NEW PERUVIAN REGULATION FOR SEISMIC DESIGN OF UNDERGROUND STRUCTURES

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Abstract

This paper describes a proposal for the new Standard for earthquake-resistant design of underground structures for road and railway infrastructure in Peru. The considered structures are: road and railway tunnels, by-passes, and, mainly, metro and railway stations. Other buried structures not corresponding to road and railway infrastructure (underpasses for pedestrians, foundations, building basements, mining infrastructure, industrial facilities, pipes, culverts, sewers, rainwater storage tanks, sewage treatment stations, etc.) are not contemplated.

This Norm will incorporate the best of international documents (ISO [1]) and regulations of advanced countries (China [2], Japan [3], New Zealand [4], European Union [5], USA [6]), but has a clear Peruvian vocation; this means to be harmonized with the legislation and construction practices of the country, and to fit the usual level of knowledge, expectations and mentality of its potential users. Of course, the Standard cannot contradict any Peruvian legal text. The Code will be developed to be easily updated in order to incorporate the latest research advances, to include the conclusions derived after observed seismic damages, and to benefit from the experience acquired by its application.

Many norms propose discrete classifications; then, small changes generate abrupt changes in the design parameters. This occurs in the soil classification into categories (A, B, C, D, etc.), and in the territory partition into seismic zones. To avoid this problem, continuous categories will be proposed instead: soil can be classified according to the shear wave velocity (or other equivalent parameters), and the site seismicity through the bedrock acceleration (PGA). Regarding this last issue, the Peruvian institutions (CISMID, IGP, SENCICO, among others) provide extensive information on the seismicity of each point inside the national territory.

The Design Life of the analyzed structures is stated as 100 years. The importance of the analyzed structures is organized in levels that express the severity of the failure consequence; three levels are considered (low, normal and high). This regulation lies in the context of the PBD (Performance-Based Design). In this sense, five Damage Levels (equivalent to Limit States or Performance Requirements) are considered; four of them are structural (correspond roughly, to FO, IO, LS and CP in the American documents) and the fifth one is related to functionality (refers especially to derailment). For important structures (high importance), resilience will be considered; it is understood as the capacity of the construction to be repaired and put back into operation within a reasonably short time. Each Damage Level and Importance Level is associated with a certain seismicity, quantified in terms of return period (probability of being exceeded during the construction Design Life). As well, the seismic action during the construction stage is defined.

The main analysis and design strategy will be displacement-based static equivalent formulations. The use of more sophisticated strategies (e.g. nonlinear dynamic analysis) will only be allowed if carried out properly. In any case, simplified analyses (approaches based on forces or displacements) will be required, and large reductions with respect to them will not be permitted.

Keywords: precast buildings; energy-based seismic design criteria; energy dissipators; parametric study.

1. Basis of design

This section discusses the conceptual principles that govern the seismic-resistant analysis and design of underground structures.

1.1. General strategy for earthquake-resistant design of underground structures

The basic approach is that the construction under consideration, during its design life, should satisfy certain performance requirements for seismic intensities that are established according to its importance. The analysis formulations are equivalent static simplified methods based on forces or displacements, or different types of non-linear dynamic analyses taking into account the soil-structure interaction (SSI) [7]. The results of advanced methods (nonlinear dynamic analysis) will have to be compared with more simplified procedures, and large differences between them will not be accepted.

1.2. Design life

In general, given the difficulty, cost and importance of the underground constructions, **100 years** is proposed as their default design life. This value is used as a reference for all aspects of design, in particular the seismic action selection. The design life must be established (and can be modified) by the infrastructure owner, although respecting the minimum of 100 years proposed. In some particular cases, shorter terms may be considered, especially in provisional constructions.

1.3. Importance

The importance of a certain underground construction is as in [8], classifying them as low (I1), high (I2) and critical (I3); these categories correspond roughly to "*other bridges*", "*essential bridges*", and "*critical bridges*", respectively. The low importance will be applied exclusively to structures that do not pose any risk to infrastructure, systems or people. The high and critical importance correspond to tunnels and containment structures that "are not" and "are" crucial in case of seismic action, respectively. Deep excavations will be considered as I3; The depth from which they will be considered deep will be defined based on the soil characteristics. The assignment of importance must be made by the construction owner based on the severity of the consequences in case of lack of operability of the construction.

