Fragility Functions for Pipeline in Liquefiable Sand: a Case Study on the Groningen Gas-Network

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ABSTRACT: Any pipeline network is of strategic importance for the country in which it is located, both because its correct functioning is of vital importance for today's functioning of the community and because any disruption, malfunctioning or rupture represents a hazard for the community. In this paper the fragility function for a simple segment of a high pressure pipeline installed in loose, saturated sand is investigated with respect to shallow earthquakes. These earthquakes can cause local permanent ground deformation due to liquefaction effects.

Since 1986, a low intensity seismic activity is present in the Groningen gas-field area (Netherlands), due to the tremors following the compaction of the gas reservoir due to stress decrease. An extensive study performed by the Dutch Meteorological Insititute (KNMI), see Dorst et al. (2013), shows that in the last decades (2003-2013), the seismic activity changed from low intensity activity with constant events rate per year to a higher rate with slightly increasing magnitude. The depth of the earthquakes is at 3 km, being the depth of the gas reservoir. On 16 August 2012 an earthquake with a local magnitude of M=3.6 occurred near Huizinge and it is the largest event that occurred until now.

A large pipeline network is present in the area affected by the induced earthquakes and it is a strategically important network for the Netherlands and Europe, representing an important node from which the natural gas is transported to several European countries. The importance of this network rises the need to investigate the fragility of this transportation network with respect to both seismic action and the effect of possible ground failure (liquefaction) due to the characteristics of the soft soil.

Fragility functions for pipelines are available in literature (O'Rourke et al. (2007) or Pitilakis et al (2010)) based on observational analysis of the performance of lifelines subjected to earthquakes of large magnitude. However, in this paper we want to investigate the performance of a pipeline segment subject to a seismic activity that is not of tectonical nature and is characterized by short duration of the signal, a local amplification and possible ground failure. Therefore, the effect of soil-pipe interaction in presence of loose saturated sand that is typical for delta-regions is considered. Furthermore, additional uncertainty is related to the definition of the fragility function in the ranges of magnitude above the maximum measured event (M3.6). This uncertainty is all of epistemic nature, since no observation of M>3.6 is available.

Our study aims to the definition of fragility curves for a high pressure pipeline, in absence of available data, and is based on the results of a fully probabilistic model that takes into account the performance of a gas-pipeline element with respect to seismic shake and the local response within the pipe-soil interaction.

1. SEISMIC ACTION AND GMPE

The Dutch Metereological Institute (Dorst et al. 2013), performed the data analysis and a Probabilistic Seismic Hazard Analysis (PSHA) with all data available until 2013. The Ground Motion Prediction Equation (GMPE) used for the prediction of the ground motion characteristics (peak ground acceleration and peak ground velocity), as function of the magnitude M_w and source-site distance R, is the GMPE of Akkar et al. (2013) recently derived on a large dataset that includes shallow and low magnitude events and a correction factor to take into account the faults typology and amplification for local seismic response in soft soil that makes it more suitable for the typology of the events in the Groningen area.

The general expression of the GMPE of Akkar et al. (2013) is reported in Eq.(1), while we refer to Dorst et al. (2013) for the specific expressions and the coefficients to be used. In Eq.(1) the ground motion characteristics X (peak ground acceleration or peak ground velocity) is a function of the ground motion parameter X_{ref} at bedrock (that depends on magnitude, fault geometry and source-site distance), of the parameter S function of velocity of propagation of shear waves at the considered site, of σ , the standard deviation of the lognormal distribution of X and ϵ a standard normal error. The standard normal error between peak ground acceleration and peak ground velocity is herein considered strongly correlated.

$$\ln X = \ln(X_{ref}) + \ln(S) + \epsilon\sigma \qquad (1)$$

2. SOIL-PIPE INTERACTION

Generally, the behavior of a pipeline segment during an earthquake event, depends on the earthquake intensity and on the material and geometry characteristics of the pipe but also on pipe placement technique and soil property of the more superficial layers.

Buried pipelines are subject to deformations due to the effect of the shear waves (s-waves) generating mainly horizontal oscillation with a certain period and amplitude. The soil deformation is transferred to the pipe to a degree that depends on the soil-pipe interaction and interface. However, the dynamic effect of the s-waves is not so severe for a large pipe section of high steel grade, while more severe effects can be generated by permanent ground deformations due to soil liquefaction.

