Seismic Design for Infrastructures

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ABSTRACT

Seismic design for infrastructures such as building and non-building is designed based on design ground shaking shall be characterized by the design spectrum. This study aims to evaluate seismic design for infrastructures following SNI 1726-2019. Several site investigations are taken by conducting boring to a depth of 30 m to count the soil site classification. It might be summarized that the area of investigation is classified as SC/hard soil, very solid and soft rock, SD/medium soil and SE (soft soil).

Keywords: seismic design, infrastructure, site investigation.

1. INTRODUCTION

This study aims to evaluate seismic design following SNI 1726-2012. Ground shaking is evaluated by the design spectrum of soil. Information of ground strength is necessary. Soil characteristics, soil consistency are primarily important. According to CALTRANS soil is categorized as S1 and S2 and classified soil as types A, B, C, D, E, and F. According to SNI 1726-2019, soil has characteristics as SA/hard rock, SB/rock, SC/hard soil, very solid and soft rock, SD/medium soil, SE/soft soil, and SF/special soils that require specific geotechnical investigations and response analysis.

Several site investigations by drilling and testing are conducted in order to evaluate site classes to the proposed infrastructures. The drilling bore hole is taken up to 30 meter in dept and completed with Standard Penetration Testing (N value) and laboratory testing. The site is located around Jimbaran and Kuta area, in Badung regency, Bali province, Indonesia.

2. THEORY AND METHODS

2.1 Seismic Design

Seismic design for building and non building in Indonesia should follow SNI 1726-2019. SNI 1726-2019 categorized soil as SA/hard rock, SB/rock, SC/hard soil, very solid and soft rock, SD/medium soil, SE/soft soil, and SF/special soils that require specific geotechnical investigations and response analysis.

Caltrans (2019) characterized soil as class S1 and class S2 soil. Soils with all the following characteristics shall be classified as Class S1: Standard penetration test, $(N1)60 \geq 30$ (Granular soils), Undrained shear strength, $su > 2000$ psf (Cohesive soils), Shear wave velocity, $vs > 886$ ft/sec, Not susceptible to liquefaction, lateral spreading, or scour. where: $(N1)60$ = penetration resistance corrected for overburden pressure and hammer efficiency. Any soil that does not satisfy the requirements of Class S1 shall be classified as "Class S2." Lateral analysis shall be required for foundations in Class S2 soils. Caltrans (2019) also classified soil as types A, B, C, D, E, and F.

Table 1. Site Classification (SNI 1726-2019)

Site Class	$\bar{v}_s (m/\det ik)$	$\bar{N} atau \bar{N}_{ch}$	$\bar{s}_u (kPa)$
SA (hard rock)	>1500	N/A	N/A
SB (rock)	750 - 1500	N/A	N/A
SC (hard soil, very solid and soft rock)	350 - 750	>50	≥ 100
SD (medium soil)	175 - 350	15 - 50	50 - 100
SE (soft soil)	<175	<15	<50
	Or any soil profile containing more than 3 m of soil with the following characteristics: 1. Plasticity Index, *PI*>20 2. Water content w≥40% 3. Undrained shear strength $\bar{s}_u < 25 kPa$		
SF (special soils that require specific geotechnical investigations and response analysis)	Any soil layer profile that has one or more of the following characteristics: - Vulnerable and has the potential to fail or collapse due to earthquake loads such as susceptible to liquefaction, very sensitive clay, weakly cemented soil - Very organic clay and/or peat (thickness H>3m) - Clay with very high plasticity (thickness H>7.5 m with plasticity index PI>75) - Soft/semi-firm clay layer with thickness H>35 m with 5 < 50 kPa $\bar{s}_u < 50 kPa$		

2.2 Response Spectrum Design

According to SNI 1726-2019, to determine the spectral response of MCE_R earthquake acceleration at the ground surface, a seismic amplification factor is required at a period of 0.2 seconds and a period of 1 second. Amplification factors include acceleration-related vibration amplification factors for short period vibrations (F_a) and acceleration-related amplification factors representing 1 second period vibrations (F_v). Acceleration spectral response parameters in short periods (S_{MS}) and 1 second periods (S_{M1}) which are adjusted to the influence of site classification, must be determined using the following formulation:

$$S_{MS} = F_a S_s \quad (1)$$

$$S_{M1} = F_v S_1 \quad (2)$$

With site coefficients F_a and F_v following Table 2 and Table 3.

Table 2. Site coefficient, F_a

Site class	**The maximum considered risk-targeted earthquake acceleration spectral response (MCE_R) parameters are mapped to the short period, T = 0.2 seconds, S_s**					
	$S_S \leq 0,25$	$S_S = 0,5$	$S_S = 0,75$	$S_S = 1,0$	$S_S = 1,25$	$S_S \geq 1,5$
SA	0,8	0,8	0,8	0,8	0,8	0,8
SB	0,9	0,9	0,9	0,9	0,9	0,9
SC	1,3	1,3	1,2	1,2	1,2	1,2
SD	1,6	1,4	1,2	1,1	1,0	1,0
SE	2,4	1,7	1,3	1,1	0,9	0,8
SF	SS[(a)]					

Table 3. Site coefficient, F_v

Site class	**The maximum considered risk-targeted earthquake acceleration spectral response (MCE_R) parameters are mapped in period 1 seconds, S_1**					
	$S_1 \leq 0{,}1$	$S_1 = 0{,}2$	$S_1 = 0{,}3$	$S_1 = 0{,}4$	$S_1 = 0{,}5$	$S_1 \geq 0{,}6$
SA	0,8	0,8	0,8	0,8	0,8	0,8
SB	0,8	0,8	0,8	0,8	0,8	0,8
SC	1,5	1,5	1,5	1,5	1,5	1,4
SD	2,4	2,2	2,0	1,9	1,8	1,7
SE	4,2	3,3	2,8	2,4	2,2	2,0
SF	SS[(a)]					

