New methodology for the Seismic Design of Large Underground Structures

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SUMMARY:
Structural design principles for large underground structures subjected to severe seismic actions are presented, as well as a consistent analysis methodology.

Underground structures under seismic actions are subjected to horizontal displacements mainly controlled and imposed by the surrounding soil. Structures have to accommodate these horizontal displacement fields regardless of the resistance of individual structural elements. Taking into account that one major action is kinematic and that material failure is controlled by strain rather than stress, design methodology should be based on kinematic variable control, such as displacement or strain. Internal forces due to the imposed displacements are consequence of material constitutive relations, but explicit flexural resistance checks are neither necessary nor relevant, as long as material strain limits are not exceeded.

The analysis methodology is based on physically non-linear structural analysis to adequately evaluate post-yield behavior. Material strain limits are checked explicitly during analysis.

Recommendations for structural conception of underground structures are discussed.

Keywords: Seismic design, Underground structures, Capacity design, Strain control

1. INTRODUCTION

The analysis of damage induced by earthquakes in large underground reinforced concrete structures in the past shows that in general these structures are less sensitive to earthquake actions than structures that develop above ground level. [Gomes, 1999; Hashash et al, 2000]. In fact if the soil deformability is small, underground structures are subjected to almost rigid body motions, which induce little internal forces in the structures. Therefore, no damage or collapse is observed in these conditions.

However, the reality shows that this type of structure can also be vulnerable to seismic actions. During the Hyogoken-Nanbu earthquake, which hit the town of Kobe in Japan in 1995, a total of 6 out of 21 underground stations suffered strong damage [Iwatate et al, 1998].

Based on numerical studies and shake table tests of small scale models, Iwatate et al [1998] attributed the cause of collapse of Dakai tube station (see figure 1) to the distortion field imposed by the soil to the structure during the earthquake movement.

Figure 1 - Collapse of the central columns of Dakai tube station, Kobe, Japan [Iwatate, 1998]

The configuration of the station in plan is similar to a rectangle 120m long and 17m wide in the narrowest zone and 26m in the largest zone [Yoshida et al, 1997]. The perimeter walls are almost undeformable in their own plan, therefore with little capacity to accommodate significant relative displacements in this plan. However the available information is that collapse of the structure was not triggered by the collapse of the perimeter walls. This means that the stiffness of the structure in the longitudinal direction counteracts the displacements soil profile in the free-field, and may even lead to three dimensional soil flow in the vicinity of the extremities of the structure or lack of kinematic compatibility between soil and structure, as schematically represented in figure 2.

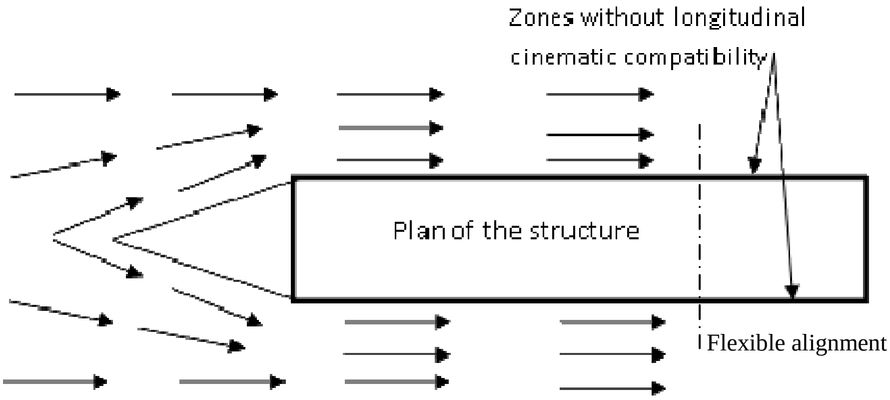

Figure 2 - Eventual three-dimensional soil flow near the extremities of the structure

The collapse of the central columns of Dakai tube station occurred essentially due to deformations in the transversal direction of the station [Iwatate et al, 1998], in particular in zones not near the extremities, where three-dimensional effects are not relevant and the structure tends to be more flexible than in the longitudinal direction. The critical transversal cross sections of the global structure, identified in figure 2, will be referred to as flexible alignments and will be the only ones considered in this paper.

