Seismic Loads on Tunnels and Buried Pipelines

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Summary

In soil dynamics analysis under earthquake excitation it is an accepted procedure to assume that:

a) the free field seismic waves propagate vertically as plane waves;

b) the soil local or "instrinsic" dissipation is represented by a viscous damping, the constant ranging from 0.05 to 0.1 relative to the critical one;

c) a horizontal rigid bedrock is present at a depth of the order of some embedment lenghts.

Assumptions (a), (b) and (c) simplify the computational procedure and besides are proved to be conservative for shallow embedded structures. Under the same assumptions underground structures can benefit of a reduction in accelerations and in equipment dynamic loads of a factor of approximately two, if compared to similar structures but shallow embedded.

However the present paper shows that the above assumptions are not always at the safe side when a long span embedded structures is concerned. In particular, body waves - P or S - propagating both vertically and horizontally may provide larger loads than a vertically propagating wave does. Besides, for the same free field surface motion, higher damping values of soil will result in higher earth pressures. On the order hand, the hypothesis of the presence of a rigid bedrock may overestimate the seismic effects.

Therefore a soil dynamic model was developed allowing to remove the assumptions (a), (b) and (c) above. It works in time domain, and plane geometry. A lumped parameter model was also developed to the same purpose. Typical results are shown.

Introduction

In the solution of earthquake engineering problems concerning soil-structure interaction, seismic waves are usually assumed to propagate vertically with plane wave front. Such assumptions are generally conservative in the case of shallow embedded structures.

However, for the case of long span embedded structures (as tunnels and buried pipelines), the assumption of vertical propagation leads to underestimate stresses in the structure. Indeed, inclined waves may produce differential displacements along the longitudinal axis of the structure, yielding an additional state of stress which would not be considered in the assumption above.

A few field observations during recent earth shakings, in particular San Fernando 71, show deviations from the above assumptions, thus suggesting, as design rule, to investigate the seismic behaviour under different wave front inclinations and curvatures. The variety of source mechanisms and propagation paths likely to be met, precludes a more deterministic assessment of the seismic input.

Under these premises, Fig.1 shows in a schematic way the problem dealt with. This will be analyzed by two different models, each working in the elastic range for both the soil and the structure: 1) a model with lumped springs, masses, and dashpots, and 2) a finite element model. The differences between the two will be explained through the equilibrium equation of the tunnel masses along the Z axis.

Let the following be:

$\{z\}$ the displacement vector in direction Z for the tunnel nodal points;

$|M|$ the mass matrix of the tunnel elements;

$|C|$ the matrix of the viscous coefficients associated with the tunnel deformation. The term $c_{ij}(\dot{z}_i - \dot{z}_j)$ represents the viscous force arising within the tunnel against the relative motion of the ith and jth nodal points;

$|K|$ the stiffness matrix of the tunnel. The term $k_{ij}(z_i - z_j)$ represents the elastic force arising within the tunnel against the relative motion of the ith and jth nodal points;

$|R|$ the soil radiation matrix. The term $r_{ij}[(\dot{z}_i - \dot{u}_{zi}) - (\dot{z}_j - \dot{u}_{zj})]$ represents a force exerted by the soil against the motion relative to the free-field of the ith and jth points;

$|B|$ the soil compliance matrix. The term $b_{ij}[(z_i - u_{zi}) - (z_j - u_{zj})]$ represents a force exerted by the soil against the motion relative to the free-field of the ith and jth points;

$\{u_z\}$ the free-field motion assumed for the tunnel nodal points.

The equilibrium equation along the Z direction yields:

$$|M|\ \{\ddot{z}\} + |C|\ \{\dot{z}\} + |R|\ \{\dot{z} - \dot{u}_z\} + |K|\ \{z\} + |B|\ \{z - u_z\} = 0 \quad (1)$$

and, putting into evidence the free-field motion,

$$|M| \{\ddot{z}\} + |C| \{\dot{z}\} + |R| \{\dot{z}\} + |K| \{z\} + |B| \{z\} = |R| \{\dot{u}_z\} + |B| \{u_z\} \quad (2)$$

It is worth noting that it is not correct to express the internal action within the tunnel in terms of displacements relative to the ground, since the ground motion is different point by point.

