Seismic design of large underground structures Application of a new methodology to structures with feasible layout

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SUMMARY:
A consistent design methodology is proposed and applied to an underground structure with feasible layout and adequate conception. It is shown that the extrapolation of current code procedures applicable to the design of structures that develop mainly above ground leads to structures with poorer performance and sometimes even to unsafe results. The proposed design methodology is based on Capacity Design Principals and therefore requires definition of a feasible mechanism with its plastic hinges, as well as adequate resistance checks for the rest of the structure to prevent yield in these zones due to overstrength of the plastic hinges. Basic reinforced concrete behavior is taken into account, prescribing well confined structural elements with dimensions strictly necessary to resist all actions but the seismic action and high resistance materials to reduce cross-section dimensions increasing the overall deformation capacity. Failure is checked limiting explicitly material strain. Structural behaviour may be improved increasing local deformation capacity or changing relative stiffness of the structural elements to change the ductility demand.

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

1. INTRODUCTION

Underground structures, such as 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 and applied to an underground structure with feasible layout and adequate conception with the objective of maximizing the overall structural deformation capacity. It is also shown that the extrapolation of current code procedures applicable to the design of structures that develop mainly above ground leads to structures with poorer performance and sometimes even to unsafe results.

The maximum horizontal relative displacement between the top and bottom of the structure can be used as performance indicator. Therefore, in this paper, instead of checking if a given structure has a safe design for a given seismic action, the maximum relative horizontal displacement was evaluated highlighting the efficiency of the proposed design method.

2. BASIC CONCEPTS

To provide deformation capacity to a structure or a structural element, it is not necessary to increase its strength, as imposed displacements reflect on imposed curvatures and this on imposed strains at section level. Since rupture is conditioned by material strains exceeding the respective ultimate values, adding flexural capacity, by means of adding flexural reinforcement, does not increase the capacity to withstand imposed curvatures. Therefore it is not necessary to design the reinforcement to provide resistance to internal forces but only to provide local ductility and enforcement of the appropriate deformation mechanism in the nonlinear range [Brito, 2011, Brito and Lopes, 2012].

It results from the above that the traditional safety verification format, by means of comparing demand and resistance in terms of internal forces (bending moments, axial and shear forces) is not adequate. Therefore safety verification must be performed explicitly in terms of the variables that define rupture, the material strains, or by means of other cinematic variables that can be directly related with strains, such as curvatures or displacements. This means that strain demand must be evaluated by means of physically nonlinear analysis of the structure.

3. DESCRIPTION OF THE EXAMPLE STRUCTURE

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 [Brito and Lopes, 2012]. This means that along those alignments the structure must be as flexible and ductile as possible. Obviously there are restrictions to the 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 between opposite perimeter walls strong axial forces due to soil and water horizontal pressures;
- 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.

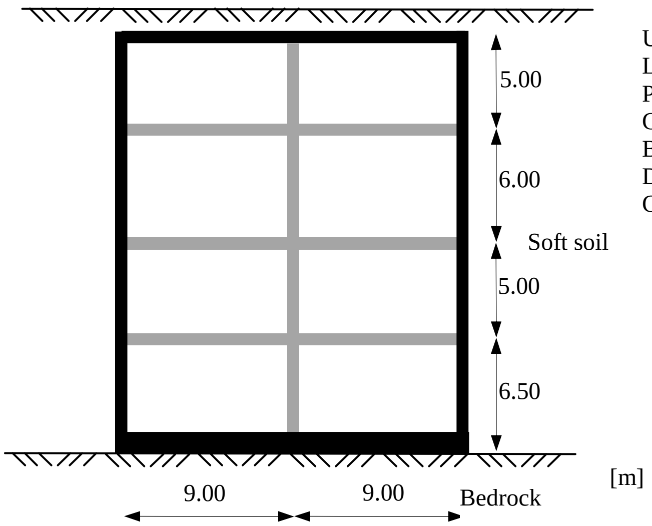
For the purpose of providing structural flexibility and ductility, the main structural elements must respect the following conception criteria [Brito, 2011, Brito and Lopes, 2012]:

- the cross-section dimensions of the elements that may develop plastic hinges, in the plan of flexural alignments, must be only the ones strictly necessary to resist to the actions except the seismic action;
- large soil covers should be avoided to do not induce high axial forces in the columns, as these have a negative effect on the available local ductility;
- elements with low shear ratios (short elements) should also be avoided, as this geometry may induce premature shear failure that reduces the deformation capacity in the non-linear range.

