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# Deterministic Analysis and Numerical Modeling of Liquefaction in Bridge Pile Abutment

Shailendra Poudyal [a], Santosh Kumar Yadav [b]

[a, b] *Department of Civil Engineering, Pulchowk Campus, IOE, Tribhuvan University, Nepal*
[a] 076msgte014.shailendra@pcampus.edu.np, [b] santosh.yadav@pcampus.edu.np

**Abstract**
This paper studies the analysis of liquefaction in bridge pile abutment. In this study deterministic liquefaction analysis is done and also this result is verified by Slope/W 2018. NCEER-1996/1998 method is used for the deterministic analysis of liquefaction and in Slope/W 2018 model is prepared and pseudo static analysis is done. In deterministic analysis factor of safety is calculated and in Slope/W 2018 general limit equilibrium method is used to calculate the pseudo static factor of safety and compare these two results. From the deterministic approach the severity of one borehole of bridge site is high and three borehole of the same bridge site is very high. The pseudo static factor of safety at the 0.3 PGA of all four boreholes of the same bridges site is less than one which indicates that all four bridge abutments are susceptible to liquefaction.

**Keywords**
Liquefaction, Abutment, Slope/W, Factor of Safety (FOS)

## 1. Introduction

Soil liquefaction is the phenomenon in which fully saturated or partially saturated soils loses its strength and stiffness in response of suddenly applied stress such as shaking during earthquake or other suddenly applied stress, in which soil solid material behaves likes liquid. This liquefaction is normally observed in saturated and loose fine and medium sand because the sand has capacity to compress when sheared. The term soil liquefaction is one of the very complex, controversial, interesting and important topic in the Geotechnical Earthquake Engineering [1].

Nepal is one of the well identified seismically active nations located in the Himalayan seismic belt [2]. Nepal is tectonically active region in between Indian and Eurasian plate so that there are so many past earthquakes history in Nepal. The recent past major earthquake was Gorkha Earthquake of moment magnitude Mw 7.8 happened at 06:11 UTC on April 25, 2015 with epicenter of about 77 km northwest from capital Kathmandu with the focal depth of approximately 15km [3]. In Gorkha earthquake there are many liquefaction cases in the Kathmandu valley and other parts of the Nepal. In Kathmandu valley Manamiju, Khokana, Kaushaltar, etc are some liquefaction due to Gorkha earthquake.

The main cause of the liquefaction is the loss of shear strength of soil due to the increase in pore water pressure and decrease in the effective stress where the sand display the fluid like characteristics [4]. There are mainly two types of liquefaction namely flow liquefaction and cyclic mobility [1]. Flow liquefaction can occurs when soil shear stress of soil mass at static equilibrium is greater than the shear strength of soil at liquefied state and flow mobility happen when the static shear strength is less than shear strength of liquefied soils [1].

The liquefaction induced by static or dynamic loading in saturated sandy soils can damage the buildings, existing structure, economic loss and even loss human life [4]. The soil deformation and lateral spreading due to liquefaction can damage the bridge abutment, pier and other bridge structural elements [5]. In the context of Nepal, there are so many bridges are built every year and lacks of proper geotechnical investigation many bridge are lateral spreading and settlement hazards. Nowadays, the government agency Department of Roads (DOR) conduct the liquefaction analysis of bridge foundation, however there is lack of liquefaction induced lateral spreading of bridge abutment.

In this paper the authors done the liquefaction analysis by deterministic approach by NCEER 1996/1998 and this liquefaction is verified from the model preparation. The liquefaction analysis of two bridge site of four abutments with two abutment of each bridge namely kajala River bridge at sijuwa road and Katnu Khola River Bridge Milan chowk South at Ratuwamai Municipality, Morang District is done and also this liquefaction is verified by model preparation in Geostudio (Slope/W) 2018.

## 2. Literature Review

### 2.1 Major Factors Goeverning Liquefaction

#### 2.1.1 Earthquake magnitude and duration

The main factors for liquefaction are ground acceleration and shaking of the ground. The highest magnitude of earthquake produces largest ground acceleration and largest time for ground shaking.

