Geotechnical Design Parameters for p-y Curves

Bridge Design (BD) may use p-y curves, or p-y springs, to analyze and design laterally loaded piles from earth pressures or earthquakes. These analyses incorporate the principes of soil-structure interaction (SSI), in which the response of the pile foundation and the surrounding geomaterial are evaluated as an integrated system. This module presents the procedures to develop and report the geotechnical information required by BD to generate p-y curves to perform their typical SSI analysis.

At a pile location, Geotechnical Services (GS) develops the representative geologic profile, selects the applicable p-y curve model for each soil/rock layer, and provides geotechnical design parameters required for the selected p-y curve model to BD. These parameters characterize the interaction between the pile and surrounding ground and are used by BD to model the lateral resistance and deformation behavior of the soil-structure system.

GS provides this information for all bridge supports founded on piles, except when the support is founded on a pile group with five or more piles in Soil Profile Class S1, as defined in the Seismic Design Criteria (SDC). This information is also provided for special design earth retaining systems supported on piles as requested by BD.

The following foundation design information is needed from BD to determine the approximate depth below the finish ground surface for which geotechnical design parameters for p-y curves are required:

- pile diameter
- pile cut-off elevation
- finish ground surface elevation at the support location

The p-y curve models presented in this module are developed for static loading and are also considered applicable to certain types of seismic lateral load analysis for piles founded in soil that do not experience liquefaction during earthquakes. Simplified p-y curves models are presented for liquefied soils. Modifications that may be considered for more complex structural analysis are beyond the scope of this module.

P-Y Curve Models

BD typically uses the computer program, LPILE (Ensoft) to perform the lateral analysis of deep foundations. Table 1 presents the p-y curve models recommended to be used in the program. GS is responsible for assigning a curve model(s) to each geologic layer in the geologic profile.

Table 1: p-y Curve Models

Geomaterial Type / Condition	Model No.	Model Identification / Description
Clay (Cohesive Soil)	1	Soft Clay Model (Matlock,1970)
	2	API Soft Clay Model with User Defined J (API, 2011)
	3	Stiff Clay with Free Water Model (Reese, et al., 1975)
	4	Stiff Clay Without Free Water Model (Reese and Welch, 1975)
	5	Modified Stiff Clay Without Free Water Model (Ensoft LPILE 2022)
Sand (Cohesionless Soil)	6	Reese Sand Model (Reese et al., 1974)
	7	API Sand Model (API, 2011)
Cemented (c-ϕ soil)	8	Cemented Soil Model (Ensoft LPILE, 2022)
Liquefied Soils	9	Equivalent Soft Clay Model (Wang and Reese, 1998)
	10	p-Multiplier Model (modified after Brandenberg et al. 2007)
Rock	11	Massive Rock Model (Liang et al., 2009)

Refer to the *Caltrans Soil and Rock Logging, Classification, and Presentation Manual* for guidance for geomaterial classification. Cohesive and cohesionless intermediate geomaterials (IGMs) where geotechnical characteristics and engineering behavior are transitional between hard or strong soil, and soft to very soft or weak rock, may be considered a clay (cohesive soil) or sand (cohesionless soil) for the development of p-y curves, respectively. Refer to the *Cast-in-Drilled-Hole* module for guidance for geomaterial design categories.

The following sections provide investigative and analytical best practices for selection of the appropriate curve model and the applicable geotechnical design parameters for the selected p-y curve model from Table 1.

Investigation

The number of exploration points and depths of explorations performed for axial load design should be adequate for lateral pile analysis (see *Driven Pile Foundation* module and *CIDH Pile Foundation* module). Obtaining targeted samples and/or more laboratory testing should be considered to capture changes in subsurface conditions in the upper layers of soil, particularly for foundation systems consisting of fewer than five piles per pile cap or a single column shaft (i.e., non-redundant).

The geomaterial properties/parameters required for p-y curve models and the determination of the applicable design groundwater elevation at appropriate limit states are typically evaluated as part of the routine site investigation and site characterization. Additional testing requirements are presented in the Model-specific sections below.

Determining Geotechnical Design Parameters and Selection of p-y Curve Models

Geotechnical Design Parameters for Clay

Table 1 presents five clay curve models applicable for cohesive soils. Table 2 presents the required parameters for each of the five clay curve models. Cohesive soils include clays and high-plasticity silts (classified as CL, CH, and MH), coarse-grained clayey sand (SC) and gravel (GC) with relatively high cohesion and very low friction angle under short-term static loading conditions, and cohesive IGM (e.g., shale, mudstone, over-consolidated clay).

Table 2: Geotechnical Parameters for Clay p-y Curve Models

Model No. (see Table 1)	Effective Unit Weight (γ')	Undrained Shear Strength (S_u)	Dimensionless Parameter J	Initial soil-pile lateral reaction modulus factor (k_i)	Strain Factor (ε_{50})
1	x	x			x
2	x	x	x		x
3	x	x		x	x
4	x	x			x
5	x	x		x	x

<u>Effective Unit Weight (γ')</u>

Determine the unit weight of clay per ASTM D7263, *Standard Test Methods for Laboratory Determination of Density and Unit Weight of Soil Specimens*. In the absence of measured values, the unit weight may be estimated based on SPT blow counts (see *Soil Correlations* module).

