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Soil Liquefaction and Ground Settlement in Chi-Chi Taiwan, Earthquake

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SOIL LIQUEFACTION AND GROUND SETTLEMENT IN CHI-CHI, TAIWAN, EARTHQUAKE

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ABSTRACT

This paper presents an investigation of soil liquefaction and ground settlement in the 1999 Chi-Chi, Taiwan, earthquake. The quake killed more than 2400 people and caused a great destruction to buildings, bridges, dams, highways and railways. One of the causes for heavy damages to the structures is soil liquefaction and ground settlement during the earthquake. Six sites that were observed to experience liquefaction are investigated through cone penetration testing (CPT), and the liquefaction potential of each site and the settlement of the liquefied soil strata are analyzed.

INTRODUCTION

In the early morning (01:47 Taiwan time) of September 21, 1999, a great earthquake (M_w=7.7, M_L=7.3) struck central Taiwan near the small town of Chi-Chi, Nantou County. The epicenter was located by the Central Weather Bureau (CWB) to be at 23.87°N, 120.75°E, which is near Chi-Chi. The earthquake killed more than 2400 people and caused a great destruction to buildings, bridges, dams, highways, and railways. Evidences of soil liquefaction such as sand boils were widely observed in this earthquake. Figure 1 shows a scene at the Changhwa Coastal Industrial Park where sand boils were observed. Figure 2 shows tilting of a residential dwelling at Wufeng caused by soil liquefaction. Damages caused by liquefaction include failure of riverbanks, damages to roadways, pipelines, and port facilities, and building settlement and tilting. More than 900 dwellings were severely damaged by the effect of soil liquefaction.

The writers have carried out an investigation of soil liquefaction in this earthquake. Six sites with strong evidence of liquefaction were examined in this paper. The results of field investigation and the subsequent analysis are reported.

FIELD INVESTIGATION AND ANALYSIS

Cone penetration test (CPT) is a relatively rapid and reliable in situ test that can provide a continuous soil profile (Lunne et al., 1997). In a typical sounding, the profiles of cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (P_w) are recorded. A useful parameter, termed friction ratio (R_f), defined as $R_f = f_s/q_c$, is derived. The profiles of R_f and P_w may be used to identify the changes in soil layers. In fact, soil behavior types can be quite reliably classified using an empirical chart developed by Robertson (1990). In addition, shear wave velocity (V_s) of soils may be measured by using a seismic cone (SCPT).

Use of CPT to investigate liquefaction susceptibility and potential has been proposed by many researchers (Olsen, 1997; Robertson and Campanella, 1985; Robertson and Wride, 1998; Seed et al., 1985; Suzuki et al., 1995; Stark and Olsen, 1995; Juang et al., 2000a). In the present study, a spreadsheet module is created for the Juang et al. (2000a) method that has been shown to be able to predict accurately liquefaction resistance of soils using CPT. The spreadsheet module is used to analyze liquefaction potential of six sites (Figure 3) where evidences of liquefaction in the 1999 Chi-Chi earthquake were observed. The developed spreadsheet module, CPT.xls, is available from the third author upon request.

Use of shear wave velocity (V_s) for evaluating liquefaction susceptibility and potential is getting greater attention in recent years. The use of V_s as an index of liquefaction resistance is solidly based since both V_s and liquefaction resistance are influenced by many of the same factors such as void ratio, state of stress, stress history, and geologic age (Andrus et al., 1999). During the past decade, several methods for predicting liquefaction resistance based on V_s have been developed using field performance data (Robertson et al., 1992; Kayen et al., 1992; Lodge, 1994; Rollins et al., 1998; Andrus et al., 1999; Juang et al., 2000b). In the present study, a spreadsheet module, VS.xls, is created based on Juang et al.

