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# Ground Waves Generated by Pile Driving, and Structural Interaction

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ABSTRACT

Pre-formed steel or concrete pile elements are installed by high energy impact or vibro-driver, which causes outgoing ground waves. In severe cases, adjacent buildings are at risk of damage. Assessment of risk is conventionally by reference to threshold limits of vibration. The global approach considers neither the interactive effects between ground and structure, nor frequency and duration.

Here, firstly, the dynamics of a pile head impact and of the transmission of a portion of the energy into the ground were modelled by a combination of finite elements (FE), springs and dashpots. The boundary disturbances were then applied to a second model of the soil as an elastic half space. This outer model was constructed of axisymmetric finite and infinite elements for calibration against on-site measurements. The infinite elements (IE) represented a wider zone, and avoided spurious wave reflections at boundaries.

Next, the verified ground disturbances adjacent to the pile were used as input to a three-dimensional FE/IE wedge-shaped model of a 'slice' of the axisymmetric system. Various structural forms, of steel frame structures and of brick walls, were added, giving a dynamic soil-structure analysis. Results show the responses of flexible and stiff structures to outgoing waves caused by impact pile driving and vibro-driving.

INTRODUCTION

Pre-formed pile elements, either interlocking sheet piles or bearing piles of steel, concrete or timber, are normally installed deep into the ground by heavy impact or by vibratory means. Both methods require high energy devices for driveability. Some of the energy is transmitted into the surrounding ground, which sets up outgoing ground waves, eg Attewell & Farmer [1973], Attewell *et al* [1991] and Selby [1991]. These attenuate with distance from the pile. A large number of site measurements of surface vibrations have been made, e.g. Uromeihy [1990] and Hiller [2000]. In severe cases, buildings and buried services in the near vicinity may be at risk of cosmetic or minor structural damage, Head & Jardine [1992], Wiss [1967] and Todd [1994].

A well-established method of assessing the severity of risk is by reference to threshold values of vibration impinging onto the building perimeter. Different values are applied to various building types – heavy industrial, domestic, ancient or historic structures, buried pipelines, retaining walls, BS5228 [1992], BS7385 [1993] and Eurocode 3 [1996]. This global approach considers neither the interactive effects of foundation and structure, nor detailed frequency and duration.

The approach presented here is an attempt to address some of these points in detail, by using computational methods to generate ground waves, and then to include simple structures so as to achieve an interactive dynamic analysis of structural response.

The computation of ground waves caused by piling has to be broken down into sub-systems. For impact driving, a spring-mass-damper system models the pile head impact; a second system represents the pile shaft, toe and immediate soil; thirdly an FE axisymmetric mesh is used to compute the outgoing waves. For vibratory driving a two stage system is required. The first stage models the sinusoidal excitation of the vibrodriver, pile and adjacent soil; next, a finite/infinite element mesh is required to compute the sinusoidal ground waves, while avoiding spurious reflections from artificial mesh boundaries. The two approaches, for impact and vibrodriving have been verified against site data by Ramshaw *et al* [2000], and small further refinements are reported in the following section.

In this next phase of the work, the verified disturbances around the immediate pile soil analysis were used as input to a 3D FE/IE wedge-shaped model of a 'slice' of the axisymmetric system. This model, too, was verified against the site-measured vibrations. It was now possible to add in to the latter model, various structural forms typical of steel frame structures and of domestic brick walls. This approach gives a full soil-structure analysis of the system, allowing appreciation of the effects of different wave types and structural forms.

METHOD FOR IMPACT HAMMERS

The approach used for an impact hammer is based on the work by Deeks & Randolph [1993], and is explained in more detail by Ramshaw *et al* [2000]. Briefly, the first stage to examine the impact between the hammer, the cushion and the pile head uses the spring-mass-damper scheme shown in Figure 1.

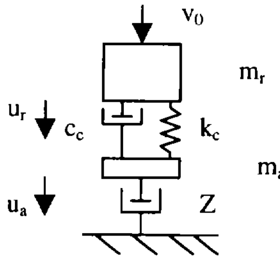



Figure 1. Pile head impact.

The symbols in the figure define the hammer mass and velocity, $m_r$ and $v_0$ , characteristics of the dolly, $k_c$ , $m_a$ and $c_c$ and Z is the impedance of the pile.

The wave travelling down the pile shaft is next modelled by a further system of spring-mass-dampers, from which the energy transmitted out into the soil can be estimated. A schematic is shown in Figure 2.