Design spectral acceleration parameters for short periods, S_{DS} and at 1 second period, S_{D1}, must be determined through the following formulation:

$$S_{DS} = \frac{2}{3} S_{MS} \tag{3}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{4}$$

If a design response spectrum is required by this ordinance and a site-specific ground motion procedure is not used, then a design response spectrum curve must be developed by referring to Figure 1 and following the provisions below:

1. For periods smaller than T_0, the design acceleration response spectrum, S_a, should be taken from the equation;
   $$S_a = S_{DS}\left(0{,}4 + 0{,}6\frac{T}{T_0}\right) \tag{5}$$
2. For periods greater than or equal to T_0 and less than or equal to T_s, the design acceleration response spectrum, S_a, is the same as S_{DS};
3. For periods greater than T_s but less than or equal to T_L, the design acceleration spectral response, S_a, is taken based on the equation:
   $$S_a = \frac{S_{D1}}{T} \tag{6}$$
4. For periods greater than T_L, the design acceleration spectral response, S_a, is taken based on the equation:
   $$S_a = \frac{S_{D1} T_L}{T^2} \tag{7}$$
   where:
   S_{DS} = design acceleration spectral response parameter at short periods;
   S_{D1} = design acceleration spectral response parameter at a period of 1 second;
   T = period of fundamental vibration of the structure.
   $$T_0 = 0{,}2\frac{S_{D1}}{S_{DS}} \tag{8}$$

$$T_s = \frac{S_{D1}}{S_{DS}}$$

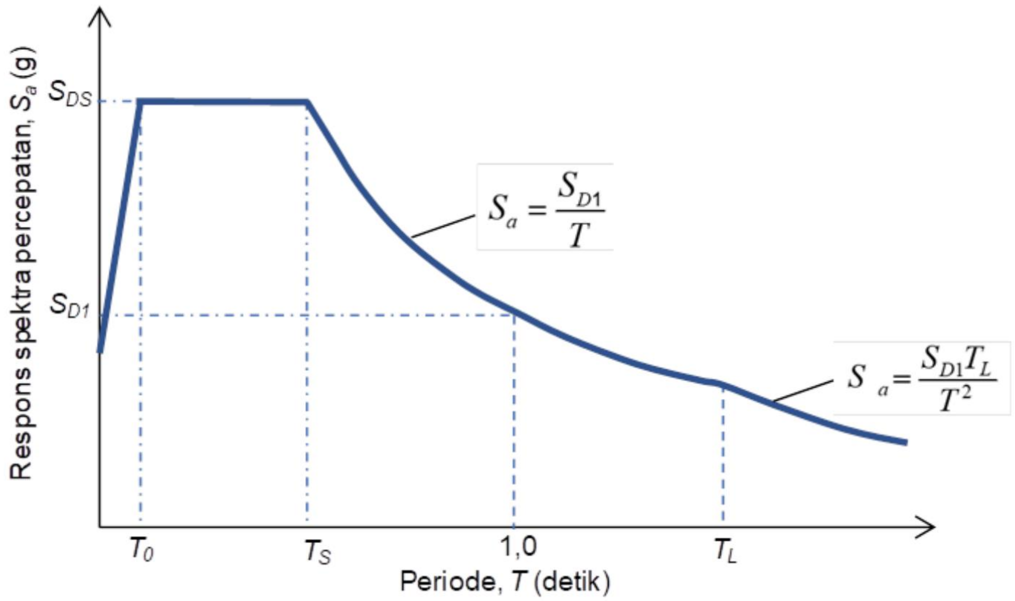


Figure 1. Design response spectrum

2.3 Foundation Design

Following site classification as indicated in Table 1, evaluation of bearing capacity of foundation should follow the equation for soil and the equation for rock where applicable. Therefore, below is given few formula to estimate bearing capacity of foundation founded on soil and foundation founded on rock.

2.3.1 Bearing Capacity on Soil

Shallow Foundation-Shallow foundation with Laboratory Shear Test values as pproposed by Terzaghi's formula:

For Clay: $$\sigma_{ult} = 1.2cN_c + \gamma D_f N_q \quad (10)$$

For Sand: $$\sigma_{ult} = 1.2cN_c + \gamma D_f N_q + 0.5\gamma B N_\gamma \quad (11)$$

where: σ_{ult} = ultimate bearing capacity; c = cohesion, γ = unit weight of soil; D_f = foundation depth, B = foundation width, SF= 3, N_c , N_q *dan* N_γ = bearing capacity factor depends on the soil friction angle ϕ, values as in the graph Fig.2 below.

