Design and installation of buried large diameter HDPE pipelines in a coastal area

Project et installation de tuyaux enterrés de grand diamètre en zone côtière

Bellezza I., Mazzier F., Pasqualini E.
Dep. SIMAU – Università Politecnica delle Marche, Ancona, Italy

D'Alberto D., Caccavo C., Serrani C.
SPS - Società Progettazione Servizi s.r.l., Ancona, Italy

ABSTRACT: The present paper deals with the main geotechnical aspects of the design and installation of two adjacent HDPE large diameter pipelines along the Adriatic Sea (Italy) coastline. The pipelines -270 m in length and 2 m in diameter - are conceived as buried collectors of polluted runoff water, to convey to sanitation prior to discharge into the sea. Considering that pipes are below the water table uplift analysis is detailed, showing three possible approaches in static conditions, whereas in seismic conditions a method is proposed that include the build-up of pore-water pressures during earthquake. As far as prediction of vertical deflection is concerned, the backfill loosening due to sheet piles extraction has been modelled by assuming no compaction (dumped backfill). Despite this assumption, theoretical short term deflection represents a lower bound of measured deflections.

RÉSUMÉ : Cet article décrit les principales problématiques géotechniques du projet et de l'installation de deux tuyaux enterrés adjacents de grand diamètre sur le littoral adriatique italien. Les tuyaux – longs de 270 m - ont la fonction de collecteurs enterrés pour l'eau de ruissellement polluée, pour permettre sa dépuration avant du déchargement dans la mer. En considérant que les tuyaux se trouvent au-dessous du niveau de la nappe d'eau on a analysé le problème du possible soulèvement en conditions statiques utilisant trois différentes méthodes. En conditions sismiques on a proposé une méthode qui considère le développement de pressions interstitielles positives excessives durant le tremblement de terre. L'ovalisation du tuyau a été calculée par une méthode de littérature en considérant un remblai sans compactage pour tenir compte de l'extraction des palplanches utilisées pendant l'excavation de la tranchée. Les valeurs calculées de l'ovalisation initiale représente un limite inferieur de l'ovalisation mesurée.

Keywords : uplift, pipe of large diameter, deflection

1 INTRODUCTION

Urban and infrastructural development often involves vulnerable areas such as coastlines. To prevent pollution of the sea from runoff water of a nearby urban area and crowded roads a system of buried collectors are to be built along a stretch of the Italian shoreline of the Adriatic Sea. In such a way, the collected runoff water will be conveyed to sanitation before discharging into the sea. A preliminary hydraulic study allowed to define different drainage basins, and for each basin an adequate collector is required. This paper deals with the design and execution of the first part of the system, concerning a collected water volume of about 1300 m^3, for which the designers opted to realise the collectors by two adjacent buried pipelines of 2000 mm in internal diameter and 270 m long.

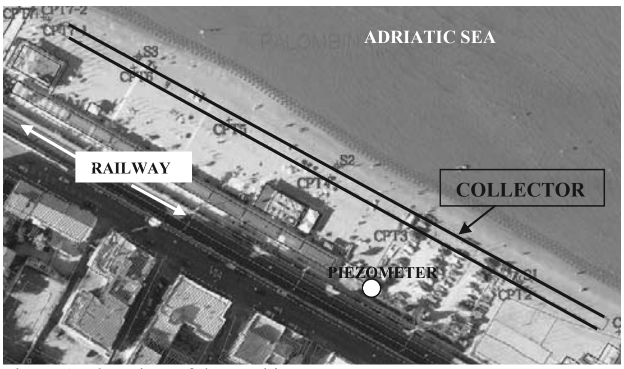

Figure 1. Plan view of the working area

The design and execution of geotechnical works in coastal area must in general face regulatory requirements, environmental and aesthetic concerns, public attention.