#### 2.1.2 Ground water table

Generally the liquefaction is highly susceptible near the ground water surface. Fluctuation of ground water table, the liquefaction is also fluctuated. Past or historic highest ground water table is used to analysis the liquefaction potential.

#### 2.1.3 Degree of saturation

The degree of saturation has great importance to liquefaction and we know that in dry soil there is no liquefaction. The soils having low degree of saturation has a little chance to liquefaction and the soil resistance to liquefaction increases to decrease the degree of saturation.

#### 2.1.4 Particle size distribution

Well graded soils are less susceptible to liquefaction than uniformity graded non-plastic soils. Uniformly graded non plastic soils tend to form more unstable particle arrangement.

#### 2.1.5 Drainage condition

Exit of pore water pressure is important issue for the liquefaction. If excess pore pressure is quickly dissipated from site, there is less chance to liquefaction. The permeable soil layers like gravel is less susceptible to the liquefaction.

### 2.2 SPT Based Liquefaction Analysis

In SPT based liquefaction analysis Factor of Safety (FS) against liquefaction is calculated in terms of CSR (Cyclic Stress Ratio) and CRR (Cyclic Resistance Ratio) as Equation 1 [6].

$$FS = \frac{CRR_{7.5}}{CSR} * MSF \tag{1}$$

Where, $CRR_{7.5}$ = Cyclic Resistance ratio for magnitude 7.5 earthquakes.
CSR = Calculated Cyclic Stress Ratio generated by the earthquake shaking
If, FS<1, the soil at the given depth is liquefied, FS=1, the soil is in critical state and FS>1, the soil is not liquefied.
MSF is the magnitude scaling factor which is calculated as Equation 2 [6].

$$MSF = \frac{10^{2.24}}{M^{2.56}} \tag{2}$$

Where, M = Moment magnitude of earthquake which is take the Gorkha Earthquake moment magnitude 7.8 According to Seed and Idriss (1971) the cyclic stress ratio (CSR) is calculated as Equation 3 [7].

$$CSR = 0.65 * \left(\frac{a_{max}}{g}\right) * \left(\frac{\sigma_{\sigma 0}}{\sigma'_{vo}}\right) * r_d * \frac{1}{K_\sigma} \tag{3}$$

Where, $a_{max}$ is the maximum ground acceleration due to earthquake, g is the acceleration due to gravity, $\sigma_{\sigma o}$ is the total vertical overburden stress, $\sigma'_{\sigma o}$ is the effective vertical overburden stress and $r_d$ is the stress reduction coefficient and $K_\sigma$ is the overburden correction factor for cyclic stress ratio. According to T.F. Blake (1996) the stress reduction coefficient is estimated as Equation 4 [6].

$$r_d = \left(\frac{1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2}\right) \tag{4}$$

Where z = depth beneath ground surface in meters. The cyclic resistance ratio (CRR) is calculated as Equation 5 [6].

$$CSR_{7.5} = \frac{1}{34 - (N_1)60} + \frac{(N_1)60}{135} + \frac{50}{(10 * (N_1)60 + 45)^2} - \frac{1}{200} \tag{5}$$

The Equation 5 is valid for $(N_1)_{60} < 30$. For $(N_1)_{60} >= 30$, the value $(N_1)_{60}$ is converted to $(N_1)_{60cs}$ and calculated as Equation 6 [6].

$$(N_1)60cs = \alpha + \beta * (N_1)60 \tag{6}$$

The $\alpha$ and $\beta$ are the coefficient, which are influenced by fineness content (FC) and calculated as Equations 8, 9 and 10.

$$\alpha = 0, \beta = 1 \qquad FC \leq 5\% \tag{7}$$

$$\alpha = exp\left(1.76 - \left(\frac{190}{FC^2}\right)\right) \qquad 5\% < FC < 35\% \tag{8}$$

$$\beta = \left(0.99 - \left(\frac{FC^{1.5}}{1000}\right)\right) \qquad 5\% < FC < 35\% \tag{9}$$

$$\alpha = 5, \beta = 1.2 \qquad FC \geq 35\% \tag{10}$$

The value $(N_1)_{60}$ is calculated as Equation 11 [8].

$$(N_1)60 = N_m C_N C_E C_B C_R C_S \tag{11}$$

Where, $N_m$ is SPT value obtained from field test, $C_N$ is Normalization factor of $N_m$ against overburden stress, $C_E$ is correction factor ratio for energy hammer, $C_B$ is the correction factor for drill hole diameter, $C_R$ is the correction factor for rod length and $C_S$ is the correction factor for the samplers.