1.4. Performance requirements, damage levels and limit states

This proposal is framed in the context of "*Performance-Based Design*". This design philosophy basically consists in establishing a number of construction performance requirements based on its importance and the severity of the considered seismic action. These requirements are expressed conceptually, to be understandable to the non-specialized public. Each performance requirement is associated with a damage level; it can be classified into structural or functional. Five performance requirements are considered; four of them are related to structural or non-structural damage and the other one refers specifically to damage that affects the functionality. The first four performance requirements (and associated structural damage levels) are (in increasing order of affectation): optimal performance (negligible damage), good performance (limited damage), safety performance (significant damage) and no collapse performance (generalized damage). The fifth binomial of desired performance and damage level is known as functional. The desired performance requirements are conceptually defined next:

- **Optimum performance**. The construction undergoes only minimal effects, giving no sense of danger; can, therefore, be used immediately.
- **Good performance**. The construction undergoes moderate effects, being reasonably easy and economical to repair.
- **Safety performance**. The construction undergoes significant damage but remains safe. In the case of railway tunnels, it includes absence of derailment.
- **No collapse performance**. The construction undergoes very important effects, but it can still be safely evacuated.
- **Functional resilience**. The construction can recover its operation in a reasonably short term (a few days).

Noticeably, these returns are expressed only in a positive way, since they correspond to the objectives to be achieved. The corresponding damage levels are defined, in an unquantified manner, next:

- **Negligible damage**. The structure has only slight damage and can be clearly economically repaired, allowing continuous operation.
- **Limited damage**. The structure has only moderate damage and still can be economically repaired. Permanent lateral displacements (drifts) are negligible, the ability to resist new earthquakes is intact, and the structural elements experience only a slight stiffness reduction. Non-structural elements can also be economically repaired.
- **Significant damage**. The structure has significant damage, perhaps with moderate permanent drifts, but retains a good part of its resistant capacity. Non-structural elements are damaged but hold their integrity.
- **Generalized damage**. The structure has serious damage but still maintains some resistant capacity. Most non-structural elements are heavily damaged or destroyed.
- **Functional damage**. The functionality may be seriously interrupted but can be restored quickly.

Limit states are defined for each performance requirement and damage level; they are quantified in terms of engineering parameters. For the performance requirements and structural damage levels, this parameter is especially the deformation (strain) of the tensioned steel. The negligible and limited damage levels can be interpreted as serviceability limit states, and the significant and generalized damage levels as ultimate limit states. For cut-and-cover tunnels, the structure behaves transversely basically like a frame; the bounds that define the different damage levels will be established based on moment-rotation (or moment-curvature) laws in the sections in which the formation of plastic hinges is expected (usually at the bars ends); obviously, the influence of the axial force will be taken into account. To ensure a ductile failure mode, shear failure in the plastic hinges will be prevented by local equilibrium capacity conditions; that is, it will be verified that the flexural failure (either simple or composed) precedes the shear failure. The bounds corresponding to the desired performance and level of functional damage will also be defined.

1.5. Design levels

In the American regulations, three design levels (structural detailing) are considered: Ordinary, Intermediate and Special; they provide different ductility. The structures object of this Standard shall be designed as Special.

1.6. Seismic actions for each damage level and importance

Each damage level and importance is associated with a certain seismicity level, quantified in terms of return period; the probability of exceedance in 100 years (selected design life) is also indicated.

Table 1 – Return period (years) and probability of exceedance in 100 years (%) of the seismic actions

Performance requirement	**Importance**		
	I1	**I2**	**I3**
PR1	60 (80%)	110 (60%)	195 (40%)
PR2	110 (60%)	195 (40%)	450 (20%)
PR3	195 (40%)	450 (20%)	950 (10%)
PR4	450 (20%)	**950 (10%)**	2450 (4%)
PRO	195 (40%)	450 (20%)	950 (10%)

In Table 1, the PR1, PR2, PR3, PR4 and PRO performance requirements correspond to Optimal, Good, Safety, no Collapse and Functional Resilience, respectively. The return periods have been selected from the value for PR4 performance and I2 importance (950 years, highlighted in bold), since this period coincides approximately with the one indicated in [8] (1000 years or 7% of probability of being exceeded in 75 years).

The number of performance requirements is the one usually indicated in the most advanced codes (American and European, both having been revised very recently). The consideration of five levels allows information on the construction behavior for earthquakes of different seismicity levels. It is noteworthy that, although this extensive information cannot be totally "free" (in terms of cost, effort and time), the increase in design complexity and construction cost are quite moderate. Both issues are discussed next.

- **Design vs. number of limit states**. Regarding the design, the number of verifications to be carried out (five) does not represent any significant increase in the difficulty and extension of the design process; on the other hand, it is quite simple to design structures that satisfy all these demands, since they are "scaled" in a natural way (the more severe the seismic action, the more accepted damage). In relation to this last statement, the usual process is to separate the verification of the ultimate and service limit states. In the ultimate limit states, the structure is designed to satisfy the safety performance, and subsequently the verification of the no collapse performance is guaranteed by providing sufficient ductility; this is usually achieved with widely known structural detailed criteria (consistent with the provisions in subsection 1.5). The serviceability limit states are subsequently checked, paying attention not only to the structure itself but also to the non-structural elements. Finally, the functional limit state is verified. In summary, not only is it not difficult to design constructions that meet the five aforementioned performance objectives, but what is actually complicated is to find examples of constructions meeting only some of these requirements.
- **Construction vs. number of limit states**. The number of limit states (the number of requirements to be met) might generate a slight increase in the design cost, but is not expected that the construction cost will rise too high. In any case, this possible moderate increase in the initial cost would be more than compensated by reducing the repair needs after a strong earthquake.