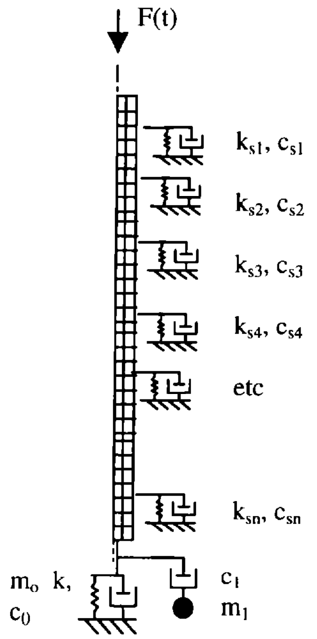



Figure 2. Schematic of pile-soil model.

The boundary disturbances are then applied as input to a second model of the soil in the wider sense. This outer model was constructed of axisymmetric finite elements for calibration against on-site measurements of surface vibrations.

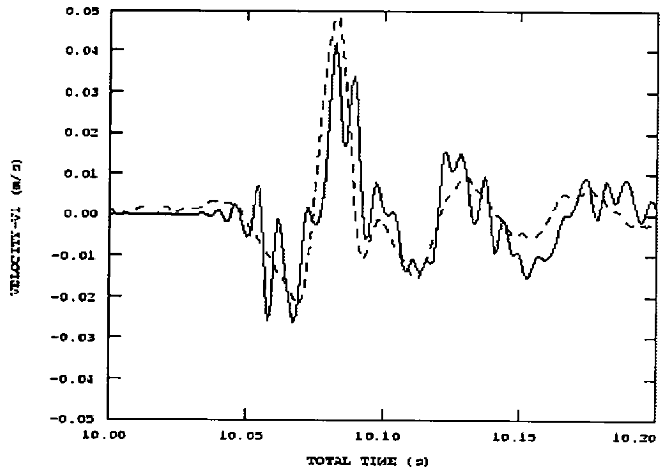
An example of a calibration of the procedure against site data is given for a site at Flitwick, Bedfordshire, UK where a 12m long steel H-pile (305 x 305 x 89kg/m) was installed to a depth of 7m by a 3200kg hammer falling through 1.0m. The parameters used in each stage of the analysis are summarised in Tables 1 and 2.

| SOIL TYPE | E (MPa) | ν | ρ (kg/m$^3$) | $k_s$ (N/m) | $c_s$ (N/ms) |
|---|---|---|---|---|---|
| Soft Clay (0.0 – 2.4m) | 5 | 0.45 | 1920 | 5.65e$^6$ | 57.55e$^3$ |
| Loose Sand & Gravel (2.4 – 4.8m) | 20 | 0.33 | 1750 | 24.65e$^6$ | 114.7e$^3$ |
| Dense Sand (>4.8m) | 50 | 0.25 | 2000 | 65.57e$^6$ | 200.0e$^3$ |

Table 1. Soil and shaft resistance parameters

| Pile head parameters | Pile toe parameters |
|---|---|
| $m_r$ = 3200 kg | $m_0$ = 0.0 |
| $k_c$ = 63.85e$^6$ N/m | k = 16.27e$^6$ N/m |
| $c_c$ = 226e$^3$ N/ms | $c_0$ = 19.85e$^3$ N/ms |
| $m_a$ = 320 kg | $m_l$ = 3.97 kg |
| Z = 452e$^3$ Ns/m | $c_l$ = 4.78e$^3$ N/ms |

Table 2: Parameters for pile head impact and pile toe resistance



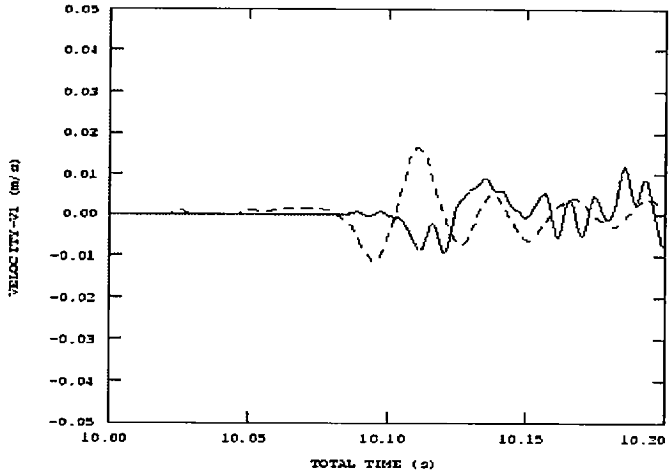




Figure 3. Radial ppv's at 7m and 16.5m respectively. (Measured = dashed line).