The hammer correction is applied for the SPT value correction as shown in 1.

| Factor | Equipment variable | Term | Correction |
|---|---|---|---|
| Overburden pressure | — | $C_N$ | $(P_a/\sigma'_{vo})^{0.5}$ |
| Overburden pressure | — | $C_N$ | $C_N \leq 1.7$ |
| Energy ratio | Donut hammer | $C_E$ | 0.5–1.0 |
| Energy ratio | Safety hammer | $C_E$ | 0.7–1.2 |
| Energy ratio | Automatic-trip Donut-type hammer | $C_E$ | 0.8–1.3 |
| Borehole diameter | 65–115 mm | $C_B$ | 1.0 |
| Borehole diameter | 150 mm | $C_B$ | 1.05 |
| Borehole diameter | 200 mm | $C_B$ | 1.15 |
| Rod length | <3 m | $C_R$ | 0.75 |
| Rod length | 3–4 m | $C_R$ | 0.8 |
| Rod length | 4–6 m | $C_R$ | 0.85 |
| Rod length | 6–10 m | $C_R$ | 0.95 |
| Rod length | 10–30 m | $C_R$ | 1.0 |
| Sampling method | Standard sampler | $C_S$ | 1.0 |
| Sampling method | Sampler without liners | $C_S$ | 1.1–1.3 |

**Figure 1:** Hammer Correction Table for SPT Value [6]

The normalization factor of $C_N$ against overburden stress is calculated as Equation 12 [6].

$$C_N = \frac{2.2}{1.2 + \frac{\sigma'_v}{P_a}} \tag{12}$$

Where, $P_a$ is the atmospheric pressure and which is taken as 100 KPa.
The overburden correction factor for cyclic stress ratio (K) is calculated as Equation 13 [6].

$$K_\sigma = \left(\frac{\sigma'_{v0}}{P_a}\right)^{(f-1)} \tag{13}$$

Where, f is the exponent which is the function of site condition. The value of f is depends on relative densities [6]. For relative densities between 40 and 60% its value is 0.7-0.8 and for relative densities between 60 and 80% its value is 0.6-0.7. For our analysis the value of f is taken as 0.75.
The liquefaction potential index (LPI) gives the severity of the site towards liquefaction and is tabulated in Table 1 [9]

**Table 1:** Liquefaction Potential Index (LPI) by Iwasaki et al., 1981

| LPI | Severity of Site |
|---|---|
| 0 | Very Low |
| $0 < LPI < 5$ | Low |
| $5 < LPI < 15$ | High |
| $15 < LPI$ | Very High |

The PGA value for the pseudo static analysis and for deterministic approach is taken from the national building code and seismic hazard map of Nepal.

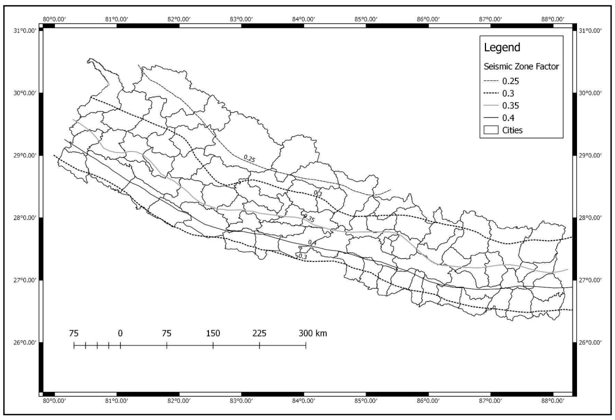



**Figure 2:** Seismic Hazard Map of Nepal (National Reconstruction Authority, 2020)

From the seismic hazard map of Nepal from Figure 2, the PGA value for the pseudo static general limit equilibrium analysis is taken as 0.3 [10].