The term “liquefaction” indicates a phenomenon for which a saturated and zero cohesion soil loses its shear resistance due to the accumulation of plastic deformations caused by transient and cyclic force actions in un-drained conditions. Indeed, the development of excess pore water -pressure reduces the effect of in situ confinement of the soil. Sand boils, cracks and lateral spread phenomena are a sign of liquefaction. When liquefaction occurs the strength of the soil has nearly vanished.

Liquefaction is a phenomenon that arises only when a seismic event has an intensity that can induce such deformations in the soil that can generate a significant increase of the neutral pressure and when the soil shows significant degradation of resistance properties under cyclic load. Therefore, liquefaction occurs only for earthquake events of certain magnitude and durations. In addition, it can occur only in saturated non-cohesive soils (sand), with low plasticity index and low relative density.

2.1. Liquefaction

2.1.1. Potential of liquefaction

The most used approach to evaluate if a soil at a certain location can show liquefaction due to seismic shake was developed in 1971 by Seed and Idriss (see Idriss et al. 2008). This simplified method is of semi-empirical nature and was developed on the basis of the comparison between mechanical properties of the soil and the occurrence of the liquefaction event. The mechanical properties of the soil are evaluated by means of in-situ tests. The effect of the earthquake is modelled through the expected maximum acceleration at the ground-level with a certain probability of exceedance for a certain return period. The acceleration needs to be multiplied by the importance factor of the structure to obtain the design value of the peak acceleration.

The method is based on the comparison between the effect of the seismic shake, expressed as Cyclic Stress Ratio (CSR), and the capacity of the soil given by the Cyclic Resistance Ratio (CRR). Both can be derived from graphical abacuses or computed with semi-empirical equations (e.g. see Figure 1).

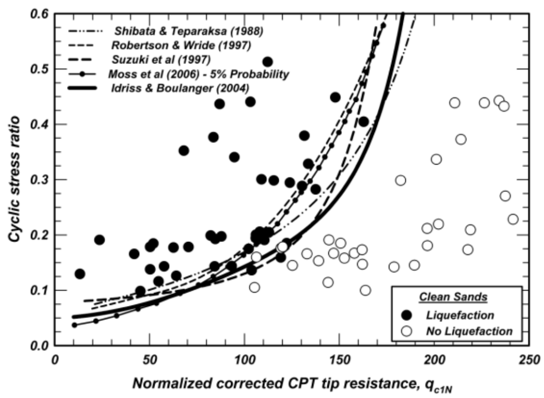

Figure 1: Abacus for CSR vs normalized CPT resistance from Idriss-Boulangier (2008).

The cyclic stress ratio at the depth z_i is computed according to the expression in Eq.(2)

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_v} r_d \frac{1}{MSF} \quad (2)$$

Where a_{max} is the peak acceleration at ground level, g is the gravity acceleration, σ_v and σ'_v are respectively the total static vertical stress and the effective vertical stress at depth z_i, 0.65 is a reduction for the irregularity of the seismic record. The factor r_d is a reduction factor that takes into account the reduction of the effect of the seismic shake with the depth of the layer and deformability of the soil and is a function of both depth and (moment) magnitude of the seismic event. The semi-empirical expression for r_d is given in Eq.(3).

$$r_d = \exp\left[\left(-1.012 - 1.126\sin\left(\frac{z}{11.73} + 5.133\right)\right) + \left(0.106 + 0.118\sin\left(\frac{z}{11.73} + 5.142\right)\right) M\right] \quad (3)$$

The factor MSF is a Magnitude Scaling Factor that corrects the CSR value for events of moment magnitude different from M=7.5. Indeed, the expression of CSR was derived on a dataset collecting events of magnitude between 5.9 and 8.3. A correction was applied to get an equivalent value for Mw=7.5.The expression for MSF is in Eq.(4).

$$MSF = \begin{cases} 6.9\exp\left[\frac{-M}{4}\right] - 0.058 \\ MSF \leq 1.8 \end{cases} \quad (4)$$

The evaluation of the resistance of the soil in the method of Seed-Idriss is based on in-situ tests such as SPT (standard penetration test) and CPT (cone penetration test) and the measure of the propagation velocity of shear waves Vs.

