16TH EUROPEAN CONFERENCE ON
EARTHQUAKE ENGINEERING THESSALONIKI
18 - 21 JUNE 2018

ROLE OF EEPS IN SEISMIC DESIGN AND PERFORMANCE OF THE EUROPEAN SPALLATION SOURCE TARGET BUILDING

Giovanni LI DESTRI NICOSIA[1]

ABSTRACT

Utilization of expanded polystyrene (EPS) to improve the static performance of earth retaining structures dates back to four decades but their application for seismic design maybe considered more recent. The present paper describes the role of the elasticized EPS (eEPS) in the design of the target building of the European Spallation Source (ESS) a new generation particle accelerator that is being constructed in Lund (Sweden). Several aspects are treated in detail. The reduction of earth pressure due to eEPS is described for both static (compressible inclusion function) as well as dynamic case (seismic buffer function). For the seismic conditions it is shown that the due to eEPS compressibility and movement, a reduction of earth pressure is calculated both following the approach included in EN1998-5 and elastodynamic methods and that in both cases similar results are obtained. In presence of cohesive backfill the Mononobe Okabe approach included in EN1998-5 is considered overly conservative and the pseudostatic method proposed by Iskander (2013) is used instead. The second aspect that is analyzed is related to inertial interaction forces: it is shown that the overall reduction of constraints on the building due to the presence of the eEPS along basements walls that can reach up to 5 m depth below ground and the consequent lengthening of the vibration period reduces the base shear force. Finally quantitative evidences and qualitative considerations are provided related to durability with respect to creep, chemical and environmental agents and radiations.

Keywords: eEPS; Retaining structures; Elastodynamic; EN1998-5

1. INTRODUCTION

The European Spallation Source (ESS) will be the most powerful linear proton accelerator ever built and is also the largest science facility being built today. Due to the criticality of the facility the design needs consider an earthquake return periods up to 1e6 years (H4 event). Required performance levels vary depending on the level of radiation to which the specific building structure is exposed to. In this work the focus is on the four target building structures represented in Figure 1, for which the performance level for an H4 event (having magnitude between 5 and 6) is no collapse.

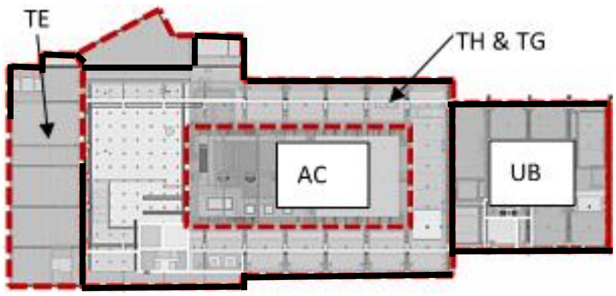

Figure 1. Buildings layout (red borders) and extent of eEPS (black borders)

[1]Senior Engineer Specialist, COWI BTM, Copenhagen, Denmark, glni@cowi.com

Description of the methodology and results of analysis and design for the soil pressure assessment using eEPS at the Utility Building (UB), transport Hall (TH), Technical Gallery (TG) and Target Entrance (TE) is included in the following paragraphs.

2. EEPS PROPERTIES

Elasticized Expanded Polystyrene (eEPS) is EPS that has undergone a uniaxial unloading-reloading cycle, resulting in an extended linear-elastic range for the material in one direction and with a reduced stiffness. Hence with relation to material durability of buried material including decomposition in soil, the eEPS manufacturer (TerraFlex, private communication) assures that eEPS has the same life as expanded polystyrene (EPS). Isochronous strain-stress plots of eEPS as those shown in Figure 2 below show that eEPS stress strain behaviour of the eEPS is loading and duration dependent, with stiffness reducing with increased loading time due to creep. Three subgrade moduli were calculated from three curves to represent SLS (1e4 hours), ULS (1e2 hours) and ALS (rapid loading) load cases.

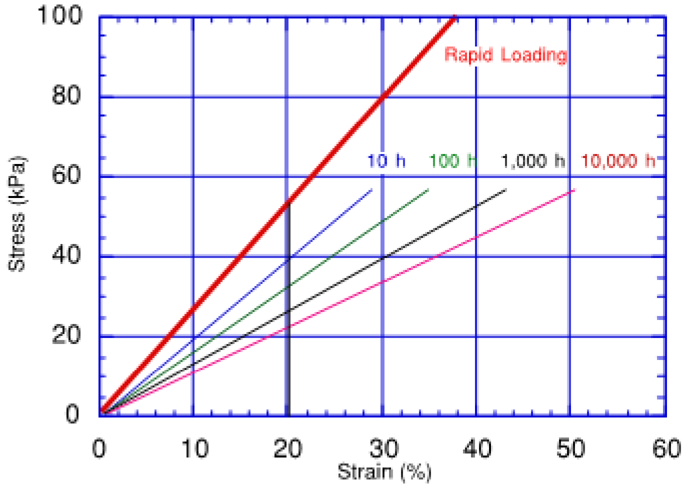

Figure 2. Stress strain behaviour of eEPS material (GeoTech Systems Corporation, 2003)