The dynamic behaviour of the soil/structure systems along flexible alignments is dominated by the soil inertia forces, which are dominant as compared to the inertia forces generated in the structure. The structure being flexible in these alignments does not oppose significant resistance to the soil deformations. Therefore there is kinematic compatibility in the vertical soil/structure interface and the structure is forced to deform along the height following the displacement field imposed by the soil on the interface.

These underground stations, subjected to seismic actions don´t need to transfer inertia forces to the foundations, since those forces are directly transferred to the surrounding soil. However under severe seismic actions those structures, especially the ones embedded in soft soils, may be subjected to large relative horizontal displacements mainly controlled by the surrounding soil. Therefore the seismic design of those structures should aim at providing enough deformation capacity while maintaining the bearing capacity for the permanent loads.

A consistent design methodology with this purpose is presented, based on basic reinforced concrete behaviour and capacity design principals.

2. BASIC CONCEPTS

It can be shown that reinforced concrete sections have a yield curvature that depend very little on the flexural reinforcement amount (figure 3). As yield and rupture is controlled by strain, this behavior of reinforced concrete leads to the conclusion that even large changes of the flexural reinforcement amounts do not prevent yield if the curvature is externally imposed.

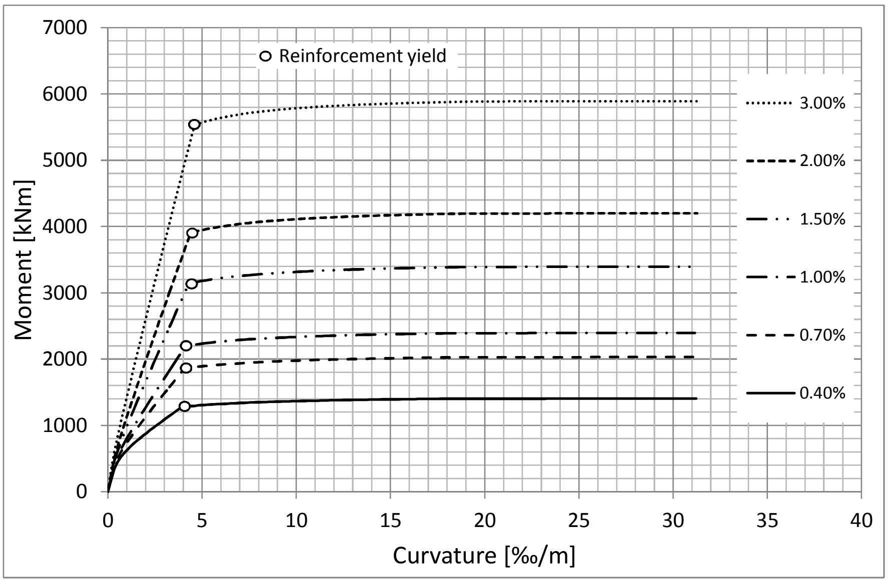

Figure 3 – Moment-curvature diagrams of a sample cross-section for several reinforcement amounts

The enforcement of relative horizontal displacements due to the seismic action by the surrounding soil on underground structures leads to the imposition of curvatures at section level. This will induce additional curvatures to bended structural elements, and therefore additional strain to steel and concrete. These additional curvatures (or strains) will have as a consequence an increase of internal forces (or stresses), as long as yield does not occur. But in this situation, structural resistance is assured by the design for permanent and live loads (except seismic load) and the seismic action is not relevant for safety verifications.

When yield due to seismic actions cannot be prevented, as strains are imposed, additional curvatures (or strains) will induce small or no increase in internal forces (or stresses), because stress increase in the post-yield range is rather small. Therefore, underground structures subjected to severe seismic actions (and effectively conditioned by this action) have to withstand imposed displacement fields, but do not have to resist to seismic forces, because the eventually emerging forces are small (they are simply consequence of imposed strain) and the horizontal inertial forces are transferred directly from the structure to the surrounding soil by axial forces in beams and slabs.