## 3. Methodology

The methodology applied for this paper is collection of the geotechnical data, liquefaction calculation by deterministic approach and this result is verified in Slope/W 2018. The flow chart is shown in Figure 3.

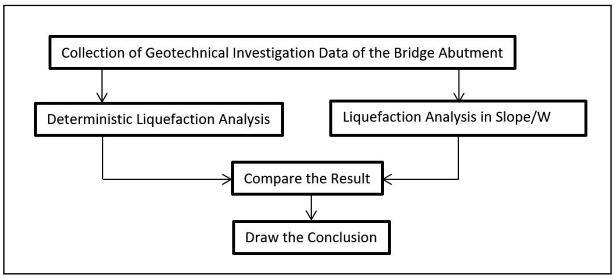



**Figure 3:** Flowchart for methodology

### 3.1 Data Collection

The geotechnical field and laboratory test data of the Kajala River Bridge at sijuwa road and Katnu Khola River Bridge milan chowk south is collected from the Government organization Ratuwamai Municipality, Morang. The available these data were filtered and used in numerical model.

### 3.2 Deterministic Liquefaction Analysis

Liquefaction of the two bridges abutment Kajala River Bridge at Sijuwa road and Katnu Khola river bridge Milanchowk South is done by the deterministic approach. NCEER 1996/1998 method is used to the liquefaction analysis. Various input data is used for the liquefaction analysis which are listed in Table refDLA.

**Table 2:** Input Value for Deterministic Liquefaction Analysis

| Description | Input value | Remarks |
|---|---|---|
| Moment Magnitude ($M_w$) | 7.8 | |
| PGA | 0.3 | [10] |
| Exponent (f) | 0.75 | |
| Atmospheric Pressure | 100 KPa | |

### 3.3 Material Modeling

For the numerical modeling of the bridge pile abutment, the material is modeled according to the Mohr-Coulomb criterion and the pseudo static factor of safety is calculated by General Limit Equilibrium by using the Morgenstern Price method. For the material modeling, the unit weight and cohesion of the soil calculated by the consultant is used whereas the internal frictional angle is calculated by using the empirical relation. The internal frictional angle of the soil material is calculated as [11],

$$\phi = 15 + \sqrt{(12 * N_{corr})} \qquad (14)$$

Where, $\phi$ is the internal frictional angle of the soil and $N_{corr}$ is the corrected SPT value.

### 3.4 Model Introduction

The numerical modeling of bridge pile abutment starts from the collection and review of the available site and bridge foundation design information to develop the geometric model of the abutment soil and pile foundation system analysis.

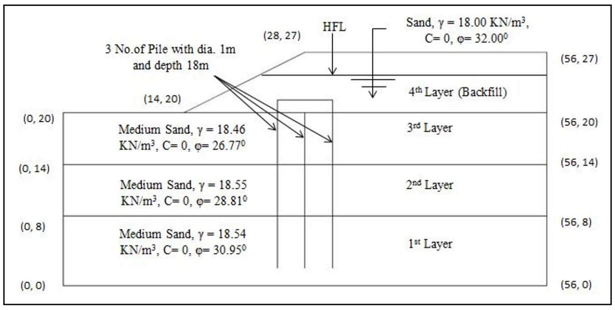



**Figure 4:** Sample Model of Katnu Khola River Bridge Abutment-1

From the Figure 4, the model dimension at the left and right should be at least 2H and 4H from toe and top of the fill end slope, respectively. The H here represents the fill height in the model. For the model prepared in Slope/W, the soil profile should be at least 2H below the toe elevation or 3H from top of the abutment embankment.

In the model, the total depth of the soil profile is 20m excluding the backfill. For the easy model preparation and the analysis the pile is placed in middle throughout the lateral distance. The total 20m depth soil profile is divided in 3 layers as shown in the above figure, two layers of each 6m from the top and remaining 8m. In the model, the high flood level (HFL) is considered critical water level for the analysis.