Manufacturer specifies that to retain linear elastic behavior of the compressible material, there is a requirement to limit strain below 20% for long term loading hence a strain versus time plot generated from Figure 2 is presented in Figure 3 below. Based on the data in Figure 3 a linear relationship can be assumed between strain and the log of time.

For an upper bound SLS earth pressure equal 17 kPa, found at 5 m depth, it is shown that the strain at 100 yrs (876e3 hours) will not exceed 12 %×1.7=20 % which is the elastic strain limit as stated by the manufacturer.

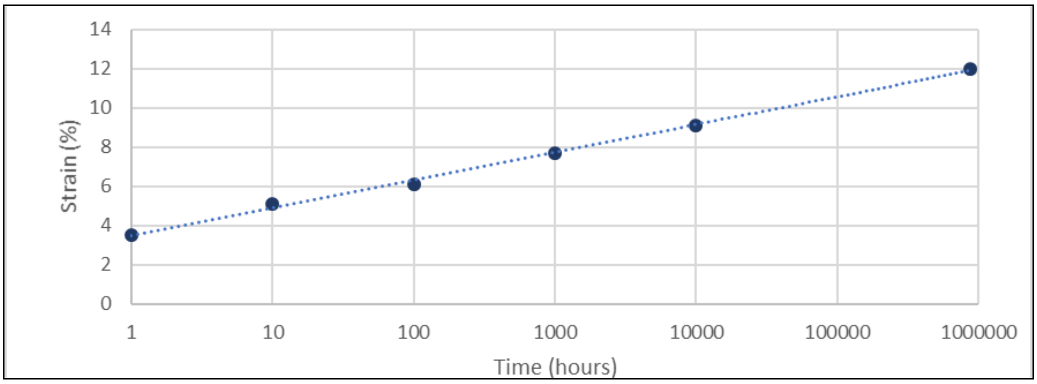

Figure 3. – Extrapolation of strain time plot for eEPS, stress=10 kPa

With respect to durability of buried material including decomposition in soil mentioned above, Norwegian Public Roads Authorities adopted the use of EPS in road embankments in 1972. A monitoring program has been undertaken over a 40 year period, with results summarized in (Aabøe, 2001) which states that "it may be fair to conclude that no deterioration effects are to be expected from EPS embankments placed in the ground for a normal life cycle of 100 years".
With respect to fire rating, the minimum concrete cover for durability means the basement walls generally have a fire rating of 120 minutes which, as a reference, is considered acceptable for retaining walls supporting lightweight fill in the British Columbia Ministry of Transport and Infrastructure “Supplement to Canadian Highway Design Bridge Code S6-14,” (British Columbia Ministry of Transport and Infrastructure, 2016).

3. EARTH PRESSURE UNDER STATIC AND DYANAMIC LOADING

3.1 Earth pressure coefficient for static loading and modified EN1998-5 method for dynamic loading

The low stiffness of the eEPS allows compression under lateral static soil pressures, resulting in the generation of horizontal displacement in the soil and concomitant shear strength mobilization within the retained soil mass (controlled yielding). This function sometimes referred to as compressible inclusion function (Horvarth, J.S., 2010) is shown in Figure 4 with a sketch of the proposed displacement mechanism. For a maximum 5 m basement height at UB building, the 73 mm eEPS/soil displacement (1.4%) has the effect of reducing lateral soil pressures from at rest to active.

