7.METHODOLOGY OF ANALYSIS FOR UNDERGROUND STRUCTURES IN SOFT SOILS

In this part of the report the seismic behaviour of large underground reinforced concrete structures in soft soils is analysed. It is shown that code procedures for structures that develop above ground are inadequate and may lead to the design of unsafe structures. A new design methodology and conception criteria are proposed for these structures. A practical example is shown.

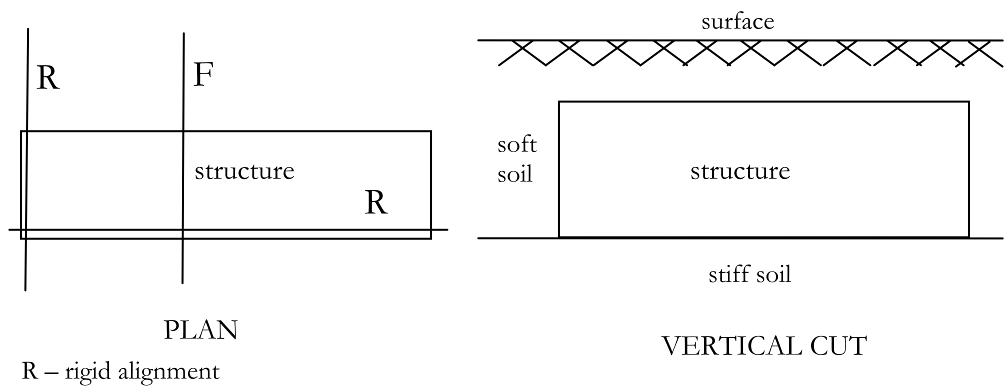
7.1 HISTORICAL BACKGROUND

The analysis of damage in large underground structures due to large earthquakes shows that these structures are in general less vulnerable than structures that develop mainly above ground (Gomes, 1999, Hashash et al. 2000). However, recent events have shown that these types of structures may also be vulnerable to earthquake actions. During the Hyogoken-Nanbu earthquake, that in January 1995 hit the town of Kobe in Japan, 6 out 21 tube stations were strongly damaged (Iwatate, 2000). Figure 7-1 shows the example of Dakai tube station, in which collapse was triggered by rupture of the central row of columns. Iwatate et al (2000) attributed this to the large horizontal displacement field imposed to the structure by the surrounding soil in the transverse direction.

Figure 7-1. Collapse of Dakai tube station

7.2 ANALYSIS OF THE SEISMIC VULNERABILITY

Under earthquake actions the deformation of underground structures, such as the tube stations previously mentioned, is essentially conditioned by the surrounding soil, as the inertia forces in the soil are much larger than the inertia forces in the structure. Therefore, the dynamic behaviour of the soil/structure system is essentially controlled by the mass and stiffness of the soil. For structures embedded in competent soil, the soil deformations are small and the structures present reduced seismic vulnerability. If the soil is soft, the soil horizontal displacements can be large. In these situations the seismic vulnerability of the structures increase and may even lead to collapse if the displacements imposed by the soil exceed the structure deformation capacity. The seismic behaviour of these structures is also influenced by their shape in plan. In tube stations of approximately rectangular shape one dimension is generally larger than the other and two types of vertical cuts can be distinguished: (i) the ones designated as rigid alignments, that are close or that contain very stiff elements, such as perimeter walls in their own plan, and oppose significant resistance to the soil movement and therefore undergo very reduced displacements, and (ii) the ones away from the zone of influence of the rigid alignments and that undergo horizontal displacements similar to the soil displacements in the free-field, designated as flexible alignments. Figure 7-2 helps to distinguish flexible from rigid alignments.



R – rigid alignment
F – flexible alignment

Figure 7-2. Schematic representation of rigid and flexible alignments

The observation of the seismic behaviour of Kobe tube stations indicates that collapse was triggered by the columns of the flexible alignments, unable to withstand simultaneously the permanent loads and the displacements imposed by the surrounding soil.

7.3 SEISMIC BEHAVIOUR

The seismic performance of an underground structure depends essentially of the ability of the flexible alignments to withstand the imposed displacements. Therefore, an adequate performance can be enforced by means of the following strategies: (i) to treat the soil to control the respective horizontal displacement field along the height during the design earthquake, or (ii) design the structure to withstand the displacements imposed by the soil, while maintaining the ability to resist to the permanent loads.

It results from the above that the design of the structure (always referring to the flexible alignments) aims essentially at providing deformation capacity while it maintains the ability to sustain the permanent loads. Current code procedures, essentially derived for structures that develop mainly above ground, assume that the seismic performance of a structure is a combination of its ability to resist to inertia forces and its ductility and energy dissipation capacity. Since engineers are essentially used to design structures to resist to applied forces, code procedures are usually based on the explicit evaluation of the effects of the inertia forces, the effects of ductility and energy dissipation capacity being accounted for approximately by means of a global factor (q factor in EC 8). The application of this procedure to underground structures is inadequate since these do not need to resist horizontal inertia forces (except for some minor local effects, usually irrelevant) which can be transferred directly to the soil on the sides of the structure and do not need to be transferred to the foundation. Within the usual code framework this would be equivalent to consider the behaviour factor infinite. This highlights the inadequacy of applying code procedures for structures that develop above ground to underground structures. This derives from the fact that providing resistance to horizontal displacements is qualitatively different from the resistance to horizontal forces. The difference between applying displacements and forces can be illustrated in terms of a reinforced concrete section in bending, considering two situations: (i) if a bending moment is applied, the higher the area of flexural reinforcement, the lower will be the respective stresses, the strains and the curvature, which are output of section analysis, (ii) if a curvature is applied the higher the area of reinforcement the higher is the associated bending moment. Note that being the curvature applied by an external source the strains can be evaluated as a function of section geometry, this is, do not depend on the amount of flexural reinforcement (assuming that the position of the neutral axis does not change much with the amount of flexural reinforcement, as generally happens in flexure without axial force).