The pile is 1m diameter and there is 3*4 pile group, the row distance is 3m and the pile spacing is 3.2m.

The pile used is concrete pile and the in the slope/w model, the pile is considered as a 1D reinforcement.

### 3.5 Input parameters of Model

For the all four abutment the different input parameters are listed in Table 4 and Table 3

**Table 3:** Pile Input Parameter for Slope/W

| S.No. | Pile Parameter | Input Value |
|---|---|---|
| 1 | Pile Length | 18 m |
| 2 | Pile Direction (thita) | 90 degrees |
| 3 | Pile Spacing (S) | 3.2 m |
| 4 | Shear Force in Pile ($V_c$) | 1.06 * $R_{Tot}$ |
| 5 | Shear Force Reduction Factor ($f_g$) | 1.0 |
| 6 | Direction of the Shear Force | Parallel to Slip |

## 4. Results and Discussion

For the deterministic approach of liquefaction analysis NCEER-1996/1998 method is used and the factor of safety is calculated which is listed below for the two bridges of each two abutment site, altogether there is four borehole in the research. The value of deterministic approach of liquefaction is listed in Table 5. From the deterministic liquefaction of the four borehole geotechnical data of two bridges, the liquefaction susceptible of BH-1 of Kajala River bridge at sijuwa road found to be high and remaining other three boreholes are very high. So that, all these four boreholes are susceptible for liquefaction potential.

The pseudo static factor of safety obtained from the slope/w is shown in Figure 6

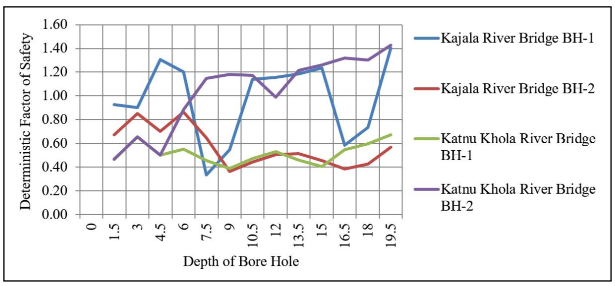



**Figure 5:** Deterministic Factor of Safety VS Depth of Bore Hole

From the Figure 6 of pseudo static factor of safety vs PGA, the factor of safety from slope/w at 0 PGA is greater than one for all four abutments which means that at in the initial condition there is no susceptible of liquefaction. Similarly, at 0.3 PGA the pseudo static factor of safety for all four abutments is less than one which says that there is liquefaction potential at zero horizontal pile load.

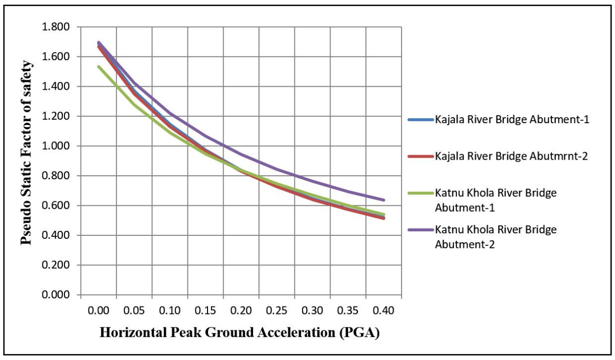
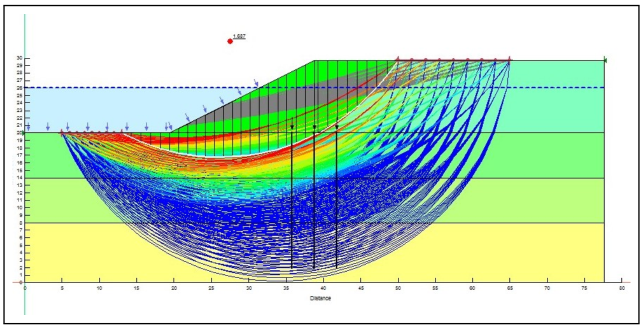