Another significant aspect of Table 1 is the seismicity associated with each performance requirement and importance. It seems clear that these levels of seismicity will not significantly affect the difficulty and duration of the design process but might clearly influence the cost (and perhaps the difficulty of execution) of the structure; obviously, lower levels of seismicity involve lower cost, and vice versa. It should be taken into account that it does not seem possible to significantly reduce the prescribed seismicity levels without contradicting [8]. Finally, the required performance for the structural elements that are in contact with the soil should be higher, because their reparability is more difficult; this consideration applies also for upper slabs that support traffic loads.

1.7. Response modification factor

The Peruvian standard [9] considers a "*Basic Reduction Coefficient*" (R_0) that affects the seismic actions. In the structures considered by this Standard, no such coefficient is used, since the accepted damage is that specified in subsection 1.5.

1.8. Load combinations

The structural designs will be carried out as the American version of the LRFD ("*Load and Resistance Factor Design*"). In this strategy, in order to achieve a uniform level of safety for all actions, they are affected by safety coefficients (> 1, unless their effect is favorable) that depend on their degree of variability. Also, the local (sectional) resistance is reduced by a coefficient ϕ that depends on the internal force considered. For the seismic verifications, the safety coefficients shall be taken equal to those considered by the most recent American documents; in general, dead and live will loads be multiplied by 1.2 and 1.6, respectively. Dead and live loads will be defined as [10]. The values of the ϕ coefficient will follow [11] and [12]. The seismic action, given it is accidental, will be taken with its characteristic value and will be combined with dead loads (also multiplied by 1) and with the percentage of live loads that are to be expected during the seismic excitation. In railway stations, it is initially proposed that the load due to the users (people) be multiplied by 0.5; under other conditions of lower intensity of use, the live load will not be taken into account. In summary, for seismic actions, the combination to consider is $D + \psi L + E$, where D, L and E represent dead and live loads and seismic actions, and ψ is a combination coefficient that depends on the type of live load, being $\psi \leq 1$.

1.9. Construction stage

Conditions during construction may be different than in the final situation; they can often be more demanding in some aspects. Given this reality, the conditions required at this stage will be defined. Since the consequences will affect the construction process more directly than the final functionality of the structure, the importance will not be distinguished. For simplicity (taking into account the brevity of the construction stage) different damage levels (performance requirements) will not be considered. For construction processes do not exceeding 2 years, 195-year return period seismic action will be taken; this corresponds to a 1% probability of being exceeded in 2 years. For longer construction processes, a special statistical study will be required.

2. Seismic action

This section discusses the level of seismic excitation for which the considered underground constructions should be designed. The seismic action to be considered in the engineering bedrock and on the soil surface for a return period of 450 years will be taken from the applicable Peruvian regulations, in particular [8-9].

2.1. Basic expression of the seismic action

Obviously, the design ground motion must be consistent with the current Peruvian design codes [8-9]; however, the comparison between both codes shows relevant discrepancies regarding the quantification of the seismic hazard in Peru. Moreover, the E.031 standard (currently under development) contains new proposals. Following common trends (also in Peru), the seismic action will be quantified preferably in terms of the 5% damped pseudo-acceleration response design spectrum (ordinarily known as response spectrum). The proposed spectrum will have four branches: (i) an initial linear growing branch (with exponent 1 for the period), (ii) a plateau (horizontal, with exponent 0 for the period), (iii) a descending branch (exponent –1), and (iv) a faster descending branch (exponent –2). This spectrum is characterized by the corner periods between such branches (T_0, T_P and T_L), and the spectral ordinates for zero period (structures with infinite stiffness), short periods (0.2 s) and intermediate periods (1 s). Noticeably, in [8] such parameters (F_{PGA}, F_a and F_v, respectively) refer to the values in the former American document [13], and have not been updated to the values in the new document [14]). Regarding these parameters, on the Sencico website there is an application delivering rock acceleration (type B) for any point of the national territory. The provided seismic hazard information includes: (i) Annual exceedance probability curves vs. spectral acceleration, for periods ranging between 0 and 3 s, with 0.1 s increments, and for damping between 2% and 10%, in a mesh with a resolution of 0.10° geographical degrees; (ii) uniform acceleration hazard spectra for 475 years, 2500 years and any other return period; and (iii) seismic design spectra determined taking into account [9] (considering both generic and specific Z values) and spectra of seismic design determined according to [14] referring to buildings. As the Norm cannot rely on an Internet application that depends on an external institution, such information will be incorporated in the Norm itself. Moreover, the studies leading to the application by the Sencico do not consider the last strong earthquakes occurred in Peru. Also, the results on the influence of damping in the spectral ordinates are not totally reliable, given the absence of studies in Peru regarding this issue; the results of studies for close countries (Colombia, but mainly, Chile) will be considered instead. This quantification of the seismic action will be applicable to procedures that do not involve nonlinear dynamic calculations; this includes both analyses based on applied forces and imposed displacements. The propagation of the seismic action from the bedrock to the ground surface can be quantified simply by means of F_{PGA}, F_a and F_v coefficients indicated in [8] or in [9]; given that [9] relies on studies actually conducted for Peru (instead of relying mainly on the American regulations, as [8]), such Standard will be generally preferred. The seismic actions for intermediate layers can be approximated by interpolating between the bedrock substrate and the surface; this interpolation should not be linear, but proportional to the shear wave propagation velocity in the layer under consideration.