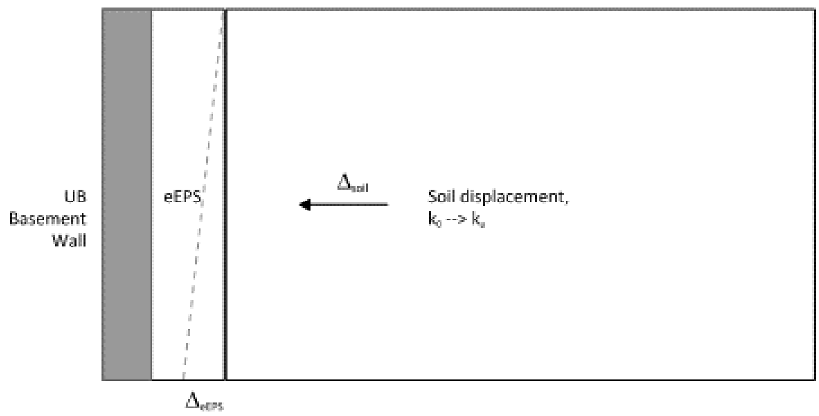

Figure 4. – Compressible inclusion methodology

The seismic analysis was undertaken mainly using a pseudo-static approach, where horizontal and vertical seismic effects were considered as a single static force or pressure applied to a soil mass. For seismic loading the deformation of the eEPS material due to lateral soil pressures enables them to function as seismic buffers (Horvarth, J.S., 2010)) leading to a reduction in the seismic pressure which, in this framework, means a reduction in the pseudo-static seismic coefficients that are applied to the system in analysis. This reduction is calculated in accordance with the procedure set out in EN1998-5 which defines

$$k_h=\alpha\times S_h/r \tag{1}$$

$$k_v=\alpha\times S_v/r \tag{2}$$

Where k_h (k_v) is the peak horizontal (vertical) seismic coefficient, αS_h (αS_v) is the horizontal (vertical) Peak Ground Acceleration (PGA) for geotechnical analysis (in g) derived from Site Response Analysis (SRA) and where r is a reduction factor selected based on wall type and allowable displacement (see EN1998-5 Table 7.1). Un-factored and factored design PGAs for the target buildings basement walls with eEPS are presented in Table 1 below.

Table 1. Un-factored and factored design PGA.

Direction	Unfactored PGA from SRA, αS	Reduction factor, r	Factored PGA, $\alpha S/r$
Horizontal, h	0.73 g	2	0.37 g
Vertical, v	0.33 g	2	0.17 g

The allowable displacement required to use the (maximum) reduction factor of 2 is calculated as: $S_r=300\times\alpha\times S_h$ (mm)=219 mm. This amount of compression of the eEPS is acceptable for short term loading hence the target buildings basement walls can be designed allowing the use of the largest PGA reduction factor, r=2.

The Mononobe-Okabe (Okabe, 1924; Mononobe and Matsuo1929, 1932) method for calculating seismic earth pressures for a flexible wall is referred to in Eurocode 8. This method is considered overly conservative for this site as it applies to cohesionless materials only. To account for the cohesion in the clay backfill, an alternative method was adopted for calculating the dynamic lateral earth pressure.

The alternative method employed was the procedure set out in Iskander et al. (2013). A Mathcad calculation sheet was set up to perform the calculation for the seismic lateral earth pressure coefficients and corresponding earth pressure and thrust.

Consistently with the linear pressure distribution case with no tension crack presented in the Iskander paper [8], a triangular seismic soil pressure distribution was assumed.

The Iskander based parameters influencing the calculated seismic pressure are the soil-wall friction angle, α, and the seismic lateral earth pressure coefficient, k_{ae}. Both parameters are calculated as a function of the maximum wall height.

The values of k_{ae} and α relating to a maximum retained height of 5m (at UB) were selected as design values therefore any basement wall heights below this would have a conservative estimate of seismic lateral soil pressure. For depths greater than 5m below ground level, strain is expected to exceed 20% therefore some plastic behaviour of the eEPS is expected. However this case occurs only in a limited area below the corner of the base slab at the UB lift pit and cannot have any relevant effects on the structural response.

3.2 Comparison between modified EN-1998 5 method and elastodynamic methods

A parallel, alternative, calculation was undertaken for the UB wall using the elastodynamic method outlined Psaropoulos et al. (2005) to check the modified EN-1998 5 seismic pressures.

This elastodynamic method considers an un-reduced PGA (α_0) at wall base and accounts for the rotational relative wall/soil stiffness (d_θ) and for the translational relative wall/soil stiffness (d_W) [7]. For the determination of d_θ as suggested by (Horvath, 1998) the methods were applied assuming that pressures generated due to the rotational displacement of a wall rotating around the base are equivalent to pressures generated due to the linear displacement (compression) of eEPS due to soil pressured. Similarly was assumed considering a translational displacement for the determination of d_W. The maximum values considered by [7] for d_θ and d_W are lower than the ones calculated and provide a safe (upper bound) estimate of the wall pressure.

For the case of a 5m wall at UB it is observed that while the (modified) EN1998-5 estimate of the maximum pressure with r equal 2 is 34 kPa, for Psaropoulos et al 2005, assuming α_0 equal to the PGA at bedrock it is approximately 17+14=31 kPa. Moreover in both cases a similar shape (triangular) of soil pressure distribution is obtained.