<u>Undrained Shear Strength (S_u)</u>

Determine S_u by laboratory testing of undisturbed samples and select the appropriate p-y curve model from Table 3. Unconsolidated undrained (UU) or consolidated undrained (CU) triaxial testing should be performed. Alternatively, in-situ measurements using calibrated cone penetration testing (CPT) may be used to determine S_u.

Pocket penetrometer or torvane measurements are considered unreliable and should not be used to determine final design values for S_u unless calibrated to laboratory results.

Table 3: Selection of Clay p-y Curve Models

Undrained Shear Strength, S_u (psf)	Access to Free Water	p-y Curve Models[a][b]
< 500	N/A	Soft Clay (1)
500 ≤ S_u < 1,000		Soft Clay (1 or 2)
1,000 – 8,000	Yes	Stiff Clay with Free Water (3)
	No	Stiff Clay without Free Water (4 or 5)

(a) Numbers in parentheses represent the p-y curve model numbers in Table 1.
(b) Cohesive IGM may use Stiff Clay models.

Engineering judgement is required for selecting the p-y curve Model 3 “Stiff Clay with Free Water”. This option should be selected only when the free water present at a support location has access to the clay layer under consideration. Such accessibility will depend on many factors, including the depth and position of the clay layer relative to the free water source and other soil layers, source of the free water, and presence of cracks or fissures within the clay layer. Sources of free water include surface water and/or groundwater within overlying free draining granular soil layer(s). Deeper clay layers may not have access to the free water source, or such access may have little significance on the lateral response and can be ignored.

Model No. 5 offers an option to incorporate the parameter k_i but requires a site-specific lateral load test. This option is typically reserved for a project with numerous piles where BD and GS agree that use of the model and lateral load test may optimize the design.

Dimensionless Empirical Coefficient J

When this model is selected (Model 2), the recommended J value is 0.25 for medium stiff clay and 0.5 for soft clay.

Initial Soil-Pile Lateral Reaction Modulus Factor (k_i)

The modulus factor k_i is required for two of the p-y curve models for stiff to hard clay soils (Models 3 and 5). It depends on the soil initial modulus (E_{si}) and the Poisson’s Ratio (ν) as determined from triaxial compression tests, the pile characteristics, loading type, and the soil-pile interactions.

Determine the design k_i value using the formula below, which is based on an empirical correlation with S_u, adopted from limited number of field lateral load tests on piles of diameter 24 inches or less.

$$k_i(\text{pci}) = 0.3333\, S_u(\text{psf}) \quad \text{(for 1000 psf < } S_u \text{ < 8000 psf)}$$

Strain Factor (ε_{50})

Determine the strain factor (ε_{50}) using the stress-strain curve from laboratory triaxial UU or CU testing. The strain factor ε_{50} is taken as equal to the axial compressive strain corresponding to a deviator stress equal to 50% of the deviator stress at failure or the maximum deviator stress.

If stress-strain curves from UU or CU tests are not available, use the empirical correlations presented in Table 4 to estimate the strain factor (ε_{50}). These empirical strain factors are subject to similar limitations as the factor k_i discussed above.

Table 4: Values for ε_{50} for Clay Soils (LPILE Technical Manual)

p-y Curve Model (Model No. in Table 1)	Average Undrained Shear Strength, S_u (psf)	Strain Factor[a] (ε_{50})
Soft Clay (1, 2)	250 - 500	0.02
	500 - 1,000	0.01
Stiff Clay (3, 4, and 5)	1,000 - 2,000	0.007
	2,000 - 4,000	0.005
	4,000 – 8,000	0.004

(a) The empirical strain factor (ε_{50}) may be estimated as $\varepsilon_{50} = 0.48\ S_u(\text{psf})^{-0.564}$
(b) For Cohesive IGM, the above equation may be used.

Geotechnical Design Parameters for Sand

The Reese Sand and API Sand models (Models 6 and 7, respectively) are included in Table 1 for sand with no cementation. Both models are applicable for cohesionless soils above or below the groundwater table. Cohesionless soil includes gravel and sand (classified as GW, GP, GM, SW, SP, SM), non- plastic silts (ML), coarse-grained clayey sand (SC) and clayey gravel (GC) soils with relatively low cohesion, and cohesionless IGM (e.g., poorly indurated sandstone, siltstone, tuff, granular residual soil, granular till).

The p-y curves from the two models differ in the way the shape of the p-y curve is defined. The shape of the p-y curve based on the API model (Model No. 7) is hyperbolic and, thus more closely represents the most common shape of the soil stress-strain curves.

The following soil parameters are required for the Reese Sand Model and API Sand Model.

- Effective unit weight (γ')
- Effective friction angle (ϕ'), and
- Initial soil-pile lateral reaction modulus factor (k_i)

Effective Unit Weight (γ')

Use empirical correlations with SPT blow counts to determine the unit weight (see *Soil Correlations* module). The term "sand" as defined for p-y curves include soil which may contain fines, and it may be possible to obtain relatively undisturbed samples to determine the unit weight in the laboratory.

Effective Friction Angle (ϕ')

Determine the effective friction angle (ϕ') using established empirical correlations with normalized SPT blow count $(N_1)_{60}$ (see *Soil Correlations* module), or normalized CPT tip resistance (Q_{tn}) (see *Cone Penetration Test Design Parameter Correlations* module).