Another feature that results from the above is that adding flexural reinforcement is useless to prevent yielding, as this depends on the fact that the imposed strain reaches the steel yield strain or not. This clearly contradicts current code concepts according to which a structure can be designed to remain elastic under earthquake actions by designing it with q=1. This is valid in general in structures that develop above ground, but not for structures or elements under applied deformations. Reasoning again at section level, for a given imposed curvature an elastic analysis yields a bending moment. If the amount of flexural reinforcement necessary to resist to that moment (equivalent to consider q=1) or slightly more is provided, the section should remain elastic. The contradiction can be explained with the help of Figure 7-3, as follows: point 1 represents the yield point of a section in bending without axial force. If for instances the flexural reinforcement was duplicated maintaining the same distribution, the flexural capacity would increase in such a way that, if the flexural stiffness was constant as assumed in linear analysis, the new situation would be represented by point 2'. In fact what happens is that the flexural capacity increases but the curvature almost does not increase (as the neutral axis remains in the same position and the steel yield strain does not change) and the point representative of the new situation is point 2 and not 2'. Therefore, the increase in the flexural capacity does not increase the yield curvature and does not avoid yielding.

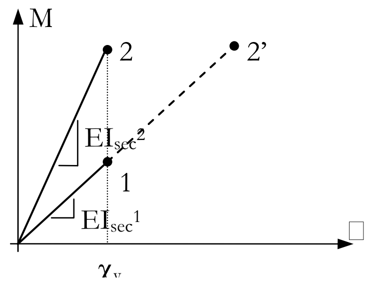


Figure 7-3. Schematic representation of change in the yield moment and curvature by increasing flexural reinforcement (N=0)

7.4 CONCEPTION

The conception (in the usual sense of defining the geometry of the structure and its elements) of large underground reinforced concrete structures in soft soils to withstand earthquake actions must aim at ensuring that flexible alignments are provided with the required deformation capacity under horizontal displacement fields without losing the ability to sustain the permanent loads. For this purpose along the flexible alignments the structure must be as flexible and ductile as possible, within the limits associated to the need to resist to all other actions.

Under imposed deformations the larger the cross section dimensions the higher will be the strains imposed both on steel and concrete. Therefore the dimensions of structural elements of the main resisting structure (elements whose collapse would induce irreparable damage or global collapse) in the plan of flexible alignments should be as reduced as possible, in practical terms should be the ones strictly necessary to resist to permanent and other actions but the earthquake action. Therefore the use of counterforts or other rigid elements, such as short beams, should be avoided. Besides, rigid elements tend to generate higher shear forces what makes them more vulnerable to seismic actions, as shear tends to reduce the available ductility, an effect that is magnified by the cyclic nature of the load history. In order to minimize section dimensions both concrete and steel of higher strength should be used. In the case of elongated tube stations in which one direction corresponds to a flexible alignment and the perpendicular direction to a rigid alignment, rectangular sections with the lowest dimension in the direction of flexible alignment are recommended.

In order to maximize the ductility it is necessary to minimize compressive axial forces in elements where yielding can be expected. Therefore in order to minimize axial forces in the columns it is recommended that large soil covers on top of the structures are avoided. Exceptions to this recommendation may be justified in some cases in which this could lead to higher imposed deformations due to the insertion of the structure in superficial soil layers of worse characteristics.

Secondary structural elements (whose collapse yields repairable damage), such as stairs, or others, and non structural elements, such as masonry partition walls, may have a negative effect by restricting the deformation of the main structure. Therefore their geometry and location should be chosen avoiding these types of interferences. For instances stairs should develop preferably in the plan of rigid alignments and should not be supported at middle height of the main columns.

7.5 DESIGN METHODOLOGY

The evaluation of the deformations imposed to the structure requires the analysis of the soil/structure system under the design earthquake action. The overall dynamic behaviour depends essentially on the properties of the soil, which usually presents stiffness and damping that are highly dependent on the level of distortion. Therefore a common methodology consists of analysing first the soil alone and estimates a value of an equivalent damping coefficient and distortion stiffness for the expected distortion amplitudes under the design seismic action. On a second stage this properties are used as input for the linear analysis of a soil/structure model. The simulation of the structure assuming linear behaviour is an approximation with little influence on the result, since the dynamic behaviour of the soil structure system depends essentially on the soil properties. The deformations obtained from this analysis can then be imposed on a structure model to evaluate its effects.

The analysis of the deformation capacity of ductile structures involves the analysis of the behaviour in the post-yield range, this is, a physically non-linear analysis. For this purpose, it is necessary to know the yield capacity of structural elements and sections, therefore global structural analysis requires not only the knowledge of the geometry of the structure and its elements but also knowledge of amounts and details of reinforcement. These are necessary to allow the definition of confined concrete constitutive relationships and evaluation of yielding and rupture. The analysis considers flexural and axial deformations but not shear deformations, which are not relevant in well conceived structures. Therefore, the analysis requires as input the definition of the geometry of the structure and its elements and the explicit definition of the flexural reinforcement, including the full monotonic constitutive relationship until rupture. The transverse reinforcement does not need to be defined explicitly, but knowledge of amounts and details of this reinforcement are necessary to evaluate confining stresses and the constitutive relationships for confined concrete. The detailed characterization of the structure allows performing the global structural analysis and the evaluation of axial stresses and strains anywhere in the structure simultaneously. Thus, it allows doing safety verifications at material level by comparing the maximum strain demands with the corresponding acceptable limits.