2.2. Expression of the seismic action for dynamic analysis

For nonlinear dynamic analyses, the seismic action will be initially quantified in terms of several surface accelerograms (i.e. corresponding to the natural ground level). These accelerograms cannot be obtained as artificially generated signals fitting the Standard design spectrum. This prevention is based on three considerations: (i) the design spectra are generated from individual spectra obtained from particular accelerograms, consequently, in any case, such accelerograms should be taken instead; (ii) the different artificial accelerograms that are compatible with the same spectrum have similar characteristics, not being useful to provide results that represent the strong variability of actual accelerograms; and (iii) in underground structures, the spectra (representing the effect of the seismic action on the construction as a function of its period) do not constitute a particularly significant quantification of the seismic action. On the other hand, these accelerograms cannot be obtained by scaling small real records, since accelerograms corresponding to earthquakes of strong and low intensity are quite different; in the final version of the Standard, the maximum allowed amplification coefficient shall be specified. The generation of design accelerograms will be based either on severe recorded accelerograms (registered) in the area, or on severe accelerograms recorded in other areas with similar tectonic characteristics (whether from Peru or elsewhere) identified by seismic disaggregation techniques. It should be made clear that the purpose of using more complex methods (nonlinear dynamic analysis, basically) is to obtain results that are more accurate than with more simplified methods; consequently, it will be required that all the involved operations (selection of the accelerograms to be used, modeling of the non-linear behavior of the soil and the structure, numerical calculation of the dynamic response) are carefully, accurately and reliably performed. The use of more complex methods to obtain more economical designs should be rather discouraged: (i) the cost and effort of these operations will be significant, (ii) large reductions in the analysis results with respect to those obtained from simplified methods will not be allowed. In particular, incremental dynamic analysis (IDA) is discouraged; this is due, among other reasons, to the fact that it requires a large number of operations to obtain reliable results. In nonlinear dynamic analyses, a sensitivity study to the sampling period (Δt) will be required.

2.3. Seismic action for underground structures

The seismic action in the ground lower layers will be obtained from the action in the engineering bedrock, by means of numerical convolution techniques [15]. If soil linear behavior is assumed (moderate seismicity), the analysis can be carried out in the frequency domain by means of a linear convolution; this operation does not entail greater difficulty and can be performed with commercial software (such as DeepSoil). Where the seismicity is more intense soil linear behavior no longer can be assumed and, consequently, the deconvolution becomes more cumbersome [16-20]; thus, simplified criteria will be proposed. These operations can be limited to shear waves (S), not being necessary to extend them to longitudinal waves (P). Alternatively, in-depth seismic action could be considered directly if in situ measurements are available.

2.4. Variation of PGA with the return period

As discussed in subsections 1.6 and 2.1, the seismicity will be quantified basically in terms of the values corresponding to 450 years return period; thus, it is necessary to propose criteria to escalate (to scale) the accelerations corresponding to other return periods (in principle, ranging between 60 and 2450 years, Table 1). As stated in subsection 2.1, this scaling will be made preferably based on the study leading to the application that is available in the SENCICO web site. Alternatively, it can be also proposed to use the simplified expression $(T / T_r)^k$, where T is the selected period, T_r is the reference period (450 years) and k is an exponent; it is initially proposed to consider $k = 0.3$, although a deeper study will be performed prior the final version of the Norm.

3. Soil geological characteristics

This section describes the soil categorization (in terms of its geological structure) in everything that concerns the effect of seismic actions on the underground construction under consideration.

3.1. Site

Refers to the location characteristics that may be relevant to the objective of the Standard.