Initial Soil-Pile Lateral Reaction Modulus Factor (k_i) – Reese and API Sand Model

Determine the parameter k_i for the API Sand (API, 2011) p-y curve model using empirical correlations of k_i with friction angle as shown in Figure 1.

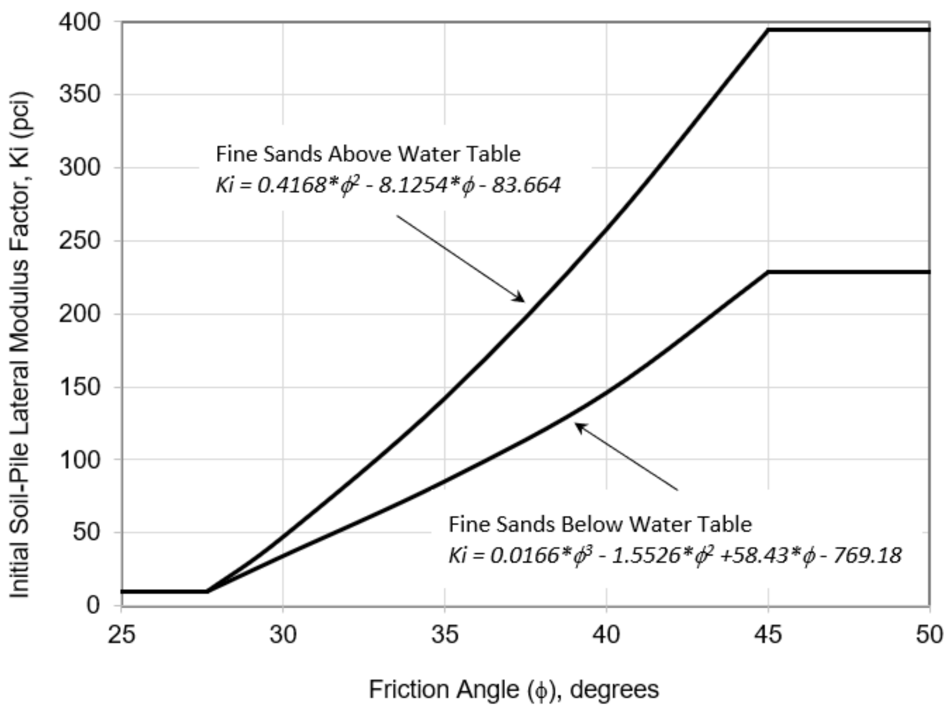

Figure 1: Parameter k_i versus Friction Angle

Geotechnical Design Parameters for Cemented (c-ϕ) Soils

The cemented soil model (Model No. 8) is included in Table 1 for soils exhibiting both cohesive and frictional strength components, commonly referred to as c–ϕ soils. Applicable geomaterial types for this model include partially cemented granular soils, low-plasticity silts, and cohesionless IGM including decomposed rock. These materials show a stiffer response than uncemented soil but can lose strength rapidly when cementation breaks down or when subjected to large-strain cyclic loading.

The following soil parameters are required for this model.

- Effective unit weight (γ')
- Effective Cohesion (c'),
- Effective Friction angle (ϕ'),
- Strain factor (ε_{50}), and
- Initial soil-pile lateral reaction modulus factor (k_i)

Effective Unit Weight (γ')

Determine the effective unit weight using the methods presented for clay and sand models (see previous sections).

Effective Cohesion (c') and Effective Friction Angle (ϕ')

Determine effective cohesion (c') and effective friction angle (ϕ'), using direct shear tests, CU triaxial tests, or consolidated drained (CD) triaxial tests.

Strain Factor (ε_{50})

Determine the strain factor (ε_{50}) from stress-strain curves from shear tests for the parameters c' and ϕ'. In the absence of test data, empirical correlations presented in Table 4 can be used to determine ε_{50} by replacing S_u with c'.

Initial Soil-Pile Lateral Reaction Modulus Factor (k_i)

The initial soil-pile lateral reaction modulus factor (k_i) of a cemented c-ϕ soil consists of a frictional component (k_ϕ) and a cohesion component (k_c).

$$k_i\ (\text{pci}) = k_c(\text{pci}) + k_\phi(\text{pci})$$

The parameter k_c may be determined from the following equation, presented in the Clay section above, using effective cohesion (c') for S_u:

$$k_c\ (\text{pci}) = 0.3333\, S_u(\text{psf}) \quad \text{(for 1000 psf < } S_u \text{ < 8000 psf)}$$

The parameter k_ϕ may be determined from the correlation for the API Sand model in Figure 1.

The default option available in LPILE to determine k_i for cemented soil may not be reliable and should not be used.

Geotechnical Parameters for Liquefied Soils

Table 1 includes two simplified p-y curve models (Models No. 9 and 10) for liquefied soils. Liquefied soils include saturated loose to medium dense cohesionless soil, including non-plastic to low-plasticity silts with PI<7 and evaluated as "liquefiable" per the *Liquefaction Evaluation* module.

The lateral response of piles in liquefied soils is highly uncertain due to the complex and rapidly changing soil behavior during and after liquefaction. Laboratory and field evidence indicate that both the shape of the p–y curve and the magnitude of the peak resistance are difficult to characterize reliably. Two simplified modeling approaches are available to represent the post-liquefaction soil behavior: the Equivalent Soft Clay Model and the p-Multiplier Model. Each approach incorporates different assumptions about the residual strength and stiffness of the liquefied soil. The following subsections outline the required parameters and procedures for each of the two models.