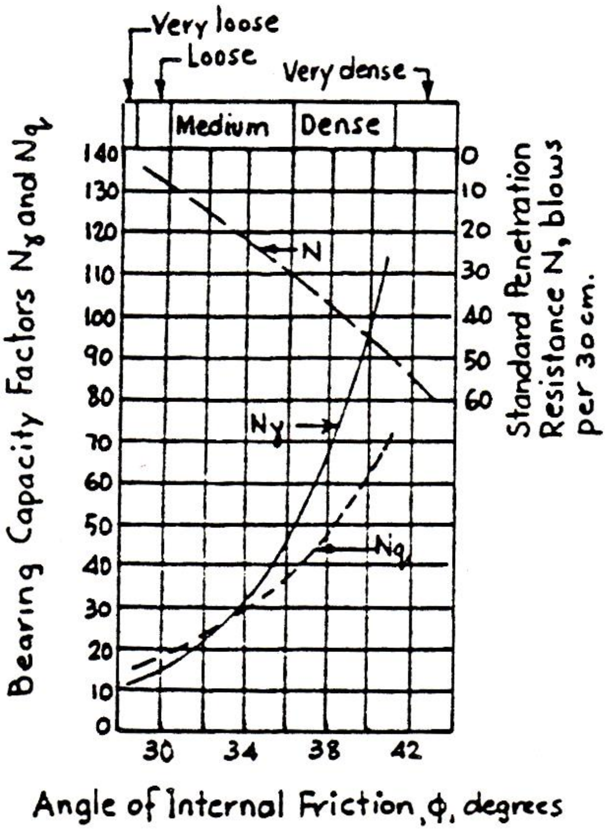


Figure 2. Graph of the relationship between bearing-capacity factors and ϕ, and the empirical relationship of the standard penetration resistance value N. (After Peck et al (1974))

Bearing Capacity of Shallow Foundations using the corrected N value as follows:

$$\sigma_{ijin} = \frac{N}{0.05\left(1 + 0.33\frac{D_f}{B}\right)} \tag{12}$$

where: B = Width (<1.20 m), D_f = Depth, N = Corrected SPT value.

Peck Hanson and Tornburn (1974) suggested using a value of σ'_v = 100 kPa (1 TSF) as a standard value while correcting the field N value. The corrected N value is proposed to be:

$$N_{corr} = N_{field} \;\; x\, C_N \tag{13}$$

where: C_N is coreection factor, $C_N=1$ *for* σ'_v = 100 kPa (1 TSF).

Peck et al (1974) proposed the following relationship:

$$p = 11.0\; x\; N_{corr}\; (kPa) \quad \approx \quad q_{net\ ijin} \tag{14}$$

where: p = net vertical pressure acting on the footing with a maximum drop of 25 mm provided that the water level is below B (B is the width of the footing).

Peck et al. (1974) proposed an empirical groundwater level correction factor C_W of:

$$C_W = 0.5\left(1+\frac{D_W}{D_f + B}\right) \quad (15)$$

where: D_w= Depth of the ground water surface from the ground surface, B =Smallest foundation width, D_f= Foundation depth.

Thus the equation becomes:

$$p = 11.0\ x\ C_W\ x\ N_{corr}\ (kPa) \approx q_{net\ ijin} \quad (16)$$

DEEP FOUNDATIONS AND PILE FOUNDATIONS

Bearing Capacity of Pile Foundations in Clay by applyingTotal Pressure Analysis

End Resistance

The ultimate bearing capacity at the end of the pile foundation is stated as below.

$$q_f = c_u N_c \quad (17)$$

where: c_u = undrained shear strength of clay, N_c = Soil bearing capacity factor N_c = 9 (based on Skempton for $D/B>4$)

Friction Resistance

The friction bearing capacity around the pile foundation is expressed as:

$$f_s = \alpha \bar{c}_u \quad (18)$$

where: $\bar{c}_u$ = Average undrained cohesion value, α = coefficient that depends on the type of clay and pile material; α = 0.3 to 1.0

Bearing capacity of pile foundation by applying Effective Stress Analysis

End Resistance

The end of pile foundation bearing capacity is the same as in sand

$$q_f = \sigma'_o N_q \quad (19)$$

Friction Resistance

Skin friction resistance or friction is expressed by the following equation:

$$f_s = K\bar{\sigma}'_o \tan\phi' \quad \text{or} \quad f_s = \beta\bar{\sigma}'_o \quad (20)$$

where: β = 0,25 - 0,40 for clay and silt, β = 0,8 for coarse and dense sand

Hence, Total Bearing Capacity as Ultimate bearing capacity becomes:

$$Q_u = A_b\sigma'_o N_q + A_s\beta\bar{\sigma}'_o \quad (21)$$

The allowable bearing capacity is:

$$Q_{ijin} = \frac{Q_u}{FS} = \frac{A_b\sigma'_o N_q + A_s\beta\bar{\sigma}'_o}{3} \quad (22)$$

Deep Foundations (Piles, Drilled Piles) Based on Corrected SPT Values

a. End Resistance $q_f = 40N \frac{D_b}{B} \leq 400N \ldots\ldots (kN/m^2)$ (23)

b. Friction Resistance $f_s = 2\overline{N} \ldots\ldots\ldots (kN/m^2)$ (24)

Allowable Bearing Capacity: $Q_{ijin} = \frac{A_b.q_f}{3} + \frac{A_s.f_s}{2} \ldots\ldots (kN)$ (25)

where: A_b = Base area of the pile, A_s = Cover area (Perimeter area) of the pile

2.3.2 BEARING CAPACITY ON ROCK

Bearing capacity of Shallow Foundation founded on rock might be estimated by equation as stated in Figure 3. Table 4 provide guidance to estimate pressure on rock mass.

