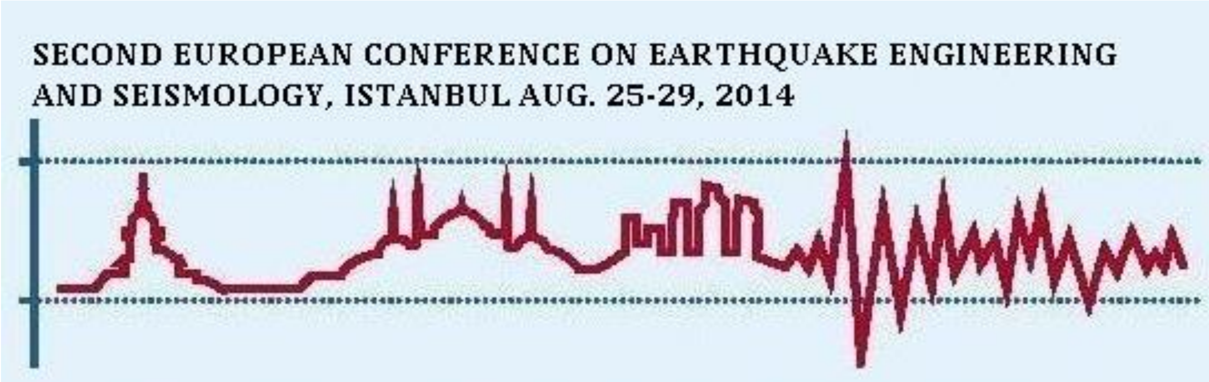

EARTHQUAKE-INDUCED RESPONSE OF SUBMERGED FLOATING TUNNELS WITH TENSION LEG ANCHORAGE

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ABSTRACT

This contribution presents the response of a submerged floating tunnel (SFT) anchored by tension legs subjected to seismic excitation. One of the proposals of a tunnel crossing the Hogsfjord in Norway is used herein as a case study. The proposed tunnel is 1345m long with a cross section diameter of 11.3m. The tunnel is proposed to lie 25m under the sea surface to enable regular sea traffic. Seismic analysis of the SFT is performed using the pseudo-excitation method (PEM) within the framework of random vibration theory. The ground acceleration spectrum based on the modified Kanai-Tajimi spectral density function is used. The coherency of ground motion between different supports and incoherence between different components of motion at a support is included in the analysis. Response of the SFT is examined for three cases of ground motion incoherence, namely (i) fully coherent, (ii) incoherent, and (iii) non-coherent. The analysis considers different velocities and angles of approach of the propagating seismic waves. The results indicate that the displacements are unlikely to impose risk to the safety of the tunnel. On the other hand, the resulting peak accelerations are quite large, in the order of acceleration of gravity. This might impose problems to the traffic inside the tunnel. It was observed, however, that such high accelerations occur as a series of impact, with much smaller peak accelerations during most of the strong shaking. The results indicate highest displacement response when the motion is non-coherent, while fully coherent motion yields the smallest response. It appears that wave passage effect is the dominating factor and that inter-component coherence has significant effects on the overall response.

INTRODUCTION

A submerged floating tunnel (SFT), also known as Archimedes´ bridge, is an innovative waterway-crossing solution consisting primarily of a tubular structure kept floating under water surface through anchorage systems—either cables, rods or piles attached to the bed of the water body, or buyos floating on the water surface. Figure 1 displays an artist's sketch of an anchored SFT. The floating depth can be adjusted to allow free passage of vessels navigating on the surface. SFTs have been conceived to be more attractive than traditional waterway-crossing structures such as bridges, immersed tunnels, and underground tunnels in certain situations, for example, in crossing a long and deep fjord. The advantages and competitive features of SFTs over other crossing solutions have been discussed by Østlid (2010).