As mentioned, yield and collapse are controlled by strain, but standard resistance verification procedures transform those strain limits in internal force limits, such as bending resistance and shear resistance, and compare them to internal design forces. Safety is guaranteed if the internal design forces are smaller than the respective resistances. Usually this comparison is used to evaluate reinforcement amounts, assuming that internal forces of the structure do not change significantly with reinforcement changes, what might be acceptable for structures in service conditions. But for structures subjected to large imposed displacements, this procedure is inappropriate, as it hides the real nature of collapse control and the implicit assumption previously mentioned is not nearly valid.

Safety verification of underground structures subjected to severe seismic loads has to be performed checking explicitly material strain and limiting them to ultimate strain limits. And as significant incursions in the post-yield branch are expected, physically non-linear analysis has to be performed to evaluate those strains.

The use of q-factors to evaluate seismic action effects, as prescribed by EC8 for structures developing above the ground, is inadequate for underground structures. The seismic action applied to underground structures do not imply any relevant additional bending moment above the yield moment, and therefore q-factor would have to be infinite in order to yield a seismic nil moment. This shows that extrapolation of code procedures concerning seismic design applicable to structures developing above ground is not adequate for underground structures.

3. STRUCTURAL CONCEPTION

General structural layout design of underground structures subjected to severe seismic actions should follow some basic rules in order to provide adequate overall displacement capacity.

The conception of large underground structures in soft soils to resist seismic actions must aim essentially at providing deformation capacity to the flexible alignments of the structure. This means that along those alignments the structure must be as flexible and ductile as possible. Obviously there are restrictions to this structural conception that derive from the need to provide resistance to other actions. Therefore structural elements must have minimum dimensions necessary to provide the necessary levels of stiffness and resistance to permanent actions, live loads and other actions (except seismic action). However, even with these restrictions, the designer is left with many options.

For the purpose of the recommendations discussed in this section it is convenient to separate structural members in two groups:

- main structural elements: elements whose collapse leads to unacceptable damage. Examples of these elements are the perimeter walls, columns from top to bottom of the structure, beams that transfer strong axial forces due to soil and water horizontal pressures between opposite perimeter walls;
- secondary structural elements: elements whose collapse leads to acceptable damage. Examples of these elements are stairs, small columns that support other secondary elements, platform slabs, etc.

One of the most important criteria is to minimize the dimension in the plane of flexible alignments of the main structural elements where plastic hinges (incursions in the post-yield branch) may develop. In general, there is no interest in increasing the flexural capacity beyond what is necessary to resist to other actions but the seismic action, in these elements. In fact to increase the flexural capacity beyond this limit would only contribute to increase the shear demand and the strength demand in elements intended to remain elastic. Therefore section dimensions of elements that are supposed to develop plastic hinges should be the ones strictly necessary to resist to the other actions. And since it is important to minimize section dimensions, the designers will be led to design sections with reasonably high percentages of flexural reinforcement. However it may not be advisable to design sections near the allowable upper limits of flexural reinforcement as in general the sections deformation capacity is lower than if lower percentages of flexural reinforcement are used. The designer must balance all these effects in order to maximize the elements deformation capacity. The above criteria of minimizing section dimensions in the plane of the flexible alignments may not apply in the cases of elements that are intended to remain elastic, as in these cases it may be necessary to provide these elements with a reserve strength (beyond the one necessary to resist to other actions but the seismic action) in order to avoid the formation of plastic hinges in those elements.

Considering that deformation capacity of those structures is largely controlled by the behavior and distribution of plastic hinge zones, brittle structural elements should at all cost be avoided. Therefore, axial compression of the structural elements should be limited to reasonable values, because large compression reduces cross section ductility. Large soil covers should be avoided, as it will induce large compression in the columns.

Structural elements with low shear ratio (short elements) should also be avoided for two major reasons. These short structural elements have little flexibility, even when PHZ are developed at element extremities, reducing overall deformation capacity. On the other hand, high shear forces might arise at those elements due to equilibrium requirements, and induce a more brittle failure mode than that associated to ductile flexural collapse.

As an example, consider the transverse vertical cut of underground stations shown in figure 4. According to the above criteria, structural conception a) is clearly the the best.