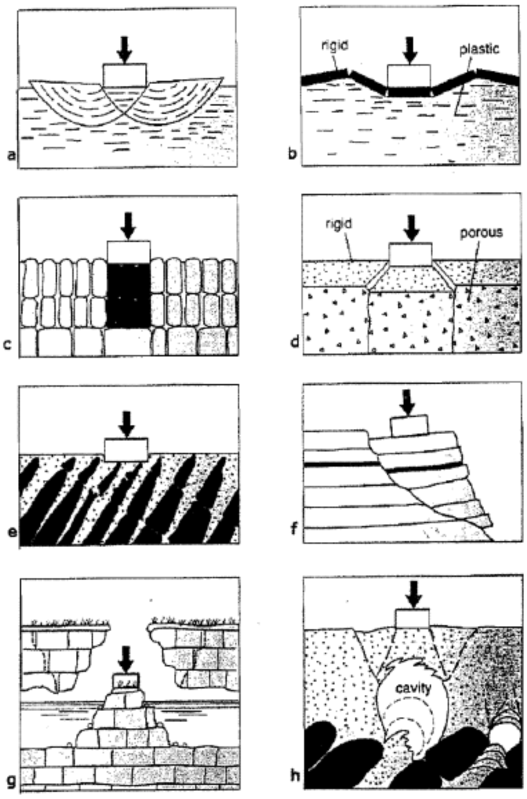


Figure 3. Mechanisms of foundation failure (from Franklin and Dusseault, 1989; adapted from Sowers, 1976): a) Prandtl-type shearing in weak rock; b) shearing with superimposed brittle crust; c) compression of weathered joints; d) compression and punching of porous rock underlying a rigid crust; e) breaking of pinnacles from a weathered rock surface; f) slope failure caused by superimposed loading; g) collapse of a shallow cave; and h) sinkhole caused by soil erosion into solution cavities.

Table 4. Applicability of Methods for the Determination of Design Bearing Pressure on Rock depending upon Rockmass Quality

Rockmass Quality	Basis of Design Method
Sound rock Rockmass with *wide* or *very wide* discontinuity *spacing*	Core strength

Rockmass with closed discontinuities at *moderately close*, *wide* and *very wide spacing*	Core strength
Low to very low strength rock Rockmass with *close* or *very closely spaced* discontinuities	Pressure meter
Very low strength rock Rockmass with *very closely spaced* discontinuities	Soil mechanics approach

In all cases, field tests may also be used to assess the capacity and load-deformation characteristics of the rock mass.

The final determination of the design bearing pressure on rock may be governed by the results of the analysis of the influence of the discontinuities on the behavior of the foundation. As a guideline, in the case of a rock mass with favorable characteristics (e.g., the rock surface is perpendicular to the foundation, the load has no tangential component, the rock mass has no open discontinuities), the design bearing pressure may be estimated from the following approximate relation:

$$q_a = K_{sp} \times q_{u-core} \tag{26}$$

where: q_a = design bearing pressure, q_{u-core} = average unconfined compressive strength of rock (as determined from ASTM D2938), K_{sp}= an empirical coefficient, which includes a factor of safety of 3 (in terms of working stress design) and ranges from 0.1 to 0.4 (see Table 5 and Figure 4)

Table 5. Coefficients of Discontinuity Spacing, K_{sp}

Discontinuity Spacing		$\boldsymbol{K_{sp}}$
Description	**Distance (m)**	
Moderately close	0.3 to 1	0.1
Wide	1 to 3	0.25
Very wide	>3	0.4

The factors influencing the magnitude of the coefficient are shown graphically in Figure 4. The relationship given in the figure is valid for a rock mass with spacing of discontinuities greater than 300 mm, aperture of discontinuities less than 5 mm (or less than 25 mm, if filled with soil or rock debris), and for a foundation width greater than 300 mm. For sedimentary rocks, the strata must be horizontal or nearly so.

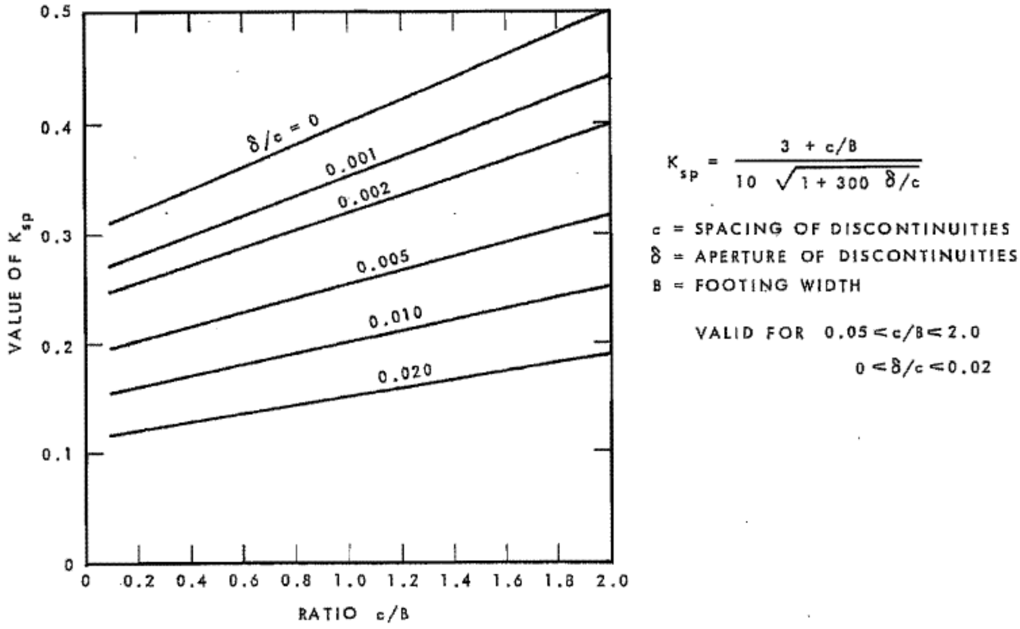


Figure 4. Bearing pressure coefficient K_{sp}

The bearing-pressure coefficient, K_{sp}, as given in Figure 4, takes into account the size effect and the presence of discontinuities and includes a nominal safety factor of 3 against the lower-bound bearing capacity of the rock foundation. The factor of safety against general bearing failure (ultimate limit states) may be up to ten times higher. For a more detailed explanation, see Ladanyi et al. (1974). Franklin and Gruspier (1983) discuss a special case of foundations on shale.