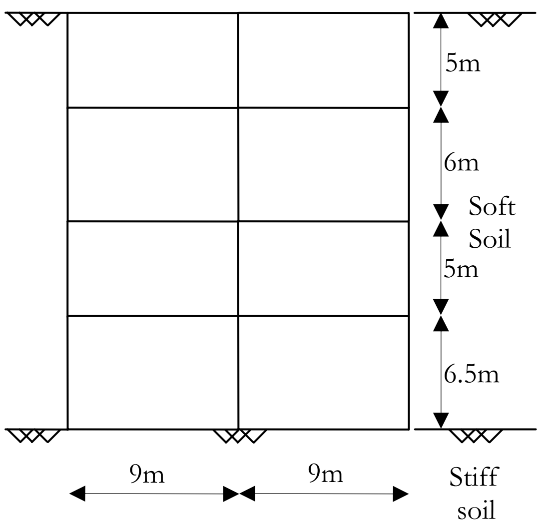
Global analysis of the structures, linear or nonlinear, are usually performed assuming average material properties. However, safety checkings at element and section level are usually based on design values of material properties to account for the possibility that at some locations in the structure the materials properties are worse than average. In order to follow this analysis and safety verification methodology it is necessary to decouple the global structural analysis from the section or element analysis using separate models. However, since there is an interest in performing both the global analysis and safety verifications with the same model, it is necessary to make a choice of what material properties to use. Since the action upon the structure is represented by imposed displacements, the evaluation of curvatures is essentially a cinematic problem whose result is almost not influenced by material constitutive relationships. Therefore, it is not relevant what material properties are used in the global analysis. Since design values should be used for safety verifications at section or material level, these properties will be used in the analyses that will be presented in the next sections.

Current design practice is usually based on a procedure with the following phases: (i) conception of the structure, (ii) global structural analysis based on a constant stiffness of all structural elements, (iii) safety verification at section or material level. In reinforced concrete structures, the third phase is transformed in the calculation of the amounts of reinforcement necessary to ensure the prescribed safety verification. In the methodology that is proposed in this work the third phase is a really verification phase, as the reinforcement needs to be known before the analysis and safety checkings are done, by comparing strain demands with the corresponding limits. As the structure has to withstand all other actions, the first phase of design must be the design of the structure, including the calculation of the necessary amounts of reinforcements to resist all load combinations in which the seismic action is not the main variable action. The next phase is the increase of the amounts of reinforcement to increase the ductility of the structure. This can be considered a second "conception" phase. The third phase is the analysis and safety verification. The second and third phases may need to be repeated, if the first verification does not yield suitable results, yielding an iterative procedure.

7.6 PRACTICAL APPLICATION EXAMPLE

In this section the application of the proposed methodology to an underground structure with appropriate conception (in terms of geometry and dimensions) is shown, complemented by the presentation of criteria for the "conception" of the reinforcement added (to what is necessary to resist to other actions) to increase its ductility. The application of the proposed methodology is also compared with code procedures for structures that develop above ground, namely EC 8 – Part 1, both in what regards seismic performance and economy.

The geometry of the example structure, with a conception considered adequate, is shown in Figure 7-4. A reduced width of 3.80m of the exterior walls was used in the calculations. The materials chosen are steel A500 and concrete C35/45.



Top slab - thickness: 1.20m

Bottom slab – thickness: 2.00m

Perimeter walls – thickness: 1.20m

Columns: 0.7 x (1.4) m²

Beams: 0.9 x (1.4) m²

Distance between flexible alignments: 7.0 m

The first dimension of beams and columns is the one on the plan of the flexible alignments

Figure 7-4. Example underground structure

The seismic action can be simulated by means of applying to the structure horizontal displacement fields with the profiles represented in Figure 7-5. The profile shown in Figure 7-5 a intends to represent the effect of a soil with increased stiffness with depth. The profile shown in Figure 7-5 b consists of a sinusoidal variation of the displacements along the height, and corresponds to the first mode shape of a soil with constant stiffness along the height. However, it is not uncommon to find strong variations of soil stiffness along the height, for instances due to the existence of more than one soil layer. This can be simulated by a displacement profile as shown in Figure 7-5 c, in which the deformations are concentrated at an intermediate soft soil layer. The examples shown next are based on the linear profile; the effects of the other profiles are discussed only qualitatively. The maximum distortion γ_{max} is used as a measure of the deformation capacity of the structure.

