# Failure mechanisms of underground structures during earthquake: an overview

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GEOQuébec 2015
Challenges from North to South
Des défis du Nord au Sud

ABSTRACT
Geotechnical structures buried near the ground surface have a wide range of applications, from small-scale pipelines such as means of gas transmission, telecommunications, water supply, and sewerage pipelines, to large-scale structures including tunnels for various transportation systems. This paper provides an overview of the current understanding of the failure mechanisms of these structures due to earthquake loadings. Based on post-earthquake investigations, experimental laboratory data as well as numerical simulations of underground structures conducted in the current study by means of computer code, FLAC, it was found that movement of ground at seismic load may cause serious damage to those infrastructures. These serious damage is represented in two main types of failure has been occurred. First, stress-strain failure of the underground structure due to extra-stress and extra-deformation which as a result of soil movement at seismic. Second, state the stress failure of soil which lead to an uplift of underground structures and collapse of surround soil then disconnection of pipe joints between buried structure and tubes.
RÉSUMÉ
Les structures enfouies près de la surface du sol ont un large éventail d'applications, allant de tuyaux pour le transport de gaz, de télécommunications, d'approvisionnement en eau, et de canalisations d'assainissement domestique, à des structures à grande échelle, incluant les tunnels qui servent comme systèmes de transport. Cet article donne un aperçu sur les mécanismes de défaillance de ces types de structures lorsque soumises à des charges sismiques. Basé sur des observations post-séisme, des données de laboratoire ainsi que des simulations numériques d'une structure souterraine menées à l'aide du code informatique, FLAC, il a été constaté que le mouvement du sol sous chargement sismique peut causer d'importants dommages aux infrastructures souterraines. Ces dommages sont représentés sous deux types principaux de défaillance. Tout d'abord, l'augmentation excessif des contraintes dans la structure souterraine provoqué par les mouvements du sol durant le séisme et à l'interaction sol-structure. Deuxièmement, l'augmentation des pressions interstitielles et la perte de capacité du sol qui conduisent à un soulèvement de ces structures et à l'effondrement du sol qui les entoure suivie de la rupture ou la déconnexion des raccords de tuyaux entre la structure enterrée et les tubes.

## 1 INTRODUCTION

Buried geotechnical structures are becoming more and more prevalent in the modern world because of the decreasing availability of ground space due to fast growing population (Yue and Li (2007). Underground infrastructure, serving for transport (e.g., highway tunnels and subway metro), utility (e.g., gas and water pipelines) and storage purposes (e.g., fuel storage and water tanks) has been a widespread alternative in redeveloping urban spaces to ease land congestion pressures. However, in the event of an earthquake, the functionality of these lifelines could be put in to risk especially in the liquefied soils (Chian, and Tokimatsu 2012). Many earthquakes (e.g. 2004 Niigata Chuetsu, 2007 Noto Hanto and 2007 Niigata Chuetsu-oki) caused serious damage to buried structures such as uplift of manholes and settlement of pavement above backfill soil for pipes (Yoshida et al. 2008). Haiti 2010 Earthquake resulted in severe destruction of essential systems (e.g. transportation and lifeline systems). One of losses is a lost 60% of the nation's infrastructure (DesRoches et al. 2011).

In fact, Public infrastructure in Canada appears highly vulnerable following decades of underinvestment, and may be severely challenged by a large earthquake. Because of the losses experienced, significant investments are required to retrofit these ageing systems to a better level of performance (Kovacs 2010). *The majority of Canada's public underground structures included no modern seismic engineering knowledge during the design and construction because almost 60 percent of Canada's underground structures was put in place before 1960 (CSCE 2003).*

To study the mechanism of failure of underground structure, a series of tests to underground structures have been modeled. e.g., small-scale model tests was conducted with centrifugal test (Zhou et al. 2015; Kang 2010) and another models was solved by numerical analyses (Jung et al. 2013; Liu 2012; Xia et al. 2010; Liu and Song 2006; Liu and Song 2005). This model tests resulted an enormous failure to the ground (e.g. uplift of soil with the structure and settlement of surround soil) and/or the underground structures (e.g. deformation and extra stresses for the underground structure).

In this paper, examples of underground structures damaging under earthquake loadings including uplift and/or collapse of the buried geotechnical structures will be presented first to be followed with method of physical and numerical modeling of infrastructures. Then, the failure mechanisms of underground structures due to earthquake loadings will be reviewed and discussed. Among failure mechanisms presented and detailed discussed in this paper, we can mention: 1) Shear failure of soil which occurs when the pore water pressure increase and the effective stress decrease which lead to uplift to the underground structure and/or settlement to surround soil which can make disconnection between the structure and tube. 2) Deformations and stresses influence on the structures which happens due to the interaction between the structures and surround pressured soil.