In this paper the results of CPT tests will be considered, in which case the Cyclic Resistance Ratio is given in Eq.(5).

$$CRR = K_\alpha K_\sigma CRR_{\alpha=0,\sigma=1} \quad (5)$$

Where $\mathrm{CRR}_{\alpha=0,\sigma=1}$ is the value of CRR for low stress state and horizontal ground level and a reference stress of 100kPa (1bar) (see Eq.(6)), K_α is the correction coefficient for the slope of the ground and K_σ is the correction coefficient for the stress state.

$$CRR_{\alpha=0,\sigma=1} = \exp\left[\frac{q_{c1,n}}{540} + \left(\frac{q_{c1,n}}{67}\right)^2 - \left(\frac{q_{c1,n}}{80}\right)^3 + \left(\frac{q_{c1,n}}{114}\right)^4 - 3\right] \quad (6)$$

Since $\alpha = \tau_{st}/\sigma'_v$, i.e. equal to the ratio between tangential stress and vertical effective stress, K_α is defined by an exponential expression function of the relative density, angle α and quality of the sand (quarz, feldspar, chalk etc.) with coefficients having a polynomial expression. For simplicity we do not report those equations and refer to Idriss et al. (2008). The correction factor K_σ is a function of the effective stresses σ'_v and of the normalized CPT resistance $q_{c1,n}$.

2.1.2. Permanent displacement

The quantification of the displacements during liquefaction is a very complex problem. However the largest displacement will occur due to floatation of the pipeline in the liquefied sand and due to lateral spread in post-liquefaction condition. The horizontal level ground displacement can be computed by numerically integrating the expected shear strains along the depth of the soil layers. To perform a one-dimensional integration on the volumetric strain is of course a simplified approach. However, it is a general approach that has been extensively used in research and practice. Usually the integration over the depth is applied on the maximum shear strain given the linear dependency of maximum displacement on the shear strain (maximum potential displacement). Actual lateral displacements will depend on several other factors (ground slope, heterogeneity, etc.).

The simplified method developed by Seed (Idriss et al. 2008) relates the maximum shear strain to the safety factor against liquefaction defined as ratio between CRR and CSR. The expression of the maximum shear strain in Eq.(7) is also of semi-empirical nature and it is applicable in a limited range of Dr and CPT resistance (Dr≥0.4 and $q_{c1,n}$≥69).

$$\gamma_{max} = \begin{cases} 0, if\ FS \geq 2\ (no\ liquefaction) \\ min\left(\gamma_{lim}, 0.035(2-FS)\left(\frac{1-F_\alpha}{FS-F_\alpha}\right)\right), 2 > FS > F_\alpha \\ \gamma_{lim}, FS \leq F_\alpha \end{cases} \quad (7)$$

Where F_α and γ_{lim} can be computed as function of Dr or $q_{cpt,n}$ (see Idriss et al. 2008).

The maximum potential displacement during liquefaction is computed with a one-dimensional integration of γ_{max} along the depth. Herein, the computed displacement is then applied to the pipeline segment, together with a uplift buoyancy effect. It is also assumed that the liquefaction involves the pipe for a length equal to 10 times the external diameter.

2.1.3. Springs Interaction model

2.1.4. Description

The pipeline we intend to study is considered as installed on the bottom of a trench at the depth of 2.3m (pipe diameter 1219mm). The interaction with soil sub layers is modelled by a set of nonlinear springs along the pipe (the segment length considered is 20 times the external diameter). The pipe is therefore constrained by springs on the top, the lateral sides and at the bottom (Helmholt et al. 2013). They differ from each other and depend on the soil properties of sub layers and top layer of soil (Figure 3). Their properties are defined as depending on the passive, neutral and reduced vertical stresses of the soil, the vertical coefficient of subgrade reaction, and the ultimate bearing capacity of the soil layers above and below the pipe (Figure 3 and Figure 4).

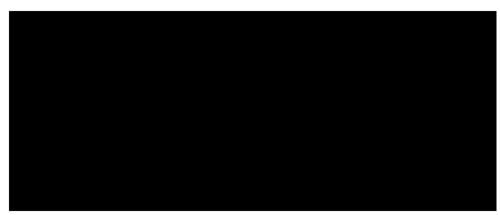

Figure 2: Springs along the pipe.