**Figure 6:** Pseudo Static Factor of Safety VS PGA at Zero Horizontal Pile Load

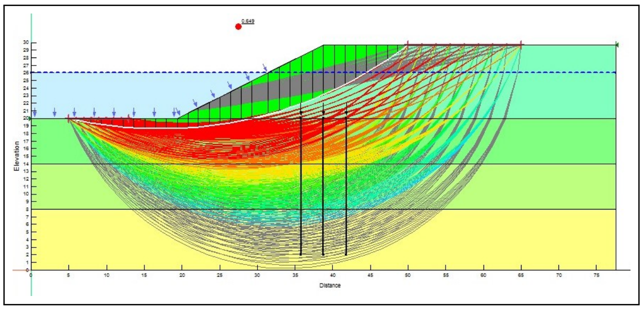
## 5. Conclusion and Future Recommendation

### 5.1 Conclusion

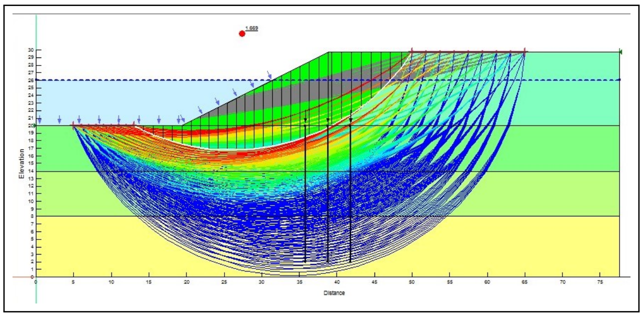
Deterministic approach for the liquefaction analysis is done and this liquefaction is verified by the numerical method in slope/w 2018. It is concluded that the factor of safety calculated by deterministic approach and pseudo static factor of safety calculated from numerical modeling in slope/w 2018 depend on the peak ground acceleration and earthquake magnitude. The factor of safety calculated by deterministic approach for the liquefaction susceptibility is smaller in the case of higher peak ground acceleration and higher earthquake moment magnitude and vice versa. The factor of safety is decreased by increasing the ground water table as the pore water pressure is increased resulting in reduced effective stress and shear strength of soil. It is observed that the pseudo static factor of safety calculated by slope/w 2018 depends on the soil material property, water table in the model and the peak ground acceleration at the site. The higher value of the internal frictional angle of the soil and the higher value of peak ground acceleration gives the higher value of the pseudo static factor of safety and vice versa. The pseudo static factor of safety also depends on the ground water table in the model. Higher value of water table depth from the bottom of the model yields lower value of pseudo static factor of safety during numerical modeling and



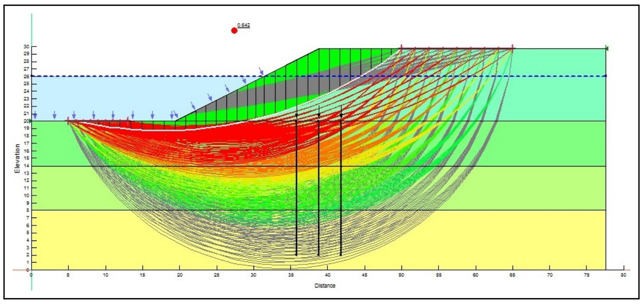
**(a)** Slip Circles and Critical Pseudo Static FOS of Kajala River Bridge, Abutment-1 at 0 PGA



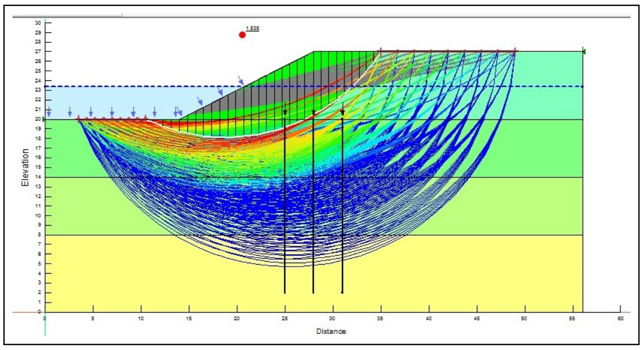
**(b)** Slip Circles and Critical Pseudo Static FOS of Kajala River Bridge, Abutment-1 at 0.3 PGA