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The concept of SFT as a waterway-crossing has attracted engineers for over a century—Sir Edward James Reed (Reed, 1886) had documented a detailed description of such structures. Interest in such structures resurged in the late 1960s. SFT was cosidered feasible for crossing many wide and deep fjords in Norway (Brandtzæg et al., 1971). The proposal by Alan Grant, in 1696, of the Messina Strait Crossing in Italy marks an important milestone in the development of the SFT concept. The Norwegian Public Road Administration (NPRA) initiated studies, participated by the four largest Norwegian contractors, on the design of SFT to cross the Høgsfjord. These studies led the NPRA to consider SFT as a feasible option (Jakobsen, 2010; Larssen and Jakobsen, 2010). Other feasibility studies of SFT include Storfjord crossing in norway (Jakobsen et al., 2009), Lugano Lake crossing in Switzerland (Haugerud et al., 2001), Rion-Antirion strait crossing in Greece, Como Lake crossing in Italy, Bosforo strait crossing in Turkey, Lake Washington crossing in the United States, etc. In addition, several SFT feasibility studies have been conducted in Japan (Kanie, 2010). An important milestone in the development of SFT concept is the Sino-Italian Joint Laboratory if Archimedes Bridge (SIJLAB) project, which started in 2004 aiming to build a SFT prototype in Qiandao Lake in the People´s Republic of China (Mazzolani et al., 2007).

Mechanical as well as environmental aspects of the behaviour and design of SFTs have been extensively studied in the past (see, Martire, 2010). SFTs are susceptible to dynamic oscillations due to different physical effects such as earthquake-induced motion at anchorage points; water waves induced by wind, seismic motion of seabed, or differential density effects; steady current vortex shedding, dynamic traffic loads, etc. A proper understanding of these dynamic effects in the safety and reliability is essential to design and build SFTs. Several research studies have addressed these effects. Remseth et al. (1999) presented alternative approaches for stochastic dynamic response analysis of SFTs under wave loads. They concluded that structural and hydrodynamic damping and optimal buoyancy are crucial parameters. Mazzolani et al. (2008) performed numerical analysis of a 100m prototype SFT proposed for Qiandao Lake (PR of China). Di Pilato et al. (2008) implemented a 3D numerical analysis of SFTs anchored to the seabed by slender bars. They considered seismic loads and nonlinear hydrodynamic loads due to steady current and wind waves, and used geometrically nonlinear finite elements to model the anchor bars.