In the assumptions of the Winkler model, the matrix $|B|$ is diagonal and, analogously, the radiation matrix $|R|$ is diagonal too. By an extension of the viscous boundary theory, it will be assumed that the viscous force perpendicular to the horizontal surface A_i^h, associated with the ith nodal point, is $\rho V_p A_i^h(\dot{z}_i - \dot{u}_{zi})$, where ρ is the soil density, and V_p is the compressional wave velocity in the soil. The concurrent viscous force parallel to the lateral surface A_i^l is $\rho V_s A_i^l(\dot{z}_i - \dot{u}_{zi})$, where V_s is the shear wave velocity in the soil. For brevity, in the following these assumptions will be referred to as generalized Winkler model assumptions.

Similar equations hold for the equilibrium along the X and Y axes.

Eq. 2 will be solved through the lumped parameters model of Fig. 2, within the realm of the generalized Winkler assumptions. The fixed ends of the supporting springs and dashpots are supposed to move with the free-field motion at the level of the tunnel axis, which is equivalent to assume the fixed ends at rest, and the ith nodal point acted on by forces $f_z = r_{ii}\ \dot{u}_{zi} + b_{ii}\ u_{zi}$.

Eq. 2 with also be solved through a two-dimensional f.e. model, Fig. 3. In this case the matrices $|R|$ and $|B|$ are evaluated by the f.e. procedure, which is consistent with continuum mechanics.

The above assumptions are worth commenting. As to the input excitation, let Fig. 4 be considered, where u_x and $\dot{u}_x$ are reported for vertically incident waves. The corresponding ground acceleration is in agreement with USNRC R.G. 1.60, normalized to 0.22 g.

It can be recognized that, as far as the soil is sufficiently stiff ($G > 30000\ t/m^2$ in the present case), the contribution to the input force supplied by $b_{ii}\ u_{zi}$, is of a few orders of magnitude greater than the contribution given by $r_{ii}\ \dot{u}_z$. Consequently this last term may be disregarded. Similarly, the inertial and damping terms of the left side of Eq. 2 may be neglected also, thus reducing the analysis to a "static" case, although subject to time varying displacements, as proposed by [6]. On the other hand, it must be observed that in the range of values of the soil stiffness for which such assumptions hold, the maximum strains and curvature of the tunnel do not necessarily coincide with the maximum strains and curvature of the free-field soil, as proposed by Newmark [6]. In particular, this is the case when an upper bound is assigned to the bond strength between soil and concrete. [4,8].

A variety of simplifying assumptions are thus feasible, depending on the soil compliance around the tunnel. However, as far as a lumped parameters model is concerned, there is no particular need of disregarding any term or to keep the analysis in the static realm. Therefore, the feasibility of the above assumptions will be no further investigated, and the following will concern only the comparison between f.e. and lumped parameters models.

The radiation matrix |R| and the compliance matrix |B| are likely to be represented inaccurately in both models: in the f.e. model, because the approach is two dimensional; and in the lumped parameters model, because non diagonal terms are disregarded. Besides, according to the literature [1,4,5,7] there is not a unique way to assign values to b_{ii} (see table I). Mainly for this reason, the reference model will be considered the one relying on a f.e. representation.

Only a few comparisons, limited to SV excitation only, have been carried out up to now. The main conclusions are the following:

1) when all other parameters are kept constant, and only the vertical component of the Winkler moduli are changed, then the axial stress N remains unchanged, in practice, but both shear and bending stresses change. An amplification of the moduli by a factor 4 - see table I - provides an amplification of the maxima of shear and bending by factors of 3 and 2.4 respectively.

2) The f.e. model and the lumped parameters model lead to resulting stresses in qualitative agreement. However, the first model provides more optimistic values of stresses, probably due to a more realistic representation of radiation damping and soil compliance.

