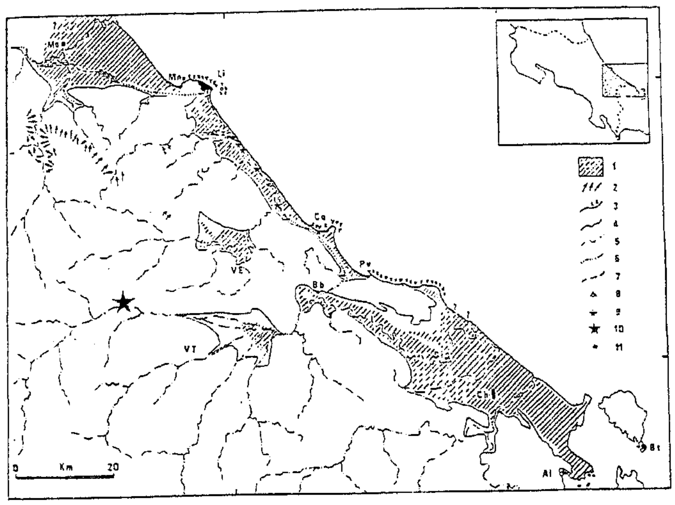

Figure 1. Areas where liquefaction occured during the Limón-Telire Earthquake, 1991. After Soulas, 1991.

The affected area in Costa Rica is congruent with the alluvial plain of the Caribbean watershed, and ranges from the Panamanian border in the south to somewhere north of the river Pacuare. Within a limited area, liquefaction was also seen to occur on the alluvial fan.

APPLICATION OF EVALUATION PROCEDURES

The two methods used are the ones proposed by Seed, H.B. (1966) and Iwasaki, T. (1981) respectively. These methods were chosen since they employ relatively simple procedures of calculation and estimate the soil strength from in-situ measurements, SPT-tests.

Some of the input data used was already existing from companies and institutions in Costa Rica. Additional geotechnical data was obtained by field investigations done by a so called DPL-equipment.

The DPL - equipment

Though smaller, the principles of this German equipment is the same as for the SPT-equipment. A steel rod is driven into the ground by the energy delivered from a 10.125 kg weight falling from a height of 50 centimeters. There is no engine to operate the weight. The number of blows needed to penetrate ten centimeters into the ground are continously noted and called the N10-value.

The formulas for transforming the DPL - values to SPT - values were taken from the German Standards (DIN 4094). For details on the transformation the reader is reffered to the work by Hafström - Skogsberg (1994).

The calculations have been made using two levels of acceleration; 0.25 g and 0.40 g. An attenuation formula for soft soils based on data from this earthquake was produced by eng Taylor (1993) at ICE.

THE SEED METHOD

The algorithm for the Seed method is presented as follows.

Input

for the hole profile: water table during penetration test, w_p (m) (when using DPL-values), water table during earthquake, w_e (m), horizontal acceleration at surface, a (m/s2), delivered rod energy from SPT-equipment, ER_m (%).

for each level: depth under surface, z (m), SPT or DPL blowcount value N_{30} or N_{10} resp, content of fines (D<0.074 mm), FC (%), soil density, ρ_s (ton/m3).

1 Calculations of N_{30}:

at each level: (if N_{10} is given as input)

If FC > 98 then $N_{30} = 0.6*N_{10}$

else if $z\text{-}w_e < 0$ and $z\text{-}w_p < 0$ then $N_{30} = 0.476*N_{10}$

else if $z\text{-}w_e \geq 0$ and $z\text{-}w_p < 0$ then $N_{30} = 0.433*N_{10} - 4.55$

else if $z\text{-}w_e \geq 0$ and $z\text{-}w_p \geq 0$ then $N_{30} = 0.865*N_{10} - 3.68$

else if $z\text{-}w_e < 0$ and $z\text{-}w_p \geq 0$ then $N_{30} = 0.952*N_{10} + 0.952$

else $N_{30} = 0$

If $N_{30} < 1$ then $N_{30} = 1$ (1)

2. σ_0, vertical overburden pressure:

$$\sigma_{0,n}=\sigma_{v,n-1} + \rho_s(z_n - z_{n-1})g \quad (2)$$

where n denotes the number of the calculated layer.