Bearing capacity of Pile Foundation founded on rock might be estimated as follows:

Bearing Pressure from Strength of Rock Cores

The method described is applicable to deep foundations. According to Ladanyi and Roy (1971) the effect of depth is included and the formula becomes:

$$q_a = \sigma_c K_{sp} d \tag{27}$$

where: q_a = allowable bearing pressure, σ_c = average unconfined compressive strength of rock core, from ASTM D2938, K_{sp} = empirical factor, as given in Section 9.2 and including a factor of safety of 3, d = depth factor = $1 + 0.4\frac{L_s}{B_s} \leq 3$, L_s = depth (length of the socket), B_s= diameter of the socket

For limit states design, it is suggested that the ultimate axial capacity be calculated as multiplying the allowable value by three. The factored geotechnical resistance at ultimate limit states would then be obtained by multiplying the ultimate capacity by the geotechnical resistance factor of 0.4 and 0.3 for compression and uplift conditions respectively.

3. RESULTS AND DISCUSSION

The research results are presented in full and in accordance with the scope of the study. The results of the research can be completed with tables, graphs (images), and / or charts. Tables and figures are numbered and titled. The results of the data analysis were interpreted correctly.

The purpose of the Results and Discussion is to state your findings and make interpretations and/or opinions, explain the implications of your findings, and make suggestions for future research. Its main function is to answer the questions posed in the Introduction, explain how the results support the answers and, how the answers fit in with existing knowledge on the topic. The Discussion is considered the heart of the paper and usually requires several writing attempts.

All tables should be numbered with Arabic numerals. Headings should be placed above tables, center. Only horizontal lines should be used within a table, to distinguish the column headings from the body of the table, and immediately above and below the table. Tables must be embedded into the text and not supplied separately. Below is an example which authors may find useful.

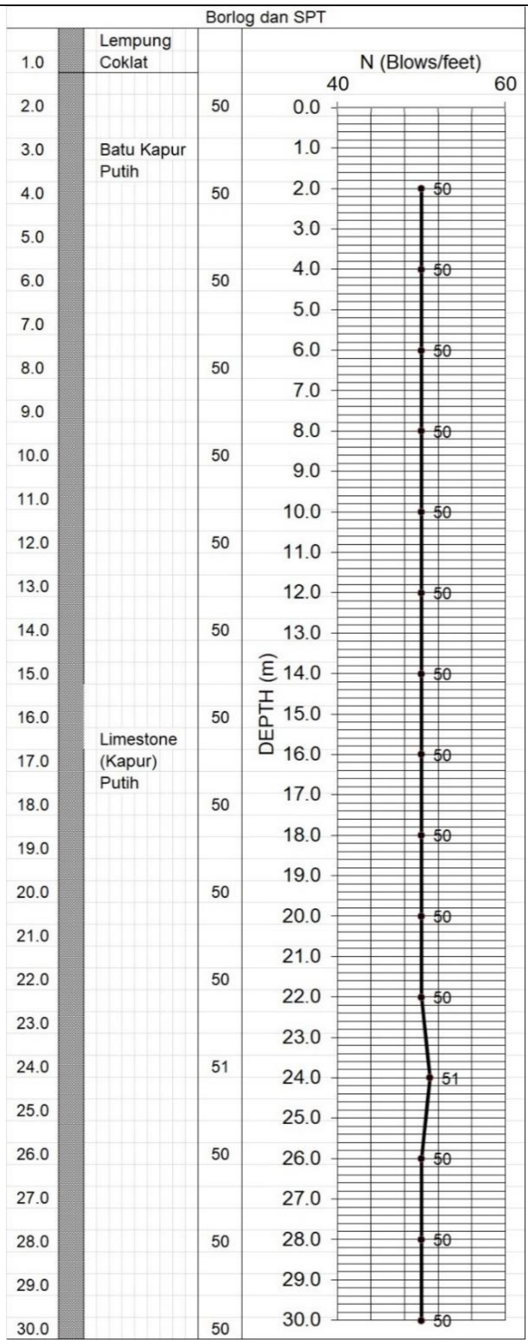
Some site investigation available to be evaluate consisting boring test and Standard Penetration Test to a dept of 30 m as required in SNI. The area of soil investigation is around Jimbaran and Kuta, Badung, Bali, Indonesia. Table 6 shows boring investigation in batu meguwung and Bingin Beach. Bore hole in batu meguwung shows limestone hard with N=50, insipte of brown clay layer from ground surface to a depth of 1 meter. Lime stone layer from depth of 1 m to 30 m consisting of hard limestone with UCT test c_u=40 kg/cm^2=400 t/m^2=4000 kPa. Figure 5 shows the core drilling of the hard limestone in the area of Batu meguwung. According to Tabel 1 site classification is SC (hard soil, very solid and soft rock).

Table 6 also shows boring investigation in Bingin beach. Bore hole in Bingin Beach reveal N values 28 to 50, give average N=33. Hard limestone is only with UCT test c_u=40 kg/cm^2=400 t/m^2=4000 kPa is found in a depth 1m to 5 m as shown by coring drilling in Figure 6. According to Tabel 1 site classification is SD (medium soil).