3) For small incident angles ($\alpha < 20°$), shear and bending stresses are directly proportional to α. Axial stress also increase with α but to a smaller extent.

As a final comment, by looking at the function u_x in Fig.4, it appears that assigning the maximum value of ground displacement proportional to the maximum acceleration, as R.G. 1.60 does, is overconservative for stiff soils. For these, high values of acceleration but relatively low values of soil displacements are expected.

Table 1 - Values of the Winkler constant b_{zi} according to different authors for the soil in Fig.1

	Ref. 4	Ref. 7	Ref. 5	lower bound considered in the analysis
b_{zi} (t/m/m)	60.000	50.000	66.000	13.500

2. Generalized input excitation

The input excitation is defined by a train of obliquely incident, plane seismic waves having the following characteristics:

1) The three orthogonal components of the motion of a point at the free field surface are defined on the basis of USNRC Regulatory Guide 1.60. According to this, two acceleration response spectra, $s_h(\omega,\nu)$ and $s_z(\omega,\nu)$, are assigned to the two horizontal components and to the vertical respectively.

2) The motion mentioned above is originated by compressional and distortional plane waves, impinging upon a horizontally layered soil, lying on a homogeneous isotropic halfspace, where the waves are propagating along a prescribed direction, defined by the angle α, Fig. 1.

The proposed procedure requires the solution of two problems. The first is the well known problem of generating an acceleration time history, u(t) given the acceleration response spectrum. The second one is the evaluation of the amplitude of obliquely incident waves corresponding to the assigned time history.

In particular let $\ddot{u}_x(t)$ be a generic function of a stochastic process $\{\ddot{u}_x(t)\}$, the response spectrum of which approaches the R.G. 1.60 horizontal response spectrum, $s_h(\omega,\nu)$. Several methods are available to generate such function, yet they will not be discussed here.

The only assumption is that, given a value ν of the damping coefficient, the specified target response spectrum can be approached as close as necessary by a series of subsequent iterations.

The chosen function will be represented with a complex Fourier expansion

$$\ddot{u}_x(t) = \sum_{1}^{N} {}_n\, u_{xn} \quad e^{j\omega_n t},$$

where

$$\omega_n = \frac{2\pi}{T}\, n,$$

T ground motion duration

$\omega_n = \frac{2\pi}{T} N$, maximum frequency of the significant Fourier components. The $\ddot{u}_y$ and $\ddot{u}_z$ components are defined similarly.

As to the second problem, a mathematical model in one dimension, z, capable of reproducing a free field surface motion shall be used to derive the P, SV, and SH incident wave amplitudes. Two different formulations are available for this operation, [3,10]. Both of them work in frequency domain, i.e., for this case the steady state, harmonic components of the surface acceleration, for the horizontally layered soil of Fig. 1, are provided once the displacement amplitude s(t) of a harmonic SV wave, and the one, p(t) , of a compressional wave,

$$s(t) = s_n \quad e^{j\omega_n t}$$

$$p(t) = p_n \quad e^{j\omega_n t}$$

are given.

Let $H_{sx}(\omega)$ and $H_{px}(\omega)$ be the complex ratios

$$H_{sx} = \frac{u_{xn}}{s_n} \ ; \quad H_{px} = \frac{u_{xn}}{p_r}$$

It can easily be proved that the impinging SV wave able to induce a surface motion $\ddot{u}_x(t)$ is

$$s(t) = \sum_{1}^{N} {}_n \ \frac{u_{xn}}{H_{sx}(\omega_n)} \quad e^{j\omega_n t}$$

and analogously for p(t). The proof -here omitted- is limited to show that the "transient portion" of the response to s(t), which is disregarded in this operation, is meanigless from a numerical point of view.

It must be pointed out that both s(t) and p(t) are able to produce the same $u_x(t)$. However, let $u_z^s(t)$ and $u_z^p(t)$ be the vertical components of the surface motion associated with s(t) and p(t) respectively. In general, neither of them reproduces the USNRC R.G. 1.60 vertical response spectrum $s_z(\omega,\nu)$.