The actions considered were the structural self-weight, the additional dead load of the underground station, soil weight and lateral pressure, as well as hydrostatic pressure. The seismic action itself was

applied by means of imposition of and horizontal displacement field, variable along the height of the structure. In this paper the structure was designed to withstand relative horizontal displacements as large as possible. For a real structure the allowed maximum horizontal displacement of the structure will have to be compared with the one imposed by the surrounding soil under the earthquake action.
In what regards material properties, in principle global structural analysis should be performed using average material properties, which are the ones that allow a better assessment of the relative stiffness between structural elements. However the global analysis is not separated from the safety checking procedures and this should be performed using reduced values of material properties. Previous studies [Brito, 2011] indicated that none of the above yields always the more conservative result of the structure potential seismic performance. As such, in a conservative approach, the lower value obtained considering separately both types of material properties in the analysis will be considered the one representative of the structure´s performance.

In this paper results of the analysis of the structure shown in figure 1, that respects the above referred conception criteria, are presented.



Upper slab: Thickness: 1.20m
Lower slab: Thickness: 2.00m
Perimeter walls: Thickness: 1.20m
Columns: 0.7 x (1.4) m^2
Beams: 0.9 x (1.4) m^2
Distance between flexible alignments: 7.0 m
Concrete: C35/45   Steel: A500

Figure 1 – Sample structure with adequate conception

4. EXTRAPOLATION OF EC8 DESIGN PROCEDURE

According to EC8 – Part 1 [2004] – reinforced concrete building structures may be designed for seismic actions assuming Low, Medium or High ductility class.

Structures belonging to Ductility Class Low (DCL) are designed to resist seismic induced inertia forces without any need to provide or detail for ductility. A q-factor of 1.5 is referred by the code, assuming a minimum overresistance level of the structural materials. Design criteria for DCL structures are essentially the same of those for structures not subjected to seismic actions.

Structures belonging to the other two ductility classes assume that seismic resistance is obtained by mean of a combination of internal force resistance and energy dissipation and ductility. Energy dissipation mechanisms allow structures to resist earthquakes with less resistance to horizontal inertia forces, allowing the use of larger q-factors in design.

As the seismic resistance of underground structures depends on the overall deformation capacity, and this depends of the available ductility, low ductility class was considered, in order to emphasize the differences between the extrapolation of EC8 and the proposed methodology.

For the extrapolation of EC8, internal design forces of the structure are evaluated assuming constant stiffness, and reinforcement was provided in order to resist to those internal forces. The maximum relative horizontal displacement allowed by the structure was evaluated assuming that internal forces cannot exceed the ones associated with a maximum flexural reinforcement amount of 4% of the area of the respective cross-sections.

5. PROPOSED DESIGN METHODOLOGY

The proposed design methodology [Brito, 2011, Brito and Lopes, 2012] comprises two phases.In a first phase the structure designed to withstand all actions but the seismic action. The second phase starts by increasing the deformation capacity by means of the following procedure:

- 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 available local ductility.

Note that there is no seismic design bending moment at the plastic hinges. This is mathematically equivalent to consider an infinite value of the q-factor, showing that the seismic design of underground structures should not be based on the extrapolation of code methodologies derived and applicable to structures that develop essentially above ground level.

The nonlinear deformation mechanism adopted is the one shown in figure 2.