σ_0', effective overburden pressure:

$$\sigma_0'=\sigma_0 - \rho_w (z - w_e)g \quad (3)$$

where ρ_w is the density of water

3. r_d, reduction factor for soil stiffnes: $r_d = 1 - 0.015z$ (4)

4. It is now possible to calculate the cyclic load induced by the earthquake.

$$\frac{\tau_{av}}{\sigma_0'} = 0.65 \cdot \frac{a}{g} \cdot \frac{\sigma_0}{\sigma_0'} \cdot r_d \quad (5)$$

5. Continuing with the correction of the N_{30}-value.

c_n, correction due to the effective overburden pressure:

$$c_n = \frac{1}{\sqrt{\sigma_0'/100}} \quad (6)$$

6. $(N1)_{60}$, N_{30}-value corrected with cn, and the fraction of energy delivered to the drill rod in the SPT-test, ER_m:

$$(N1)_{60} = c_n \cdot \frac{ER_m}{60} \cdot N_{30} \quad (7)$$

When the $(N1)_{60}$-value was established the correction for fine contents was made:

7. If FC < 10% then $\Delta(N1)_{60} = 0$

else if FC < 25% then $\Delta(N1)_{60} = 1$

else if FC < 50% then $\Delta(N1)_{60} = 2$

else if FC < 75% then $\Delta(N1)_{60} = 4$

else $\Delta(N1)_{60} = 5$ (8)

$$(N1)_{60corr} = (N1)_{60} + \Delta(N1)_{60} \quad (9)$$

8. The soil resistance, CRS, is calculated and corrected for the influence of effective overburden pressure.

K_σ, correction for overburden pressure in resistance to cyclic loading:

$$K_\sigma=1.6-0.007637*\sigma_0'+0.000017687*\sigma_0'^2- 0.000000013*\sigma_0'^3 \quad (10)$$

CRS, in-situ resistance:

$$CRS=(0.028234*(N1)_{60corr}-0.001724(N1)_{60corr}^2+ 0.000042 *(N1)_{60corr}^3)*K_\sigma \quad (11)$$

Output

The factor of safety is finally calculated as the ratio of the soil strength to the cyclic load for each level:

F, factor of safety $F = \frac{CRS}{\tau_{av}/\sigma_0'}$ (12)

THE IWASAKI METHOD

The algotithm for the Iwasaki method is as follows.

Input

for the hole profile: water table during penetration test, w_p (m), water table during earthquake, w_e (m), horizontal acceleration at surface, a (m/s^2).

for each level: depth under surface, z (m), SPT or DPL blowcount value, N_{30} or N_{10} (blows), mean particle diameter, D_{50} (mm), contents of fines (D less then 0.074 mm), FC (%), saturated soil density, ρ_s (ton/m3).

1. Calculations of N_{30}:

at each level(if N_{10} is given)

If $D_{50} < 0.002$ then $N30 = 0.6*N_{10}$

else if $z\text{-}w_e < 0$ and $z\text{-}w_p < 0$ then $N_{30} = 0.476*N_{10}$

else if $z\text{-}w_e \geq 0$ and $z\text{-}w_p < 0$ then $N_{30} = 0.433*N_{10}-4.55$

else if $z\text{-}w_e \geq 0$ and $z\text{-}w_p \geq 0$ then $N_{30} = 0.865*N_{10} - 3.68$

else if $z\text{-}we < 0$ and $z\text{-}w_p \geq 0$ then $N_{30} = 0.952*N_{10} + 0.952$

else $N_{30} = 0$

If $N_{30} < 1$ then $N_{30} = 1$ (13)

2. σ_0, vertical overburden pressure:

$$\sigma_{0,n} = \sigma_{0,n-1} + \rho_s(z_n\text{-}z_{n-1})g \quad (14)$$

σ_0', effective overburden pressure:

$$\sigma_0' = \sigma_0 - \rho_w*(z - w_e)g \quad (15)$$

Now the soil strength, or resistance to liquefaction, can be calculated.

3. R_1, in-situ resistance:

$$R_1 = 0.0882 \cdot \sqrt{\frac{N_{30}}{\sigma_0' + 0.7}} \quad (16)$$

4. R_2, in-situ resistance:If $D_{50} \leq 0.05$ then $R_2 = 0.19$

else if $D_{50} \leq 0.6$ then $R_2 = 0.225\log(0.35/D_{50})$

else $R_2 = -0.05$ (17)

5. R_3, in-situ resistance: If $FC < 40$ then $R_3 = 0$

else $R_3 = 0.04*FC - 0.16$ (18)

6. R_t, total in-situ resistance:

$$R_t = R_1 + R_2 + R_3 \quad (19)$$

7. L, dynamic load: $L = \frac{a}{g} \cdot \frac{\sigma_0}{\sigma_0'} \cdot (1 - 0.015z)$ (20)

Output: F_l, factor of safety:

$$F_l = \frac{R_t}{L} \quad (21)$$

ANALYSIS OF THE RESULTS

Input parameters with the greatest impact on the results are the ground acceleration and the SPT value. It can also be seen that the Iwasaki method to a higher extent than the Seed method is dependent on the input parameters reflecting the soil texture. For soils with a grading curve within the boundaries for most liquefiable soils, the Iwasaki method generally gives lower safety factors than the Seed method. For soils with a larger fraction of fine material the safety factors calculated with the Iwasaki method tend to increase.