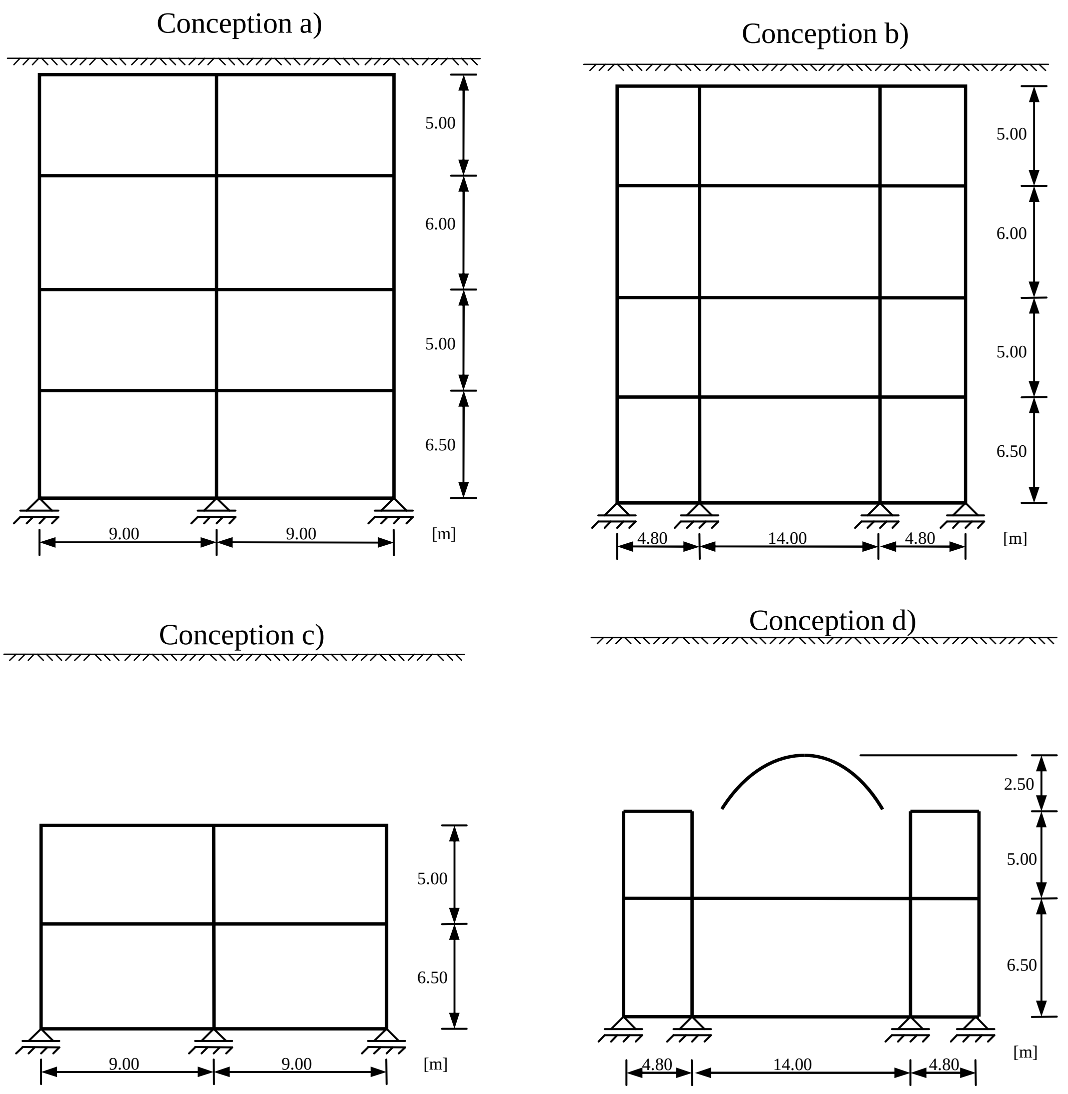

Figure 4 – Possible structural layouts of flexible alignments

4. PROPOSED DESIGN METHODOLOGY

The proposed design methodology comprises two phases. At the first phase the underground structure should be designed for all actions except the seismic action following standard design procedures. As result of this phase, minimum structural element dimensions and reinforcements are evaluated.