Equivalent Soft Clay Model (Wang and Reese, 1998)

This p-y curve model for liquefied soil is the same as the Soft Clay Models (Models No. 1 and 2). In this case, the liquefied soil is considered as an “equivalent soft clay” characterized by an undrained shear strength equal to the undrained residual shear strength (S_r) of the liquefied soil.

Wang and Reese (1998) proposed this model based on the reasoning that liquefied soil retain some undrained residual shear strength (S_r) that will provide some lateral resistance over a relatively large deformation. This model assumes that the liquefied soil experiences no dilation upon further deformation imposed by the laterally loaded pile.

The following geotechnical design parameters are required for the liquefied soil layers when using this model:

- Effective unit weight (γ')
- Undrained Residual Shear Strength (S_r)
- Strain factor (ε_{50})

<u>Effective Unit Weight (γ')</u>

Determine the effective unit weight using the methods presented for the sand models (see previous sections).

Undrained Residual Shear Strength of Liquefied Soils (S_r)

The following empirical correlation proposed by Kramer and Wang (2015) is recommended for the determination of the undrained residual shear strength S_r.

$$S_r(\text{psf}) = 2116\ \exp\left\{-8.444 + 0.109(N_1)_{60}\ + 5.379\left(\frac{\sigma'_{vo}}{2116}\right)^{0.1}\right\}$$

- $(N_1)_{60}$ = The corrected field measured SPT normalized to 60% hammer energy and to an initial effective overburden stress equal to one (1.0) atmospheric pressure (≅ 2116 psf). See Youd et al (2001) or the *Liquefaction Evaluation* module. Note, the SPT blow count is not corrected for fines content of the liquefied soil.
- σ'_{vo} = Initial in-situ effective overburden stress (psf) at the depth under consideration evaluated based on the ground surface, considering scour and groundwater elevations applicable to seismic design (Extreme Event Limit State I).

Strain factor (ε_{50})

Use the empirical correlations presented in Table 4 to estimate the strain factor (ε_{50}) and replace S_u with S_r. The equation below may be used.

$$\varepsilon_{50} = 0.48\ S_r(\text{psf})^{-0.564}$$

P-Multiplier Model

Based on this model the p-y curve for a liquefied soil is determined by reducing the static “backbone” p-y curve for the same soil in the non-liquefied condition, all other p-y curve computation related factors and conditions remaining the same. The static backbone p-y curve for the non-liquefied condition is computed by using one of the static p-y curve models for sand, i.e., Reese Sand Model or API Sand Model (Model No. 6 or 7, respectively, in Table 1). The reductions are achieved by multiplying the discrete p-values computed for the static backbone p-y curve by a constant factor (m_p). The y-values do not change. Mathematically, the procedure can be expressed as follows:

$$p_{Liq} = m_p \times p_{static}$$

- p_{Liq} = p-value for the p-y curve for the liquefied soil
- m_p = p-multiplier for soil liquefaction effects (See Figure 2)
- p_{static} = p-value for the static backbone p-y curve

The p-multiplier (m_p) for liquefied soils is not related to, and should not be confused with, the p-multiplier (p_m) used to represent shadowing effects of adjacent piles in a pile group.

This model may be used for all liquefied soil irrespective of the initial state, i.e., $(N_1)_{60}$.

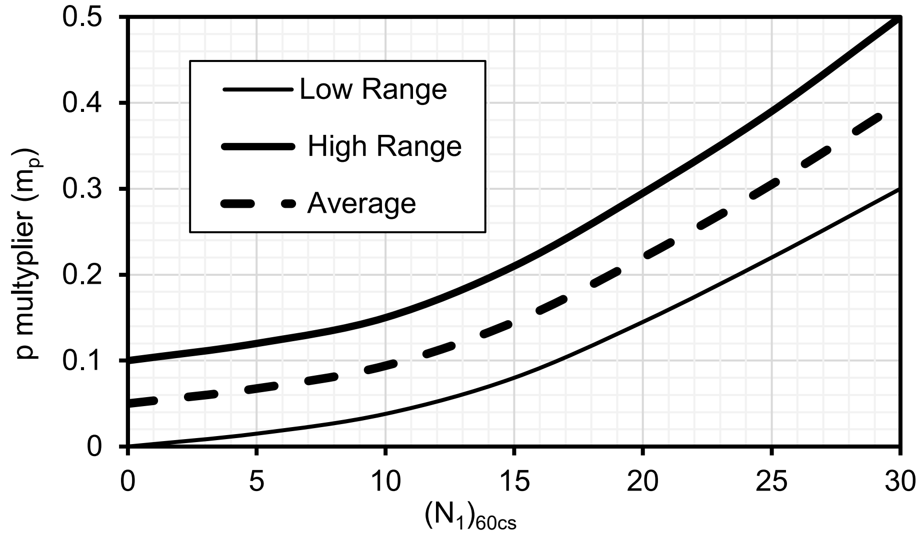

Figure 2: p-multipliers (m_p) for Liquefied Soil (Modified after Brandenberg et al., 2007)

The dashed line curve in Figure 2 represents the average value based on the low-and high-range of the p-multiplier (m_p) for a given $(N_1)_{60cs}$ after Brandenberg et al 2007. The dashed line is represented by the following equation:

$m_p = 3 \times 10^{-4}{(N_1)_{60CS}}^2 + 2.3 \times 10^{-3}(N_1)_{60CS} + 5 \times 10^{-2}$ with $(N_1)_{60CS} \leq 30$

Geotechnical design parameters required for a liquefied soil layer when using this model include:

- Geotechnical design parameters for the static backbone p-y curve model
- p-multiplier (m_p) for liquefied soil.