As far as engineering problems are concerned, the designer must take into account the objective difficulties connected with the critical location (e.g., underwater excavation, tidal and storm waves, risk of uplift). For the specific case, additional constraints are represented by the closeness of the working area to an important railway (Figure 1), and consequent limited accessibility for materials and machinery. Moreover, local authorities required to minimize the working area, avoiding the occupancy of the beach for the overall length of the collectors through the entire duration of the works. To comply with this requirement, a staged execution was envisaged.

2 OUTLINE OF THE DESIGN

2.1 *Pipe material*

Corrugated HDPE pipes were selected (Table 1). HDPE offer in general significant advantages in terms of costs, corrosion resistance, ease of handling and jointing over more traditional materials such spheroidal cast iron. The selected HDPE pipes are manufactured in modules 6.0 m long (less than other materials, e.g., cast iron) which allowed solving the problem linked to area accessibility. In particular with the use of 6 m long modules it is possible to reach the beach area passing through a narrow railway underpass. For longer pipe modules, a more expensive marine transportation would be requested.

Table 1. HDPE Pipe properties

Parameter	
Weight for unit length (kN/m)	2
External diameter (mm)	2240
Internal diameter (mm)	2000
Moment of Inertia (mm^4/mm)	45899
Young modulus short term(MPa)	1185
Young modulus long term (MPa)	288

2.2 *Underwater excavation*

The soil stratigraphy is essentially composed by a sandy layer 3.5-4.0 m thick overlying a cohesive bed. An open standpipe piezometer installed close to the working area (Fig.1) indicated that the groundwater table is located 1.0-1.5 m below the ground level. Fig.2 shows a typical CPT profile with the characteristic values of geotechnical parameters obtained by laboratory and in situ tests.

Considering the large pipe diameter, the bedding layer and a minimum soil cover to counteract buoyancy (as detailed later), an excavation depth of at least 4.70 m was necessary. Moreover, a minimum inclination of 0.5% to the horizontal is required to ensure gravity flow. This results in an excavation depth ranging form 4.7 m to 6.0 m from ground surface.

Various techniques were considered for the excavation. Unsupported trench with inclined sidewalls was excluded due to excessive breadth to ensure stability and the need for continuous dewatering by well-points. Other equally suitable technologies, (e.g., soil freezing), were incompatible with the budget.

The selected solution consisted in a 6.1 m wide trench supported by strutted sheet piles, embedded in the impervious clay layer. The total length of the sheet piles varied between 8 m and 10 m as depending on excavation depth. Sheet piling allowed retaining the vertical trench walls, minimizing seepage into the trench and protecting the working area from tidal and storm waves (the top of sheet piles was +1 m above g.l., Fig. 2).

To comply with the requirement of minimizing occupation of the area, the installation of the collectors (270 m) was realized in four distinct segments (i.e. the excavation in a zone starts only after the work in the previous zone is completed). For the first segment, sheet piles were preliminarily installed to enclose a rectangular excavation zone, creating a continuous barrier to groundwater along the entire perimeter. For the subsequent segments, the presence of the installed pipes prevented to create rectangular hydraulic barrier by sheet piles only. Therefore, cast-in-place concrete waterproof screens were designed around the protruding edge of the pipes to block seepage due to extraction of sheet piles from the adjacent completed segment.

2.3 *Pipe uplift*

During the service phase the pipelines are expected to be only rarely filled by runoff water but permanently submerged by groundwater and then subjected the buoyancy. Consequently, the design shall be checked against failure by uplift.

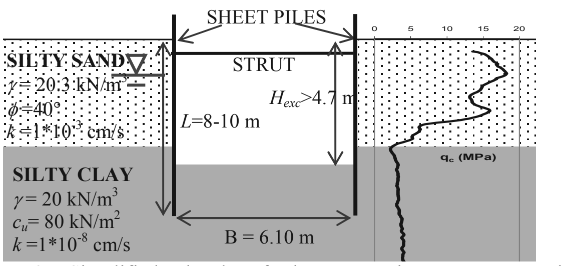

Figure 2. Simplified sketch of the excavation geometry with geotechnical soil characterization and a typical cone resistance profile.