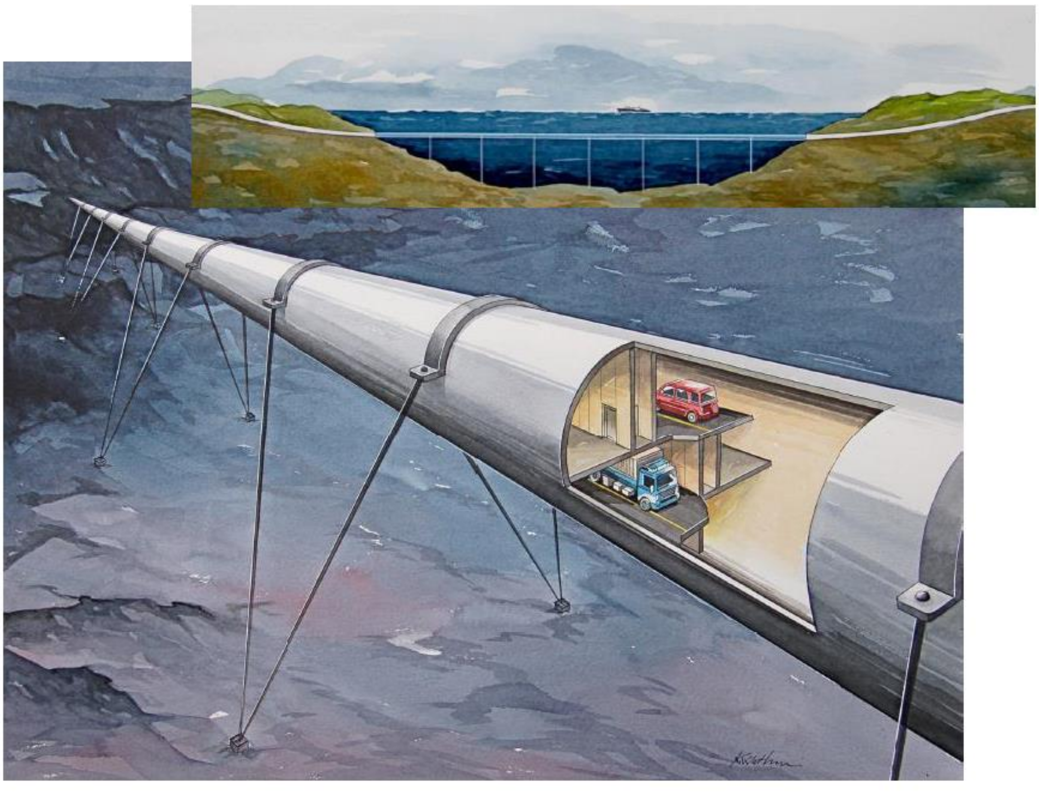

Figure 1. Artist’s sketch of an anchored submerged floating tunnel (NPRA, 2011; NRPA, 2012)

Because SFTs typically extend cover long spatial distance, an important effect in their seismic response is the asynchronous motion of the anchor points. The incoherence of motion at different supports of a structure is known as inter-support incoherence. Incoherence is commonly modelled by using spectral coherency functions (Zerva, 2009; Martinell et al., 2011). Differential motion of anchoring bars or tethers due to inter-support incoherence results in asynchronous loading effects. Modelling and analysis of such effects is important for designing SFTs in seismically active regions (see, for example, Chen and Huang, 2010, and Fogazzi and Perotti, 2000). In addition to inter-support incoherence of ground motion in a given direction, there exists a second type ground-motion incoherence. This is the incoherence of motion in different directions at the same support point. In the context of a three-dimensional description of ground-motion at a given support, motions along the three orthogonal coordinate directions are called ground-motion components. At a given support, ground-motion components are generally incoherent to each other. This type of incoherence is termed here as inter-component incoherence. This incoherence may potentially have important effects in the overall response of a SFT. As far as we are aware of, this kind of incoherence as not been yet considered in published studies of seismic action on SFTs. The main objective of this study is to present a methodology for seismic response analysis of SFTs considering the effects of both inter-support and inter-component incoherence. The methodology is applied as a case study using the proposed Hogsfjord SFT (see, for example, Larssen and Jakobsen, 2010).

METHODOLOGY

This section describes the modelling and analysis methods used in the present study. Formulation of the dynamic finite element equations for hydrodynamic and seismic loads is outlined, and the solution procedure is briefly described. The solution is based on the pseudo-excitation method (Zhang et al., 2009).

Equation of motion

The dynamic equation of motion cast in the finite element form is

$$\mathbf{M}\ddot{\mathbf{r}} + \mathbf{C}\dot{\mathbf{r}} + \mathbf{K}\mathbf{r} = \mathbf{Q}_s + \mathbf{Q}_h \tag{1}$$

where $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ are the structural mass, damping, and stiffness matrix, respectively. The structural displacement, velocity and acceleration vectors are denoted as $\mathbf{r}$, $\dot{\mathbf{r}}$, and $\ddot{\mathbf{r}}$, respectively. The vectors $\mathbf{Q}_s$ and $\mathbf{Q}_h$ represent seismic and hydrodynamic loads respectively. Note that the structural matrices, response quantities, and the loads are, in general, time dependent. This time dependence is not explicitly denoted, for example by using $\mathbf{Q}_s(t)$ instead of $\mathbf{Q}_s$, but is assumed as understood.

Hydrodynamic action

The hydrodyamic action resulting from fluid-structure interaction is modelled using the Morison´s equation (Morrison et al., 1950). In this study external water motion is not considered. In such cases, the transverse force $d\mathbf{Q}_h$ acting on an infinitesimal strip dx of a cylinder of diamter D is given by

$$d\mathbf{Q}_h = -\frac{1}{2} C_D D dx \dot{\mathbf{u}} \left|\dot{\mathbf{u}}\right| - \frac{\pi}{4} \rho \left(C_M - 1\right) D^2 dx \ddot{\mathbf{u}} \tag{2}$$

where ρ is the density of water, C_D is the coefficient of drag, C_M is the coefficient of inertia, $\dot{\mathbf{u}}$ is the velocity vector field (components orthogonal to the axis of the cylinder) at the strip, and $\ddot{\mathbf{u}}$ is the corresponding acceleration vector field. The first part of Equation **2** represents hydrodynamic damping and is nonlinear in the velocity of motion. Stochastic linearization of E1uation 2, with an assumption of Gaussian distributed velocity process, leads to the following representation of hydrodynamic force.