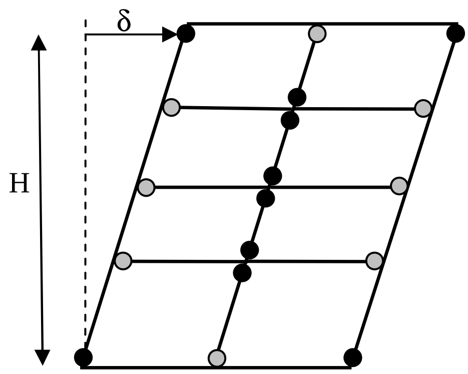


Figure 2 – Global mechanisms

The location of the hinges in beam-exterior wall column-slab connections cannot be chosen, as walls and slabs are much stronger than beams and columns. On the other cases the location of the hinges must be chosen based on other criteria, such as maximizing the deformation capacity, easiness of construction (placing the confinement reinforcement), and existence of other displacement profiles imposed to the structure.

The second phase is a verification phase, because the nonlinear analysis of the deformation capacity of the structure requires the knowledge of the complete structure, including amounts and detailing of reinforcement, as concrete stress-strain relationships depend heavily on these parameters and ultimate concrete strain may vary significantly.

6. STRUCTURAL ANALYSIS

Pushover analysis may be performed to evaluate the structural deformation capacity, as deformation increase due to cyclic nature of the seismic action is usually very small if axial compression forces and shear forces are not too high [Brito, 2011], as it is the case in well-conceived structures.

Concrete stress-strain relationship proposed by Mander et al [1988] was used (see figure 3). Structural deformation capacity is usually limited by concrete and therefore, ultimate concrete strain (ε_{cu}) is one of the most important parameters that control the overall deformation capacity of the underground structures. Ultimate concrete strain was evaluated as referred in Eurocode 8 - Part 2 [2005] (E2.1.c):

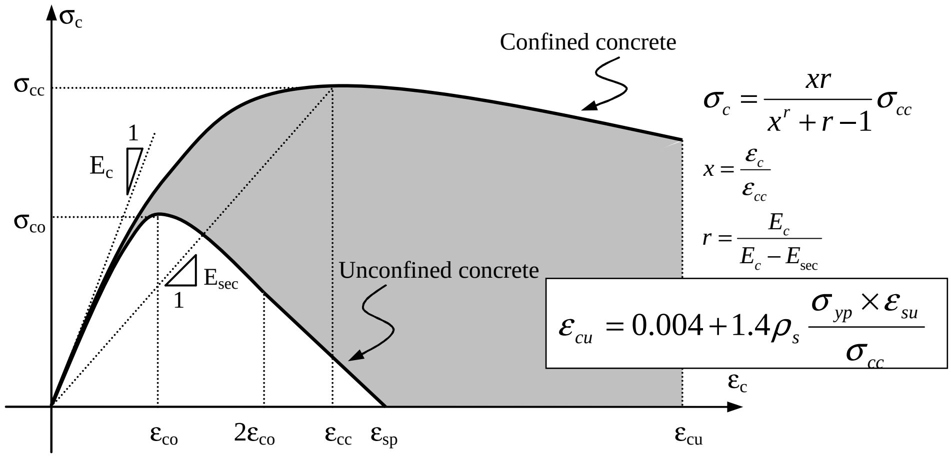


Figure 3 – Stress-strain relationship of confined and unconfined concrete [Mander et al, 1988]

Confinement (ρ_s) plays a major role in ultimate concrete strain (ε_{cu}). It is worth mentioning that unconfined concrete has an ultimate strain of about 4‰, as cross sections with reasonable confinement might achieve ultimate strains of about 10‰, and if a cross section is heavily confined ultimate strains of 30‰ may be achieved. The other parameters, such as hoop yield stress (σ_{yp}), ultimate steel strain (ε_{su}) and maximum concrete compression stress (σ_{cc}) are also relevant for the evaluation of ultimate concrete strains, but have less influence.