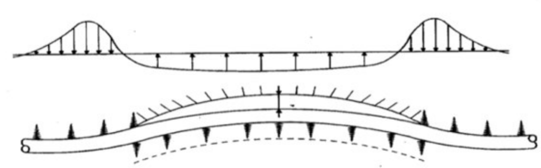

Figure 3: Longitudinal view of pipeline displacements.

The stresses in the soil and its bearing capacity are related to the mechanical properties of the soil (saturated and effective unit weight, friction angle, cohesion, Young's modulus etc.) but also from some geometrical conditions such as external diameter, installation depth, groundwater level. The soil mechanical parameters are derived for two locations in the Netherlands northern region by means of Cone Penetration Test (CPT) and Triaxial tests.

2.1.5. W-Tube FEM solver

The W-Tube (Helmholt et al. 2013) tool is a TNO developed solver that computes the Finite Element Solution (FEM) for a 3D continuous, segmented and even a full pipe network installed in trench on a spring bed as described in 2.2.1. Pipeline geometry and material, constraints, length of segments, springs stiffness and external action (forces or displacement patterns) are the input for the solver that can be managed from Matlab.

3. CASE STUDY

3.1.1. Pipeline Seismic Fragility

The fragility function expresses the probability of exceedance P_{LS} of a certain limit state Y_{LS} with respect to a certain Intensity Measure (IM). The fragility is also known as conditional probability of exceedance of the limit state LS and can be defined in Eq. (8), where the intensity measure is denoted with S.

$$P_{LS}(s) = p(Y_{LS} > 1|S = s) =$$

$$p\left(S_{Y_{LS}=1} \leq s\right) = \Phi\left(\frac{\ln s - \mu_{\ln S_{Y=1}}}{\sigma_{\ln S_{Y=1}}}\right) \qquad (8)$$

The parameters of the curve are the mean $\mu_{\ln S_{Y=1}}$ and standard deviation $\sigma_{\ln S_{Y=1}}$ of the logarithm of the seismic intensity $S_{Y_{LS}=1}$ that causes the achievement of the limit state $Y_{LS} = 1$. Common practice is to derive the parameters $\mu_{\ln S_{Y=1}}$ and $\sigma_{\ln S_{Y=1}}$ from observational analysis. For the pipeline we are investigating, we have to operate in absence of observations due to the non-tectonic nature of the earthquakes, therefore we need a model to simulate the behavior of the pipe segments. Monte Carlo simulations are carried out to generate the necessary data to perform an hypothetical observational analysis to compute the seismic fragility of the specific pipeline in the north of the Netherlands. Herein, we present the results of the benchmark study on two segments of the high pressure pipeline (diameter 1219mm, welded steel X60, continuous) at two different locations. In further work the study will be extended to the full network of high pressure pipeline with valves and operational stations.

3.1.2. Stochastic simulations

The GMPE of Akkar et al. (2013) as in Eq.(1) is used as to sample peak ground acceleration and velocity of earthquake events occurring with a magnitude uniformly distributed in the range $M_w(4 \div 6)$ and with source-site distance of 3km. The two locations are considered independent from each other.

The full characterization of the soil layers is available in two locations in the region of Groningen, (Meijers 2014). The subsoil is characterized in the first 13m of soil by alternate layers of loose sand, peat and clay and deeper layers of sand. The saturated unit weight of the sand and peat, effective unit weight, friction angle, Young's modulus, Cone Penetration Test (CPT) resistance is provided in Meijers (2014) for each layer of the stratigraphy at two locations and those values are considered as mean values in the single layer. The stratigraphic distribution of the layers is available at steps of 50cm. The same discretization is used in the computations to compute the static tensional state in the soil layers. To compute the stiffness of the springs, the values of the soil parameters at 2.3m depth are used (sand). Soil properties are sampled as uncorrelated with exception of the saturated and effective unit weight of each layer. The relative density is computed as function of the CPT resistance with the Lunne-Christoffersen relation. The main random variables are listed in Table 1.