According to Italian Building Code (NTC, 2008), as well as Eurocode 7 (2004), for any mass potentially subjected to the failure mechanism, the following inequality must be satisfied:

$$V_d \leq G_d + R_d \tag{1}$$

where V_d is the design destabilizing action acting upwards (obtained by a partial factor η_1 = 1.1 in static conditions), G_d is the design stabilizing permanent action including the weight of the mass subjected to uplift (obtained by a partial factor η_2 = 0.9 in static conditions) and R_d is the design soil resistance by friction along the vertical contours of the assumed block.

Considering the closeness of pipes to the sea (Fig. 1) it can not be excluded that in the future a portion of the soil above the pipe can be eroded. To confer protection against erosion a cast-in-place concrete slab (6.05 m wide and variable thickness) is realized above the pipes, as illustrated in Figure 3. This solution allows also to increase the average unit weight of the material above the pipes and enlarge the size of the block subjected to uplift. Finally, it represents a protection against accidental damage due to anthropic activities and the superficial sand layer enables to continue the recreational use of the beach.

In the application of Eq. (1) different approaches can be adopted to calculate the term V_d and in G_d. Eurocode 7 indicates a total stress analysis for uplift problems (EC7, 2004 §10.2). According to this approach, V_d is the upward resultant of pore water pressure acting on the *lower* boundary of the assumed block. Consistently, G_d includes the *total* weight of the soil block above the pipes. However, following this approach, the resultant of pore water pressure acting downwards is multiplied by a partial safety factor (η_1 =1.1) different to that applied to the vertical upward resultant (η_2 = 0.9). This results in a violation of the "single source principle" enunciated by Eurocode 7 (EC7, §2.4.2). According to this principle, when destabilising and stabilising permanent actions come from a single source, "a single partial factor may be applied to the sum of these actions or to the sum of their effects". Based on the above considerations, in the second approach the destabilizing action is assumed to be the buoyancy force on the two submerged pipes (i.e. the weight of the water displaced by the pipes W_w). Consistently, G_d includes the submerged weight of the block above the pipes. Finally, a third approach can be used in which the destabilizing action is assumed to be the *resultant* buoyant force of the submerged pipes, i.e. the algebraic sum of weight of displaced water W_w and weight of pipes W_p (WSSC, 2008). This latter approach implies that the check against failure by uplift is automatically verified when $W_w < W_p$.

The three approaches described previously are applied assuming the simplified sliding surface shown in Fig. 3, which implies a failure mechanism involving pipes, slab and soil above and between the pipes as well (hatched zone in Fig.3). The results were obtained for the worst-case scenario of complete erosion of the superficial sand layer (h_3 = 0 in Fig 3) and minimum cover thickness above the pipes ($h_1 + h_2$ = 0.5 m, s = 0.6 m). The unit weight of concrete and saturated soil were 23.5 kN/m^3 and 20.3 kN/m^3 respectively.

The R_d term was calculated as the sum of the friction forces along the vertical planes on each side of the assumed block (BC, B'C', DE, D'E')

$$R_d = \gamma' K_s \frac{\tan\delta_{BC}}{\eta_3} s^2 + \gamma' K_s \frac{\tan\phi_k}{\eta_3}\left[(s + h_1 + h_2 + R_e)^2 - s^2\right] \tag{2}$$

where δ_{BC} is the interface friction angle between concrete and the sandy soil (δ_{BC} = 30°), ϕ is the shear resistance angle of the granular backfill, η_3 = 1.25 is the partial safety factor applied to the shear strength parameters. The angle ϕ_k after backfilling is assumed to be 40°- 42°, but a reduced value of 38° is assumed in eq. (2) owing to potential loosening induced by sheet pile extraction. K_s is the lateral earth pressure coefficient assumed conservatively equal to 0.5 by neglecting the effect of compaction.