- **General conditions**. The objective of the study will be to characterize the soil stratigraphic profile and its geological and geotechnical properties. The aim is to identify seismic sensitivity (for instance, potentially liquefiable sands or degradable clays by cyclic loading) and to find the basic resistance and stiffness parameters. Then, the corresponding analysis methodologies can be applied.
- **Potentially active faults**. In constructions with importance I2 and I3, potentially active faults can be only crossed if a seismic joint is provided and the vertical distance to the bedrock exceeds a minimum value. This value depends on the seismicity level and the soil stiffness (in terms of shear wave velocity). The

crossing of an active fault is dangerous and can be avoided, wherever possible. If the fault needs to be crossed anyway, it is recommended to do it perpendicularly. Then, there are two major strategies: (i) the fault displacement is restrained by stiffening with intense transverse piling, and (ii) the fault displacement is accommodated by encircling the tunnel with flexible (soft) material and placing a seismic joint in the tunnel lining. Regarding the sizing of this joint, it was mentioned that it is very difficult to estimate the seismic displacement of a fault; thus the design of a seismic joint can be only based on the judgement of the design engineer and not on reliable theoretical considerations. Therefore, these analyses cannot be framed in the context of the PBD (Performance-Based Design) given that the verifications cannot be quantified. Studies to detect active faults will be required if: the return period of the considered seismic action is greater than 1000 years, the maximum moment magnitude (M_w) of the earthquake that can be generated the fault is greater than 6.5, and the fault is less than 5 km away.

3.2. Slopes stability

The seismic stability of slopes will be only considered when can affect underground structures, especially the tunnels sections near their portals (openings). The analysis will be based on the residual displacement; it is proposed to use simplified approaches, particularly improved (coupled) versions of the Newmark method. Alternatively, dynamic limit equilibrium methods may be considered. The slope reinforcing (i.e. preventing earthquake-triggered sliding) is ordinarily considered as a rather difficult task; thus, it is not recommended to undertake it. Conversely, the tunnel openings should be designed to resist the “unavoidable” sliding.

3.3. Potentially liquefiable soils

The liquefaction generated by seismic actions is a substantial decrease in the soil shear strength caused by a sudden increase in the pore water pressure; this occurs in loose and saturated granular soils during the shear waves propagation. Sinking, lifting (flotation) and lateral flow will be considered. The relevant data for the evaluation of liquefaction should be the cyclic undrained shear strength and the soil grain size distribution (or other equivalent indices). Potentially liquefiable soils are loose granular soils located under the water table, with a plasticity index not exceeding 10. In any case, if the percentage of clay exceeds 15%, it is not necessary to consider liquefaction. It should be noted that in Peru there is a map of soils susceptible to liquefaction. In tunnels and similar constructions, the analysis of the liquefaction in the seismic zones 1 and 2 of Peru can be omitted if the liquefiable soil is more than a certain depth below the affected structure; this depth will depend on the structure and soil types.

3.4. Analysis of the site specific response

This part refers to special soil conditions that do not fit into the general classification: hard rocks, very deep basins, very shallow and soft sites, soil profiles in irregular layers, among others. In these situations, the seismic action will consist of historical or generated accelerograms as indicated in the corresponding section of the Standard. The propagation of the waves through the different layers of the soil will also be considered.

4. Soil geotechnical characteristics

This section describes the mechanical categorization of the soil in everything that concerns the effect of seismic actions on the underground construction under consideration. A Table summarizing the quantities obtained at each experiment and their utility in the analysis and design the underground structure under consideration will be incorporated in the final version of the Norm. This Table will allow to emphasize that the parameters to be measured depend on the type of analysis to be performed (simplified static, nonlinear dynamic, etc.).

4.1. Soil classification

The classification of the soil is of paramount importance regarding the Norm and, thus, particular attention is paid to this issue. Obviously, the soil classification must be consistent with the current Peruvian design codes; among them, only [8] and [9] contain indications on the soil classification in Peru. However, the comparison between both codes shows relevant discrepancies. Moreover, the E.031 standard (currently under development) contains new proposals. Given these circumstances, as is commonly worldwide accepted (mainly in Peru), the soil will be preferably classified by the weighted harmonic average of the shear wave velocity in the soil layers in the top 30 m above the (underground) construction site ($v_{s,30}$); however, following recent research trends, a more refined strategy involving the engineering bedrock depth is going to be proposed. In general, the engineering bedrock is identified with a shear wave velocity of 800 m/s (although particular studies will be performed for extremely soft soils) corresponding to soil B according to the American standards; accordingly, such depth will be named as H_{800}. Regarding the choice of the shear wave velocity for the engineering bedrock, it is worldwide accepted; moreover, the soil in Lima is rather stiff, thus, the selection of any lower velocity would lead to the practical impossibility of distinguishing between most of the zones in Lima. If H_{800} is less than 30 m, $v_{s,30}$ will be replaced with v_{sH}, where H refers to H_{800}; in other words, if the engineering bedrock is less than 30 m deep, the average shear wave velocity will be referred to its depth (instead of being referred to the top 30 m). In any case, the soil will be generally classified on $v_{s,30}$ (or v_{sH}) and H_{800}; the lower the average shear wave velocity and the deeper the engineering bedrock, the softer the soil classification. Should H_{800} not be available (this will happen either in very soft soils or in case of lack of soil information), it can be replaced with the soil fundamental period (T_G); this magnitude can be easily measured (after Nakamura strategies or other field studies). Regarding the Nakamura approaches, criteria to avoid under or over-estimation of the soil fundamental period will be provided. Although the soil will be initially classified into discrete groups (S_0, S_1, S_2, etc.), interpolation in terms of $v_{s,30}$ (or v_{sH}) and H_{800} (or T_G) will be required in order do not generate artificial discontinuities in the parameters to be considered. When the shear wave velocity of each layer, v_i, needed to calculate $v_{s,30}$ are unknown, such velocities can be estimated by correspondence with other parameters obtained commonly after soil testing: SPT (Standard Penetration Test), FVT (Field Vane Test), pressuremeter, or CPT (Cone Penetration Test).