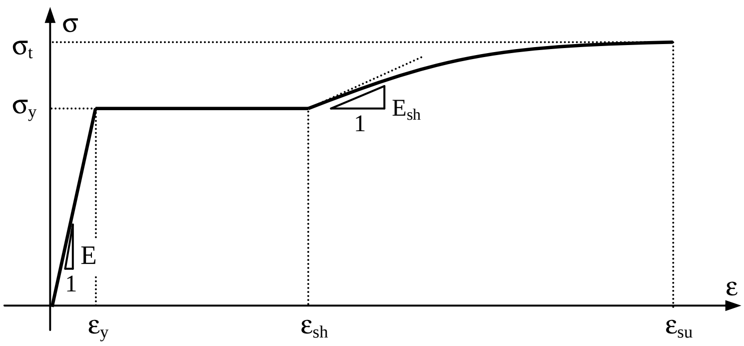


Figure 4 – Steel stress-strain relationship [Pipa, 1993]

Steel stress-strain relationship proposed by Pipa [1993] was used (see figure 4). Ultimate steel strain (ε_{su}) depends on specific steel propreties but values between 100‰ and 150‰ are usual.

Collapse of the structure is considered to take place when at any point of the structure the strain demand in any of the materials reaches the respective ultimate strain.

7. COMPARISON BETWEEN EC8 EXTRAPOLATION AND THE PROPOSED DESIGN PROCEDURE

The example structure was designed first for all actions but the seismic action, following EC1 and EC2. The seismic action was considered equivalent to the imposition of a linear displacement profile along the height, and no irregularities in the displacement profile associated to abrupt changes in soil stiffness were considered. In this situation the maximum value of the average distortion along the height $\gamma=\delta/H$ (see figure 2) that the structure can withstand was considered an adequate parameter to assess the potential seismic performance of the structure.