$$d\mathbf{Q}_h = -\sqrt{\frac{2}{\pi}} C_D D \sigma_{\dot{u}} \dot{\mathbf{u}} dx - \frac{\pi}{4}\rho\left(C_M - 1\right) D^2 dx \ddot{\mathbf{u}} \tag{3}$$

In Equation 3 $\sigma_{\dot{u}}$ is the standard deviation of resultant velocity at the centre of the element. By integrating the force on the infinitesimal strip over the length L of the cylinder, and using shape function matrix $\mathbf{N}$ for interpolation of displacement field, $\mathbf{u} = \mathbf{N}\mathbf{r}$, the hydrodynamic load can be expressed as

$$\mathbf{Q}_h = \mathbf{Q}_m + \mathbf{Q}_c = -\left(\frac{\pi}{4}\rho\left(C_M - 1\right) D^2 \int_L \mathbf{N}^T \mathbf{N} dx\right)\ddot{\mathbf{r}} - \left(\sqrt{\frac{2}{\pi}} C_D D \sigma_{\dot{u}} \int_L \mathbf{N}^T \mathbf{N} dx\right)\dot{\mathbf{r}} = -\mathbf{M}^h \ddot{\mathbf{r}} - \mathbf{C}^h \dot{\mathbf{r}} \tag{4}$$

The matrices $\mathbf{M}^h$ and $\mathbf{C}^h$ are called added mass and damping matrix due to fluid-structure interaction. Using equations 1 and 4, the dynamic equation of motion can be written as

$$\tilde{\mathbf{M}}\ddot{\mathbf{r}} + \tilde{\mathbf{C}}\dot{\mathbf{r}} + \mathbf{K}\mathbf{r} = \mathbf{Q}_\mathbf{s} \tag{5}$$

with $\tilde{\mathbf{M}} = \mathbf{M} + \mathbf{M}^\mathbf{h}$, and $\tilde{\mathbf{C}} = \mathbf{C} + \mathbf{C}^\mathbf{h}$. Note that the hydrodynamic damping matrix $\mathbf{C}^h$ is a function of the standard deviation of velocity at the center of the element which is not known *a priori*. This implies that the equation of motion needs to be solved iteratively.

Seismic action

Denoting the constrained (at the ground) and unconstrained (at the structure) degrees of freedom as g and s degrees of freedom, the dynamic equation of motion of the s degrees of freedom can be cast in terms of the relative (to the support) displacement as

$$\tilde{\mathbf{M}}_{ss}\mathbf{r}_s + \tilde{\mathbf{C}}_{ss}\dot{\mathbf{r}}_s + \mathbf{K}_{ss}\mathbf{r}_s = \mathbf{Q}_s = -\left(\tilde{\mathbf{M}}_{ss}\mathbf{B} + \tilde{\mathbf{M}}_{sg}\right)\ddot{\mathbf{r}}_g - \left(\tilde{\mathbf{C}}_{ss}\mathbf{B} + \tilde{\mathbf{C}}_{sg}\right)\dot{\mathbf{r}}_g \tag{6}$$

where the motion vectors have been partitioned to separate the response quantities, $\mathbf{r}_s$ from the input ground motion, $\mathbf{r}_g$, and the property matrices are accordingly partitioned, i.e., matrices with subscripts ss represent the sub matrices corresponding to the structural degrees of freedom and those with subscripts sg are sub matrices formed by taking the s rows and q columns from the matrices in Equation 6. The matrix $\mathbf{B}$ in Equation 6 is defined as $\mathbf{B} = -\mathbf{K}_{ss}^{-1}\mathbf{K}_{sg}$, and is sometimes called as the influence coefficient matrix (Clough and Penzien, 1995). The spectral density matrix of seismic action can be expressed as

$$\mathbf{S}_Q(\omega) = \mathbf{A}_a \mathbf{S}_{\ddot{u}_g}\left(\omega\right)\mathbf{A}_a^T + \mathbf{A}_v \mathbf{S}_{\dot{u}_g}\left(\omega\right)\mathbf{A}_v^T + \mathbf{A}_a \mathbf{S}_{\ddot{u}_g\dot{u}_g}\left(\omega\right)\mathbf{A}_v^T + \mathbf{A}_v \mathbf{S}_{\dot{u}_g\ddot{u}_g}\left(\omega\right)\mathbf{A}_a^T \tag{7}$$