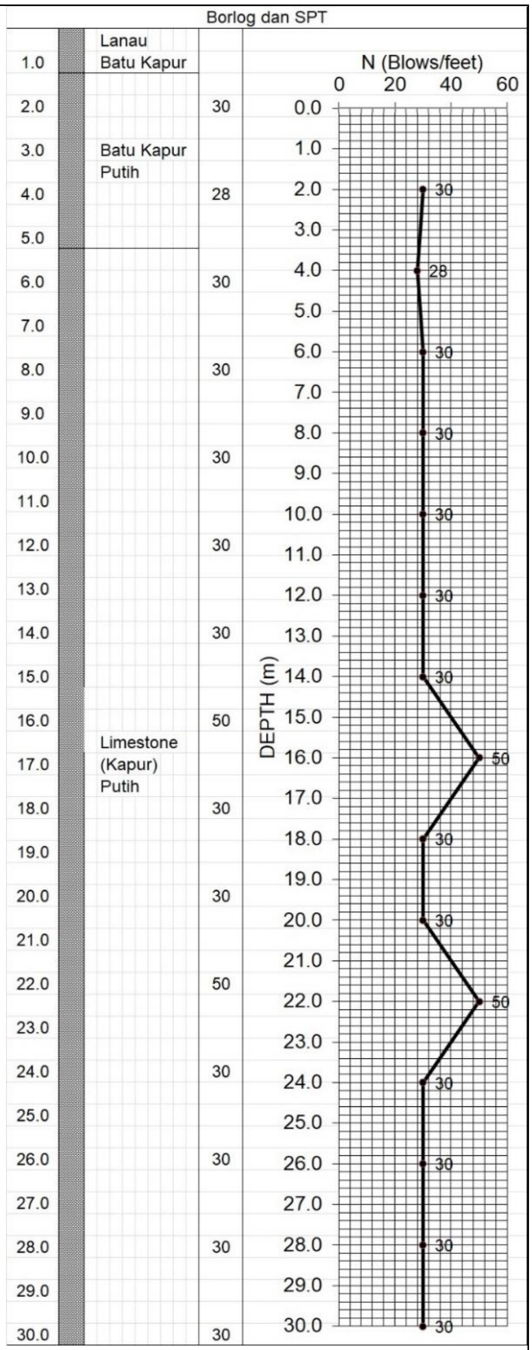
Figure 5. Core box hasil pengeboran di daerah Pura Batu Meguwung.

Figure 6. Core box hasil pengeboran di daerah Pantai Bingin kedalaman 4-5 meter.