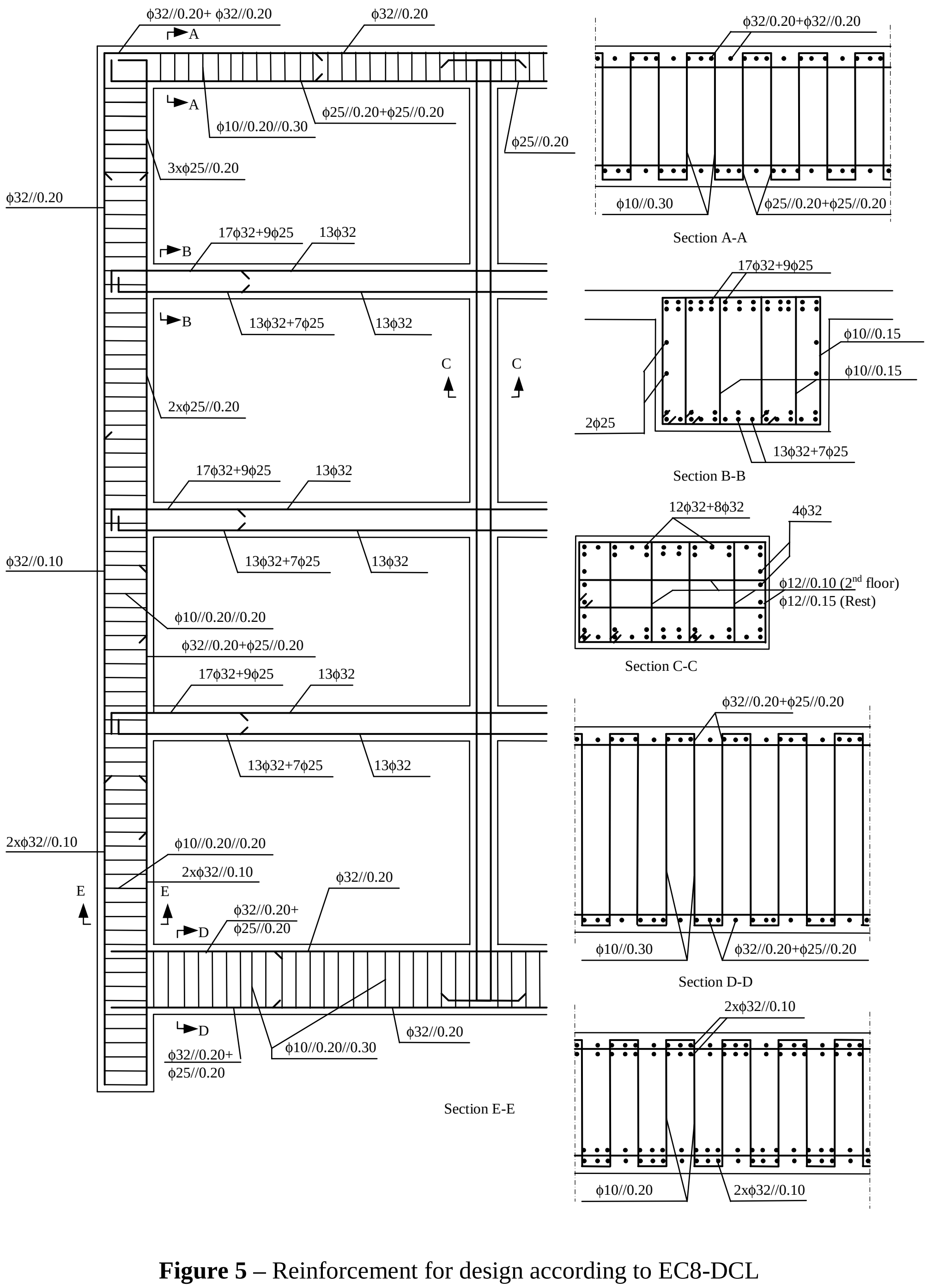


Figure 5 – Reinforcement for design according to EC8-DCL

The structural design according to EC8 – Part 1, assuming Ductility Class Low, yielded the amounts of reinforcement shown in figure 5, to which corresponds an average distortion γ=12.4‰. It was assumed a behavior factor q=1.5 and that the effect of cracking can be modeled approximately reducing the shear and flexural stiffness to half of the values corresponding to the gross concrete sections.

Figure 6 shows the amounts of reinforcement obtained by the application of the proposed design methodology. Note that this is the result of an iterative procedure, that included the application of Capacity Design principles (for instances by increasing the flexural capacity of the top slab sections near the perimeter walls) the provision of additional flexural reinforcement to increase the stiffness of the columns in relation to the beams to optimize the distribution of ductility demand, and the provision of large amounts of confinement reinforcement in the plastic hinges to increase the available local ductility capacity.