The computed peak particle velocities (ppv's) at the ground surface for the pile at 7m depth are compared to the measured ppv's in Figure 3. These computations assume a small strain stiffness of 20MPa for the soft clay layer above the water table, and 200MPa for the sands and gravels below the water table. The pile-soil interface is modelled using a surface-based contact simulation, with slip controlled by a Coulomb friction model with $\mu = 0.4$.

The correlations between the computed and measured radial ppv's in Figure 3 show very close agreement, as do the equivalent vertical ppv's. The main discrepancy appears to be due to P-wave from the pile shaft which becomes more obvious at the 16.5m geophone as the P-waves and S-waves separate due to their different propagation speeds. The P-wave generated from the shaft may be caused by an eccentric strike. This effect is not simulated in the model.

## METHOD FOR VIBRATORY DRIVING

The equivalent procedure to estimate ground waves set up by vibratory pile installation is made in two-stages. In stage one, the objective is to establish a model for rigid body vertical oscillation of the pile in response to the cyclic excitation of the vibro-driver. This is done by the use of rigid axi-symmetric elements for the pile shaft, a limited axi-symmetric FE/IE mesh representing the soil, and a mechanical model for toe reaction based on a spring and a dashpot in parallel, proposed by Lysmer & Richart (1966). The pile-soil interface comprises a two-surface contact, with a Coulomb friction model. A static computation is made for the soil/shaft normal stresses due to geostatic stress. A dynamic analysis is then conducted to set up steady state response of the rigid pile to the cyclic excitation from the hammer. The soil disturbance is characteristically sinusoidal in form.

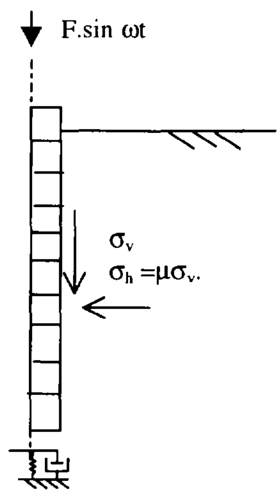



Figure 4. Schematic of vibrodriver/pile/soil.

A schematic of the first stage is shown in Figure 4. The use of *dynamic* infinite elements presents a problem when it is necessary to incorporate self-weight static stresses for the Coulomb friction, since these infinite elements offer damping but no stiffness. A technique to avoid the large rigid body displacements of the FE part of the mesh was devised which used an expansion of the pile shaft into the FE mesh. This generated the correct normal stresses between the shaft and the soil.

This two-stage procedure was then applied to an example of vibratory pile installation at the same site at Flitwick. In this case, the 12m long steel H-pile was installed using a PTC 13HF1 vibratory hammer with an eccentric moment of 13m.kg. The computed ppv's at the ground surface for the pile at 7m depth at a frequency of 19.1 Hz are shown in Figure 5.