Nowadays in Quebec, Hydro-Quebec used more than 30,000 underground structure that had been installed over the province for housing power cables and transformers during the last years. The underground structure is a cuboid concrete chamber (3.8x2.3x2.8 m) connected with concrete tubes as shown in figure 1. Replacing of these structures are needing to expansive cost and may be disrupted life. So, the challenge facing Hydro-Quebec is to design underground structures can resist any earthquake may be happen in these region. To investigation in this phenomenon, a numerical model of this underground structure using FLAC 2D is created and subjected to seismic load. Then the uplifting displacement and the stresses of the structure is calculated.

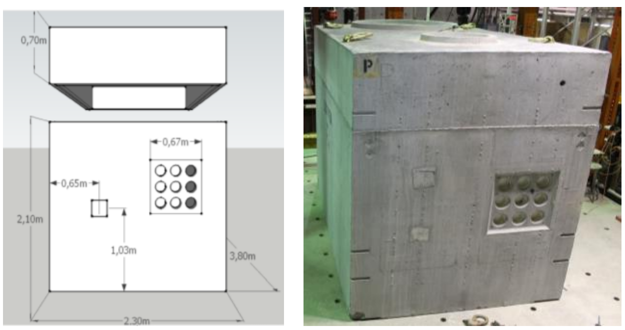

Fig. 1. Hydro-Quebec structure

## 2 EXAMPLES OF DAMAGE OF UNDERGROUND STRUCTURES DUE TO EARTHQUAKES

There are several reports on devastations caused by seismic loads in the underground structures (Tokimatsua et al., 2012; Koei 2014).

Tokimatsua et al. (2012) presented an overview of the geotechnical aspects of the building damage due to the 2011 Tohoku Pacific Earthquake, based on field reconnaissance made after the earthquake. Extensive soil liquefaction occurred along the coast of Tokyo Bay. Underground facilities, such as manholes, emergency water tanks and parking lots were uplifted (figure 2), tap water and sewerage systems were damaged, roads had dents and utility poles were toppled. The liquefaction induced floating of sewer manhole during earthquake causes serious damages to the function of sewer system. In addition, the uplift of manholes pose hazards to the traffic of ambulances and obstructs the rescue activities (Koei 2014). About 100 m of National Highway above the Daikai Station had settled by up to 3 to 4 m, over a width of 30 m as shown in figure 3.


Fig. 2. Uplift of underground parking lot (Tokimatsua et al. 2012).


Fig. 3. Daikai Station failure - Niigata-Chuetsu Earthquake (http://kobe117shinsai.jp/area/hyogo/d037.php)

## 3 METHOD OF MODELING OF UNDERGROUND STRUCTURES

### 3.1 Physical Modeling of Underground Structures

The objective of this part is to study the failure behavior of geotechnical structures buried near to the ground surface due to earthquakes. Considering the importance of lifeline facilities, the damaging behaviour of infrastructures is investigated in detail through centrifuge studies. Also, a new practical method to predict uplift displacement of the buried structure and surface settlements of backfill is developed and validated in the course of the investigation (Koseki et al. 1997b). Studies on failure of buried geotechnical structures have been started experimental modeling by (Ling et al. (2003); Sasaki and Tamura (2004); Chou et al. (2011); Tobita et al. (2011); Kang et al. (2009); Kang et al. (2013); Zhou Jian et al. (2015)).

In this regard, a geotechnical centrifuge has been widely used to perform small-scale model tests of geotechnical structures. In reduced scale, it is able to reproduce the same level of an effective confining stress with a prototype ground. A series of centrifuge tests is conducted to study the uplift mechanism of buried geotechnical structures in liquefied ground. If the tests are conducted with the centrifugal acceleration of N G, The scaling law for the centrifuge tests for N G is summarized in Table 1 (Kang 2010). Ling et al. (2003) prepared the ground with Nevada sand and shaken with a sinusoidal wave at an amplitude of 0.5g. The loose sand and gravel backfills placed around the tunnel are considered to be liquefiable (Chou et al. 2011). Tobita et al. (2011) used many factors in the experiments (e.g. ground water levels, the magnitude of input accelerations, the duration time of shaking, the relative densities of backfill and the ground, Etc.). Zhou et al. (2015) performed a centrifuge model test on saturated sand deposits with a shallow buried structure model on the geotechnical centrifuge. The photograph of the centrifuge test in (figure 4) shows that the top of the underground structure emerged on the surface of the sand layer after shaking. Sasaki and Tamura (2004) conducted a series of dynamic centrifugal model tests in order to investigate the effects of several factors on uplift movement of underground structure. Uplift of the structure initiated after the surrounding sand layer had attained liquefaction. Kang et al. (2013) found that excess pore water pressure is one of the contributing factors to the magnitude of the manhole uplift.