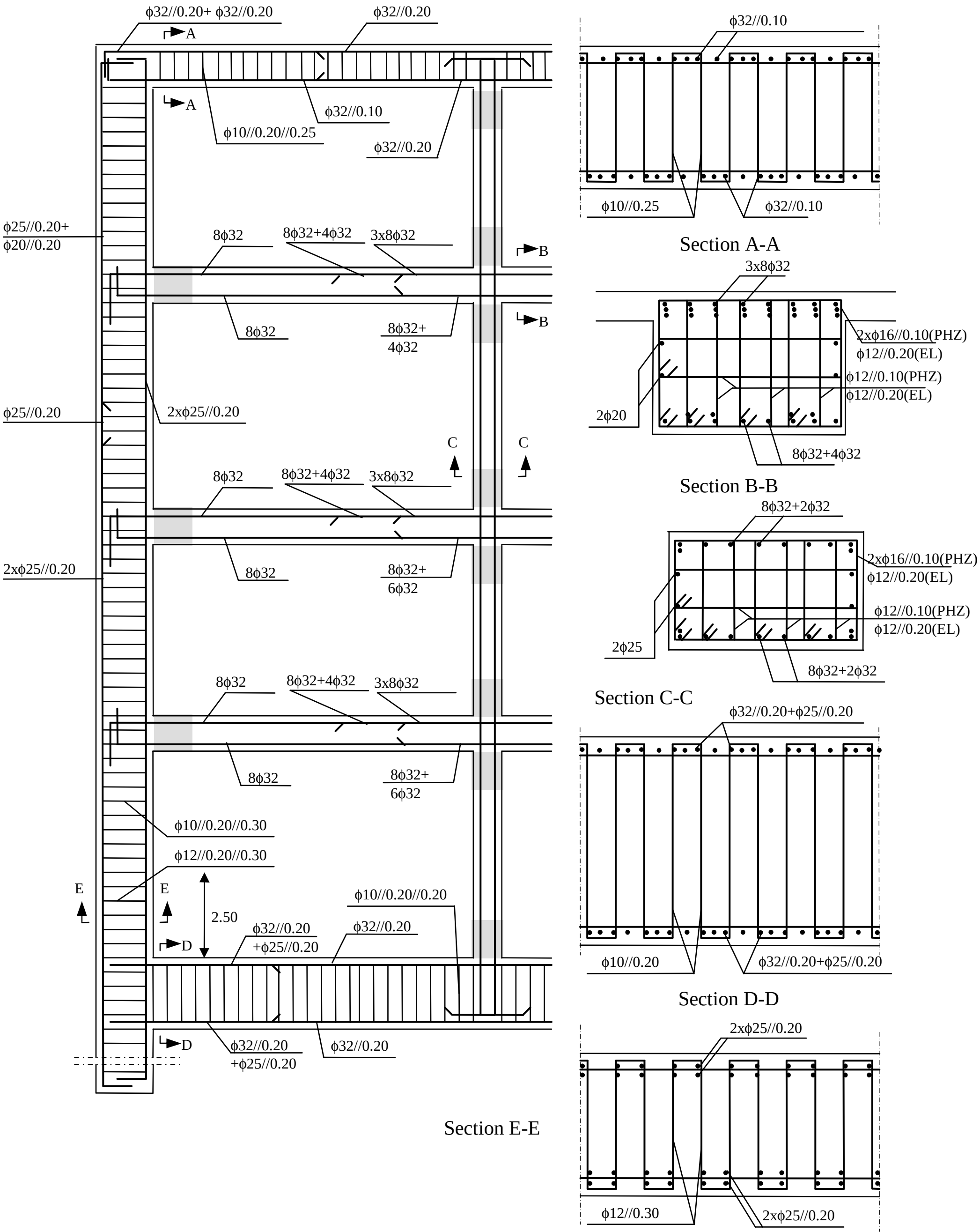


Figure 6 – Reinforcement for the design according to the proposed methodology

Comparing this structure with the structure designed according to EC8 one can notice the much lower amounts of flexural reinforcement in the structure designed according to the proposed methodology [Brito, 2011]. This is natural as flexural reinforcement was added not to provide flexural strength in the plastic hinges, as it is done according to code prescribed methodologies for structures that develop above ground, but only to control de location of plastic hinges and change the relative stiffness between elements in order to yield the desired failure mechanism. With this respect it is worth mention that bending moments in the central column due to other actions is almost zero, and therefore minimum flexural reinforcement should be enough. However since the beams need to be provided with reasonable amounts of flexural reinforcement to resist to permanent and live loads, they become much more stiff than the columns and the deformation pattern of the structure is similar to the one shown in figure 7. Essentially the columns behave as almost hinged struts due to lack of flexural stiffness associated to the lack of flexural reinforcement. This is a very negative effect as the counter clockwise rotation of beam-column nodes strongly increases the columns ductility demand, leading to rupture at low distortions. To avoid this situation it is necessary to impose a frame type mechanism, what can be done by increasing the amount of flexural reinforcement in the columns (see figure 8). However, column flexural reinforcement remained much less than according to EC8-DCL design.