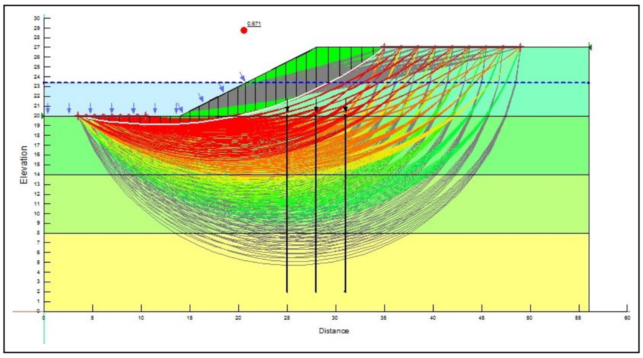
**(c)** Slip Circles and Critical Pseudo Static FOS of Kajala River Bridge, Abutment-2 at 0 PGA



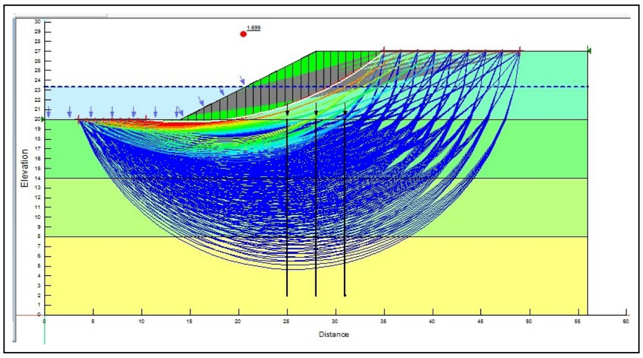
**(d)** Slip Circles and Critical Pseudo Static FOS of Kajala River Bridge, Abutment-2 at 0.3 PGA



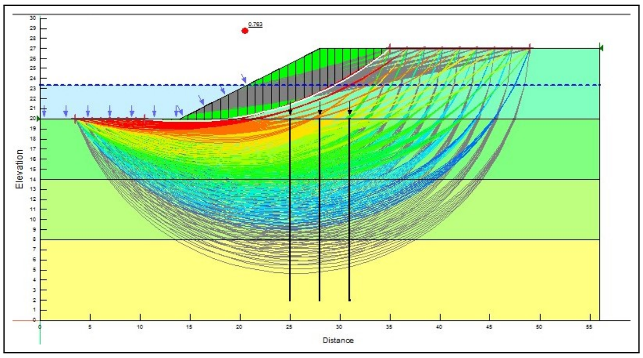
**(e)** Slip Circles and Critical Pseudo Static FOS of Katnu Khola River Bridge, Abutment-1 at 0 PGA



**(f)** Slip Circles and Critical Pseudo Static FOS of Katnu Khola River Bridge, Abutment-1 at 0.3 PGA



**(g)** Slip Circles and Critical Pseudo Static FOS of Katnu Khola River Bridge, Abutment-2 at 0 PGA



**(h)** Slip Circles and Critical Pseudo Static FOS of Katnu Khola River Bridge, Abutment-2 at 0.3 PGA

**Figure 7:** Slip Circles and Critical Pseudo Static FOS

**Table 4:** Soil Input Parameters

| S.No. | Bridge Name | Depth | Unit weight (KN/m$^3$) | Cohesion (C) | Frictional Angle $^\circ$ |
|---|---|---|---|---|---|
| 1 | Kajala River Bridge, Abutment-1 | Backfill | 18.00 | 0.00 | 32.00 |
| | | 0-6 | 17.39 | 0.00 | 28.50 |
| | | 6-12 | 18.51 | 0.00 | 31.93 |
| | | 12-20 | 18.43 | 0.00 | 35.88 |
| 2 | Kajala River Bridge, Abutment-2 | Backfill | 18.00 | 0.00 | 32.00 |
| | | 0-6 | 18.25 | 0.00 | 27.27 |
| | | 6-12 | 18.37 | 0.00 | 29.01 |
| | | 12-20 | 18.42 | 0.00 | 29.90 |
| 3 | Katnu Khola River Bridge, Abutment-1 | Backfill | 18.00 | 0.00 | 32.00 |
| | | 0-6 | 18.46 | 0.00 | 26.77 |
| | | 6-12 | 18.55 | 0.00 | 28.81 |
| | | 12-20 | 18.54 | 0.00 | 30.95 |
| 4 | Katnu Khola River Bridge, Abutment-2 | Backfill | 18.00 | 0.00 | 32.00 |
| | | 0-6 | 18.73 | 0.00 | 27.38 |
| | | 6-12 | 18.58 | 0.00 | 34.36 |
| | | 12-20 | 18.78 | 0.00 | 37.07 |