Table 1. Scaling law for the centrifuge tests for N G.

| Quantity | Sc | Quantity | Sc |
|---|---|---|---|
| Length | N | Stiffness | 1 |
| Density | 1 | Permeability | N |
| Time | N | Pore pressure | 1 |
| Frequency | $N^{-1}$ | Fluid pressure | 1 |
| Acceleration | $N^{-1}$ | EI | $N^4$ |
| Velocity | 1 | EA | $N^2$ |
| Displacement | N | B.M. | $N^3$ |
| Stress | 1 | Shear | $N^2$ |
| Strain | 1 | Axial force | $N^2$ |

Fig. 4. Observed Uplift of structure model (Zhou et al. 2015)

### 3.2 Numerical Modeling of Underground Structures

Many works is reported on numerical analyses of the seismic behavior of underground structures constructed in ground especially liquefiable soils. When the surrounding soil of the underground structure is liquefied, large deformations happen in the area leading to increase the internal stresses and deformations of the structure. The uncoupled method and the coupled method are two kinds of numerical methods are usually used to analyze the response of an underground structure under seismic loads (Wang et al. 2005). Xia et al. (2010) investigated the seismic response of an underground structure in saturated deposits using fully coupled dynamic finite element method. Khoshnoudian and Shahrour (2002) studied this problem using the (u-p) formation (displacement for solid phase and pore-pressure for the fluid phase).

Using the program (dynamic Finite Element code DYNA Swandyne-II), Liu and Song (2005) incorporated a generalized plasticity model that can simulate both cyclic liquefaction and pressure dependency of soils to model the sandy deposit. Lu et al. (2005) and Jung et al. (2013) used a two-dimensional, finite element (FE) continuum model with a Mohr–Coulomb (MC) to evaluate soil–pipeline interaction for uplift in granular soil. Kang et al. (2014) has used FLIP program to study the seismic response of underground structures that built in liquefiable soils. Azadi and Hosseini (2010) assessed the topic through the use the Finn model of the FLAC software which can assess of the liquefaction effects for the soil Beam elements are two-dimensional elements with three degrees-of-freedom at each end node. He and Chen (2011) and Yang and Wang (2013) simulated the pore water pressure until liquefied of sand by Finn model based on the Mohr-Coulomb model under dynamic load in FLAC3D software. Zhou et al. (2014) conducted a numerical model using a two-phase fully coupled distinct element code. This code incorporates a particle-fluid coupling model in PFC3D. Lui and Song (2006) and He and Chen (2011) assumed, the boundary between the soil deposit and the bedrock was fixed, in order to the reflection of waves in the model is prevented and the boundaries act as adsorbent boundaries. Liu and Song (2005) tested many trial to thicknesses of elements to ensure that the seismic events from the base could be adequately transmitted and found that the width of the tied boundaries of the model must be large enough so that the structure isn't affected by the reflected vibration. While Azadi (2011) considered free field conditions for the dynamic boundaries and the damping ratio is 5%. However, Qiao et al. (2008) and Yang and Wang (2013) selected the hysteretic damping option in FLAC.

Wang (2013) executed the model in two stage. Frist, in the static analysis to compute gravity stresses, the base boundary was fixed both horizontally and vertically and the side boundaries were fixed horizontally. Then, in the dynamic analysis, the horizontal seismic load was applied at the base boundary. The horizontal restraints of the side boundaries were released and replaced by attaching the two sides to force a rigid side boundary condition at both sides.

### 3.3 Analytically Method for Modeling of Underground Structures

The analytical method provide useful information on the uplift force and displacement of underground structures in liquefiable soils. However, knowledge gaps persist, particularly in the prediction of uplift displacement of underground structures and simulation of post liquefaction response of the soil in numerical analysis.