where $\mathbf{A}_a = -\left(\tilde{\mathbf{M}}_{ss}\mathbf{B} + \tilde{\mathbf{M}}_{sg}\right)$; $\mathbf{A}_v = -\left(\tilde{\mathbf{C}}_s\mathbf{B} + \tilde{\mathbf{C}}_g\right)$; $\mathbf{S}_{\ddot{u}_g}$ is the spectral density matrix of ground acceleration; $\mathbf{S}_{\dot{u}_g} = \omega^{-2}\mathbf{S}_{\ddot{u}_g}$ is the spectral density matrix of ground velocity; $\mathbf{S}_{\dot{u}_g\ddot{u}_g} = -i\omega^{-2}\mathbf{S}_{\ddot{u}_g}$; $\mathbf{S}_{\ddot{u}_g\dot{u}_g} = i\omega^{-2}\mathbf{S}_{\ddot{u}_g}$; ω is the frequency; and i is the imaginary unit. If $\mathbf{H}_{ss}\left(\omega\right)$ is the system transfer matrix corresponding to the s degrees of freedom, the spectral matrix of the response at s degrees of freedom can be expressed as

$$\mathbf{S}_r\left(\omega\right) = \mathbf{H}\left(\omega\right)\mathbf{S}_Q\left(\omega\right)\mathbf{H}^*\left(\omega\right) \tag{8}$$

where * denotes conjugate transpose of a matrix. The standard deviation of response at a degree of freedom i can be obtained as

$$\sigma_r^i = \sqrt{\int_{-\infty}^{\infty} S_r^{ii}(\omega)\,d\omega}$$
$$\sigma_{\dot{r}}^i = \sqrt{\int_{-\infty}^{\infty} \omega^2 S_r^{ii}(\omega)\,d\omega} \tag{9}$$
$$\sigma_{\ddot{r}}^i = \sqrt{\int_{-\infty}^{\infty} \omega^4 S_r^{ii}(\omega)\,d\omega}$$

where $S_r^{ii}(\omega)$ is the $(i,i)^{\text{th}}$ element of $\mathbf{S}_r(\omega)$ representing the power spectral density function of displacement response at degree of freedom i. The expected peak values of the response quantities can be obtained by multiplying the standard deviations (Equation 9) by peak factors. In this study, peak factors proposed by Cartwright and Longuet-Higgins (1956) are used.

CASE STUDY: THE HØGSFJORD TUNNEL

The plan and elevation of the proposed SFT are shown in Figure 2. The SFT consists of reinforced concrete tubes anchored by steel tension legs. The tunnel is designed to have a small vertical curvature, and a clearance of 25m at mid span. The seabed at Høgsfjord is quite irregular, with the deepest part close to the left end of the tunnel.