Results of static analysis of uplift failure (Table 3) indicate that, as expected, the second and third approaches are less conservative in static conditions; however they appear to better represent the physical reality (i.e. the destabilizing force on a fully submerged pipe is independent of the depth below the water table).

In seismic conditions, Italian Building Code (NTC, 2008 §7.11.1) requires that Eq. (1) shall be checked using $\eta_1 = \eta_2 = 1$, which result in uniqueness of the approach in seismic analysis (i.e. the difference between G_d and V_d is the same using a total stress analysis or an effective stress analysis).

Using a pseudo-static approach the vertical inertial force F_V acting on pipes, soil, concrete is assumed upwards and proportional to relevant weights by the seismic coefficient k_v (=0.046) defined in Italian Building Code (NTC, §7.11.3.5.2).

It is well recognized that in the presence of earthquake, a build-up of pore water pressures can occur with respect to static conditions. A simplified procedure to account for this phenomenon is the introduction of the pore-pressure coefficient $r_u = \Delta u/\sigma'_{v0}$ assumed constant with depth (e.g. Ebeling and Morrison, 1992; Kramer, 1996). Accordingly, the unit weight of water and submerged soil are given by:

$$\gamma_{we} = \gamma_w + r_u\gamma' = \gamma_w + r_u(\gamma_{sat} - \gamma_w) \tag{3}$$

$$\gamma'_e = \gamma'(1 - r_u) = (\gamma_{sat} - \gamma_w)(1 - r_u) \tag{4}$$

For r_u = 1 soil liquefaction occurs which implies that (a) upward action acting on pipes is proportional to γ_{we} (= γ_{sat}) instead of γ_w. and (b) the submerged weight of the soil block and soil resistance R_d vanish.

In seismic analysis the coefficient r_u shall be selected on the basis on seismic input (magnitude and maximum acceleration), as well as soil characteristics. In the analyzed case the presence of a coarse backfill (gravel) around the pipes (Fig.3) is expected to strongly limit the build-up of pore water pressure. Hence, the seismic analysis of uplift are performed assuming r_u = 0 and r_u = 0.1. Results of seismic analysis shown in Table 3 indicate that also for r_u = 0.1 the inequality (2) is satisfied.

2.4 Pipe deflection

Flexible conduits fail by excessive deflection rather than by rupture of the pipe wall. It is necessary, therefore, to estimate the deflection of this type of conduit and to establish limits of allowable deflection for the proposed installation.

Table 3. Uplift analysis

Action (kN/m)	Static conditions			Seismic conditions	
	appr.1	appr.2	appr.3	r_u = 0.1	r_u = 1
V_d	174.1	86.6	82.3	86.6	159.9
W_{pipes}	4.0	4.0	-	4.0	4.0
W_{soil}	87.6	44.4	44.4	40.0	0
W_{slab}	85.3	49.0	49.0	45.3	11.6
F_v	0	0	0	-8.1	-8.1
G_d	159.2	87.7	84.1	81.2	7.5
R_d	15.6	15.6	15.6	13.4	0
R_d+G_d	174.8	103.3	99.7	94.6	7.5

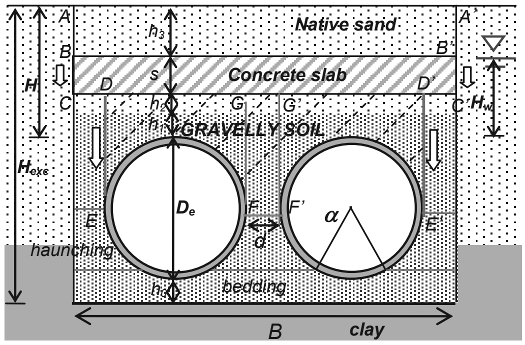

Figure 3. Details of pipe installation

For flexible conduits the vertical deflection Δy mainly depends on the actual load acting on the pipe and the stiffness of the backfill at the side of the pipes whereas the contribute of pipe stiffness is generally small (i.e. Rogers et al. 1995). In the specific case, the presence of two pipes as well as of the concrete slab makes the analysis more complex than the classical solutions available in the literature.