A simplified mechanism for the floatation of a circular underground structure consists of the weight of the structure ($F_T$), the weight of the overlying soil ($F_{WS}$), the shear contribution ($F_{SP}$), the buoyant force of the structure ($F_B$) and the excess pore pressure generated near the invert of the structure ($F_{EPP}$). From Eq. (1), Chian et al. (2014) can calculate the net uplift force ($F_{NET}$) as shown:

$$F_{EPP} = (F_B + F_{EPP}) - (F_T + F_{WS} + F_{EPP}) \qquad [1]$$

Tobita et al. (2012) inferred first approximation Eq. (2) which can calculated the maximum uplift of the underground structure ($\Delta_f$).

$$\Delta_f = (1 + \frac{\gamma_m}{\gamma_{sat}}) * h \qquad [2]$$

Where: $\gamma_m$ is unit weight of underground structure, $\gamma_{sat}$ is the saturated density of backfill soil and h is the height of the underground structure. Then, more exact Eq. (3) is proved to find the uplift of the underground structure.

$$\Delta_f = (1 - \pi(\frac{d}{2a})^2) * ((1 - \frac{\gamma_m}{r_u * \gamma` + \gamma_w}) * h - ((1 - \frac{r_u * \gamma_t}{r_u * \gamma` + \gamma_w}) * h_w - (\frac{R}{r_u * \gamma` + \gamma_w}) * (\frac{1}{\pi}) * (\frac{2}{d})^2) \qquad [3]$$

Satoh et al. (1995) found clearly from the regression analysis that a comparatively good correlation was in the maximum value of magnitude of the ground deformation ($D_{max}$) and the thickness of liquefiable layer (H). Yoshiaki (1998) proposed a simplified calculation method for maximum uplift displacement ($\Delta_f$) cause complete liquefaction of the soil by Eq. (4).

$$\Delta_f = (\frac{L^3}{160EI} + \frac{3}{2f * k_h}) * \frac{p * L^2}{b} \qquad [4]$$

## 4 FAILURE MECHANISMS

When the ground is exposed to a large earthquake, the soil will subject to huge energy and great deformation. Due to the interaction between underground structures and surround soil, the deformations and strains is affected on the structure by the surrounding ground (Hashash et al. 2001). Underground structures must be designed to support static stresses as well as to accommodate the additional deformations and stresses imposed by seismic load. The deformations of underground structures is produced by ground failure due to earthquake (Huo 2005).

Among failure mechanisms observed in the literature, we can mention: a) lifting due to the liquefaction of replaced fill soils, b) the shear failure and uplift of underground structures, and c) disconnection of pipe joints due to the liquefaction of filled soils and the surrounding ground. Each of these hazards may be potentially catastrophic (Huo 2005).

To further investigate this phenomena a numerical analysis was calculated. The static and dynamic calculations were done with the FLAC 2D (Fast Lagrangian Analysis of Continua) software. The model was done in two stage. Frist, in the static analysis to compute gravity stresses, which divide to two step. Initially, the soil was modeled with a Mohr–Coulomb and water level was at the ground surface. The boundary conditions are configured so that it is fixed vertically at the base and only fixed horizontally at the both sides. Then the structural members were represented by beam elements as shown in figure 5. Second, the horizontal seismic acceleration was input to the model at the base boundary. In this step, undrained analysis is carried out for liquefaction by using total stress model. Meanwhile, a normalized shear wave velocity, $V_{s1}$ of 200 m/s is used ($V_{s1} = (100/\sigma'_v)^{0.25}$). Then the shear modulus (G) was

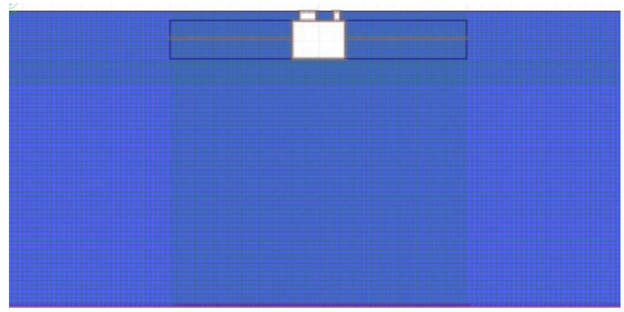

Fig. 5. Display of mesh underground structure model

calculated in to the model according to $V_{s1}$ of 200 m/s and the calculated overburden stress in the first step ($G=\rho V_s^2$). The degradation of shear modulus ($G/G_{max}$) and the associated damping was modeled according to the upper limit proposed by Seed and Idriss (1970).

### 4.1 Deformations and stresses of the structure

During the earthquake the underground structure follows the deformation of the surrounding ground, and because the structure is confined no damaging stresses are produced in the structure. This perception changed after the severe damage and even collapse of a number of underground structures that occurred during recent earthquakes (Huo et al. 2005). The internal forces of the underground structure increase due to soil-structure interaction at earthquake. For this, the final support system of underground facilities in seismic zones must be designed to support static overburden stresses as well as to accommodate the additional deformations imposed by the earthquake-induced motions. For flexible or small structures resting on a stiff soil, the effects of the interactions are usually insignificant while the interactions of heavy structures located on liquefiable soil are very critical (Tabatabaiefar et al. 2013).