Geotechnical Design Parameters for Rock

Rock is identified based on the geologic origin. For geotechnical engineering analysis and design, rock consists of cohesive cemented geomaterial with a $q_u \geq 50$ tsf. Distinctions must be made between intact rock and rock mass properties. Secondary structures of rock masses such as those pertaining to joints, cracks, fractures, and inclusions play dominant role on the load supporting behaviors of foundation in rock, including laterally loaded piles.

For piles embedded in shallow rock (at or near the ground surface or cut-off elevations), pile lateral response, may control the pile design including the required minimum pile length. Lateral response of piles in rock can vary significantly. More detailed investigation and engineering judgment than soils are required to characterize the rock mass for lateral pile analysis.

For lateral analysis, a pile or deep foundation embedded in rock generally is associated with a Cast-in-Drilled Hole (CIDH) pile or drilled shaft.

Massive Rock (Liang et al., 2009)

Liang et al. (2009) developed a p-y curve model for rock mass based on full-scale lateral load tests and 3D finite element analysis. The input for rock mass requires parameters defined using the Hoek-Brown strength criterion. The following parameters are required:

- Effective Unit Weight (γ')
- Intact Rock Strength (σ_{ci}), or Uniaxial Compressive Strength
- Geological Strength Index (GSI)
- Hoek-Brown Material Index (m_i)
- Intact Rock Modulus or Young's Modulus (E_i or E_R)
- Rock Mass Modulus (E_m)
- Poisson's Ratio (ν_m)

AASHTO LRFD BDS Articles 10.4.6.4 and 10.4.6.5, and the sections below discuss development of the parameters for the rock p-y curve model.

<u>Effective Unit Weight (γ')</u>

The recommended method for determining the unit weight of rock is to use laboratory density testing per ASTM D7263 on rock core samples.

<u>Intact Rock Strength (σ_{ci})</u>

The rock core uniaxial compressive strength (q_u) is typically determined using results of unconfined compression tests on intact rock cores. The design strength is usually selected based on the lower-bound value and as a function of depth. The methods for the determination of σ_{ci} are the same as those for q_u for rock. However, it is important to recognize that unlike q_u, σ_{ci} does not vary with depth for a given rock mass layer within the design profile developed for p-y curves**.** For a given rock mass layer, only one σ_{ci} value is specified. The model uses this σ_{ci} with other parameters to calculate variations in the rock mass shear and compressive strengths with depth.

<u>Geologic Strength Index (GSI) for the Rock Mass</u>

The Geological Strength Index (GSI) is a dimensionless parameter that was developed to provide a means to correlate field and laboratory observations and measurements to the Hoek-Brown strength criterion. It depends on the rock mass structure and surface quality expressed as function of surface conditions. GSI values range from 0 (laminated /sheared/highly fractured, highly weathered rock) to 100 (intact/massive/widely spaced discontinuity, fresh (unweathered) rock).

GSI considers two fundamental characteristics of a rock mass: the "blockiness" and the condition of the discontinuities. In combination with the uniaxial (unconfined) compressive strength of intact rock and the state of in-situ stress, these factors control the mechanical behavior of a rock mass. Higher values of GSI represent more tightly interlocking blocks and tighter/rougher discontinuity surfaces, while lower values indicate more highly fractured rock with smoother conditions of the discontinuity surfaces.

GSI determination from core logs requires input by an experienced geologist. Figure 3 presents criteria to determine GSI.