Regardless of the need to provide horizontal deformation capacity to the structure to resist to the main effect of the seismic actions, the structure has to be designed to resist to the other actions and eventually to the effects of some vibration modes, the ones with configurations not controlled by the soil dynamic behavior (vertical vibration modes, for instance). Therefore the section size and reinforcement amounts must be enough to resist to all the static actions-effects (bending moments, shear and axial forces) due to all code prescribed load combinations, including static seismic action-effects associated to the mentioned vibration modes. However it should be emphasized that in well-conceived structures, without significant soil covers, the effects of these modes are small and unlikely to condition the envelopes of static action-effects. In practical terms, this means that in such cases the seismic action is not relevant and can be disregarded at this phase. This is the first design phase, and should be performed using current design methodologies and code procedures. – (EC2 can be used at this phase).

The second phase of the proposed design methodology regards the main purpose of the seismic design: to provide the flexible alignments of the structure with enough deformation capacity to undergo the deformation that the surrounding soil may impose on the structure without losing the resistance to permanent loads and part of the design live loads. The second phase of the design process is based on capacity design principals, provision of local ductility and optimization of the distribution of the ductility demand. As the structure has to be able to withstand externally imposed displacements, a feasible mechanism has to be provided. Figure 5 shows two possible mechanisms for a structure with adequate conception, showing plastic hinge locations. Some of those locations cannot be chosen due to the large strength difference of the structural elements. For instance, the strength of the beams is much smaller than the strength of lateral walls; therefore the plastic hinge at beam ends is unavoidable. The plastic hinges at internal nodes may be placed either on the columns or on the beams, as the comparison of the two mechanisms shows.

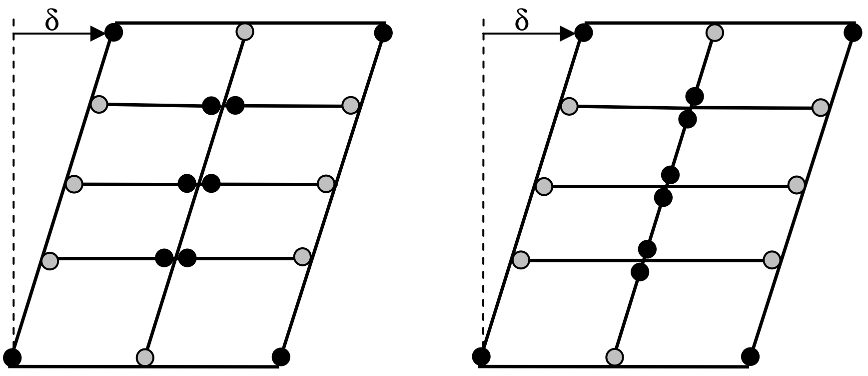

Figure 5 – Global mechanisms

A nonlinear analysis of the structure has to be performed to evaluate its deformation capacity. Structural collapse is controlled directly at material level, limiting explicitly concrete and steel strain. To accurately evaluate material strain due to external actions (applied forces and imposed displacements), structural layout, as well as the amounts of flexural and transversal reinforcement and respective detailing have to be known, as concrete stress-strain relationships strongly depend on these parameters and ultimate concrete strain may vary significantly.

The second design phase can be performed straight forward after conclusion of the first phase, but it is useful to make some structural improvements in order to try to maximize overall displacement capacity.

- A feasible mechanism should be selected, choosing adequately the plastic hinge locations;
- Capacity Design principal should be applied to force the location of the chosen plastic hinges. Whenever necessary, additional flexural and transverse reinforcement outside plastic hinge zones should be provided to avoid eventual yield of those zones due to over strength of the PHZ;
- Adequate confinement reinforcement should be provided to PHZ, in order to optimize local ductility.

The second design phase is a verification phase, because the structure has already to be fully detailed to perform the structural analysis and safety checks. If the imposed displacements are higher than the maximum the structure may sustain, it is necessary to act in one of two ways: (i) increase the structure lateral deformation capacity while maintaining its ability to sustain the permanent loads, or (ii) to treat the soil to reduce its deformability and the amplitude of the displacements imposed to the structure. The discussion that follows focus on the first objective.