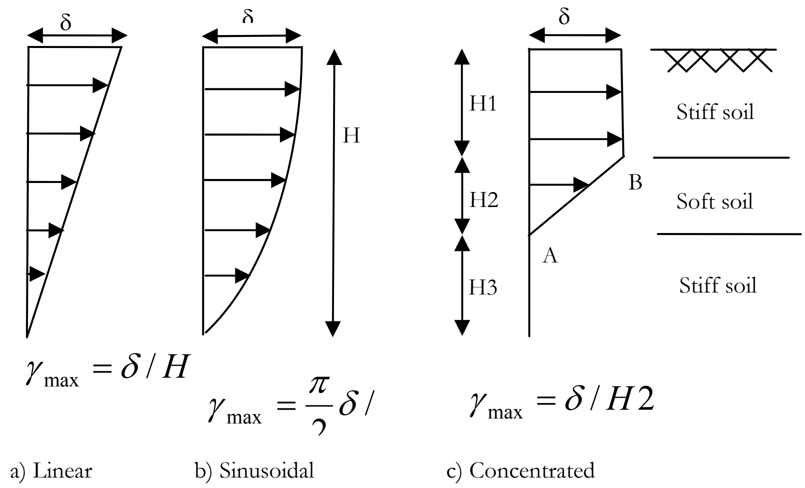


Figure 7-5. Horizontal displacement profiles

7.6.1 Structure designed according to current code concepts

Following current code procedures, seismic action-effects are obtained dividing the results of elastic analysis by a behaviour factor (q-factor in EC 8), a procedure that will be designated as Direct Design. Since EC8 does not cover this type of structures, an extrapolation of Part 1 will be made, as this is the most likely procedure designers will adopt. EC 8 – Part 1 (referred to as EC 8, from now onwards) considers three main Ductility Classes in seismic design: Low, Medium and High. Ductility Class Low structures are designed to resist earthquake effects essentially in the linear range and no procedures are applied to increase ductility. EC 8 prescribes a q-factor of 1.5 for this type of structures to account for some levels of overstrength that is assumed is always available in reinforced concrete structures. Structures of Ductility Classes Medium and High are designed to resist earthquake actions by a combination of their resistance to inertia forces with their ductility and energy dissipation capacity. This represents an intermediate type of design between the one associated with Ductility Class Low and the proposed methodology. Therefore, to highlight the differences to the proposed methodology the example structure is designed as a Low Ductility Class structure.

Since in the framework of Direct Design applied displacements result in internal action-effects on the structure (bending moments, shear and axial forces), the maximum displacement the structure can withstand is restricted by the maximum amounts of reinforcement that is possible to place in any structural member. Assuming q=1.5 and that the constant member stiffness assumed in the elastic analysis is half the stiffness of the gross concrete sections as prescribed in EC 8, the maximum allowable distortion associated with the linear profile of imposed displacements is $\gamma_{max}=8.2x10^{-3}$. The reinforcement corresponding to this distortion is shown in Figure 7-6.

The explicit evaluation of the deformation capacity of this structure was evaluated by means of a static nonlinear analysis imposing the permanent loads and the linear displacement profile. It is assumed that proper detailing ensures the anchorage and effectiveness of all reinforcement, in particular confinement reinforcement after spalling of the concrete cover. The deformability of the nodes and shear deformations were disregarded, only flexural deformations were accounted for. The nonlinear behaviour of concrete and steel were simulated using the constitutive relationships for confined concrete prescribed in EC 8 – Part 2 and constitutive relationships for steel obtained from a large statistical characterization of the Tempcore steels used in Europe (Pipa, 1993). Figure 7-11 shows the constitutive relationship for steel and an example of constitutive relationships for confined concrete. Rupture was defined by the attainment of the maximum axial strain anywhere in the structure. The maximum allowable strain for steel is $\varepsilon_{max}=7.5\%$, corresponding to steel type C and for concrete it depends on the level of confinement, according to the equation prescribed in Annex C of EC 8 – Part 2.

The results of this analysis indicate that the maximum average distortion that the structure can withstand is $\gamma_{max}=5.0x10^{-3}$. Figure 7-7 shows the curvature diagrams at this situation, indicating the yield curvature at some sections and showing that flexural yielding took place at several locations. The maximum tensile strain is $\varepsilon=27.8$ ‰, 13 times the steel yield strain ($\varepsilon=2.07$ ‰). Note that at this stage the maximum distortion was 60% of the distortion evaluated according to EC8 (Low Ductility Class), at which the structure was supposed to be elastic. If the sinusoidal profile had been applied the ductility demand would be higher at the lower part of the structure and it would withstand a lower relative displacement (δ) between the top and bottom slabs.

The above results show that the design with a low behaviour factor does not prevent yielding if the action is an imposed displacement field, contradicting widely held views and basic concepts of current code prescriptions for seismic design of structures that develop above ground. It also shows that extrapolating those procedures to underground structures can be unsafe, as lead to an overestimation of the structure deformation capacity.