The model being linear, an impinging motion characterized by A s(t) simultaneous to (1-A)p(t) also produces the target $\ddot{u}_x(t)$ at the surface. The ratio A, therefore, is available to minimize the difference between the resulting vertical response spectrum and the target $s_z(\omega,\nu)$ in the selected range of frequencies.

The method is in the main aspects consistent with USNRC R.G. 1.60 or the corresponding Standard Review Plan 3.8.1, except for the statistical dependency of $\ddot{u}_x(t)$ and $\ddot{u}_z(t)$. However, it must be noted that such dependency is an intrinsic characteristic of obliquely incident seismic waves.

As to the y component of motion, it can be generated by a SH wave, by a similar procedure. The mathematical model for obliquely incident SH wave is described in the reports mentioned above [3, 10]. The resulting motion can be, in theory, statistically independent from $\ddot{u}_x$ and $\ddot{u}_z$.

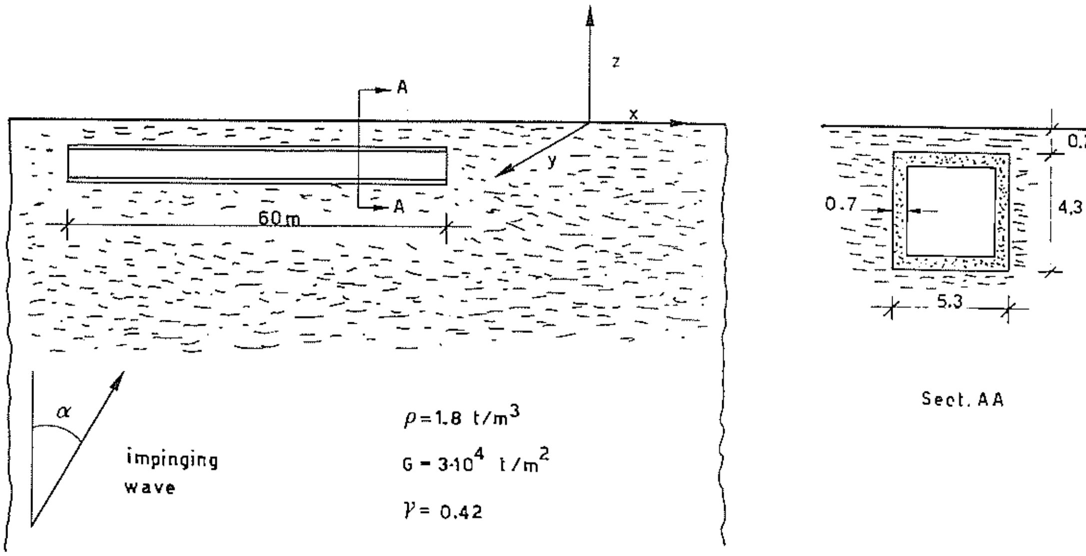

Fig.1 Representation of the problem.

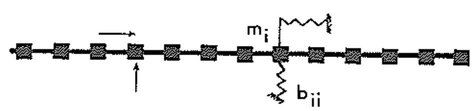

Fig.2 Lamped parameters model.