4.2. Water table level

The location of the water table that corresponds to each main geotechnical unit must be defined during the site investigation stage. In the designing phase, a characteristic ("almost permanent") position will be adopted (i.e., being exceeded during 5% of the construction design life). Variations of that position should be considered to analyze non-permanent conditions (i.e., extreme environmental actions), particularly when those variations may define a worst scenario than the current one (regarding actions on the underground structure). Causes that can modify the level will be taken into account, among them: construction and filling of reservoirs, shifting of natural or artificial channels, incorporation of facilities capable of altering underground water flows, etc.; in this sense, particular attention will be paid to densely populated urban areas, given that these changes are usually very frequent and intense. Particular care is required when a geotechnical unit behaves as a confined aquifer and its piezometric head (instead of a local water table) has to be measured. Estimation of the position of the water table can be carried out through several direct and indirect procedures. Direct procedures involve drilling boreholes. During the field investigation stage, as described in section 4.6, at least one borehole in each main geotechnical unit will be drilled, and samples shall be recovered (undisturbed if possible) for further analysis. At least 30% of the drilled boreholes will be used to install piezometers (either open or closed). Closed piezometers are recommended in low permeability soils; when open piezometers are used, special attention will be paid to the time lag required to measure a correct value of the ground water table. After installation of the piezometers, the observation of groundwater table will be recorded for some time in order to define the fluctuations during dry and wet periods. Also, if the pore water pressure varies quickly in a geotechnical unit, a continuous reading for piezometers will be adopted. Indirect procedures to measure water table position are based on geophysical methods, as electrical resistivity measurements or ground-penetrating radar. In addition, seismic refraction techniques and other geophysical techniques may be used (particularly for horizontal layers increasing stiffness with depth). A correlation of the results from any geophysical technique with drilled boreholes and direct piezometric measurements is always required. That correlation should be established for each soil or rock identified unit.

4.3. Strength parameters

It is assumed that during the high-speed seismic excitation the soil behaves under undrained hydraulic conditions. Therefore, the shear strength parameters in undrained conditions may generally be considered. For cohesive soils (such as silts and clays) the static undrained shear strength is quantified in terms of the undrained cohesion c_u. For rocks, the unconfined compressive strength $q_u = 2\ c_u$ may be used. This static parameter depends on the effective stress (it increases with depth) and the over-consolidation ratio (that parameter increases with such ratio). This variation should be assessed by appropriate laboratory or field tests, although conservative empirical criteria will be also accepted. The undrained shear strength should also be adjusted by adequate experimental tests for the rapid rate of loading of the seismic action (that strength decreases with the excitation duration), as well as by cyclic degradation effects under the earthquake action. The appropriate shear strength parameter for cohesionless soils is the cyclic undrained shear strength $\tau_{cy,u}$, which should be assessed by appropriate laboratory tests that consider the pore pressure built-up during the cyclic process. If pore pressure dissipation prevails in high permeability cohesionless soils (non-plastic silts, sands and gravels), drained conditions should be also considered in the earthquake analyses. In this case, the shear strength parameters to assess are the drained friction angle ϕ', and the drained cohesion c' (usually neglected in the case of sands and gravels). The drained friction angle should be obtained from drained laboratory tests or empirical correlations using field data.

4.4. Stiffness parameters

Given the preeminence of the shear waves (S) overt the pressure ones (*P*) in the seismic excitation, the soil shear stiffness will preferably be characterized by the shear modulus (*G*). This parameter displays an important dependence on the strain level. For small shear strains (linear behavior), *G* will be obtained as $G = \rho\ v_s^2$, where ρ is the soil bulk density and v_s is the shear wave velocity. The small-strain shear stiffness also depends on the effective stress state (it increases with depth), the over-consolidation ratio (it increases with such ratio) and the void ratio (it decreases with increasing void ratios). Values of shear modulus compatible with the strain levels induced by the design earthquake may be considered. For larger shear strains (non-linear behavior), the hyperbolic fit of modulus reduction with normalized strain may be considered. The variation of *G* with strain level and effective stress (depth) should be assessed by appropriate laboratory and field tests, in which particular care should be taken to induce small disturbances on the soil. Conservative empirical criteria will be also accepted to assess the shear stiffness at different strain levels (or ground accelerations) in the absence of specific measurements.