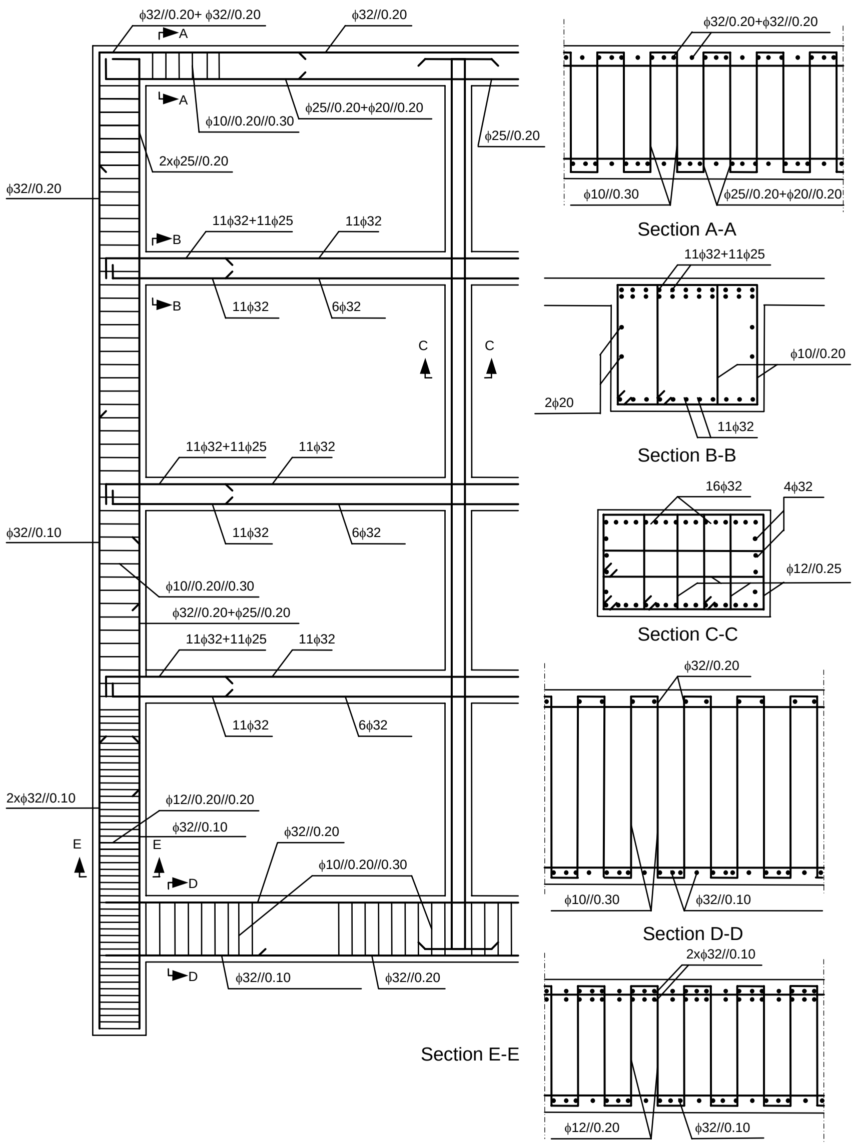


Figure 7-6. Reinforcement for maximum displacement according to Direct Design

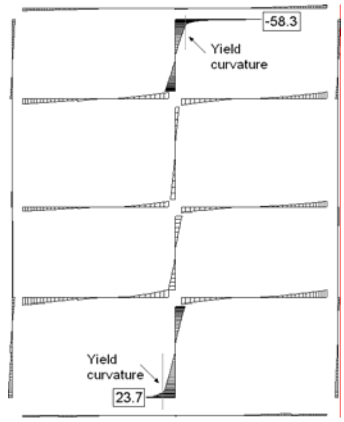


Figure 7-7. Curvature diagrams at maximum displacement – code design [/1000m]

7.6.2 Structure designed according to the proposed methodology

In order to maximize de ductility of the structure, Capacity Design principles must be applied.

7.6.2.1 Choice of deformation mechanism

The number and location of plastic hinges involves in general the choice of a partial or global mechanism (structure with fewer connections than necessary to maintain equilibrium). In structures that develop above ground the mechanism can be chosen by the designer, but in an underground structure it must compatible with the applied displacement profile. For the linear, sinusoidal or any other profile reasonably regular along the height (not the one shown in Figure 7-6 c) two main global mechanisms can be foreseen, as shown in Figure 7-8.

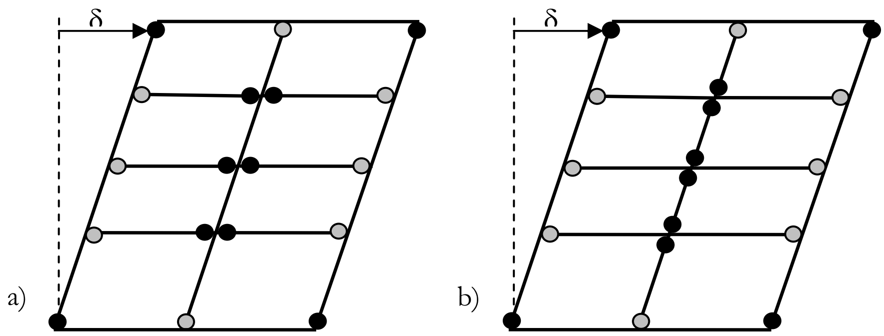


Figure 7-8. Example structure: global mechanisms

In what regards the choice of an appropriate mechanism it would be difficult to formulate standard recommendations for all cases. However, some considerations can be made, as follows. In nodes where elements with very different dimensions in the plan of the flexible alignments join, it may not be possible to choose the element in which the plastic hinge will develop. That is the case where beams or columns join slabs or perimeter walls. In general, it is very difficult to avoid that the hinges in the vicinity of this nodes develop in the beams or columns, as it is almost impossible to design these elements with more flexural capacity than the slabs or walls. This hinges are identified in Figure 7-8 by the grey colour.

In wall-slab or beam-column connections the location of the hinges is in general a designer's choice. Some criteria to support these choices can be considered. The bottom slab is usually a very thick element with considerable flexural capacity. It is therefore easier that at the connection with perimeter walls the hinges develop at the walls. At the wall-top slab connections the dimensions of both elements usually are not too different and the designer may be able to choose where to develop the hinges, as of the point of view of performance (maximization of the global ductility) both options can be acceptable. Therefore, two criteria can be used: easiness of construction and easiness of repair after a strong earthquake. The zone where the plastic hinge develops needs to be confined, what implies placing a large amount of reinforcement perpendicular to wall or slab faces to provide confining stresses in that direction. The horizontal reinforcement perpendicular to the thickness of the wall is probably easier to place than vertical reinforcement in the slabs. And since other plastic hinges develop in the perimeter walls (at the base and other locations, as will be shown later), the best options appears to be to locate the hinges in the walls. This allows maintaining the top slab elastic during strong earthquakes, avoiding the need to repair it afterwards.