**Table 5:** Summary of Deterministic Liquefaction Analysis at PGA = 0.3

| S.No. | Bridge Name | Bore Hole | Liquefaction Potential Index | Severity |
|---|---|---|---|---|
| 1 | Kajala River Bridge at Sijuwa Road | BH-1 | 13.75 | High |
| | | BH-2 | 33.99 | Very High |
| 2 | Katnu Khola River Bridge Milan Chowk South | BH-1 | 46.07 | Very High |
| | | BH-2 | 18.72 | Very High |

vice versa.

### 5.2 Future Recommendations

There are various parameters that are considered for this research so that in depth study in required for these parameters. The future recommendation for this research are given as:

- In this research geometric model is analyzed by General Limit Equilibrium method, it is recommended to approach this study by Finite Element Method of analysis.
- Calculation of pseudo static factor of safety by taking horizontal and vertical component of peak ground acceleration simultaneously can be studied.

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## References

[1] S. L. Kramer. GEOTECHNICAL EARTHQUAKE ENGINEERING Kramer 1996.pdf, 1996.

[2] Sreevalsa Kolathayar and T.G. Sitharam. Deterministic seismic hazard assessment. *Earthquake Hazard Assessment*, pages 35–44, 2018.

[3] M. Subedi, K. Sharma, I. P. Acharya, and K. Adhikari. Soil liquefaction in Kathmandu valley due to 2015 Gorkha, Nepal earthquake and assessment of liquefaction susceptibility. *11th National Conference on Earthquake Engineering 2018, NCEE 2018:*

*Integrating Science, Engineering, and Policy*, 9:5579–5583, 2018.

[4] Xiaohua Bao, Zhiyang Jin, Hongzhi Cui, Xiangsheng Chen, and Xiongyao Xie. Soil liquefaction mitigation in geotechnical engineering : An overview of recently developed methods. *Soil Dynamics and Earthquake Engineering*, 120(June 2018):273–291, 2019.

[5] A K Murashev, D K Kirkcaldie, C Keepa, and J N Lloyd. The assessment of liquefaction and lateral spreading effects on bridges. 2014.

[6] T Leslie Youd, I M Idriss, and Ronald D Andrus. Liquefaction Resistance of Soils : Summary Report from the 1996 NCEER and 1998 NCEER / NSF Workshops on Evaluation of Liquefaction Resistance of Soils. 0241(October), 2001.

[7] I. M. Idriss and R. W. Boulanger. Semi-empirical procedures for evaluating liquefaction potential during earthquakes. *Soil Dynamics and Earthquake Engineering*, 26(2-4 SPEC. ISS.):115–130, 2006.

[8] K. Onder Cetin, Raymond B. Seed, Armen Der Kiureghian, Kohji Tokimatsu, Leslie F. Harder, Robert E. Kayen, and Robert E. S. Moss. Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential. *Journal of Geotechnical and Geoenvironmental Engineering*, 130(12):1314–1340, 2004.

[9] T Iwasaki, K Tokida, and F Tatsuoka. Soil Liquefaction Potential Evaluation with Use of the Simplified Procedure. *International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, 12:209–214, 1981.

[10] National Reconstruction Authority. Seismic Design of Buildings in Nepal. *Ministry of Urban Development*, pages 1–111, 2020.

[11] John W. Dunham. Pile foundations for buildings. 1954.