Figure 1. Sand boils at Changhwa Coastal Industrial Park

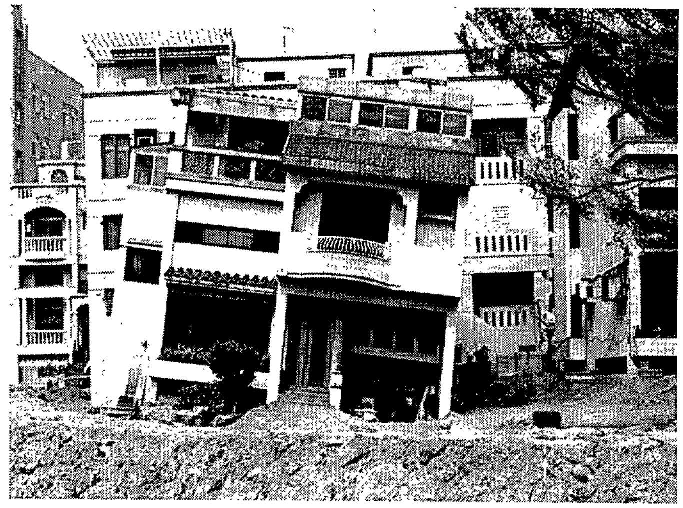

Figure 2. Tilting of a residential dwelling at Wufeng (tilted at about 15°)

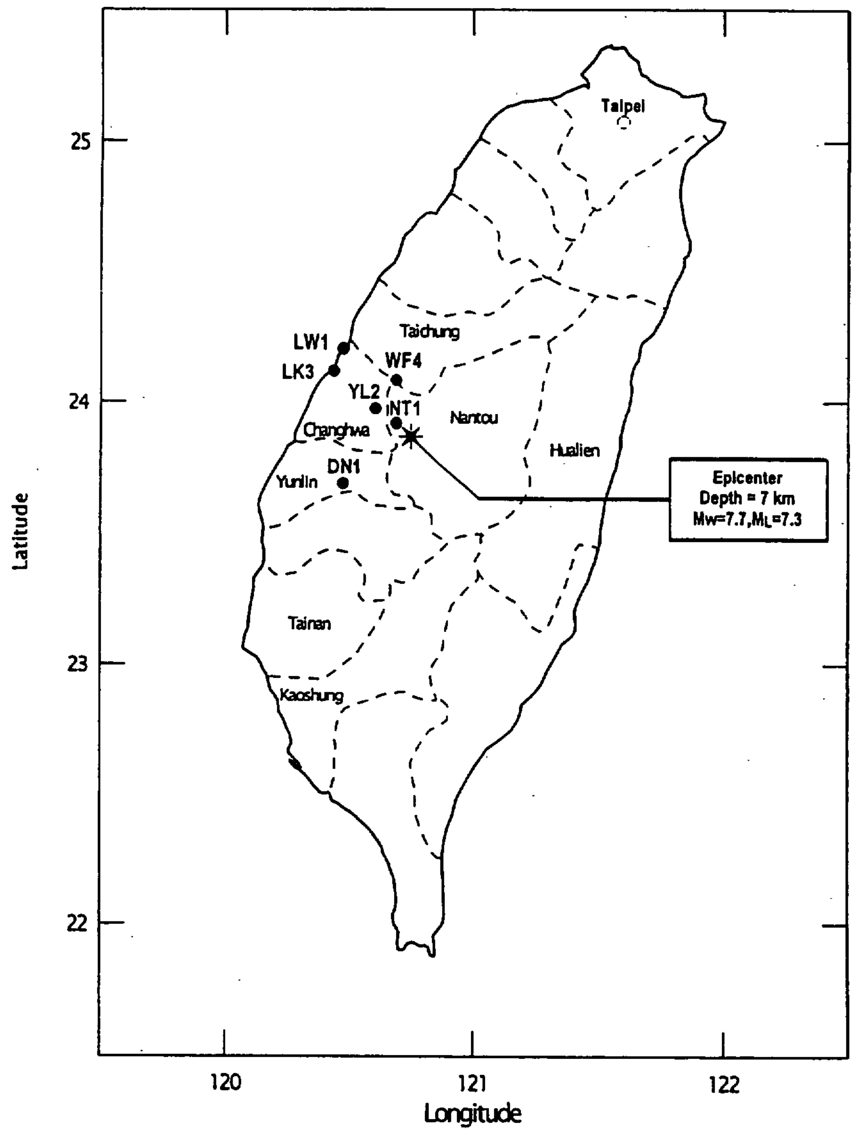

Figure 3. Liquefaction Sites Investigated

(2000b) method that has been shown to be able to predict accurately liquefaction resistance of soils using V_s data. The spreadsheet module is then used to analyze liquefaction potential of the six sites investigated. The spreadsheet module, VS.xls, is available from the third author upon request.