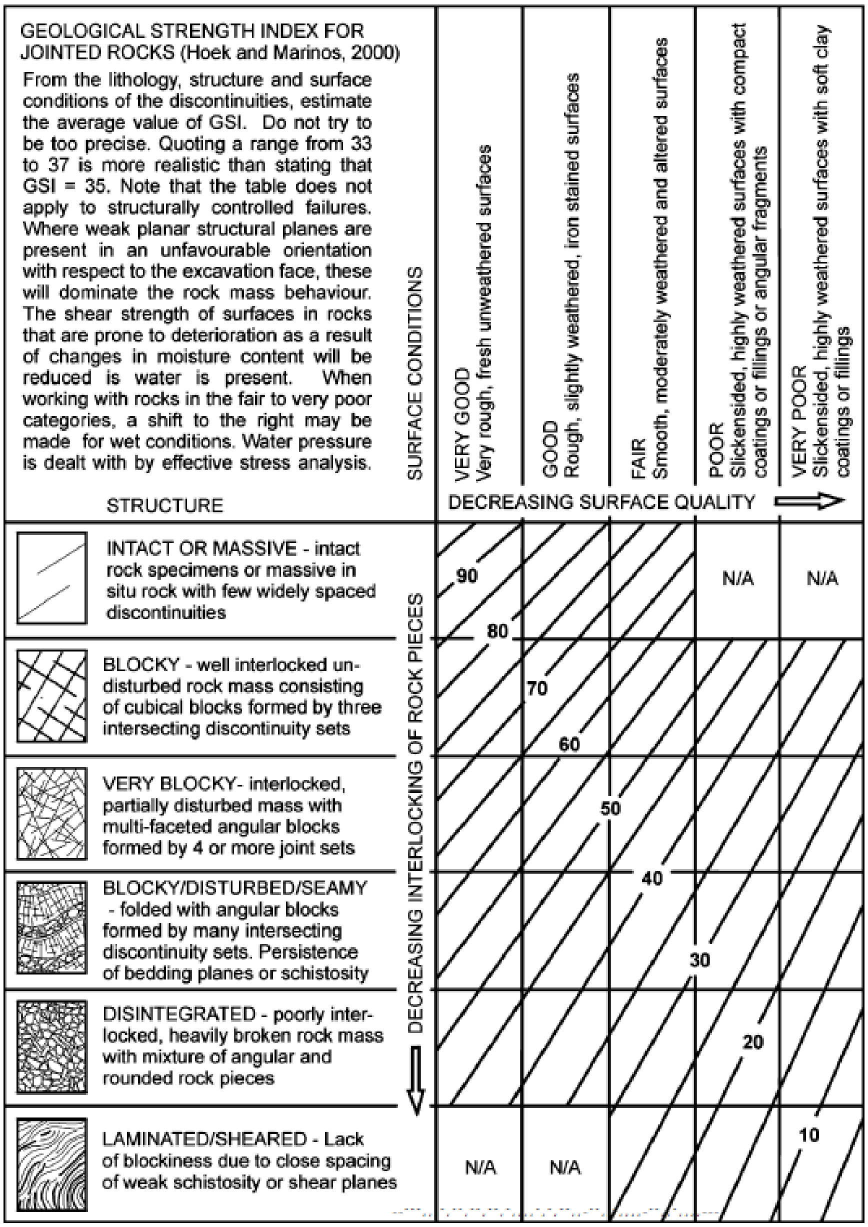

Figure 3: Chart for estimation of geological strength index (GSI) in rock (from Marinos et al., 2005)

Hoek Brown Material Index (m_i)

The Hoek-Brown Material Index (m_i) is a dimensionless parameter which depends on rock type and surface texture of intact rock cores, ranging from 2 (claystone) to 35 (granite). Values list on Table 5 of m_i can be determined with input by an experienced geologist.

Table 5: Hoek-Brown Material Index m_i for Intact Rock (AASHTO LRFD BDS 8th Ed.)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
IGNEOUS	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
IGNEOUS	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Intact Rock Modulus or Young's Modulus (E_i or E_R)

Intact Rock Modulus or Young's modulus (E_i or E_R as notated in AASHTO LRFD BDS) represents the stiffness of an intact rock core sample tested per ATSM D7012. It is derived from the slope of the measured stress-strain curve from the compression test. While different values of Young's modulus can be established, E_i herein corresponds to the slope of the line tangent to the stress-strain curve at a stress level equal to 50 percent of q_u. If the measured stress-strain curve is not available, E_i may be estimated based on measured q_u and Figure 4.