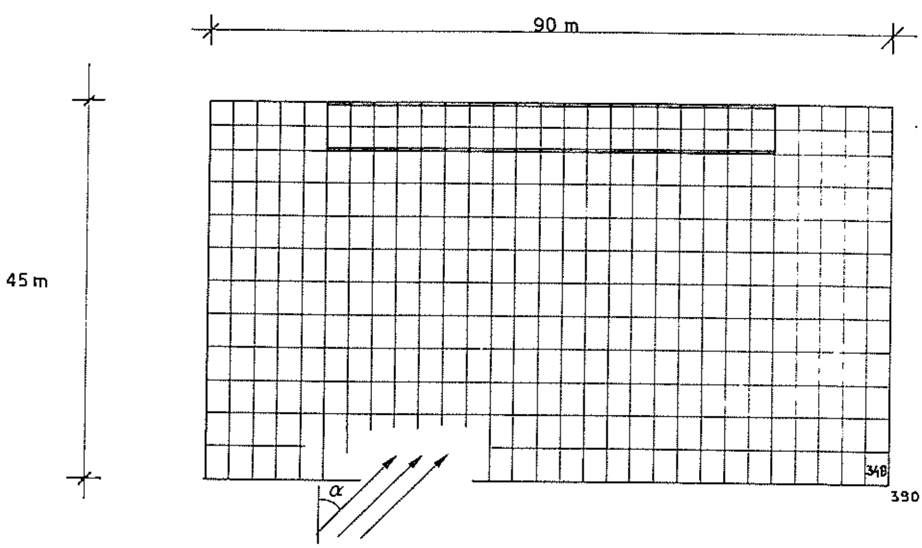

Fig.3 Finite element model. Active and passive boundaries according to (2).