In the present study, the seismic load, in terms of cyclic stress ratio (CSR), is calculated according to Youd and Idriss (1997), which is an updated version of the Seed et al. (1985) method. The liquefaction resistance of soils, in terms of cyclic resistance ratio (CRR), is calculated according to Juang et al. (2000a) when using CPT, and according to Juang et al. (2000b) when using V_s data. The probability of liquefaction (P_L) is calculated using the calculated CSR and CRR through Bayesian mapping functions (Juang et al., 2000a & 2000b). These procedures are implemented in the spreadsheet modules, CPT.xls and VS.xls. The calculated P_L may be used as an index of the liquefaction potential (see Table 1).

Table 1. Liquefaction likelihood classification

Class	Probability of liquefaction (P_L)	Description of likelihood
5	$P_L \geq 0.85$	Almost certain that it will liquefy
4	$0.65 \leq P_L < 0.85$	Very likely to liquefy
3	$0.35 \leq P_L < 0.65$	Liquefaction and no liquefaction are equally likely
2	$0.15 \leq P_L < 0.35$	Unlikely to liquefy
1	$P_L < 0.15$	Almost certain that it will not liquefy

Wufeng Site (WF4)

The site mainly consists of weathered silty sandstone. The soil deposit from the ground surface to the depth of about 5.7 m is sand to silty sand. From the ground surface to the depth of 3.5 m, the average q_c is about 1.6 MPa, and below 4.3 m, the average q_c is about 9.7 MPa. The penetration was not continued below 5.7 m. Figure 4 shows the CPT sounding profile. Note that friction ratio (R_f), instead of sleeve friction (f_s), is plotted in this figure. The maximum horizontal ground acceleration, a_{max}, is 0.41 g at this site in the 1999 Chi-Chi earthquake.

The CSR, CRR, and P_L for soils at this site are calculated using CPT data (q_c and f_s) and the spreadsheet module CPT.xls. The calculated CSR, CRR, and P_L are shown in Figure 4. In the layer between 0.5 m and 4.3 m, the calculated probabilities of liquefaction are very high, i.e., P_L > 85%. According to Table 1, liquefaction is "almost certain to occur" at this site subjected to the 1999 Chi-Chi seismic event. This "post-event" prediction agrees very well with field observations of sand boils at this site.

The shear wave velocity (V_s) is also measured by SCPT at this site. Between the depth of 1 m and 4 m, the measured V_s values range from 84 m/s to 127 m/s. The calculated P_L values using the spreadsheet module VS.xls range from 94% to 100%. Thus, the soils at the depths between 1 m and 4 m at this site is "almost certain" to liquefy in a seismic event such as the 1999 Chi-Chi earthquake. This agrees very well with the conclusion derived based on the CPT data, and both predictions agree well with field observations.

Nantou Site (NT1)

This site is located near the Li-Mei Bridge in the City of Nantou. The maximum horizontal ground acceleration, a_{max}, is 0.54 g in the 1999 Chi-Chi earthquake. From the ground surface to the depth of 4 m is a loose sand layer with a few clay seams. Between 4 m and 10 m is a loose to medium dense sand, and between 10 m and 11.3 m is a medium dense to dense gravelly sand. Below the depth of 11.3 m, it is a gravelly cobble deposit where penetration could not be continued.

The CSR, CRR, and P_L are calculated using CPT.xls. Except in a few clayey seams, the calculated P_L values for the soils at the depth between 1 m and 7 m at this site reach almost 100%. Thus, these soils are "almost certain" to liquefy when subjected to the 1999 Chi-Chi seismic event. This "post-event" prediction for the soils at this site agrees very well with field observations of sand boils.

The measured V_s values at the depth between 2 m and 8 m range from 107 m/s to 160 m/s, and the calculated P_L values using Vs.xls range from 90% to 100%. The results indicate that the soils at the depth between 2 m and 8 m are "almost certain" to liquefy when subjected to the 1999 Chi-Chi seismic event. This agrees very well with the prediction using CPT data, and both predictions agree with field observation of sand boils at this site.