4. BENEFICIAL EFFECTS ON OVERALL SEISMIC RESPONSE

As an illustrative example the TH and TG buildings only are considered below. The contribution of the compressible fill material for the overall lateral foundation stiffness depends on the direction of motion of the building. For instance, compressible fill material installed along the East walls is active when the motion of the building is eastwards only. The compressible eEPS fill subgrade modulus is k_{cf} = 0.5 MN/m^3 is obtained as the ratio between the soil pressure divided by the expected displacement.
Knowing the number of 120 mm diameter and 180 mm diameter steel core piles supporting the TH & TG buildings and knowing the (upper bound) value of their dynamic stiffness (Li Destri Nicosia et. al, 2017) a total stiffness of 2853 MN/m is found.
The ratio of the compressible fill stiffness to the total lateral stiffness due to SC piles considering the most unfavourable motion (towards North) results in a ratio of $220.59 \times 0.5 / 2853=0.039$ which is less than 4%. Therefore, it may be concluded that the dynamic properties of the structural components of target building are only marginally affected by the consideration of the additional stiffness due to the compressible fill material. In fact, the increase of vibration periods of modes dominated by foundation response is maximum $\sqrt{1.04} \approx 2.0\%$ from which marginal increase of inertia demand is expected.
Considering that the in situ soil and the backfill comprise approximately 10 m of hard clay till overlaying bedrock if eEPS was not used the subgrade modulus would increase drastically. Considering a 15 mm displacement (obtained neglecting soil constraint on basement walls) based on Eurocode7 (Figure C3) an approximate estimate subgrade modulus is 6.5 MN/m^3, a 13 fold increase which would imply a 52% increase in stiffness and a 23% increase in vibration period with a very significant increase in inertial forces and base shear.

5. CONCLUSIONS

The present study describes the main advantages of using eEPS for seismic design of earth retaining structures, based on a recent seismic design experience of the ESS project. where an innovative use of eEPS was carried out. Provision of Eurocode 8 Part 5 are shown to be conservative compared to a recent solution for active earth pressure under seismic loading proposed by Iskander et al. (2013). Comparison of earth pressure estimated by mean of the elastodynamic solution proposed by Psarropoulos et al. (2005) shows that agreement is obtained for conservative values of rotational and translational relative wall/soil stiffness. Although this is expected as Eurocode 8 Parts 5 does not specifically address the effect of compressible fill, it also shows that Eurocode 8 Parts 5 is conservative also on this respect and that the coefficient r for eEPS applications could be even higher. Finally it is shown that in presence of very stiff soil the structural period lengthening effect of the eEPS can be very relevant.

6. REFERENCES

Aabøe R., Frydenlund T. E, 2011, “40 years of experience with the use of EPS Geofoam blocks in road construction”, International conference EPS 2011 Geofoam Blocks in Construction Applications, June, Lillestrøm, Norway.

British Columbia Ministry of Transport and Infrastructure, “Supplement to Canadian Highway Design Bridge Code S6-14,” 2016

European Committee for Standardization, “Eurocode 7: Geotechnical design - Part 1: General rules,” European Committee for Standardization, 2005.

European Committee for Standardization, “Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects,” European Committee for Standardization, 2004

GeoTech Systems Corporation, GeoTech Systems Corporation, 31 08 2003. [Online]. Available: http://www.geosyscorp.com/noframes/technical/terraflexdocs.htm. [Accessed 03 2017].

Horvath J. Lateral pressure reduction on earth-retaining structures using geofoams: correcting some misunderstandings. ASCE Earth Retention Conference 3 2010: 862-869. DOI: 10.1061/41128(384)86.

Iskander, M., Chen, Z. C., Omidvar, M., Guzamn, I., Elsherif, O., 2013. Active static pressure for c-ϕ soils. Soils and Foundations 53 (5), 639–652

Li Destri Nicosia, G., Nicolai, G., Schow Jensen, A., 2017. "Foundation Dynamic Stiffness for Pile groups of non homogenous diameters and lengths". SMiRT24, International Conference is Structural Mechanics in Reactor Technology, Busan, South Korea

Mononobe, N., Matsuo, M., 1932. Experimental Investigation of Lateral Earth Pressure During Earthquakes. Earthquake Research Institute and Research Office of Public Works, pp. 884–902.

Mononobe, N., Matsuo, O., 1929. On the determination of earth pressure during earthquakes. In: Proceeding of the World Engineering Congress, vol. 9, Tokyo, Japan, pp. 179–187

Okabe, S., 1924. General theory on earth pressure and seismic stability of retaining walls and dams. Journal of Japanese Society of Civil Engineers 10 (6), 1277–1323

Psarropoulos,P.N.,Klonaris, G., Gazetas,G., 2005. Seismic earth pressures on rigid and flexible earth retaining walls. Soil Dynamics and Earthquake Engineering 25, 795–809.