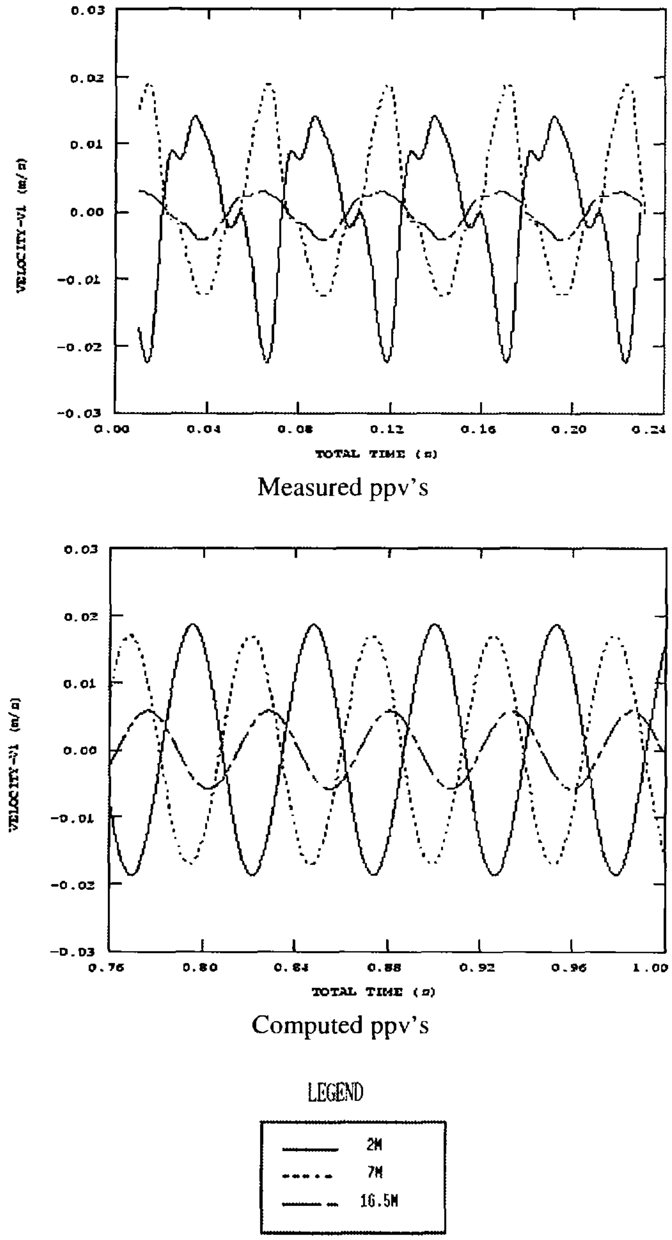



Figure 5. Measured and computed radial ppv's at 2m, 7m and 16.5m.

These computations assumed an overall small strain stiffness of 155MPa for the soil. Slip on the pile-soil interface was controlled by a Coulomb friction model with $\mu = 0.5$. A typical damping ratio of 5% was applied which is within the elastic range of soil deformations.

The form of the response is quite different to that of an impact driver, being sinusoidal, at a frequency dictated by that of the vibrodriver. Adequate agreement between measured and computed vibrations is achieved, again by correct selection of a number of pile and soil parameters.

## A SOIL-STRUCTURE MODEL

While the ideal FE/IE configuration for a pile and for the soil is axi-symmetric, the natural choice for a section through a frame structure, or for a wall element of a brick building, is plane stress. A full representation of a pile-soil-structure would require a full 3D model, which suffers from poor representation of the elastic half-space by comparison with the axi-symmetric elements, and an excessive requirement of computing resource because of the constraints of the time-stepping solution.

A compromise solution was sought comprising a 'wedge' of the axi-symmetric soil represented with 3D fanned elements, which allowed connectivity with a 2D structure. A simplified view of the system is shown in Figure 6. In practice a much finer FE mesh was used.

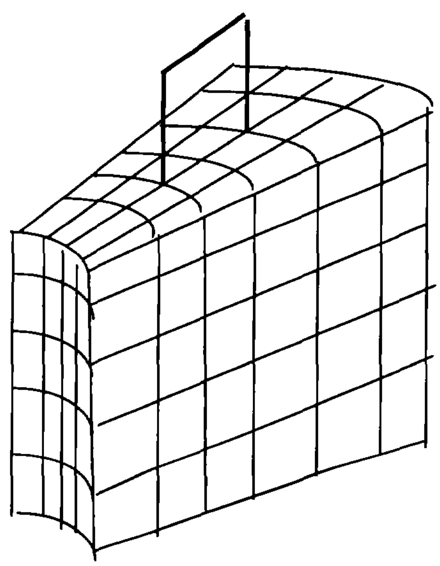


Figure 6. Simplified view of soil and structure.

Transient displacements were imposed onto the curved vertical face nearest to the pile, using values from the preliminary stages of the axi-symmetric analyses discussed in the previous sections. The system shown in Figure 6 was suitable for impact, provided that the mesh was large enough to avoid reflections from boundaries while the structure was responding to transient ground waves spreading outwards from source. For the situation of vibrodriving, infinite elements were added around the outer boundaries.

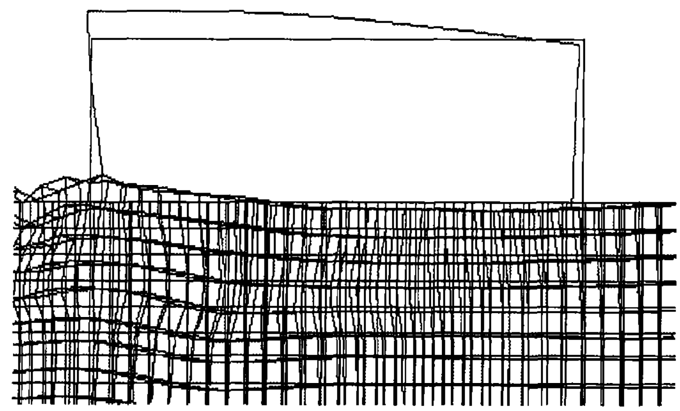
Calibrations of the 'wedge' model of the soil, without the structure, were made for the impact and vibrodriver cases of the previous sections, and very similar levels of agreement were achieved.