Table 1: Random Variables in the model

Random variable	Mean	c.o.v.
Magnitude $Mw \propto U(4,6)$	5	0.057
Saturated unit weight of sand $\gamma_{sat} \propto N$	Varies with stratigraphy and location	0.10
Saturated unit weight of peat $\gamma_{sat} \propto N$	Varies with stratigraphy and location	0.10
Effective unit weight of sand $\gamma_{sat} \propto N$	Varies with stratigraphy and location	0.10
Effective unit weight of peat $\gamma_{sat} \propto N$	Varies with stratigraphy and location	0.10
Young Modulus of Sand $E \propto N$	$3.8 \cdot 10^3$	0.10
Friction angle $\varphi \propto N$	Varies with stratigraphy, location and soil 20°-35°	0.10
CPT resistance $CPT \propto N$	Varies with stratigraphy and location	0.20

Within the Monte Carlo procedure, we compute two main limit state functions:

- Potential of liquefaction

$$LS1 = CRR - CSR \tag{9}$$

- Pipe rupture for exceeding stressed (von Mises yielding criterion) caused by soil permanent ground deformations (pgd)

$$LS2 = f_y - \sigma(pgd) \tag{10}$$

Pipe rupture for transient strain and ovalization are also computed, but they are considered of less importance in common practice, due to the fact that the rupture is more likely to occur due to the effect of pgd.

At each simulation, the mechanical parameters of the soil layers and an event of a certain magnitude is sampled and from Eq.(1), pga and pgv are derived. CSR and CRR are computed and the exceedance of the LS1 is verified (Eq.(9)). If LS1 is exceeded, the lateral soil displacement is computed by integrating γ_{max} along the depth. The displacement, geometry of the pipe and the calculated soil spring stiffnesses are used as input for the W-Tube FEM solver. The W-Tube solver computes longitudinal and cross section deformations and stresses that are used to verify if the limit state LS2 is exceeded.

3.1.3. *Simulation Results*

The results of the simulations are treated as an artificial dataset to derive the fragility functions for the two segments of the pipe at two independent locations. Figure 4 and Figure 5 show the probability of soil failure due to liquefaction conditioned on the pga value (fragility or exceedance of limit state LS1) and the expected value of lateral soil displacement at location 1 and 2. Although the probability of soil liquefaction is high, the lateral spread at location 1 is small (in the order of the cm) while a larger spread is expected at location 2. The result is consistent with the author's expectation, since the stratigraphy at location 1 is characterized by smaller layers of sand under a upper layer of peat and clay, which is absent at location 2. However, the result is judged as too conservative in relation to the large conservativism of the scaling factor MSF for events with short duration (Meijers 2014) and to the sensitivity of γ_{lim} to the value of the geotechnical parameters (mostly soil relative density).

However, the predicted displacements do not affect the pipeline segment with 1.219m diameter, for which no rupture is found. Therefore we can expect liquefaction events, but of minor intensity and with no effect on this kind of pipeline. Figure 6 and Figure 7 show the maximum lateral displacement of the pipe with respect to the correspondent soil lateral spread for the locations 1 and 2: the pipe displacement is of 1 and 2 order of magnitude lower that the lateral spread at location 1 and 2 respectively.

For the transient displacements, based on the effect of the shear waves (s-waves), rupture occurred with probability of $8.5 \cdot 10^{-3}$. However, the formulation used (traction stresses linear proportional to the pgv) is quite conservative and more detailed modelling should be done.

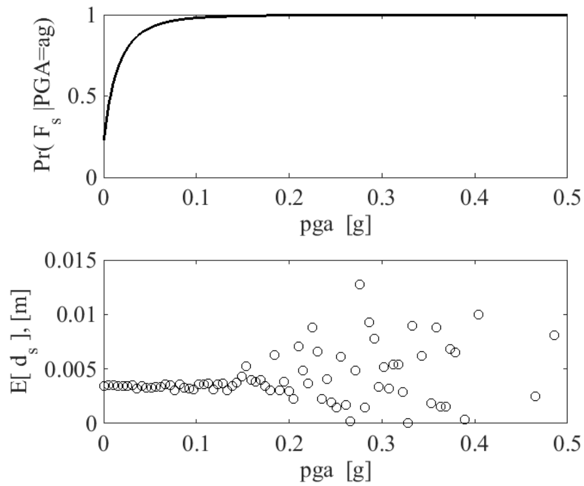

Figure 4:Conditional probability of soil failure and expected simulated lateral soil displacement at location 1 with respect to pga [g].