Dornan Site (DN1)

The site is located at Jung-Jeng chicken farm in the Town of Dornan. The maximum horizontal ground acceleration, a_{max}, is 0.23 g at this site in the 1999 Chi-Chi earthquake. From the ground surface to a depth of 5 m is a layer of very loose sand with several clayey seams. Between 5 m and 12 m is a medium-dense sand. The next layer between 12 m and 15 m is a cohesive soil with thin layers of sand. Between 15 m and 18 m is a medium dense to dense sand, which is underlain by a cohesive soil between 18 m and 20 m.

Based on the calculated probability of liquefaction (P_L), the soils in the layers between 0.7 m and 3.3 m, 8.6 m to 10 m, and 11 m to 12 m, are "almost certain" to liquefy, as P_L > 90% in these layers. The sand boils observed at the site are most likely the result of liquefaction of soils at the depth between 0.7 m to 3.3m. The measured V_s values at the depth between 2 m and 10 m range from 109 m/s to 160 m/s. The P_L values in these depths calculated from the spreadsheet module VS.xls

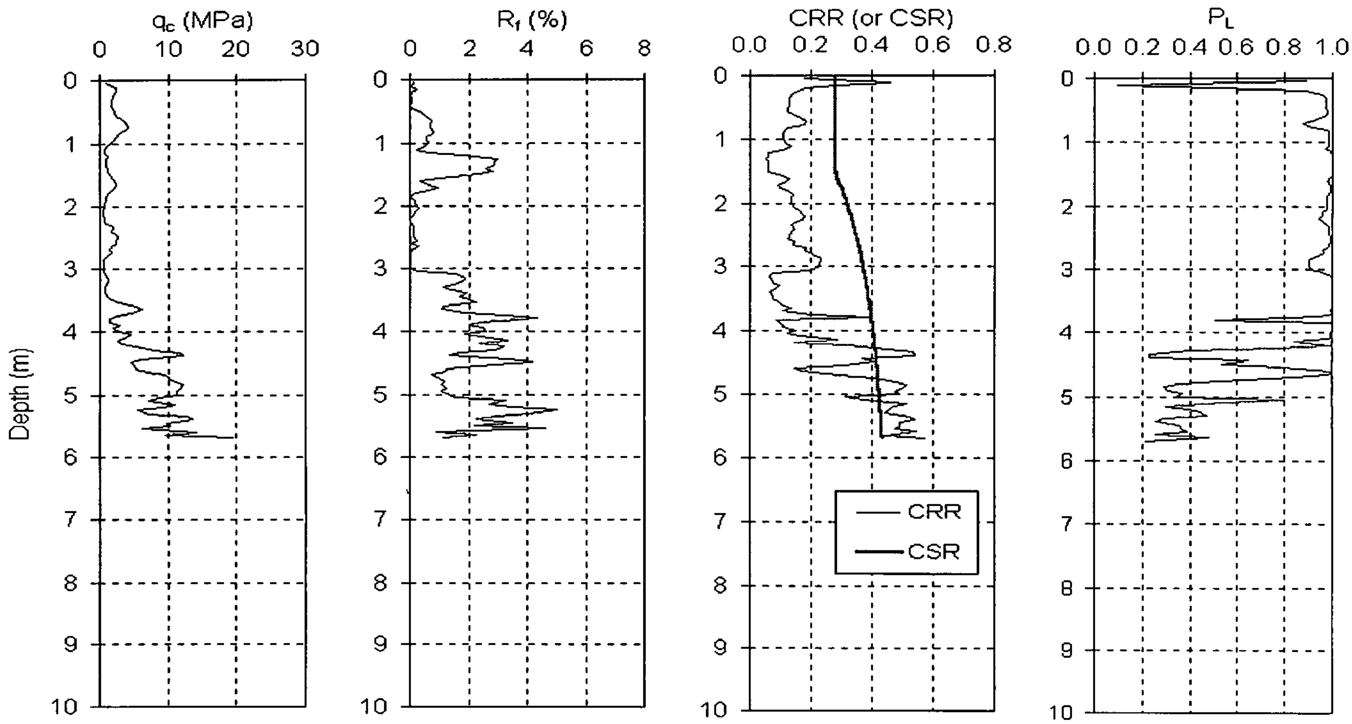

Figure 4. Profiles of q_c, R_f, CSR, CRR, and P_L at Site WF4

range from 74% to 98%. In general, this result agrees with the assessed high liquefaction potential based on CPT analysis.