A similar option about the location of the plastic hinges has to be done at the beam column joints. Note that the reasons why EC 8 prescribes the weak-beam/strong-column mechanism in building frames don't apply to underground structures: there is no need to avoid the soft storey mechanism since the deformation of the structure is conditioned by the surrounding soil and therefore large ductility demands and large second orders effects can not be triggered due to the soft-storey deformability. Another issue related with the choice of the hinges location at beam-column joints is the shape of the displacement profile imposed on the structure. If it is a profile similar to the one shown in Figure 7-5.c, it is impossible to avoid hinging at intermediate levels of the vertical elements, as shown in Figure 7-9. Note that even though in node 2 the designer can choose to locate the hinges in the beams or in the columns, in nodes 1 and 3 there is a variation of rotation between the columns converging on those nodes, which forces column hinging regardless of beam design.

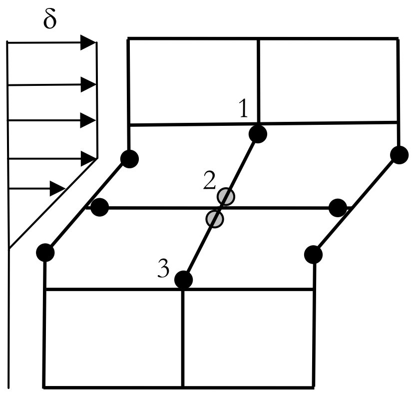


Figure 7-9. Mechanism with unavoidable hinges at intermediate locations of walls and columns

Since column hinging is unavoidable at the extremities and probably also at intermediate levels, it is the first option to consider and probably the most suitable. Another argument of practical nature in support of this option is that for the other actions the columns are essentially under axial compression, while the beams also have to withstand flexure and shear effects, leading in general to larger dimensions in the bending plan. However if the beams have similar dimensions to columns and larger aspect ratios it may be possible to provide more ductility to beams than to columns, leading to a larger deformation capacity for the structure. Another feature of behaviour highlighted in Figure 7-9 is that unless the soil characteristics are very uniform in the entire vicinity of the structure yielding can take place anywhere in the perimeter walls. Therefore, it may be necessary to provide confinement reinforcement throughout the perimeter walls.

Following the above discussion the example structure was designed to develop the mechanism shown in Figure 7-8 b and the perimeter walls were confined at all locations in order that a reasonable curvature ductility is available at any location.

It is worth to emphasize that the different constraints to the choice of the best mechanism in underground structures as compared to building frames lead to criteria different from the ones prescribed in EC 8 for those structures.

7.6.2.2 Design of reinforcement

The starting point for this phase is the structure as designed to resist to all other actions but the seismic action. According to Capacity Design principles the zones chosen to remain elastic must be designed to do not yield during the development of the plastic hinges. This implies these zones must be provided with enough reserve strength for that purpose. The plastic hinge zones must be designed for ductility as well as to avoid any brittle type of failure. Considering the chosen mechanism the main implications for the different structural elements are as follows:

- perimeter walls: it is not necessary to increase the flexural capacity as hinges are expected to develop at the walls (remind that the proposed methodology is equivalent to consider $q=\infty$). It is necessary to increase the available ductility throughout the walls: for this purpose confinement reinforcement, comprising horizontal links in the direction of the wall thickness and properly anchored at the extremities around the vertical reinforcement must be provided. Figure 7-11 shows the new design of the wall cross sections.
- slabs: the design for the other actions ensures that slabs are stronger then the columns to which they are connected. However the flexural capacity may need to be increased, particularly in the extremities of the top slab, to be higher than the maximum moment at the walls hinges, in order to avoid the formation of plastic hinges at the slab extremities. For this purpose at the extremities the slab is designed for a bending moment which is $Msd_{slab}=\gamma_0.Mrd_{wall}$, with both moments evaluated by the usual design procedure prescribed in EC 2. A value $\gamma_0=1.3$, as prescribed in EC 8 for column design, seems appropriate for the first iteration of the proposed design procedure. Figure 7-10 shows a longitudinal cut of the top slab.
- beams: in order to increase the ductility of the extreme sections where plastic hinges are expected to develop, confinement reinforcement must be provided at these zones. Flexural reinforcement on the lower face was also added in order to reduce the size of the compressive zone when the top reinforcement yields at beam extremities. The effectiveness of this extra reinforcement in increasing the curvature ductility can be easily evaluated by section analysis. In what regards interior beam column joints it was decided to develop the plastic hinges in the columns. Therefore in the first iteration the flexural reinforcement on the beams in the vicinity of these nodes provide an excess flexural capacity above the sum of the moments of resistance of the columns converging at the same node of 30%, what also depends on column design. However the analysis showed this was not enough. Figure 7-10 shows the new design of the beams. The beams were provided with more transverse reinforcement at the zones plastic hinges are expected to develop to increase the ductility of confined concrete.
- columns: since the columns are essentially under axial compression for all other loads, can be designed for that purpose with the minimum amounts of flexural and transverse reinforcement. Since the columns are not intended to remain elastic there is no need to increase their flexural capacity ($q=\infty$). However flexural reinforcement may be useful to decrease the ductility demand because of the following reasons: (i) to increase column stiffness relatively to the beams, in order to reduce the

restrictions that the beams impose to column rotations at beam-column joints, (ii) because large spacing of vertical reinforcement reduces the effectiveness of confinement, (iii) because the spacing of confinement reinforcement should be proportional to the diameter of the flexural reinforcement, therefore this should not be too small.