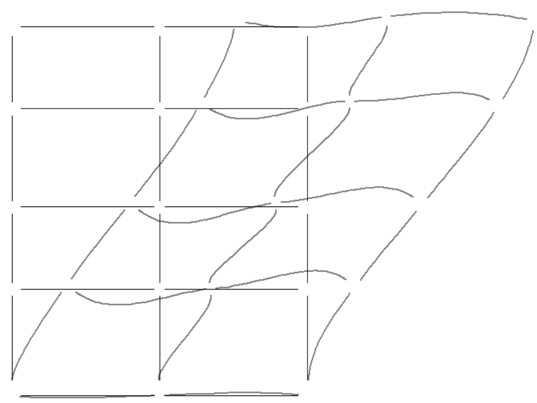


Figure 7 – Deformed shape (low stiffness column)

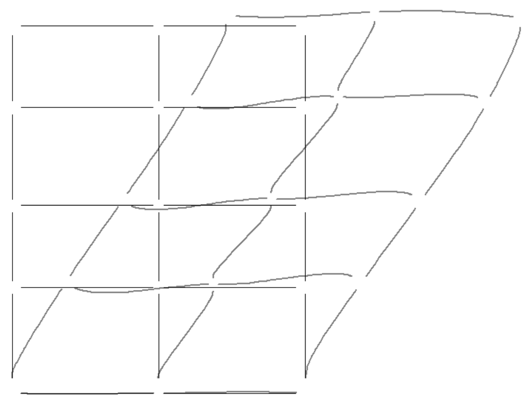


Figure 8 – Deformed shape (High stiffnes column)

However the proposed methodology leads to much higher levels of transverse reinforcement at the potential plastic hinge zones, as it should be expected.

The potential seismic performance of both structures was assessed by means of nonlinear static analysis. The horizontal displacements corresponding to the linear profile were imposed at the bottom, middle and top nodes on one side of the structure (in a way that the structure is pushed from that side). This derives from the assumption, which can be confirmed by studying the soil structure interaction [Brito, 2011], that the linear displacement profile at the free-field is adjusted in the contact with the perimeter walls of the structure. The results yielded an average distortions of γ=6.4‰ for the structure designed according to EC8 and 16.7‰ for the structure designed according to the proposed methodology. The curvatures at rupture for both cases are shown in figures 9 and 10, showing a much better distribution of plasticity in the structure designed according to the proposed methodology.

The first relevant conclusion of the above results is that the extrapolation of code methodologies applicable to the seismic design of structures that develop above ground may be unsafe, as the nonlinear analysis, which is closer to the real structural behavior, yields a structural deformation capacity of γ=6.2‰, which is half of the one obtained by code extrapolation (12.4‰).

The second main conclusion is that the proposed methodology leads to a much better seismic performance (170% increase in the global deformation capacity) despite the fact that it corresponds to a cheaper design, with less reinforcement.