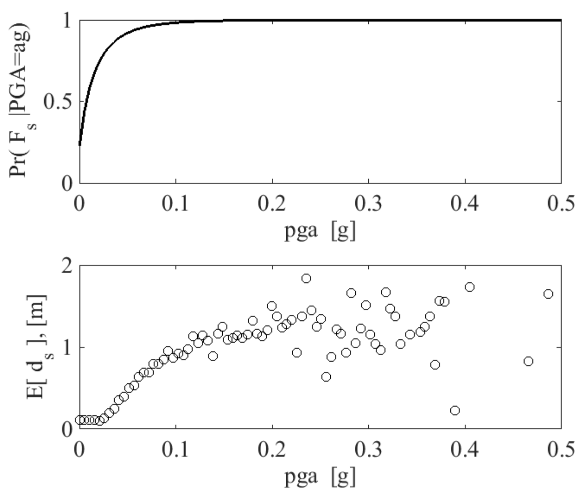

Figure 5: Conditional probability of soil failure and expected simulated lateral soil displacement at location 2 with respect to pga [g].

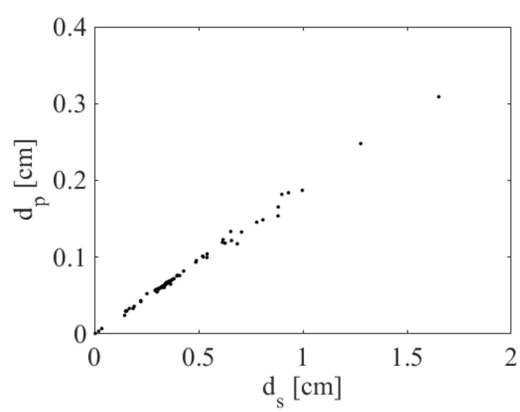

Figure 6: Pipe horizontal maximum displacement with respect to lateral spread at location 1

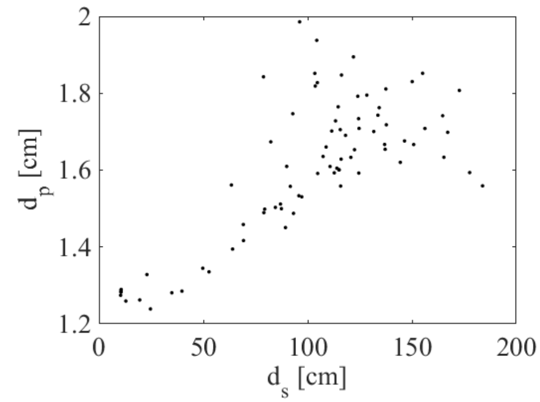

Figure 7: Pipe horizontal maximum displacement with respect to lateral spread at location 2

Minor ovalization effects occur instead with high probability and with average ovalization 0.18%, largely smaller than the operational limit state of 5%.

3.1.4. Discussion

In absence of available data to perform an observational analysis and derive fragility curves for a specific pipeline, it is common practice to refer to literature studies or data collected for similar pipelines in terms of material, geometry, soil and earthquake intensity.

For the specific geographical area of interest, the buried high pressure pipeline is subject to human induced earthquakes for which no similarity is found in other studies. Therefore, we investigated the behavior of two pipeline segments at two different locations, where liquefaction of loose sand can occur. The two locations are chosen as representative of the typical stratigraphy of delta region in the North of the Netherlands.

The simulation results show how sand liquefaction can occur even at low values of pga. However, the predicted lateral spread due to liquefaction causes only minor effects of deformation (ovalization) in the two pipe segments. Pipeline segment with smaller diameter and steel grade may show more severe ovalization or even some ruptures.

The study herein presented is however limited and further modelling needs to be done to completely investigate the behavior of the pipeline network with different diameters and steel grades.

4. CONCLUSIONS

The behavior of pipelines subject to earthquakes is investigated with a simulation based procedure that mainly considers the mechanisms of soil-pipe interaction. Strong uncertainty is related to the possible liquefaction of loose sand when the earthquake is of small magnitude and the results of the study show also a certain instability due to the limited range of validation of the approach of Idriss. The study represents a benchmark that will be refined and extended in future work to the full network.

5. ACKNOWLEDGEMENTS

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