Each site investigated has been assigned a value of liquefaction severity based on the safety factors obtained with the two methods respectively. The severity values assigned to each site are 0, 1 and 2, corresponding to no liquefaction, moderate liquefaction and severe liquefaction respectively. Furthermore, the degree of liquefaction observed at each site during the earthquake has been classified using the same scale (0, 1, 2). A comparison between the calculated and the observed severity values was made.

Presented to the left in Table 1 are the severity values based on the safety factors obtained using accelerations according to the attenuation formula. In the center of the table is the degree of liquefaction as observed in the field, and to the right the severity values obtained from the safety factors that correspond to the two levels of acceleration: a = 2.5 m/s2 and 4.0 m/s2.

If the severity values that correspond to the accelerations obtained with the attenuation relation are studied, it is seen that among the sites where SPT-investigations have been used in the calculations, the Iwasaki method gives severity values that are in accordance with the observations or severity values that are conservative. The Seed method gives non-conservative severity values for sites 4 and 5.

Among the calculations based on DPL-investigations, the Iwasaki method is non-conservative at sites 14, 17 and 18, while the Seed method shows non-conservative severity values at sites 17, 18 and 19.

At site 12 both methods give a severity value of 2 while no liquefaction has been observed in the field. The deviations may depend on an incorrect too high ground acceleration, or on the fact that the soil material on this site may have been coarse enough to prevent the occurrance of liquefaction.

The transformation of blow count values obtained with the DPL-equipment into the corresponding SPT-values is an additional source of error. It would be desirable to confirm the transformation formulas for soil deposits in Cost Rica.

CONCLUSIONS

In this work a zonation has been made for the Caribbean watershed of Costa Rica. The zonation has been based on the surface geology of the region complemented with information about historic ground motion intensities.

In this work it is shown how the two empirical methods proposed by H.B. Seed and T. Iwasaki respectively can be used to evaluate the liquefaction potential for a site. The application of the two methods at a number of sites within the area affected by liquefaction during the earthquake of April 22, 1991 shows that while the Seed method primarily is dependent on the blow count values, the Iwasaki method also shows a great dependency on the rest of the geotechnical input parameters. It is seen that for soils with a texture considered most susceptible to liquefaction, the safety factors obtained with the Iwasaki method are generally lower than those calculated with the Seed method. When finer soils are at hand the situation often is the opposite.

Based on the comparison between the calculated safety factors and the observations made in the field during the earthquake, and because of its simple procedure of calculation, the Iwasaki method is suggested as an appropiate and practical way of evaluating site specific liquefaction potentials in Costa Rica. Some caution is recommended though, when applying the method on soils with low D50-values and/or high contents of fines, since the method in these cases may be slightly non-conservative.

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Table 1. Observed degree of liquefaction and severity values estimated from calculated safety factors.

SPT testing equipment

Assuming Attenuation formula					Assuming acceleration of 0.25 g		Assuming acceleration of 0.40 g	
Site	a m/s^2	Iwasaki	Seed	Obs	Iwasaki	Seed	Iwasaki	Seed
1	4.7	2	2	2	1	0	2	2
2	4.8	2	2	2	2	1	2	2
3	5.0	2	2	2	2	1	2	2
4	5.4	2	0	2	0	0	1	0
5	3.9	2	1	2	2	0	2	1
6	4.1	2	2	2	1	1	2	2
7	4.1	1	1	0	0	0	1	1
8	3.7	2	1	1	0	0	2	1
9	3.8	1	2	1	0	0	1	2

DPL - testing equipment

10	2.5	2	2	2	2	2	2	2
11	3.0	2	2	2	1	1	2	2
12	3.0	2	2	0	2	2	2	2
13	2.9	1	1	1	0	0	2	2
14	3.4	0	1	1	0	0	0	2
15	2.9	2	2	2	2	2	2	2
16	3.7	1	2	1	0	1	1	2
17	3.2	0	0	1	0	0	1	2
18	2.9	0	0	1	0	0	1	2
19	3.5	1	0	1	1	0	2	2
20	2.2	0	0	0	0	0	0	1