Table 6. Bor logs and SPT test in Batu Meguwung and Bingin Beach



Bor Pura Batu meguwung B-3

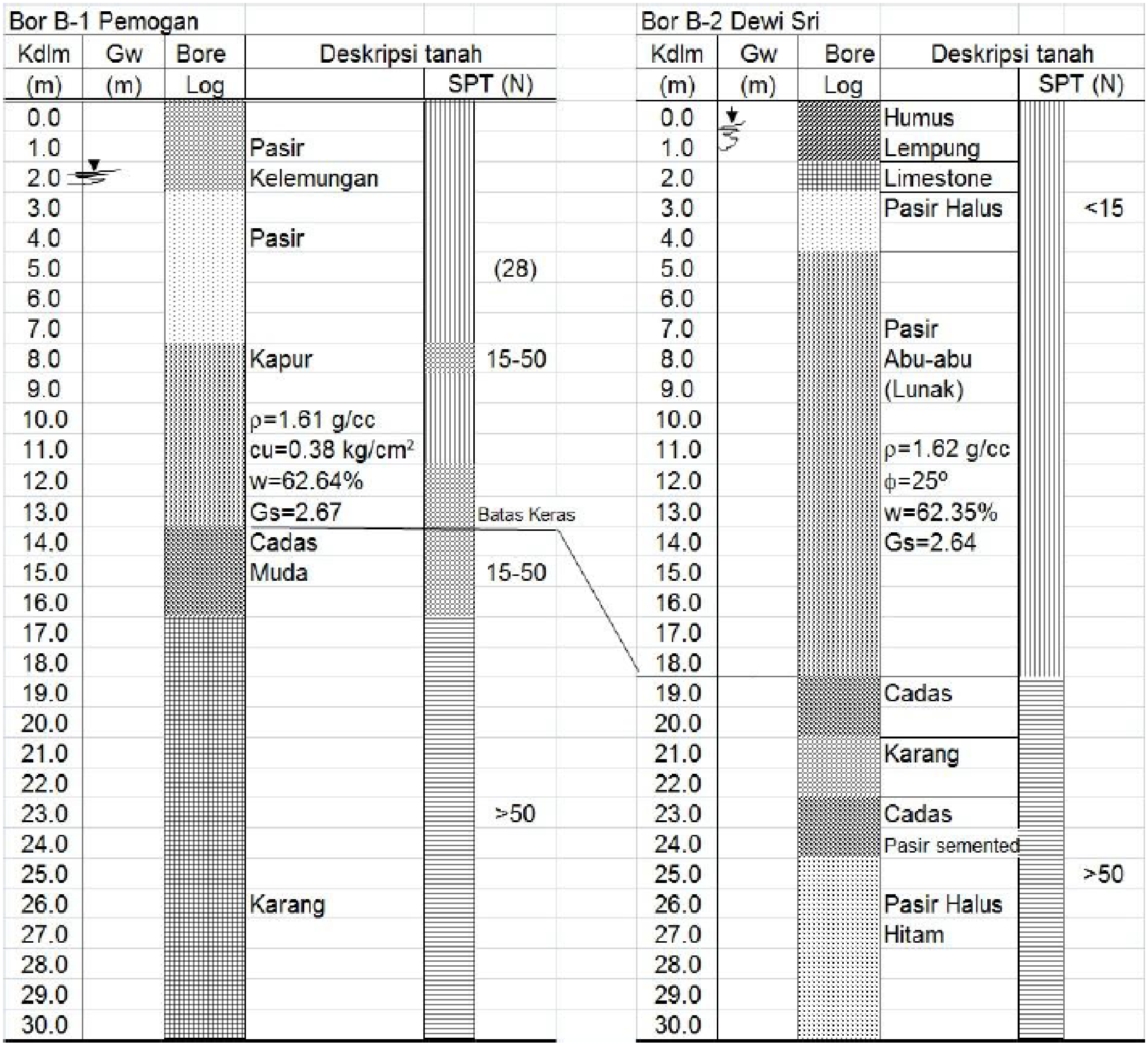


Bor Pantai Bingin B-4

Table 7 shows soil investigation in Pamogan and Dewi Sri area. Soil in this area is dominated by sand in the upper layers. The N values in the upper layers to a depth of 16 meter in Pemogan shows N-SPT value varies between N=15 to 50. In the botom layer from depth of 16 to 30 meter consisting of coral showing N>50. Average N =15 to 50 in this pamogan area. According to Tabel 1 site classification is SD (medium soil).

The values of N in Dewi Sri area N<15 from ground surface to depth of 18 meter as shown in Table 7. However, below this layer from depth of 18 meter to end of bore hole at 30 meter shows N>50. It migh be counted to site classification of SE (soft soil) in Dewi Sri area.

Table 7. Results of drill logs and SPT



Pemogan Bore Log B-1	Dewi Sri Bore Log B-2

Acording to SNI 1726-2019 maximum earthquake parameters in Bali, Indonesia S_s=0.9 to 1.0g, S_1=0.3 to 0.4g, PGA=0.4 to 0.5g, with risk coefficient C_{RS}=1 to1.05 for periode respond spectral 0.2 second and C_{R1}=0.95 to 1 for periode respond spectral 1 second. These parameters based on respond spectra of maximum earthquake considered risk-targeted (MCE_R) Indonesian region for 0.2-second response spectrum (critical attenuation 5%) as basic reference. Respond spectra for others site classification as mention in Table 1 should follow the guide line provided by SNI 1726: 2019.

Based on the soil site class test results that have been obtained, the design spectral response based on SNI 726;2019 can be determined as follows.

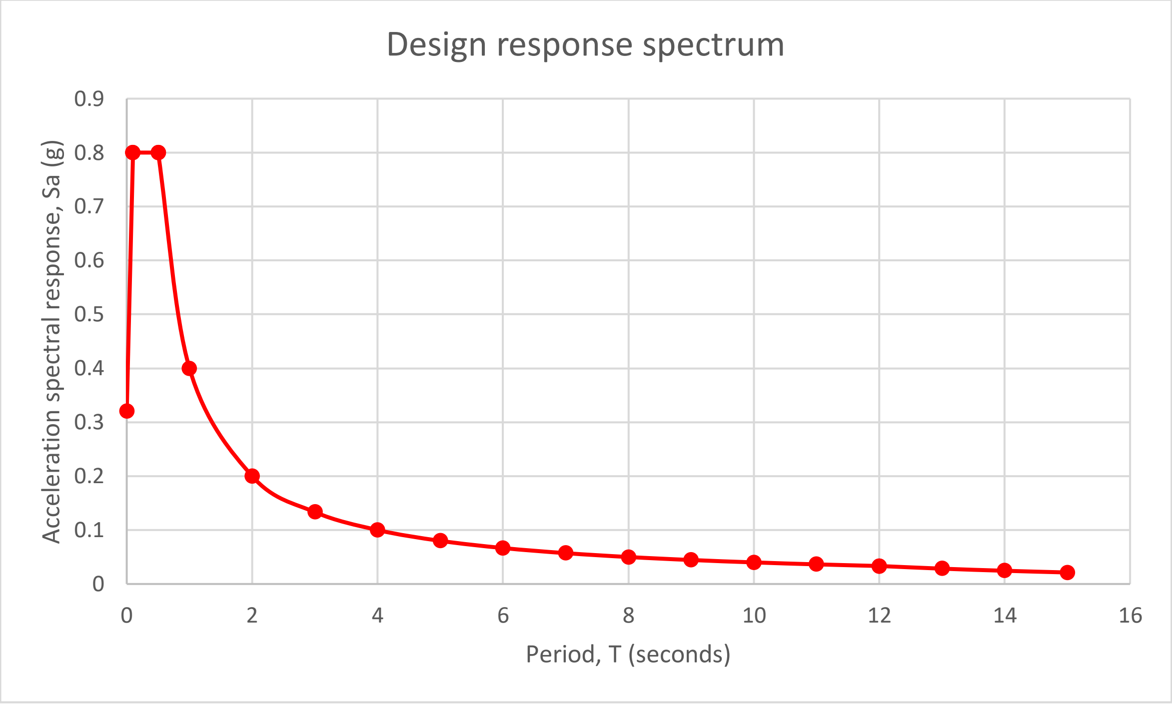


Figure 7. Design response spektrum for SC (hard soil, very solid and soft rock)