#### 4.1.1 The deformation of the structure

Underground structures are constrained by the surrounding soil and cannot move independently so are not generally subjected to significant dynamic amplification effects. They are affected by deformation of surrounding ground and by inertia forces acting on the structure. Rectangular underground structures experience transverse deformations due to earthquake-induced shear strains in surrounding soil. Deformation of the cross-section is usually more critical in design than the axial or curvature deformations induced along the axis of the structure (Wood 2004). Distribution of maximum displacement in a cut and cover structure was studied by (Matsuda and Tanaka 1996 and Sweet 1997) using Two-dimensional and three-dimensional finite element and finite difference models as shown in Figures 6. The analytical solution indicates that the structure deformation is dependent on the stiffness ratio between the structure and the ground and on the shape of the structure, which is given by the ratio between its length and height (Huo 2005).

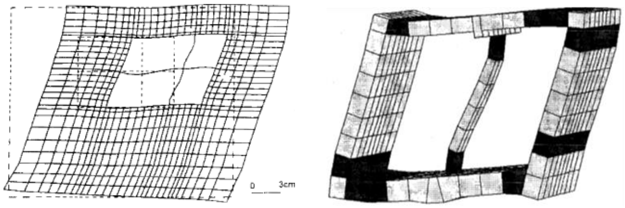


Fig. 6. Deformed cut-and-cover structures.

In our study the deformation of underground structure is determined by using FLAC 2D as shown in Figure 7. As a result of seismic forces compatible with the seismicity of Montreal (Québec, Canada) zone according to NBC 2010, deformation of soil and the structure occur. As it can be seen in Fig. 7, the deformation is variable with the time history of the earthquake with a deformation of the structure wall under 1 mm for 1 m height ($\gamma < 0.1\%$) . The deformation under the structure is more than the deformation of the surround soil at the same level because of the interaction between the soil and the structure and the effective stress under the structure in less than of it at the same level of surround soil (Fig. 7d). The magnitude of the displacements is different from joint to another joint. The relative difference leads to extra-stresses on the structure could reach times higher than the static stresses.

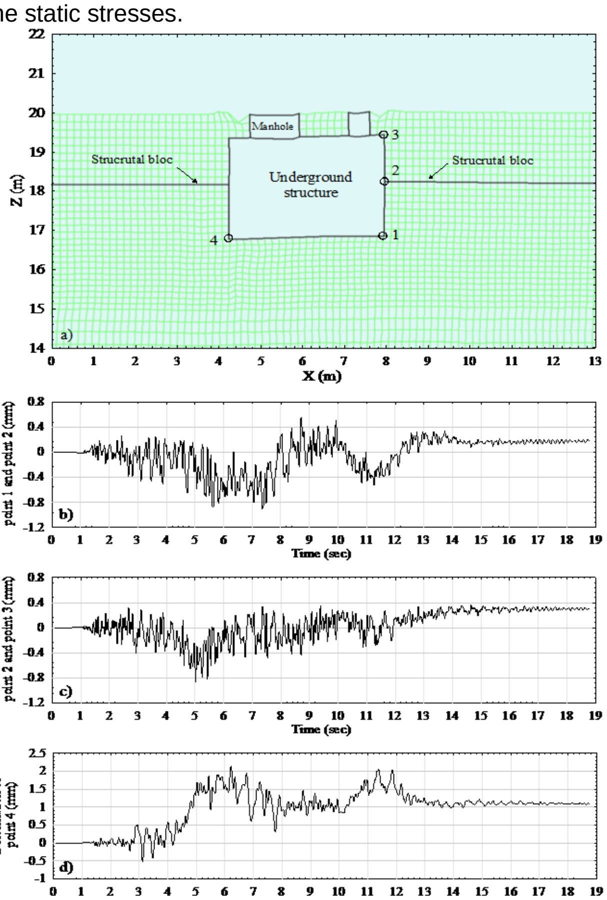


Fig. 7. a) Structure deformations subjected to earthquake compatible with the seismicity of Montreal, b) History time of deformation between point 1&2, c) point 2&3, d) History time of deformation of point 4.

4.1.2 The stresses of the structure

(Navarro 1992) analyzed the dynamic pressures acting on walls, roof and floor, due to seismic waves. Predicting underground structures' forces is the most work done, which provides relationships to evaluate the magnitude of seismic-induced stresses or strains in underground structures. The pseudo-static approach assumes that the dynamic amplification of stresses associated with a stress wave impinging on the opening is negligible. The maximum soil stresses at layer points, due to surface waves, may be computed as:

$$\sigma or(\tau) = \left(\frac{V_{particle}}{V_{wave}}\right) * E or(G) \qquad [5]$$

Where $V_{particle}$ is the maximum soil particle velocity, $V_{wave}$ is the wave phase velocity, E and G the elasticity and shear moduli of soil respectively, and σ and τ are the normal and shear stresses in the soil mass due to the seismic wave component.