Structural deformation capacity can be improved using one or more of several procedures.

- Increase ultimate concrete strain by increasing confinement ratio. This might be the single most effective and easy way to improve deformation capacity, as it acts directly on the available local ductility.
- Changes of flexural reinforcement of the structural elements, in order to change relative element stiffness and internal force distribution to reduce ductility demand of critical elements and therefore improve overall structural deformation capacity. This procedure is however less predictable.
- Eventually changes of cross-section dimensions might improve also the overall deformation capacity, but it is not expected to be an efficient procedure.

5. APPLICATION OF THE STRUCTURAL DESIGN PRINCIPALS

This section of the paper shows the application of the mentioned design principals to an underground structure with flexible alignments, the one shown earlier in figure 4a.

Application of the first design phase allows accurate evaluation of minimum cross section size of all structural elements (lateral walls, top and bottom slab, columns and beams) as well as evaluation of longitudinal and transversal reinforcement. The in-plane size of the structural elements should be maintained small, in order to maximize deformation capacity of each individual structural element, and therefore of the whole structure. Longitudinal and transversal reinforcement evaluated at the first design phase will provide the necessary resistance to the static loads.

The second phase of the design process begins with the choice of an adequate deformation mechanism. For the existing spans and cross section sizes, the second mechanism shown in figure 5, comprising plastic hinged at column top and bottom on all floors, will lead to a larger overall deformation capacity [Brito, 2011]. This mechanism is also better if big differences in soil stiffness are expected, as the imposed displacement field may vary significantly from the one shown in figure 5, imposing eventually plastic hinges at some intermediate columns.

At all plastic hinges adequate cross section confinement must be provided, in order to increase concrete deformation capacity (increase of the local ductility) and allow adequate development of the plastic hinges. The development of plastic hinges associated mainly with steel hardening will lead to bending moments at the plastic hinges somewhat bigger than the yield moment, and considering static equilibrium, transversal forces will also be bigger than evaluated during the first design phase. Therefore, even outside the plastic hinge zones, additional transversal reinforcement may eventually be necessary, in accordance with Capacity design principals.

The cross section confinement of plastic hinges is increased using transversal reinforcement and adequate detailing. Longitudinal reinforcement does not have to be changed, as mentioned earlier.

The mentioned design principals will have the following consequences for each one of the main structural elements:

- Perimeter walls:
  - At the plastic hinges at the top and the bottom of these walls, confinement reinforcement will have to be provided, increasing transversal reinforcement. Longitudinal reinforcement does not need to be changed. Usually the amount of longitudinal reinforcement of these walls will be conditioned by the construction phase.
- Bottom and top slabs:
  - These structural elements do not have any plastic hinges. Development of overstrength at the plastic hinges at the bottom and top of the perimeter walls may induce the need for additional longitudinal reinforcement at the bottom and top slabs, in order to increase their bending resistance, avoiding steel yield, as well as increase shear forces, requiring additional shear reinforcement.
- Columns:
  - Plastic hinges at top and bottom of each column require adequate confinement to allow their adequate development and increase deformation capacity. Longitudinal reinforcement does not have to be changed. Outside the plastic hinge zone, shear reinforcement might have to be increased to provide adequate resistance to shear forces due to plastic hinge overstrengh.
- Beams:
  - The plastic hinges of the beams, adjacent to the perimeter walls, require transversal reinforcement to increase confinement. Outside the plastic hinge zones, additional transversal reinforcement might be required, in order to provide adequate shear resistance, as plastic hinge overstrength might increase shear forces. Longitudinal reinforcement of the beams near the plastic hinges does not have to change. At beam-column nodes, taking into account that the plastic hinges are located at the columns, longitudinal beam reinforcement might eventually need to be increased to provide adequate flexural resistance, in order to keep plasticity in the column sections.

The application of the proposed design principals does not increase longitudinal reinforcement of the plastic hinges. Outside the plastic hinges the longitudinal reinforcement might eventually increase due to development of overstrength of the designed plastic hinges.