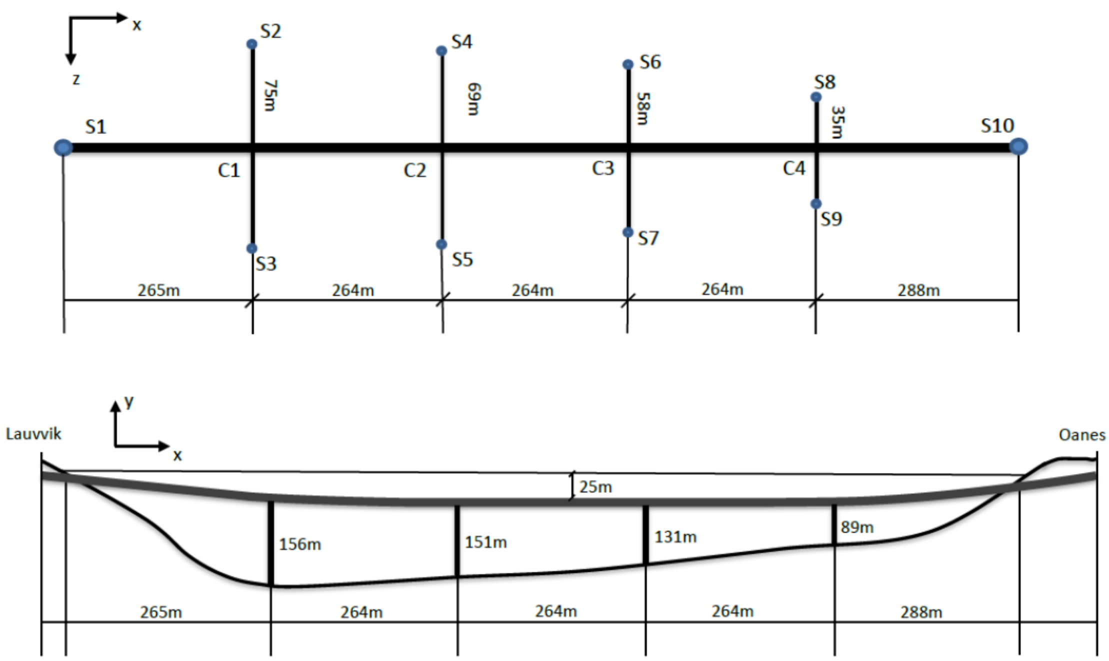

Figure 2. Plan and elevation of the Høgsfjord tunnel; S1-S10 represent the supports.

The Finite Element (FE) model

A finite element model of the SFT is constructed based on the SINTEF report (Holmas and Ferfestad, 1988). The tunnels and the tension legs are modelled as 3D Euler-Bernoulli beam elements. The cross section of the tunnel is circular with outer diameter 13.3m and thickness 0.9m. The tunnel is assumed to be made of concrete with density 3.18 kg/m3, elastic modulus

33GPa, and Poisson´s ratio 0.2. The tension legs are made from 0.4m diamter steel cabels with density 7.85 kg/m^3, elastic modulus 210GPa, and Poisson´s ratio 0.3. The tension legs are inclined at an angle of 30° to the vertical and are directed to the center of the tunnel. Geometric stiffness (Cook et al., 1982) of the tension legs is considered by including pretension corresponding to the gravity and buoyancy forces. The pretension stresses applied to the tension legs C1 – C4 (see Fig. 1) are 90, 92, 95, and 120MPa, respectively. The tunnel is assumed to lie submerged in still water; currents and waves are ignored. All the supports are considered as fixed. The FE model consists f 65 nodes, 64 elements, and 390 degrees of freedom, out of which 60 are the degrees of freedom at the 10 supports. The FE model was assembled in the FE software Abaqus. System matrices extracted from Abaqus are used to find the solution in frequency domain by using the computer program Matlab.

Since the focus of this work is on seismic excitation and effects of incoherent ground motion, loads due to waves and currents are not considered. The hydrodynamic interaction due to the earthquake-induced motion of the structure is modelled as discussed above. The coefficients for computing the added mass and damping matrices are taken as $C_M = 2$ and $C_D = 1$. It is noted that the computation of the hydrodynamic damping matrix requires an iterative procedure. The structural matrices extracted from Abaqus were used to solve the undamped free vibration problem (eigensolution) in Matlab. The wet (considering hydrodynamic effects) and dry (ignoring hydrodynamic effects) undamped natural frequencies of the first 10 modes of vibration are shown in Table 1. Structural damping is modelled by a Rayleigh damping matrix with 0.9% and 1% of critical damping in the first and the tenth mode of vibration, respectively.

Table 1. Undamped natural frequencies of the first ten modes of vibration.

Mode number	Frequency (Hz)	
	Dry	Wet
1	0.26	0.21
2	0.28	0.22
3	0.29	0.23
4	0.31	0.24
5	0.33	0.27
6	0.35	0.28
7	0.38	0.30
8	0.39	0.31
9	0.39	0.33
10	0.43	0.36

Seismic action

Seismic excitation resulting from the motion of all the ten supports is considered. The ground-motion field is modelled in the frequency domain. Power spectral density of ground acceleration is based on the modified Kanai-Tajimi model (Clough and Penzien, 2010). Power spectral densities at all the supports are taken to be the same. The spectral density functions are scaled to correspond to a peak ground acceleration of 20% of gravity. The resonant frequencies of the two filters associated with the modified Kanai-Tajimi model are taken as 15.6 rad/s and 4 rad/s, and the corresponding damping ratios are taken as 0.7 and 0.5, respectively.