4.5. Damping (energy absorption capacity) parameters

Damping refers to the energy dissipation when a travelling wave passes through a given soil or rock. The mechanisms that dissipate energy include friction between particles, plastic deformations and heat generation. Typically, all these effects are represented by a global viscous damping model. It is convenient to refer the damping level to a critical value which, providing a dimensionless damping ratio, well below 100% (under-critically damped system). The damping ratio will range between 2% (low shear strain level, usually related to stiff soil and/or low seismicity) and 20% (high shear strain level, usually for soft soil and/or high seismicity).

For very low shear strains the soil is expected to behave as linear elastic and the damping ratio is reduced to a minimum value of 2%, sometimes called “viscous damping”. For larger shear strains, the soil behavior is nonlinear and non-reversible (i.e. plastic), resulting in hysteresis loops in the shear stress-shear strain curves. In such case, the damping ratio increases with the shear strain level up to a value of about 20%. This type of damping is sometimes referred as “hysteretic damping”. In this text, the name “material damping” will be used to denominate the damping of the soil or rock, either due to viscous or hysteretic effects.

Apart from the material damping, there is a decrease of wave amplitude with distance from the source due to geometrical factors. That is, the wave spreads in all directions over a greater volume of soil or rock, decreasing the elastic energy per unit volume. This is called “radiation damping”. In Soil-Structure Interaction analyses the damping ratio should include both components, i.e. material and radiation damping.

Damping soil or rock properties will be obtained from laboratory or field tests, as described in sections 4.6 and 4.7. If possible, the dependence with the strain level will be taken into account in the calculations. Alternatively, a simplified sensitivity analysis could be carried out, considering at least two damping ratio values. If there is not specific available information, Table 2 suggests specific values; such values may be used as a reference. Those values depend on the seismic action and on the soil or rock stiffness, as this combination leads to typical shear strain amplitude consistent with the proposed damping ratio value.

Table 2 – Material damping depending on the shear wave velocity and the severity of the seismic action

Seismicity level	$150 < v_s < 250$ m/s	$250 < v_s < 400$ m/s	$400 < v_s < 800$ m/s	800 m/s $< v_s$
Very low	4%	3%	3%	2%
Low	7%	5%	3%	2%
Moderate	15%	10%	5%	2%
High	20%	15%	10%	2%

4.6. Field studies

General inspection techniques of the ground geometric, geological and geotechnical characteristics will be described. The minimum number of tests (particularly boreholes) to be carried out shall be specified; this number will depend on the previous information available, the soil characteristics (its homogeneity, mainly) and the properties of the underground structure. The decisions to be made from the tests results will also be indicated; in particular, the need to perform additional experiments.

- **Geometric characterization of the ground stratigraphic profile**. Field studies should be devoted to the understanding of the geological and geotechnical profiles involved in the construction, including the main parameters defining the geotechnical properties of the soil and rock units. In linear infrastructures as tunnels, in general, at least a borehole with continuous sample collection every 200 m should be carried out; this requirement can be relaxed (up to 400 m, for instance) depending on the local conditions, particularly soil uniformity and site accessibility. Depending on the soil characteristics, drilling boreholes will also be required at locations separated from the construction axis. The number of boreholes must be sufficient to define the geometry of the geotechnical units and their contacts. Boreholes must be capable of determining the water table depth. In complex profiles, installing piezometers at different levels may be required. Also boreholes may be used for field tests or for extraction of soil samples (undisturbed, if possible) for laboratory analysis. The number of dynamic penetration tests should be limited and always correlated to boreholes with continuous sample collection. Geophysical techniques (i.e. electrical resistivity measurements, ground penetrating radar, etc.) can also be used to define stratigraphic profiles, but should be always correlated with boreholes collecting continuous samples. That correlation should be established for each identified soil or rock unit.
- **Tests at very small strain (far from failure)**. Geophysical techniques used in site investigation allow to estimate the velocity (and attenuation) of compression waves, shear waves and/or Rayleigh waves in the ground, which is a fundamental parameter in seismic design as stated in subsection 4.4. Those techniques include refraction, MASW (Multi-Array Surface Wave Analysis), MAM (Microtremor Array Measurements), etc. It is possible to combine these techniques with the borehole drilling as in cross-hole, up-hole or down-hole tests, where P-wave and S-wave velocities are measured. In this type of tests, the strain level is very low (vibrations are involved), therefore the stiffness estimated corresponds to the initial (maximum) stiffness of the soil or rock (i.e., G_{max}, as described in subsection 4.4). The following standards from ASTM can be used as a reference in this group of geophysical techniques: D6820-18, D6429-99, D6727M-16, D6639-18, D5777-18, D5753-18, D6726-15, D7128-18, D6430-18, D6432-11, D6274-18, D6431-18, D6167-19.
- **Tests at intermediate strain**. Several tests involving a higher strain are considered here. Strength, stiffness and damping parameters indicated in subsections 4.3, 4.4 and 4.5 may be obtained directly or through correlations. Typical field tests in this group follow, including the corresponding ASTM standard which can be used as a reference.