Yuanlin Site (YL2)

The site is located at Yuansway Road, Yuanlin. The maximum horizontal ground acceleration, a_{max}, is 0.26 g in the 1999 Chi-Chi earthquake, according to the Central Weather Bureau (CWB). The top soil layer at the site is a 0.7 m-thick backfill. Underlain the backfill and to the depth of 8 m from the ground surface is a loose sand. From 8 m to 10.3 m is a cohesive soil, and from 10.3 m to 17.6 m is a medium dense to dense sand. Below 17.6 m is a cohesive soil.

Based on the calculated P_L values using CPT.xls, the soils in the layer of 1 m to 8 m, except a couple of possible "clayey seams" that were identified using the chart proposed by Robertson (1990), are "almost certain" to liquefy when subjected to the 1999 Chi-Chi seismic event, as P_L > 85% in this layer.

The measured V_s values in soils between the depths of 2.7 m and 8.7 m range from 132 m/s to 144 m/s. The calculated P_L values using the measured V_s data in these depths range from 71% to 96%, indicating high liquefaction potential. This result agrees very well with the prediction using CPT data at this site, and both predictions agree well with field observation of the occurrence of sand boils.

Lukang Site (LK3)

This site is located in the Lukang district of the Changhwa Coastal Industrial Park. At this site, the maximum horizontal ground acceleration, a_{max}, is 0.10 g in the 1999 Chi-Chi earthquake.

At this site, the top layer (from ground surface to a depth of 8.4 m) is a loose sand. From 8.4 m to 12 m is a weak cohesive soil with sand seams. Below that is a loose to medium dense sand (from 12 m to 15.1 m). At the depth between 15.1 m and 16.7 m is a cohesive soil, and between 16.7 m and 19.5 m is a loose to medium dense sand.

Based on the P_L values calculated using CPT.xls, the soils at the depth between 1 m and 6 m are "likely" to liquefy when subjected to the 1999 Chi-Chi seismic event. The soils at the depth between 6 m and 8 m, except in a possible clayey seam, are "almost certain" to liquefy. The measured V_s values at the depth between 2 m to 8 m range from 103 m/s to 137 m/s, and the P_L values calculated using VS.xls for the corresponding depths range from 61% to 97%. The results of the analyses agree well with field observations.

Lunwei Site (LW1)

This site is located in the Lunwei district of the Changhwa Coastal Industrial Park. The maximum horizontal ground acceleration, a_{max}, at this site is 0.17 g in the 1999 Chi-Chi earthquake. The top layer (from ground surface to a depth of 3 m) is a loose to medium dense sand. Between the depths of 3 m and 7.6 m is a very loose sand, and from 7.6 m to 8.3 m is a very weak cohesive soil. Below 8.3 m is a loose to medium dense sand.

The CSR, CRR, and P_L are calculated using CPT.xls. Based on the calculated P_L values, the soils are "likely to very likely" to liquefy at the depths between 2 m and 6 m, and "almost certain" to liquefy at the depths between 6 m and 7.6 m, when subjected to the 1999 Chi-Chi seismic event. The measured V_s values at the depth of 2 m to 8 m range from 110 m/s to 188 m/s. The P_L values calculated using VS.xl range from 36% to 72% for the soils at the depth between 2 m to 4 m, and 84% to 98% for the soils at the depth between 4 m and 8 m. These results agree well with field observations.

Summary of the Investigation

The average fines content of the boiled up samples at Sites WF4, NT1, DN1, YL2, LK3, and LW1 are 34%, 29%, 42%, 14%, 11%, and 6%, respectively. These fine materials are all low plasticity silt. The boiled up samples mainly consist of silty sands (SM). Table 2 shows a summary of site characteristics, soil data, and liquefaction observations at the six sites investigated. The analyses of these sites using existing methods such as Juang et al. (2000a & 2000b) yielded liquefaction predictions that agreed well with field observations.