The coherency function proposed by Oliveira et al. (1991) is used to model inter-support incoherence. Inter-component incoherence is modelled on the basis of analysis carried out by Sigbjörnsson et al. (2013). For simplicity, lagged coherency between the components is taken as 0.35.

PRELIMINARY RESULTS

In this preliminary study, certain simplifications in modelling of the structure and environmental actions are made. For example, potential effects of soil-structure interaction are not modelled. Velocity of water waves is ignored, although the hydrodynamic action due to the motion of the tunnel relative to water is considered. In this regard, the nonlinearity in hydrodynamic effect is simplified through stochastic linearization. The hydrodynamic damping depends on the velocity response of the structure, which is not known a priori. This necessitates an iterative solution procedure. In this preliminary study, the hydrodynamic damping is estimated by iterating on the velocity at the centre of the tunnel. For a future study, iterations over the whole structure are planned. In addition, variation of power spectral density at different supports needs to be considered in a more detailed study. Since the main objective of this study is to understand whether inter-component incoherence is influential in total response of the structure, we believe the afore-mentioned simplifications are justified for preliminary analysis.

Figure. 3 shows the peak transverse displacement along the length of the tunnel. The peak displacements were found to be associated mostly with frequencies in the range 0.2-0.4 Hz, which is also the dominant frequency region of the power spectral density function used in the analysis. The displacements are largest closer to the left end where the fjord is deepest resulting in the longest tension legs. To study the importance of inter-component incoherence, three different scenarios are analyzed: (1) considering both inter-support and inter-component incoherence (2) considering inter-support incoherence only (3) ignoring all incoherence. In Figure.4 the expected peak vertical displacement of the tunnel for the three scenarios are shown. It is seen that the fully coherent model results, generally, in the smallest peak displacement. The displacement response is seen to be increased due to the incoherence effect. The effect of inter-station incoherence seems to be less significant than the effect of inter-component incoherence. It is noted that the seismic waves are propagating parallel to the tunnel, and thus the inter-component incoherence along with the wave passage effect is expected to significant. However, the model of Oliveira et al. (1991) yields, for the average separation distannce between the supports of the studied model, a lagged coherency value of ~0.7. This value is approximately twice the average inter-component incoherence observed by Sigbjörnsson et al. (2013). This implies that, for the structure being studied, inter-component incoherence is significantly larger than inter-station incoherence. This seems to have resulted in larger peak displacement response when inter-component incoherence is included in the analysis.

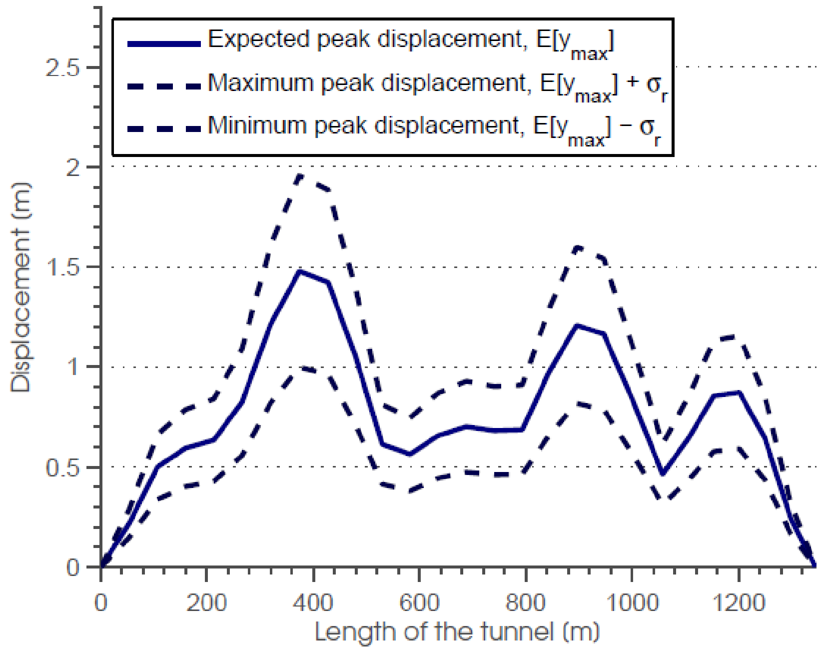

Figure 3. Peak transverse displacement along the length of the tunnel; seismic waves are propagating parallel to the tunnel axis a velocity of 500m/s. The solid trace represents the expected value while the two dashed ones correspond to the expected value plus (minus) one standard deviation.