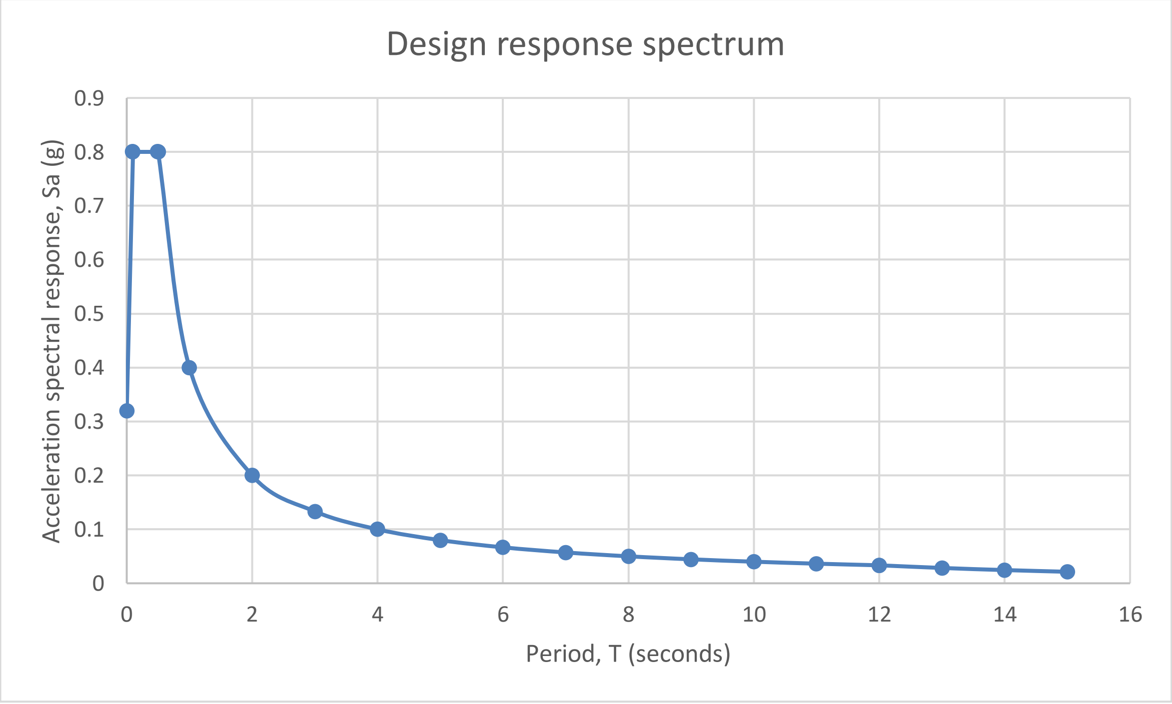


Figure 8. Design response spektrum for SD (medium soil)

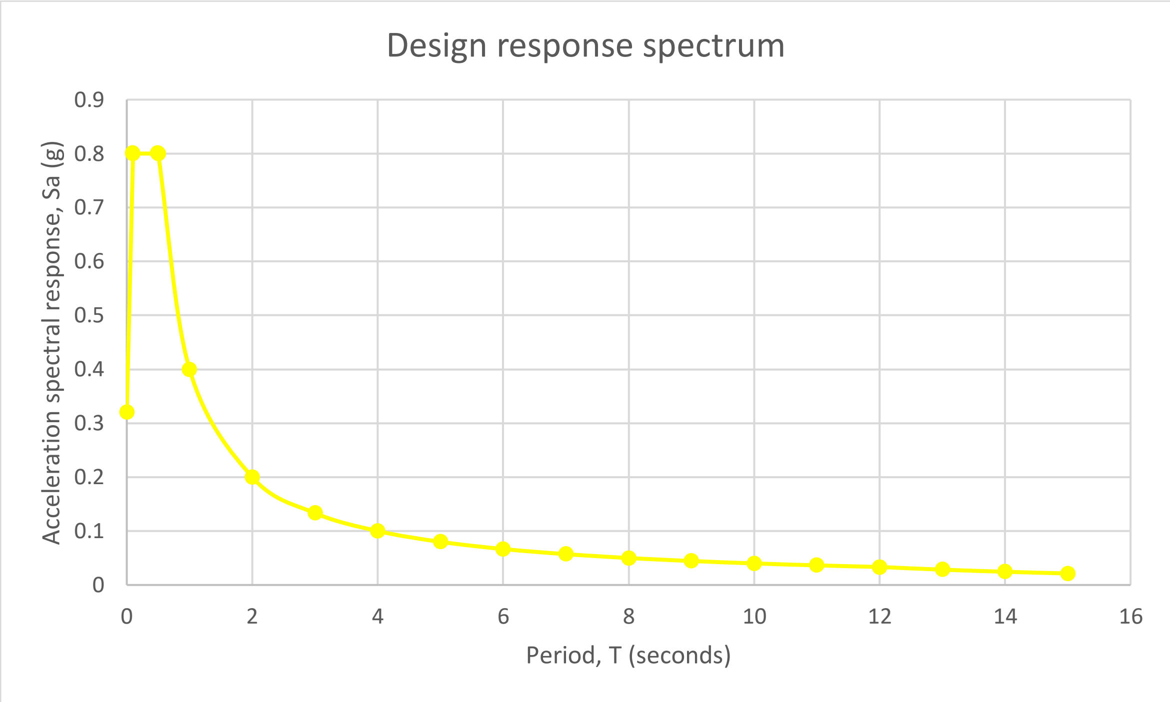


Figure 9. Design response spektrum for SE (soft soil)

4. CONCLUSIONS

Soil investigation taken from four sites in this Jimbaran and Kuta area reveal soil site classification as SC (hard soil, very solid and soft rock) and SD (medium soil) and SE (soft soil). Maximum earthquake parameters in Bali, Indonesia Ss=0.9 to 1.0g, S1=0.3 to 0.4g, PGA=0.4 to 0.5g, with risk coefficient CRS=1 to1.05 for periode respond spectral 0.2 second and CR1=0.95 to 1 for periode respond spectral 1 second.

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