2.4.1 Load on pipes

According to Young and Trott (1984), the pair of pipes can be considered as equivalent to a single pipe of overall width D' where $D' = 2D_e+d$. The load on D' is calculated by taking the lesser of the two values obtained by the complete ditch condition (P_1) and the positive projection condition (P_2). This load is taken as being shared equally by the two pipes.

The load acting on a pipe (stiffer than side fill) in a trench with a partially submerged homogeneous backfill (see Fig.3) is given by Bulson (1985):

$$P_1 = \frac{B^2\gamma}{2\lambda}\left[1 - \exp\left(-\lambda\frac{2H}{B}\right)\right] - \frac{B^2\gamma_w}{2\lambda}\left[1 - \exp\left(-\lambda\frac{2H_w}{B}\right)\right] \tag{5}$$

where λ is a coefficient ranging from 0.11 to 0.19 depending on soil type, γ is the total unit weight of backfill. In the analysed case actual backfill unhomogeneity is accounted for by assuming a weighted average unit weight (γ_{av}).

For a positive projection conduit the P_2 value depends on the relative settlement between the soil prism above the pipe and the adjacent soil, which determines positive or negative arching. In the specific case, the presence of the slab prevents the occurrence of complete ditch or projection conditions. Moreover, considering that the ratio H/D' is very small, it is reasonable to neglect arching. Therefore, the value of P_2 is assumed to be the weight of overlying prism of width D'.

Obviously, the maximum deflection is expected to occur at the section with the maximum cover (3.30 m) with the lowest groundwater level (-1.60 m below g.l.). With reference to Figure 3, for h_1 = 1 m, s = 0.30 m, h_2 =1.7 m, h_3 = 0.3 m, H_w =1.7 m γ_{av} = 20.85 kN/m^3 λ = 0.19, P_1 and P_2 are calculated as 281 kN/m and 264 kN/m, respectively. Following the suggestion of Young and Trott (1985), the load acting on a single pipe (P) is taken as 132 kN/m.

2.4.2 Backfill

A large part of ability of flexible pipes to support vertical load must be derived from the passive pressures induced as the sides move outward against the soil. Therefore, any attempt to analyse the behaviour of the flexible conduits must take into account the soil at the sides as an integral part of the structure, since such a large proportion of the total supporting strength is attributable to the side material.

Considering that compactive effort is restricted by the geometry of the trench and the difficult in compacting underneath the pipe in the haunch zone (Fig.3), as well as the sensitivity of installed flexible pipe to compaction of material around it, a clean gravelly soil has been selected as structural backfill (i.e. the part of backfill that extends from the base of the bedding to a maximum of 30 cm above the pipe, as shown in Fig.3). This coarse-grained soil is preferred over native silty sand for easy of compaction, high earth pressure response and stability when saturated and confined. The same material – well compacted - has been used also as bedding soil (Fig.3).

2.4.3 Calculation of pipe deflection

The pipe deflection is predicted by the method of Spangler (1941) or Iowa formula, although it is well recognized that this method contains some debatable assumptions.

$$\Delta y = \frac{D_L \cdot K \cdot P}{EJ/R^3 + 0.061E'} \quad (6)$$

where Δy = vertical deflection of pipe (m); P = vertical load on the pipe (MN/m); EJ = flexural pipe stiffness (MNm^2/m); R = mean radius of the pipe (m); D_L= time-lag factor (-); K = bedding constant (K = 0.1 for bedding angle α = 60°, see Fig.3). E' = horizontal modulus of soil reaction (MPa).