Referring to underground structure under a static loading, it is found that Moment is on the walls 5-10 kN.m and shear is around 10-15 kN as well moment of the basis is about 20-25 kN.m and shear about 50 kN. Figs. 8 and 9 show the distribution of history time of moment and shear force in selected structural joints of the underground structure when the earthquake is loading. It can be seen that the maximum internal forces during the earthquake were mostly found at the edges and the connections of the structural members. From static and dynamic result it is found that the shear forces and bending moments caused by an earthquake can double the static forces.

4.2 Ground Failure

Ground failure includes several types of ground instability such as direct shearing displacements of active faults intersecting the structure, landslides, liquefaction of the surrounding ground, and tectonic uplift and subsidence. Each of these hazards may be potentially catastrophic leading to failure of telecommunications, gas transmission, water supply, and sewerage pipelines addition to break down of roads (Huo 2005).

4.2.1 Uplift of Underground Structures

Underground structures in saturated liquefiable soils may be subjected to severe damages during earthquake. One of the reasons is the uplift or even floatation of underground structures due to soil liquefaction. There have been several studies concerning the liquefaction related seismic behavior of small underground structures. These studies include numerical analyses (Liu and Song 2006; Liu and Song 2005; Xia et al. 2010) and experimental investigations (Zhou et al. 2015) as well as both together (Zhou et al. 2014; Chian and Tokimatsu 2012). The numerical and experimental analysis revealed that the liquefaction of soil due to strong earthquake excitation caused the underground structures to uplift, as shown in Figure 10 (a). The different in uplift displacement from test to another is due to the different of the boundary condition of the tests. Liquefaction ratio ($Ru=\Delta u/\sigma'$) of some previous tests are explained in figure 7.

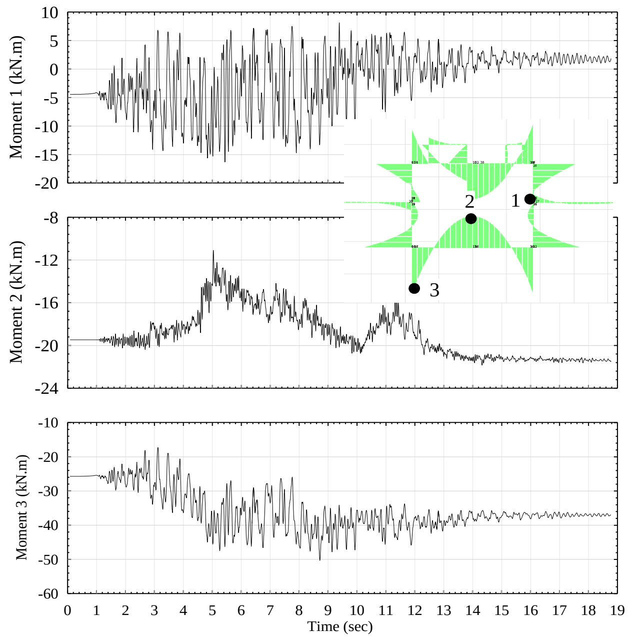


Fig. 8. History time of the moment of underground structure under an earthquake.

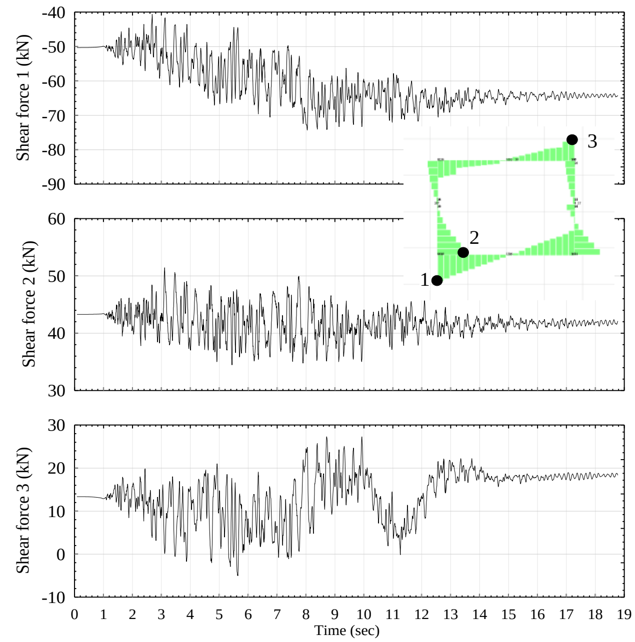


Fig. 9. History time of the shear forces of underground structure under an earthquake.