Table 2. Summary of the results of the sites investigated (M_w = 7.7)

Site	Liq. ? 1= Y 0= N	a_{max} (g)	Water table (m)	Characteristics of critical layer			Average-of-the-layer data					
				Layer (m)	Soil description	Average fines content (%)	σ_v (kPa)	σ'_v (kPa)	q_c (MPa)	R_f (%)	V_s (m/s)	CSR
WF4	1	0.41	1.55	0.5-4.3	Silty sand	34	45.2	36.4	1.96	1.1	95	0.37
NT1	1	0.54	2.05	1-7	Silty sand	29	75.6	56.0	3.45	1.6	140	0.49
DN1	1	0.23	1.50	0.7-3.3	Silty sand	42	36.7	31.8	1.52	2.5	111	0.18
YL2	1	0.26	1.55	1-8	Silty sand	14	84.9	55.5	2.63	0.68	138	0.27
LK3	1	0.17	1.55	1-8	Silty sand	11	85.0	55.6	2.11	0.39	123	0.17
LW1	1	0.17	2.05	2-7.6	Silty sand	6	92.5	65.0	2.33	0.40	128	0.16

LIQUEFACTION-INDUCED GROUND SETTLEMENT

Ground settlement due to earthquake-induced liquefaction occurs rapidly. Thus, this liquefaction-induced ground settlement often posts greater damage to structures. In the present study, ground settlement at Sites LK3 and LW1 is analyzed using the method proposed by Ishihara and Yashimine (1992). The "post-event" predictions are compared with field observations.

At Site LK3, which is in a newly reclaimed land (by fills) in the Changhwa Coastal Industrial Park, the design elevation was at 4.3 m. Sand boils were observed at this site in the 1999 Chi-Chi earthquake.

Before the quake, the average elevation was 4.22 m (measured on May 7, 1998), and its elevation after the quake was 3.8 m (measured on September 27, 1999). The difference is 42 cm. At a distance of 660 m away from the LK3 site, where no liquefaction evidence was observed, the elevation after the quake was 4.19 m (measured on September 27, 1999). Assuming the elevation at this reference location was affected only by the consolidation due to the self-weight, then the difference between this elevation (4.19 m) and the elevation at LK3 before the quake, which equals to 3 cm, represents the consolidation settlement of the fills in this reclaimed land. Thus, the ground settlement due to liquefaction at Site LK3 is about 39 cm.

The method proposed by Ishihara and Yashimine (1992) is a chart-based solution. The post-liquefaction volumetric strain can be estimated based on the soil strength (in terms of relative density of sand, or N_{60}, or q_c) and the calculated factor of safety against liquefaction. Using the CPT data at Site LK3, the Ishihara and Yashimine (1992) method yields a settlement of 36 cm, which agrees very well with the observed liquefaction-induced settlement of 39 cm.

At Site LW1, which is also in a newly reclaimed land by fills, the elevation before the 1999 Chi-Chi earthquake was at 4.38 m (measured on August 10, 1998). Sand boils were observed at this site in the 1999 Chi-Chi earthquake. After the quake, the elevation was 3.79 m (measured on November 12, 1999). The change in elevation at LW1 before and after the quake is 59 cm. At a reference location (LW2), 380 m away from LW1 and in the same reclaimed land by fills, where no liquefaction occurred during the 1999 Chi-Chi earthquake, the elevation before and after the quake are 4.42 m and 4.05 m, respectively. The change in elevation at LW2 is 37 cm. The difference between the changes in elevation at LW1 and LW2 may be considered as the effect of liquefaction. Thus, the ground settlement due to liquefaction at Site LW1 is inferred to be 22 cm.

Using the CPT data at Site LW1, the Ishihara and Yashimine (1992) method yields a liquefaction-induced settlement of 21 cm, which agrees very well with the observed liquefaction-induced settlement of 22 cm. Again, the Ishihara and Yashimine (1992) method is shown to be accurate in predicting liquefaction-induced settlement.

CONCLUSIONS

At the six sites where sand boils were observed, the occurrence of liquefaction is found to be "almost certain" based on a series of "post-event" analyses using CPT and shear wave velocity measurements. Good agreement between the predictions and the field observations indicates that the existing methods such as Juang et al. (2000a &b) are capable of predicting liquefaction potential of sandy soils. These methods may be used in routine practice to evaluate liquefaction potential at a given site.

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