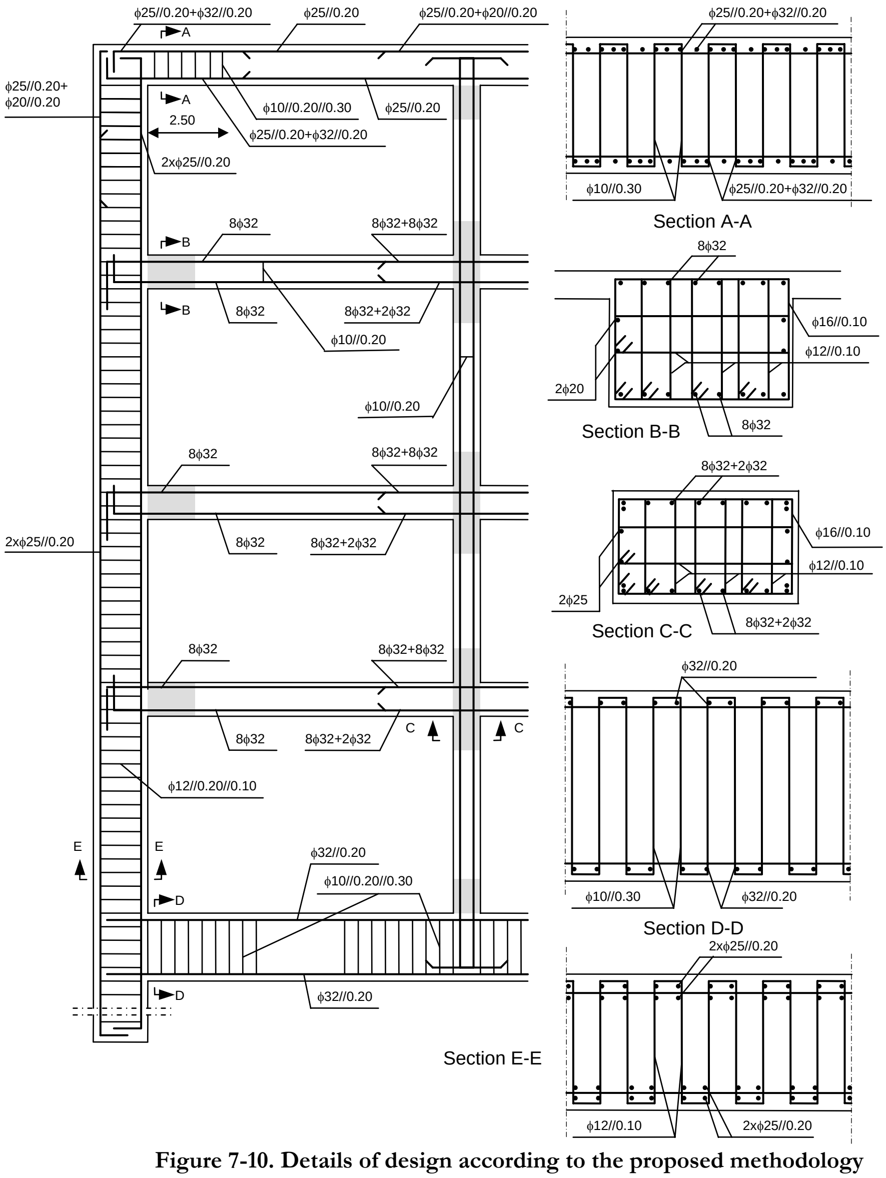
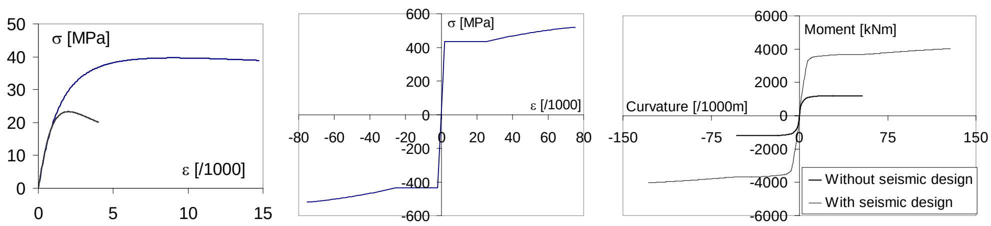


Figure 7-10. Details of design according to the proposed methodology

Besides there is the obvious need to provide confinement reinforcement in the plastic hinge zones to increase the available curvature ductility in those zones. The efficiency of the above can be evaluated by section analysis. Figure 7-11 shows the constitutive relationships for steel, confined and unconfined concrete and the moment curvature diagrams at the base of the columns before and after the increase in reinforcement, evaluated considering the axial force at maximum displacement.



a) Concrete b) Steel c) Column moment-curvature diagrams

Figure 7-11. Material constitutive relationships and moment-curvature diagrams at the column base section