From figure 10, we can observe that the uplift of underground structure does not stop when the liquefaction occurs. But the uplift continues after the liquefaction ratio is up to the unit.

The most common approach to characterization of earthquake loading is through the use of cyclic shear stress ratio (CSR). CSR is the ratio between the average cyclic shear stress generated by seismic loadings and the initial effective vertical stress. CSR of the shaking event is used to relate the underground structure uplift with the shaking event (Chou 2010). In this study the result of CSR of the FLAC model was analyzed, the value of CSR under the base of the underground structure is three to four time more than the surround soil at the same level, as shown in figure 11(a). This increase is a result of low the effective vertical stress under the structure because the unit weight of the structure is less than the surround soil. The increase of CSR could lead to failure of the soil in region. Therefore the relative density under the structure should be increased so as not to liquefaction occurring to the soil.

The liquefaction resistance was evaluated from cyclic resistance ratio (CRR). The resistance of the soil to liquefaction (CRR) depends on standard Penetration Test (SPT) (Rauch 1997). $K_\sigma$ and $K_\alpha$ is factors that are used to modify the cyclic resistance ratio (CRR) at confining pressure 100kPa and static shear stress of zero to account for higher confining pressure and static shear stress depend on; density, confining pressure and initial static shear stress ratio ($\alpha = \tau_{xy}/\sigma_V'$) (Stedman 1994). The factor $K_\alpha$ can be calculated from the relative density of the soil and initial shear stress ratio and using figure of Seed and Harder (1990). Figure 11(b) shows initial shear stress ratio distribution at the end of the earthquake. Obviously that the initial shear stress ratio of the soil is near to zero at the middle third of the soil under the structure but it is up to 0.5 under at the edges under the structure.

No shear stress ratio of the soil at the middle third under the structure lead to reduction of factor Kα which means the resistance of the soil to liquefaction (CRR) is too low which lead to increase in the pore water pressure and uplifting of the structure.

Figure 12(a) shows the underground structure at two case of water table. The first state, the water table is at the ground level, the other the water table is at the bottom level of the underground structure. In case of static, at water level (1), the pore water pressure is greater than the vertical stress of the structure by about 10 kPa therefore the structure would be uplift, but in reality this does not occur due fixed connection between the structure and side tubes that generate additional resistance stresses (assumed 20kPa). At water level (2), the pore water pressure is zero at the bottom of the structure so there aren't uplift of the structure. In case of dynamic, the pore water pressure will be increased more than the static case due to seismic load. Of that the underground structure can be uplift in both water levels but after different time of number of cyclic of the earthquake and different excess pore water pressure. From figure 12 (b, c), it is found that, the uplift of structure does not need to liquefaction stage but it will be occurred before liquefaction occurring. When the water table is at the surface, the underground structure will be uplifted when E.P.W.P. can disconnect the fixation between the tube and the structure at 10kPa. On the other, when the water table at the bottom surface of the structure, the underground structure will be uplifted when E.P.W.P. is up to 40 kPa. Out of that we conclude that the uplift of underground structure is not related to the liquefaction of soil and the uplift can be occurred before liquefaction. Also we can overcome of this problem by fixation the tube with the structure to increase the resistance stresses.

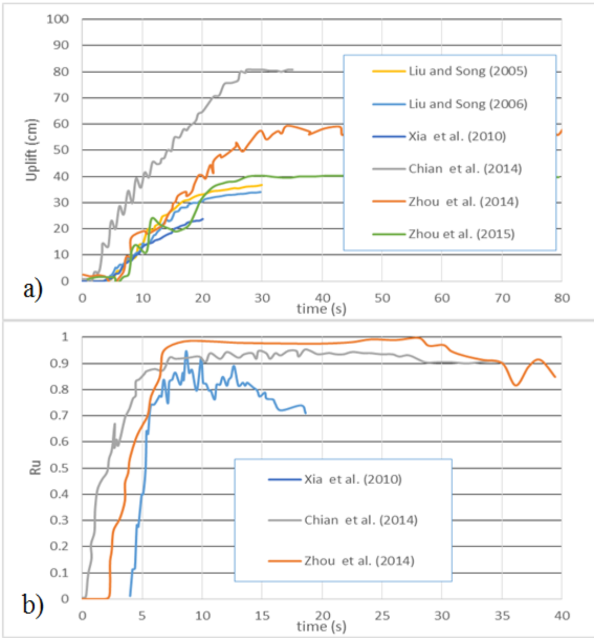


Fig. 10. a) Uplift of underground structures and b) Excess pore pressure/σ'$_{v0}$ at base of structures