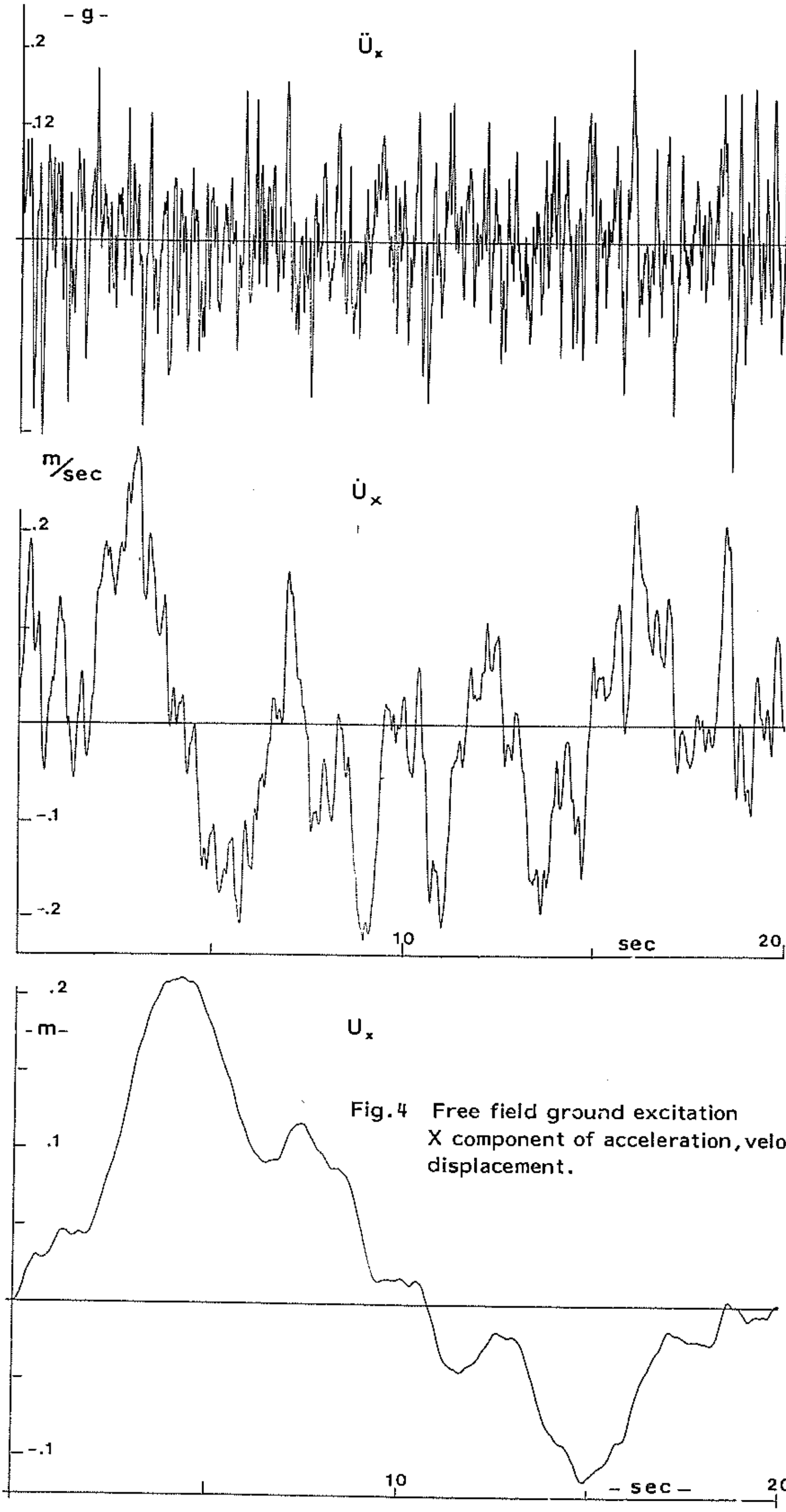

Fig. 4 Free field ground excitation
X component of acceleration, velocity and displacement.

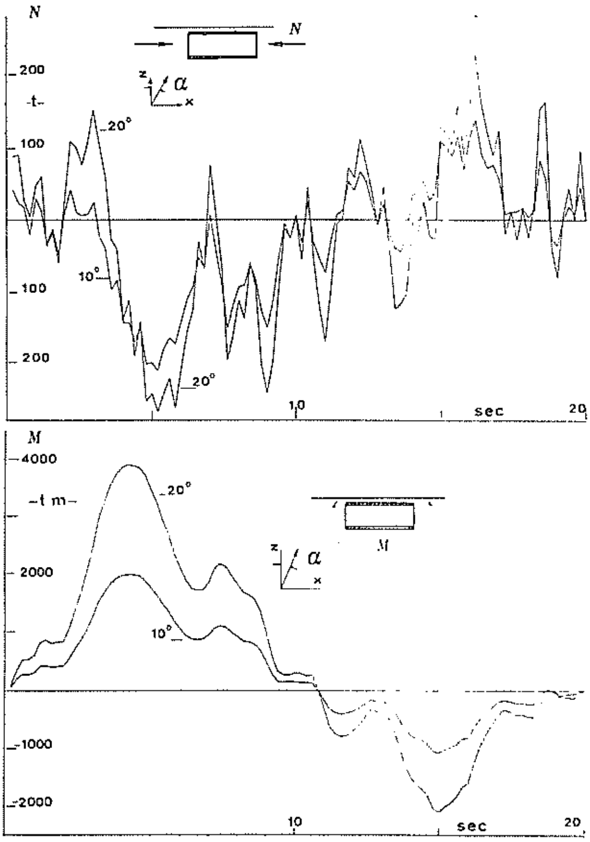

Fig.5 Time history of N,M components at the tunnel central point.Lower bound value for Winkler constant.

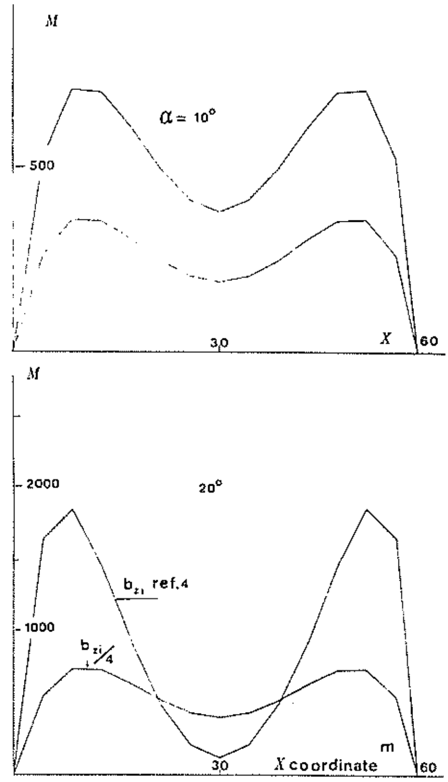

Fig.6 Profile of bending moment at t = 4.2 sec.

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