The wedge of soil plus a variety of simple structures was then analysed as described in the following sections.

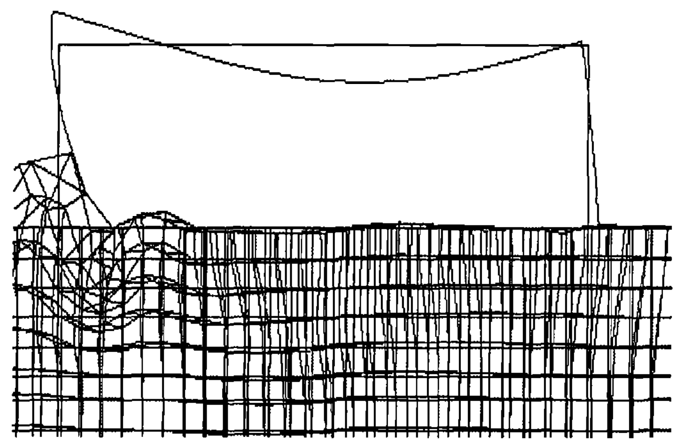
## IMPACT WAVES ON SOIL AND STRUCTURE

A single bay rectangular steel portal on pad footings was chosen for addition to the soil surface, when waves from impact driving spread outwards. The portal had a 12.5m span and was 3m high, and comprised 203 x 203 x 60 Universal Columns and 610 x 229 x 125 Universal Beams.

Typical deformations as the waves passed through are shown in Figure 7. The response is a function of ground wave length, with peak distress to the frame when the feet of the columns are in anti-phase.



Time = 0.15 secs



Time = 0.25 secs

Figure 7. Frame deformation due to a transient wave. (Magnified x 5000)

For comparison, the transient displacements of the free ground surface with no structure were compared to those with the portal in place. This showed that the portal causes hardly any modification to the ground surface waves. In such cases, it would therefore be acceptable, and time-saving, to impose the ground wave displacements directly onto the structure alone.

In a second example, a brickwork wall, 6m high, 10m long and 0.2m thick was superimposed onto the 'wedge' of soil. The in-plane response of the wall was typical of a very stiff structure, in that it showed rigid-body movements of lift and pitch, but only very small deformations, see Figure 8. Contour plots of the stresses in the wall (Figure 9) indicate that the dominant effect is due to the *horizontal* ground displacements, peaking at about 25kPa at the base of the wall. The maximum vertical stresses were of the order of 8kPa.

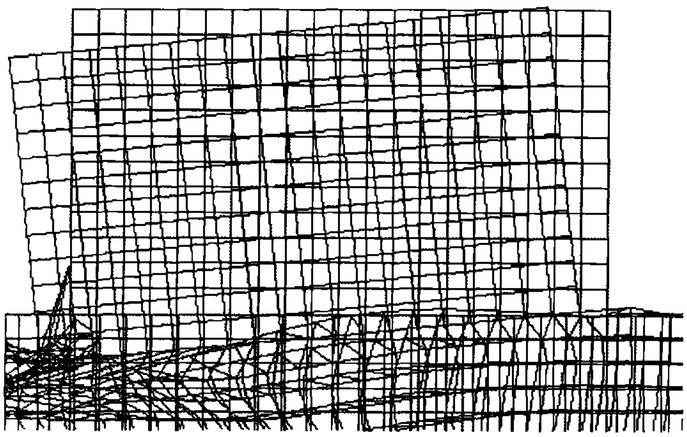


Figure 8. Wall deformation due to a transient wave. (Magnified x 16047)

Figure 9. Horizontal stresses in wall due to a transient wave.

VIBRATORY WAVES ON SOIL AND STRUCTURE

The steel portal frame on the soil 'wedge' was subjected to outgoing continuous sinusoidal waves, similar to those observed on the site at Flitwick. For the chosen combination of vibrodriver frequency and frame dimensions, there was no resonance in evidence. As before, with the transient disturbance, the frame offers very little modification to the free-ground deformations. Further cases will be studied when resonance of a frame or part of a structure will occur.