It should be emphasized that the process of maximizing the overall structural ductility is an iterative procedure, that starts from the structure as designed to resist to all other actions. Successive analysis and changes were done in order to improve the overall ductility. The following examples highlight this procedure: (i) at each analysis the rupture point and other locations close to rupture were identified and the possibility of increasing the available ductility at those locations was analysed; this was the case at beam extremities that initially were all designed with 6 vertical stirrups ϕ12, that the analysis showed were not enough to prevent rupture at the beams, limiting the overall ductility of the structure; in the final design, at the extremities the beams were designed with 6ϕ12+2ϕ16 vertical stirrups; another change of this type was the use of external stirrups ϕ16 at the three lower column hinges; (ii) column flexural reinforcement was increased in order to increase its stiffness (according to the concept discussed in section 7.3 and illustrated in Figure 7-3, the amount of flexural reinforcement influences the member stiffness) relatively to the beams, to reduce the ductility demand on the columns; note that the increase in column flexural reinforcement also led to an increase in beam flexural reinforcement to avoid beam hinging but due to the curtailment of reinforcement, the stiffness of the beams increased less than the stiffness of the columns, in which there was no curtailment of flexural reinforcement; (iii) beam overstrength at beam-column joints was increased far above the initial value of γ_0=1.3, because the balance between beam moments on both sides of the nodes changed in the non-linear range increasing the moment demand.

The above is qualitatively different from current elastic analysis in which the designer knows the exact procedure that must be followed. The design for ductility leaves the designer with much more freedom but demands more knowledge and capacity to anticipate the potential seismic behaviour of the structure in order to decide at each iteration what are the most adequate changes to the design that resulted from the previous iteration.

7.6.2.3 Results

The non linear analysis of the structure designed according to the proposed methodology showed it could withstand a distortion of γ_{max}=14.6x10^{-3}, corresponding to a horizontal relative displacement between top and bottom of the structure of δ=32.9cm. Figure 7-12 shows the curvature diagrams at this stage.

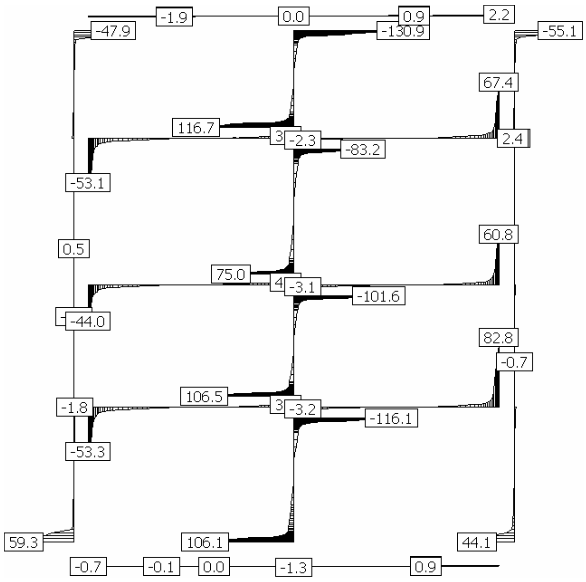


Figure 7-12. Curvatures at maximum displacement - proposed methodology [/1000m]

The comparison of this results with the ones of the structure designed according to current code concepts γ_{max}=5.0x10^{-3} shows the superior seismic performance of the structure designed according to the proposed methodology. The comparison between the curvatures at maximum displacement for both structures (Figure 7-7 and Figure 7-12) highlights the reasons for this difference: the higher ductility of the structural elements and the efficient exploration of that ductility throughout the structure designed according to the proposed methodology. A full comparison of costs cannot be done as the structure was not fully defined, neither was the constructive process. However, in terms of materials most of the difference regards the amount of steel in the perimeter walls. The proposed methodology leads to the use of less flexural reinforcement, but needs large amounts of confinement reinforcement, leading to almost equal total amounts of steel spent in the perimeter walls. In the slabs the proposed methodology leads to moderate savings, as the flexural reinforcement is conditioned essentially by the minimum levels prescribed in EC2. In beams and columns the general trend is similar to that observed in the perimeter walls, with some savings for the design of the columns according to the proposed methodology. The above indicates that in general the design according to the proposed methodology does not has a significant influence on the overall costs, and may even lead to slight savings in some elements.

7.7 SUMMARY AND CONCLUSIONS

During earthquakes underground structures do not have to resist to horizontal inertia forces, as structures that develop essentially above ground, but only to withstand the displacements the soil imposes on them without losing the capacity to resist to permanent actions. Therefore reinforced concrete structures must be designed to be flexible and ductile. For instances large underground reinforced concrete structures, such as tube stations, should be designed in the transverse direction with elements whose dimensions must be the ones strictly necessary to resist to other actions but the seismic actions. Stiff elements, such as counterforts or short beams should be avoided, as well as large soil covers. The interference of secondary or non-structural elements with the deformation of the main structure should be avoided.

The structure must be designed by stages: first for all load combinations whose main variable action is not the seismic action; second for the seismic action. Since there are no inertia forces (equivalent to consider the behaviour factor infinite) the designer must choose a suitable deformation mechanism and apply Capacity Design principles, this is, to design the potential plastic hinge zones for ductility and the remaining zones with excess strength to remain elastic. An application example is shown. The proposed procedure tends to lead to considerable savings in flexural reinforcement but more confinement reinforcement. In general terms it leads to structures with better seismic performance than the extrapolation of code procedures derived for structures that develop above ground, that may lead to unsafe underground structures. Therefore it is recommended that EC8 covers explicitly the seismic design of underground structures.

CHAPTER 7

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