It has to be noticed that the application of these principals still leaves much freedom to the structural engineer. Deformation capacity of the structures designed with a straightforward application of this procedure is not guaranteed to be adequate. Even with an adequate deformation mechanism, adequate confinement of the plastic hinges and provision for resistance outside the plastic hinge zones ductility demands might somehow overcome the available ductility. Assuming that the available ductility of the plastic hinges will not be changed, as it might be considered that confinement should not be increased, it is still possible to change the ductility demand, changing relative stiffness of the several structural elements. According to figure 3 this stiffness change can be achieved varying the flexural capacity (by means of changing of the longitudinal reinforcement) of the structural elements (keeping at least reinforcement amounts evaluated at the first phase of the design process). Therefore some tests will have to be performed to allow adequate stiffness distribution in order to reduce as much as possible the maximum local ductility demand for a given imposed transversal displacement. This procedure might lead to some increase in the longitudinal reinforcement of some structural elements, even at the plastic hinges, but it must be noticed that this reinforcement increase is only to allow relative stiffness changes and not to resist to seismic forces.

6. ANALYSIS METHDOLOGY

Analysis of several underground structures under severe seismic actions were performed assuming that the seismic actions impose a transversal displacement field. Only global static pushover analyses were performed. At first this option might seem inadequate, as the seismic action is by nature cyclic. Cyclic analyses were performed at section level to show that for well conceived structures (flexure dominated behavior) the cyclic nature of the seismic action is usually not relevant.

It was concluded [Brito, 2011] that for flexure dominated behaviour the results of monotonic and cyclic analysis are very similar, except if certain values of the axial compression force are exceeded. Those values are a function of the level of confinement, as shown in table 1. Well-conceived structures do have usually lower compression forces than those presented in table 1.

Table 1 – Minimum confinement ratio to avoid deformation increase due to cyclic nature of seismic action

Normalized axial compression force (ν=N/(A f_{cd}))	Lower than 0.5	Between 0.5 and 0.7	0.7
Minimum confinement ratio (ρ_s)	0.5%	0.5% + 5% x (ν-0.5)	1.5%

According to many codes, global analysis should be performed using average values of material stiffness, and the safety verification should be performed at local level comparing action-effects with reduced values of the respective capacities. Therefore the fact that, according to the proposed methodology, the global structural analysis and the safety verifications are done simultaneously raises the issue of which material properties should be used. This issue is relevant, as weaker concrete and steel are more ductile that stronger concrete and steel. For instances a stronger concrete will reduce the dimension of the compressive zone of a cross-section of a reinforced concrete compressed element, therefore reducing the ductility demand, expressed as strain in the extreme fibers. However it also reduces the available ductility, casting a doubt on which properties condition the deformation capacity. Several analysis of simple and complex structures [Brito, 2011] showed that both situations can occur, therefore it is recommended that at least two analysis are performed: one with average material properties (strength and deformation) and one with the respective design values, and, in conservative approach, the lower value obtained in both analysis should be taken as the deformation capacity of the analyzed structure.

7. SYNOPSIS AND CONCLUSIONS

A consistent design methodology for large reinforced concrete underground structrures subjected to severe seismic actions was presented, based on basic reinforced concrete behaviour and capacity design principals.

Underground structures under seismic actions have to withstand horizontal displacements imposed by the surrounding soil, regardless of the resistance of individual structural elements.

Material failure is controlled by strain rather than stress. Taking into account that for underground structures the seismic action is mostly an imposed displacement field, without the need to transfer inercia forces to the foundation, the proposed design methodology uses explict strain control, without any need of stress or internal force control, as those items are just consequence of material constitutive relations. The design methodology requires phisically non-linear structural analysis to adequately evaluate post-yield behavior.

For the purpose of enhancing the structure´s deformation capacity short span elements should be avoided and the main structural elements cross-section dimensions on the flexural plan should be as reduced as possible. For this purpose high resistance materials should be used. Local ductility should be enhanced by means of providing significant amounts of confinement reinforcement at plastic hinge zones and avoiding large compressive forces on the columns, by means of reducing soil cover on top of the structure to the minimum thickness possible.

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