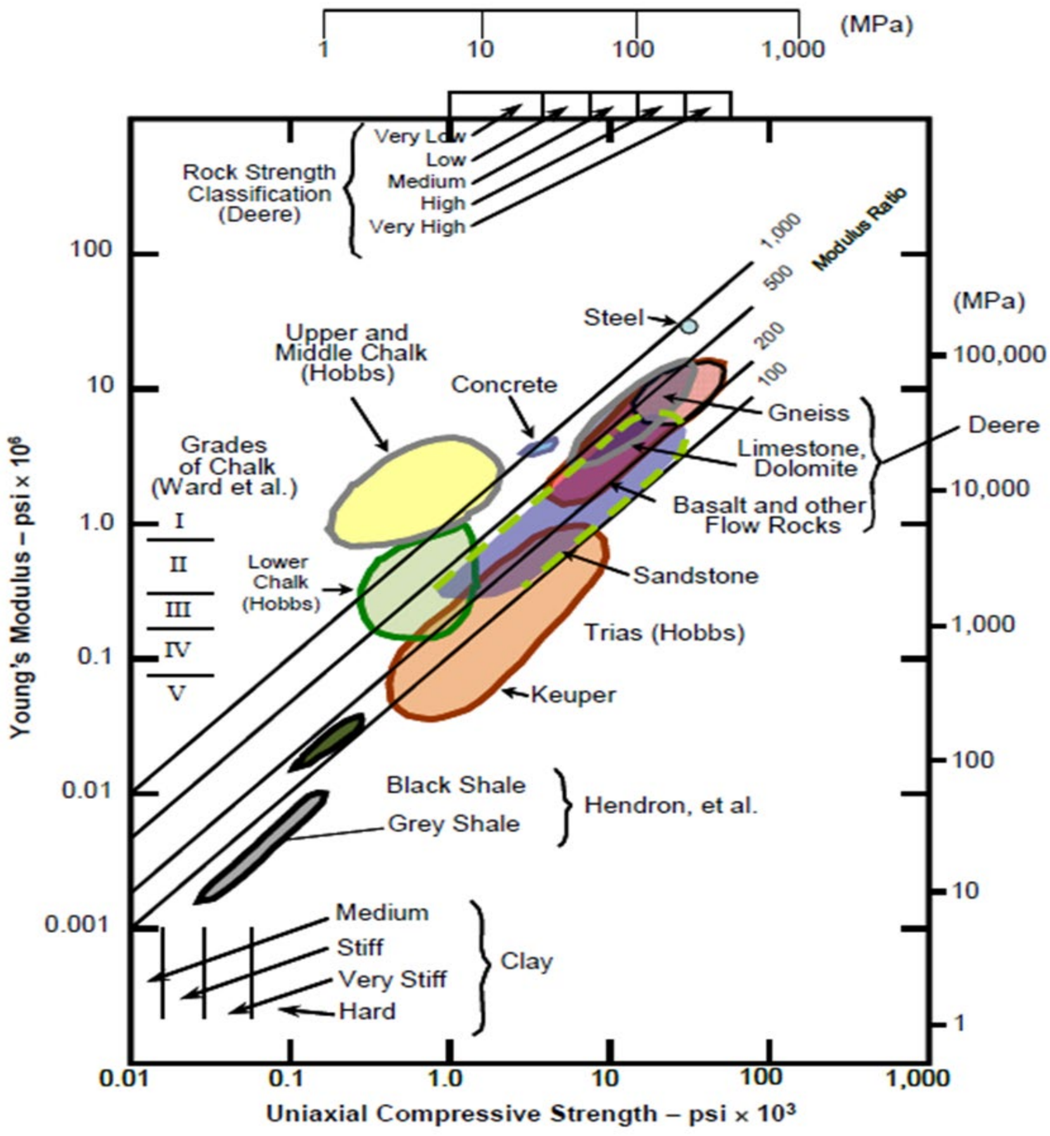

Figure 4: Engineering Properties for Intact Rocks (From Ensoft LPILE 2022)

Rock Mass Modulus (E_m)

The rock mass modulus (E_m) value is taken as the lesser of E_i or the modulus determined from the equations below (AASHTO Table 10.4.6.5-1 and Liang et.al 2009).

$$E_m(GPa) = \sqrt{\frac{q_u}{100}}10^{\frac{GSI-10}{40}}$$ for $q_u \leq 100$ MPa

$$E_m(GPa) = 10^{\frac{GSI-10}{40}}$$ for $q_u > 100$ MPa

$$E_m = E_i\left(\frac{E_m}{E_i}\right) = E_i\frac{e^{GSI/21.7}}{100}$$ reduction factor on intact modulus

For critical or large structures, E_m may be measured in the field using in-situ methods such as a pressuremeter test (ASTM D4719).

Poisson's Ratio (ν_m) for Rock Mass

The Poisson's Ratio (ν_m) for the rock mass is generally greater than the Poisson's Ratio (ν) for intact rock core and varies from about 0.1 to 0.4 (Zhang 2017b). Typical values for (ν_m) per Hoek (2001) are presented in Table 6.

Table 6: Typical Poisson's Ratio for Rock Mass (After Hoek, 2001)

Property	Symbol	Good Quality Rock Mass	Intermediate Rock Mass	Very Poor Quality Rock Mass
Intact Rock Strength	σ_{ci}	1500 tsf	800 tsf	200 tsf
Geological Strength Index	GSI	75	50	30
Poisson's Ratio	ν_m	0.2	0.25	0.3

Design Considerations

The following are design considerations when providing geotechnical recommendations to BD for lateral analysis of pile-supported structures:

- Recommended models and geotechnical design parameters for p-y curves can generally be provided from the finished ground surface to a depth of approximately 20 pile diameters for soft or loose material. Shallower depths may be appropriate for more competent material. The presence of weak material (e.g., liquefiable, very soft, very loose, or scourable soil) may warrant extending the reporting depth.
- Recommended geotechnical design parameters should be representative of the subsurface conditions and account for their variability and uncertainty (see *Geotechnical Design Parameters* module for guidance).
- For soft or stiff clays, sands, and IGM, more than one model may be applicable (see Table 1). One curve model should be assigned to each geologic layer in the geologic profile.
- For sites where liquefaction is anticipated:
  - More than one model may be applicable for the liquefiable layer(s) (see Table 1). One curve model should be assigned to each liquefiable layer in the geologic profile.
  - Subsurface conditions may require reductions in the lateral soil stiffness for the materials above and below liquefiable layer(s) (SDC Article 6.2.4.3). Refer to the *Lateral Soil Spring Reductions* module for guidance.

Adjustments to the ground surface elevation to account for scour at each limit state are typically performed by BD. Similarly, p-y multipliers used for pile group effects are typically evaluated by BD.

In some cases, BD may request that GS generate and provide p-y curves directly. When this occurs, GS should confirm the required output depths with BD. A vertical spacing equal to 5 feet or less than or equal to one pile diameter is typically appropriate. If scour is considered, separate sets of p-y curves should be developed for each applicable limit state. GS should confirm the design scour elevations with BD before adjusting the ground surface elevation for each limit state.

Since several sand and clay p-y curve models are available, and because of the inherent uncertainty in modeling liquefiable soil, it may be prudent to recommend multiple models for a geologic layer, particularly for critical or complex bridges. This allows BD to evaluate how the different models influence the predicted pile response and identify the controlling foundation design parameters. For example, the different liquefiable soil models may govern different portions of the p-y response (e.g., initial stiffness versus ultimate resistance), potentially affecting the foundation design. GS and BD should discuss whether multiple models should be considered for the project. Several iterations of structural analysis may be needed to evaluate the range of responses and achieve an appropriate level of conservatism in the final design.