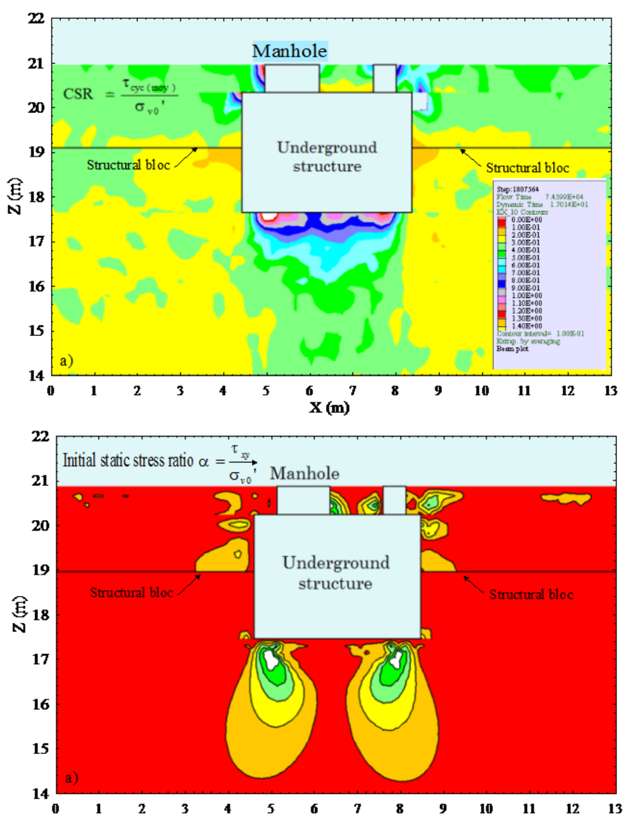


Fig. 11 a) Cyclic shear stress ratio (CSR) distribution, b) Initial shear stress ratio (α) distribution at the end of the earthquake.

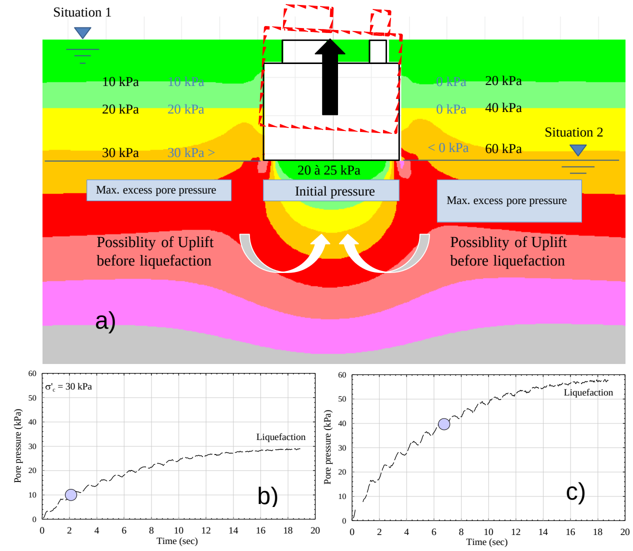


Fig. 12 a) Mechanism of uplift of the structure due to pore water pressure & b) History time of excess water pressure at situation (1) and c) at situation (2)

## 5 CONCLUSIONS

Post-earthquake investigations as well as physical and numerical modelling of underground structures imply that when the structure is subjected to seismic load, two types of failure can be emerged:

a) Failure of the soil that included to excess in pore water pressure which increase the probability of uplifted of the underground structure whether liquefaction is occurred or not. Also the surround soil can be collapsed. The uplift of the underground structure and the settlement of surround soil as well the tube in it can lead to disconnection between the structure and the tube.

b) Result of deformation of surround soil at dynamic load and interaction between the structure and the soil, the underground structure is exposed to additional stresses and extra deformation.

The result of numerical simulation using FLAC 2D for underground structure in saturated sand soil at static and dynamic state we can find the following:

1. The shear forces and bending moments caused by an earthquake can reach 1.5 to 2 times the static forces.
2. Increase CSR under the structure accompanied by decrease of the CRR at the middle third of soil under the structure which increase the probability of liquefaction occurring.
3. The PA can undergo to uplift although there are not earthquake or liquefaction.
4. The liquefaction may not be the most critical problem in the case of buried structures Hydro-Québec
5. More sophisticated analysis are required before starting to think about solutions. That said, it is necessary to find ways to dissipate the pressures below the PA.

## ACKNOWLEDGEMENTS

The authors would like to thank Hydro-Québec and the NSERC for their financial support throughout this research project.

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