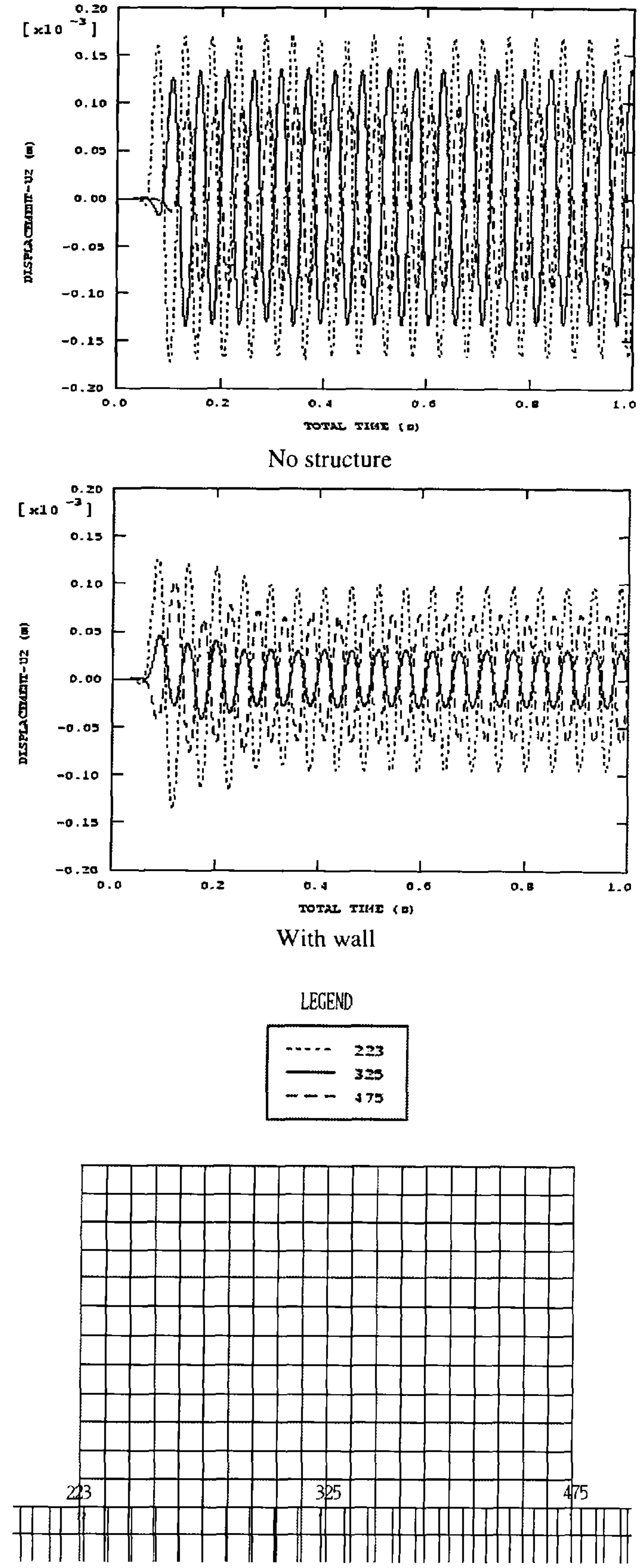



Figure 10. Vertical displacements of three nodes (223, 325 and 475) showing reduction due to presence of wall

Finally, sinusoidal excitation of the wedge plus an in-plane brick wall is examined. The presence of the wall substantially reduces the ground movements as shown in Figure 10. It is therefore inadmissible to impose the *free ground* displacements onto a very stiff structure and so the whole three-dimensional analysis is necessary.

## CONCLUSIONS

Methods have been developed which allow computation of ground waves due to either impact or vibratory pile driving. The outgoing waves have been compared with site measurements of surface velocities, and close agreement is achieved by judicious choice of several parameters.

These same waves were also generated in a 3D soil 'wedge' which was used to represent a part of the axi-symmetric half space of soil centred on the pile axis.

Finally, it has been shown that the method can be used to identify dynamic displacements and stresses induced into simple structures, which incorporates dynamic soil-structure interaction.

When the structure comprises a slender frame, then imposition of free-ground deformations gives a close representation of the coupled behaviour. However, stiff structures such as in-plane walls show substantial reduction of the free ground movements. When considering induced stresses, both vertical and horizontal wave components must be included.

## ACKNOWLEDGEMENTS.

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