  - Static Penetration Test (SPT), measuring NSPT (ASTM D1586M-18).
  - Pressuremeter Test, obtaining pressure-radial deformation curve and deriving soil shear modulus.
  - Seismic Cone Penetration Test (SCPT) measuring cone penetration and lateral friction; Cone Penetration Test measuring pore water pressure (CPTU) and measuring shear wave velocity (CPTUS). (ASTM D5778-12).

- Load Plate Test. Static, measuring force-settlement relationship, or dynamic (measuring soil natural frequency as well). (ASTM D1196M-12).

- **Settlements under cyclic loading**. Repeated loading may accumulate displacements on soils or may reduce strength parameters obtained from static tests. This risk should be considered particularly in seismic zones 3 and 4 of Peru, mainly on loose unsaturated granular soils; their densification (and corresponding settlement) under cyclic (seismic) excitation may be excessive. This phenomenon can also be significant in soft clays and saturated sands, leading in this last case to liquefaction. Field tests applying repeated loading include dynamic load plate test.

4.7. Laboratory tests

Laboratory tests to measure shear strength, shear stiffness and damping parameters will be described. These tests are classified in three categories regarding the level of strain applied to the soil, the hydraulic undrained or drained conditions prevailing during the test, and the dynamic or static nature of the experiment. Soil layers should be described according to an acknowledged geotechnical classification system, including the relevant mechanical and hydraulic parameters. The number and type of tests required will be specified; in particular, the conditions under which the tests will be carried out according to the above classification; as well as the specifications to retrieve samples to minimize sample disturbance effects. Samples disturbances may result in a change of soil structure that has important consequences on the small-strain mechanical properties.

- **Tests at very small strain for confined soil**. The main small-strain stiffness parameter related to the design under seismic actions is the shear modulus. The internal damping associated with the inelastic soil behavior under cyclic loading should be also considered. These stiffness and damping parameters mainly depend on the effective stress state, the soil strain amplitude and the void ratio. The stiffness and damping for different effective confinement stress and shear strains can be obtained by resonant column tests (excited with a dynamic torque and with fixed base) (ASTM D4015-15). Shear wave velocities can be also measured with bender elements at very small shear strain levels (ASTM WK60969). Transmitter and receiver bender elements can be installed in end platens of a triaxial cell to measure its dependence on the effective stress state.
- **Tests at intermediate strain (close to failure)**. These tests can be used to determine the soil strength parameters under static undrained or drained conditions. The undrained shear strength is the usual static reference for cohesive soils (ASTM D2166/D2166M-16, ASTM D2850 – 15; ASTM D4767-11), which should be evaluated for the fast rate of loading and cyclic degradation effects under the seismic loads. The static shear strength parameter for cohesionless soils is the drained friction angle (ASTM D7181-11). For cyclic loading tests in these cohesionless soils, the cyclic shear strength should be assessed by considering the pore water pressure build-up. A particular case is the liquefaction phenomenon in loose sands, in which excess pore pressure is built up during the shaking process. The pore water dissipation process should be also considered (long-term drained conditions) to assess the soil volume change (soil settlement and deformations affecting underground structures). The rate of this process depends on the permeability of the soil. Cyclic tests for evaluating the ability of a soil to resist the shear stresses induced by cyclic loading and the pore pressure increase can be performed using the following apparatus: (i) simple shear cell with controlled cyclic lateral displacement or controlled cyclic shear stress (ASTM D6528-17 for the static test); (ii) triaxial cell with controlled cyclic axial displacement or cyclic axial stress (ASTM D5311/D5311M-13 for load controlled tests); and (iii) torsional hollow cylinder cell with controlled cyclic torque or rotational angle. The tests may be performed at different effective stress states to provide data required for estimating the cyclic stability of the soil. Cyclic strength depends on many factors, including soil density, effective stress state, applied cyclic shear stress, stress history, shape of the cyclic wave form, etc. Close attention must be therefore given to testing details and equipment. Larger physical scale models, such as shaking table and geotechnical centrifuge tests, can be also considered for studying the dynamic response of the ground or the soil-structure interaction

5. Conclusions

This paper presents preliminary (not final) information on the forthcoming Peruvian Norm on seismic design of underground structures. The discussed parts are: (i) basis of design, (ii) seismic action, (iii) soil geological characteristics, and (iv) soil geotechnical characteristics. This regulation will incorporate the best of international documents (ISO [1]) and codes and documents of advanced countries (China [2], Japan [3], New Zealand [4], European Union [5], USA [6]), is harmonized with the legislation and construction practices of Peru, and will fit the knowledge, expectations and mentality of its users. Noticeably, this paper provides only technical and scientific information; it has no direct relation with the final content of the Norm.

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