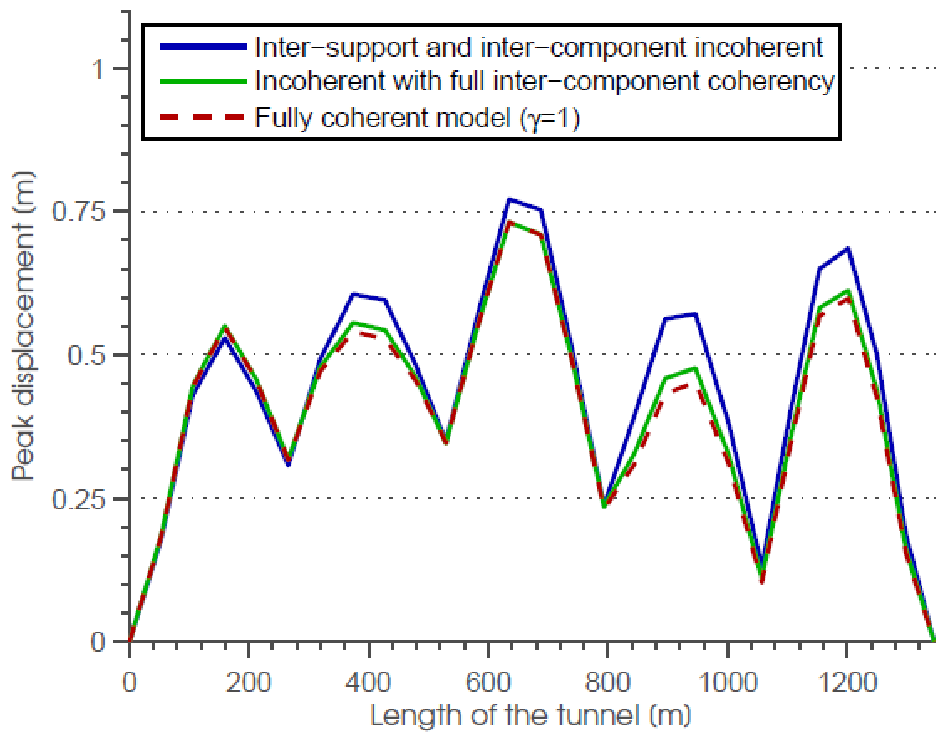

Figure 4. Expected peak vertical displacement along the length of the tunnel; seismic waves are propagating parallel to the tunnel axis a velocity of 500m/s.

CONCLUSIONS

A preliminary analysis of a submerged floating tunnel under spatially variable ground motion was carried out. The results of the analysis highlighted some interesting features. It appears that the direction of wave propagation is an important parameter that determines the overall effect of the degree of ground-motion incoherence in structural response. When seismic waves are propagating parallel to the tunnel, incoherent motion seems to produce larger structural response than fully coherent motion. But, when the waves are propagating perpendicular to the tunnel, incoherence effects seem to reduce structural response. For the structure being studied, inter-component incoherence was found to be more influential than inter-station incoherence in controlling vertical displacement response of the tunnel. Study of strong-motion array data shows that significant incoherence exists between the components (see, for example, Sigbjörnsson et al., 2013). For example, an average lagged coherency of 0.35 was observed between the components of motion recorded by the ICEARRAY during the Ölfus Earthquake in South Iceland (see, Sigbjörnsson et al., 2013 for more details). In the commonly used inter-station incoherence models, such as that of Oliveira et al. (1991) lagged coherency of 0.35 implies separation distance above 1 km at high frequencies (10 Hz), even more at low frequencies. It is then evident that for smaller separation between supports, inter-station coherency is much larger than inter-component coherency. These observations indicate that inter-component incoherence is an important feature that should be properly modelled and considered in seismic analysis of structures of the type studied here.

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