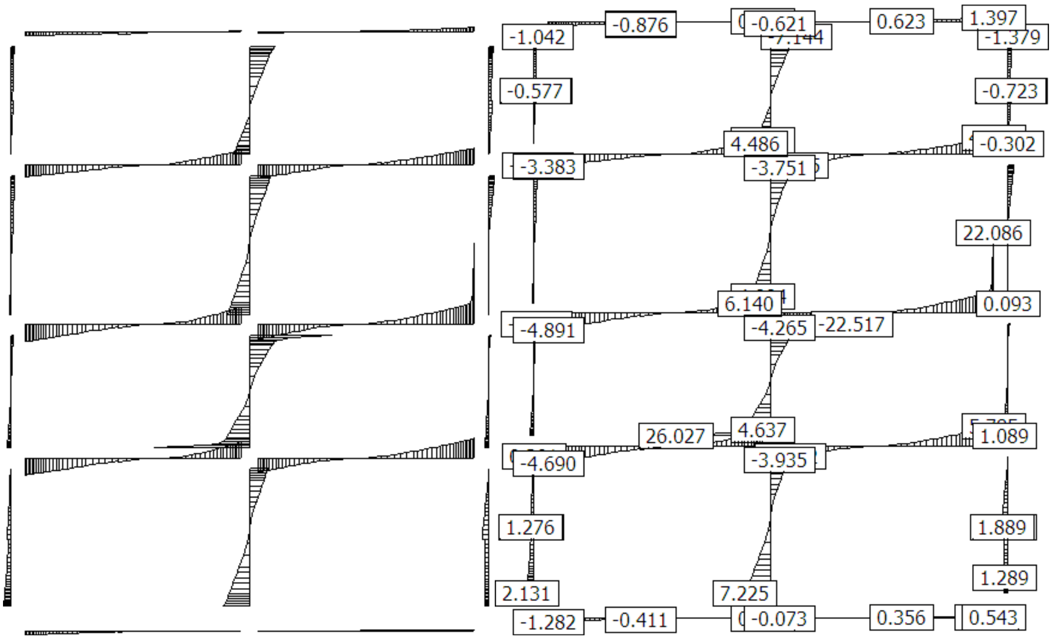


Figure 9 – Curvature diagrams [‰/m] at rupture for the EC8-DCL structure

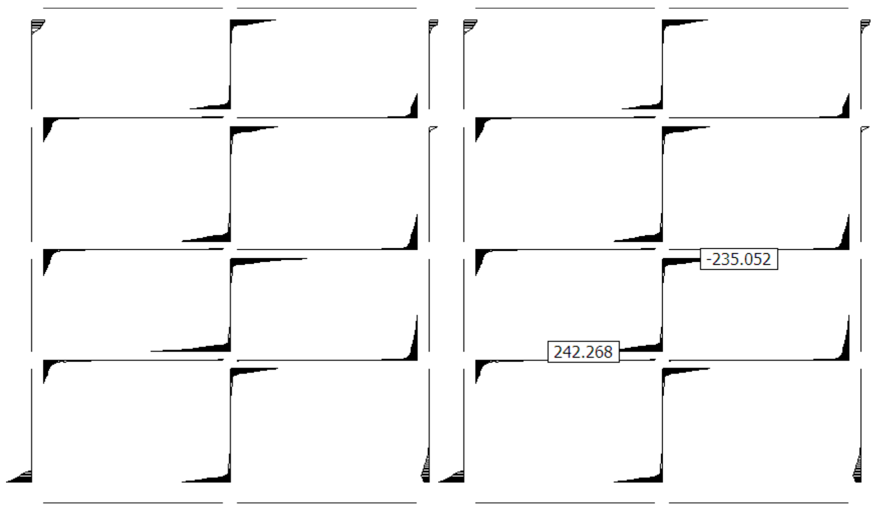


Figure 10 – Curvature at rupture [‰/m] for the structure designed according to the proposed methodology

8. CONCLUSIONS

A new methodology for the seismic design of reinforced concrete underground structures was applied to a structure with adequate structural conception and compared with the design according to EC8 prescriptions for buildings of Ductility Class Low. It should be noted that even though this is common design practice it is an extrapolation, as these prescriptions are for buildings and not underground structures. The new methodology aims at increasing the lateral deformation capacity of the structure by means of the application of Capacity Design principles and optimization of the distribution of the ductility demand by means of changing the amounts of flexural reinforcement. There is no need to design the structure to resist to horizontal inertia forces, this is, the design bending moments due to the seismic action are zero in all the structure.

The structure´s deformation capacity was evaluated by means of static monotonic nonlinear analysis. It was concluded that the extrapolation to the design of underground structures of code provisions for buildings may be unsafe, as it leads to the overestimation of the deformation capacity. It was also shown that the proposed methodology leads to better seismic performance as it increased the deformation capacity by 170% and led to a cheaper structure with less reinforcement.

The above indicates that a new part of EC8, dedicated to underground structures, should be developed, as none of the existing parts covers the design of this type of structures.

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