Considering the absence of vehicular loading and the prevalent recreational use of the site, live loads have been neglected. In design the sheet pile extraction is accounted for using the value of *E'* relevant to a dumped backfill (Table 4). In long term analysis a time lag factor of 1.5 and a reduced pipe stiffness are considered (see Table 1). With the above assumptions short term and long-term deflections are calculated as 10.1 cm and 20.5 cm , respectively.

Numerous authors have reported that pipes have been distorted by 10-20% and still continue to perform adequately. Therefore the theoretical deflections have been considered acceptable, but a monitoring activity was planned during installation.

3 COMPARISON OF ACTUAL AND THEORETICAL DEFLECTIONS

The large diameter of pipelines allowed accessibility and direct measurement of vertical diameter at prescribed positions during the various stages of installation (structural backfilling, slab, final backfilling and sheet piles extraction). The trend of measured vertical deflection versus time is not monotonic, showing an initial small deflection, followed by a slight decrease, a sharp increase and a final stabilisation. The observed trend can be ascribed to the variation of the acting loads (backfill height and groundwater level) and the different lateral support offered by the soil before and after the extraction of sheet piles. Therefore, for the comparison between actual and theoretical deflections, only the stabilised values are considered because they better represent the service conditions of the pipes, with the groundwater level certainly above the crown of the pipes.

With reference to a pipe stretch 45 m long the vertical deflections were measured in sections spaced 3 m apart. Final (stabilized) deflections are shown in Fig. 4. In spite of a quite uniform cover height the measured deflections vary considerably along the pipeline with a minimum of 7 cm to a maximum of 15 cm. This behaviour can be attributed mainly to inherent differences in compacting the soil beside the pipes and variable effect of sheet pile extraction. Moreover, variability in stiffness of native soil can influence the overall response owing to the closeness of pipes to trench sides.

Considering that measurements refer to a design cover height ranging from 2.15 m to 2.37 m, the vertical deflection is calculated by (6) for an average cover height H = 2.26 m. The load on pipe (P = 104 kN/m) is calculated following the suggestion of Young and Trott (1984) discussed previously. The groundwater level was assumed at 1.6 m below the ground surface (H_w = 0.66 m) based on measurement in the nearby piezometer. As shown in Fig. 4, the theoretical deflection calculated by Spangler method is lower than the actual average deflection. This can be ascribed to effect of sheet pile extraction which results in a loosening of backfill and a probable increase of the load on pipes to due to negative arching.

Table 4. Values of *E'* for a clean coarse-grained soil (Howard, 1977)

Degree of compaction	dumped	slight	moderate
E'(MPa)	1.4	6.9	13.8

4 CONCLUSIONS

In the present paper some aspects of design and installation of two adjacent large diameter pipelines along the Adriatic Sea coastline in Italy are described.

Uplift analysis is detailed, showing three possible approaches which lead to different results in static conditions, whereas in seismic condition a unified approach is proposed that account for build-up of pore-water pressures.

As far as prediction of vertical deflection is concerned, in the analyzed case the backfill loosening due to sheet piles extraction has been modelled by assuming no compaction (dumped backfill). Despite this assumption, theoretical short term deflection represents a lower bound of measured deflections.

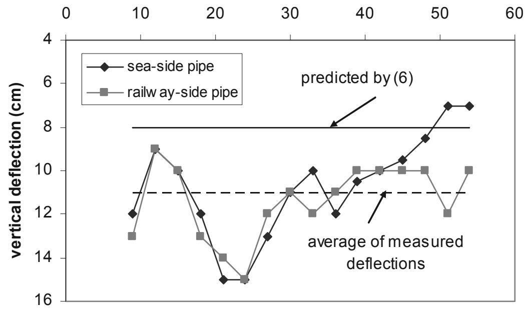

Figure 4. Comparison between measured and predicted short–term deflections

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