Communication between the Geoprofessional and the Bridge Designer throughout the design process is therefore essential to ensure that assumed site conditions, selected p-y curve models, and developed geotechnical design parameters are understood and implemented in the lateral analysis as intended. Additional support from the Geoprofessional may also be needed for sensitivity analyses performed by BD for particular calculated pile responses, such as localized peak moments or shears.

Reporting

Present geotechnical recommendations in accordance with the *Foundation Reports for Bridges* or the *Foundation Reports for Earth Retaining Systems* module.

In the *Recommendations* section of the Preliminary Foundation Report and/or Foundation Report, include selected p-y curve models and geotechnical design parameters for each foundation location, as applicable for the materials encountered. The information presented in Tables A, B, and/or C at a minimum should be provided to BD. Provide recommendations in accordance with the following:

- Report the recommended geotechnical design parameters for the selected p-y curve models from the finished ground surface to a depth confirmed with BD.
- For geologic layers that can be analyzed using more than one p-y curve model (e.g., API or Reese sand), select one model and report the corresponding design parameters. If agreed upon with BD, several models may be provided. Report the model used by BD for the final pile design in the Foundation Report.
- If liquefiable layers are present, provide two sets of geotechnical design parameters for each soil layer predicted to liquefy during the design seismic event: one representing non-liquefied and one representing liquefied soil conditions.
- For liquefied soil conditions, select either the Equivalent Soft Clay (Matlock or Reese) or the p-multiplier (API or Reese) model and report the corresponding design parameters. If agreed upon with BD, both models may be provided. Report the model used by BD for the final pile design in the Foundation Report.
- Refer to the *Lateral Soil Spring Reductions* module for guidance on presenting soil profiles where the lateral soil stiffness of the soil above and below liquefiable soil layer(s) may require reduction.
- If requested by BD, provide the direct p-y curves for each output depth and for each applicable limit state confirmed with BD.

Document considerations and discussions with BD that influenced the selection of the p-y curve model(s) (e.g., models considered for sensitivity analyses and the basis for selecting the final model), as applicable.

Table A: Example Geotechnical Design Parameters for p-y Curves

Top Elevation (feet)	Bottom Elevation (feet)	p-y Curve Model	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Cohesion, c' (psf)	Friction Angle, ϕ' (degree)	Initial Modulus Factor, k_i (pci)	ε_{50}
75.0	65.0	API Sand	52.5	N/A	N/A	29	24	N/A
65.0	55.0	Stiff Clay with Free Water	47.5	2000	N/A	N/A	667	0.007
55.0	45.0	API Sand	50.0	N/A	N/A	32	55	N/A
45.0	35.0	Cemented c-ϕ Soil	62.5	N/A	200	38	187 (k_c=67, and k_ϕ=120)	N/A
35.0	20.0	Stiff Clay Without Free Water	67.5	8000	N/A	N/A	N/A	0.003
20.0	10.0	See Table B for the recommended parameters for the Liang et al. (2009) Rock Model						

- See Table C for geotechnical parameters for p-y curves for liquefied soils where present.
- Uppermost layer (highest elevation) is the finished grade elevation provided by Bridge Design
- Pile Cut-off Elevation = 70.0 feet (depth of 0 in LPILE)
- Groundwater Elevation = 75.0 feet
- Bridge Design to modify finished grade elevation for scour conditions, if applicable
- Bridge Design to apply p-multipliers for pile group effects, if applicable

Table B: Example Geotechnical Parameters for p-y Curves for Rock

Top Elevation (feet)	Bottom Elevation (feet)	p-y Curve Model	Effective Unit Weight, γ' (pcf)	Uniaxial Compressive Strength (psi)	Hoek-Brown Material Index, m_i	Poisson's Ratio	Geologic Strength Index, GSI	Intact Rock Modulus, E_i (psi)
20	10	Massive Rock	75	2900	18	0.25	55	1000000

- See Table A for entire soil profile
- Intact Rock Modulus = "Option 1" input in LPILE; Rock Mass Modulus may be provided as "Option 2" input
- Ground Surface Elevation = 75.0 feet
- Pile Cut-off Elevation = 70.0 feet (depth of 0 feet in LPILE)
- Groundwater Elevation = 75.0 feet
- Bridge Design to apply p-multipliers for pile group effects, if applicable

Table C: Example Geotechnical Design Parameters for Liquefied Soils

Top Elevation (feet)	Bottom Elevation (feet)	p-y Curve Model	Effective Unit Weight, γ' (pcf)	p-multiplier (m_p)	Residual Undrained Shear Strength, S_r (psf)	ε_{50}
75.0	65.0	API Sand (p-multiplier)	52.5	0.08	N/A	N/A
		Matlock Soft Clay (Equivalent Soft Clay)	52.5	N/A	75	0.04
55.0	45.0	API Sand (p-multiplier)	50.0	0.2	N/A	N/A
		Matlock Soft Clay (Equivalent Soft Clay)	50.0	N/A	480	0.02

- See Table A for entire soil profile
- Ground Surface Elevation = 75.0 feet
- Pile Cut-off Elevation = 70.0 feet (depth of 0 feet in LPILE)
- Groundwater Elevation = 75.0 feet
- P-multiplier does not include pile group effects; Bridge Design to apply additional p-multipliers for pile group effects, if applicable

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