INVESTIGATION ON CHARACTERISTICS OF VIBRATION INDUCED BY CONSTRUCTION ACTIVITIES AND ITS EFFECT ON STRUCTURAL RESPONSES

W.A.M.Wanniarachchi¹, C.M. Welhena¹, G.H.M.J. Subashi De Silva², G.S.Y. De Silva²

Department of Civil and Environmental Engineering, Faculty of Engineering, University of Ruhuna,
Hapugala, Galle, Sri Lanka

¹Research Student Email: ayalmaneth@gmail.com
²Senior Lecturer Email: subashi@cee.ruh.ac.lk
³Senior Lecturer Email: sudhira@cee.ruh.ac.lk

Abstract

Vibration due to construction activities is a significant problem in urban areas. Due to the scarcity of proper lands, people tend to use soft soil lands with deep foundations such as pile foundations, and ground improvement techniques such as dynamic compaction. However, these activities induce ground vibration and make damages to nearby structures. When the energy left over from a construction activity, it transmits to the surrounding; particles in their paths are displaced by these waves giving rise to particle velocities. However, structural response to these vibrations may vary according to the soil type, soil-structure interaction and characteristics of the structure. The characteristics of vibrations may also vary according to the type of construction activity. This paper presents a study on investigating the characteristics of vibration induced by the construction activities and effect of the vibration on structural responses.

In the current study, vibrations induced by pile driving, roller compaction and rammer compaction were measured by using a seismograph. Characteristics of measured vibration are compared and evaluated with respected to the guidelines given by local and international standards. Numerical modeling was carried out with the Finite Element Modeling (FEM) using commercially available software, SAP2000 (ver16). Structural responses of a wall panel (having a size of 3m x 5m) to dynamic loads were determined. A possible method to control structural responses was determined by introducing a reinforced concrete frame to the wall. It was found that the vibration induced by pile driving has high Peak Particle Velocity (PPV)value,around20 mm/s, in the frequency range of 7 - 12Hz. Ground vibration (PPV) induced by roller compaction and rammer compaction is at around 2.6mm/s and2.0 mm/s, respectively. Effectiveness of reinforced concrete frame to control dynamic responses of structures caused by the ground vibration was evaluated and discussed in this paper.

Keywords: Ground vibration, Structural damages, Guidelines, Constructions, FEM
1.0 Introduction

With the rapid development in Sri Lanka, construction activities are increasing day by day. Due to the scarcity of proper lands, these developments projects are often carried out in densely populated areas and close to sensitive structures and installations. As a result, construction activities often generate community vibration complaints, even when it takes place over a limited time frame. For examples, there were many complaints from public on rock blasting activities nearby main quarry sites in Hambantota Harbor Project (The Nation, 2009) and the Southern Transport Development Project (Environmental Impact Monitoring Report, 2010). In addition, vibration induced by traffic and rail transport systems is also significant to generate cracks and damages to buildings. For example, there was a significant vibration percept at the “Great Vila-Reception hall” in Dodanduwa, Galle, due to the rail transport systems at the coastal region of Sri Lanka.

Most types of construction activities, such as soil and rock excavation, driving of piles and sheet piles or compaction work, generate vibrations in surrounding soil layers. During the process of pile driving at construction site, an unavoidable ground vibration generates and it can causes significant damages to the nearest structures in urban areas. The magnitude of vibration received to the building depends on the type of soil, in which the piles are driven, building type, building material, and foundation type (Massarsch & Fellenius, 2008).

Identification of structural damages due to ground vibration may be a difficult task because damage of structures may not only be caused by vibrations transmitted directly to structures but can also be caused by differential settlement in soil, bad weather conditions and design failures (Kim & Drabkin, 1995). In order to identify the mechanism responsible for damages, it is important to investigate the characteristics of ground vibration induced by different construction activities. In addition, knowing the vibration characteristics such as Peak Particle Velocity (PPV), frequency components, amplitude of the ground vibration induced by construction activities will be useful to maintain the allowable limits, as a result, the structural damages as well as the public complains will be reduced.

On the other hand, introducing a vibration control technique through the path of propagation of ground vibration or to the building will cause to reduce structural damages. For example, constructing a suitable wave barrier can be used to minimize the propagation of ground vibration to the structures (Seyhanet al., 2010). There are several vibration damage control techniques that can be used for building. Control techniques such as, installing shear walls, introducing a concrete frame and installing dampers have to be selected appropriately, considering the type of buildings. For example, the vibration damage control technique that suits to high rise buildings may not be appropriate for single storey buildings when the cost is considered. It is important to identify a suitable and effective vibration damage control technique for different type of buildings, specially the masonry buildings, as there are many masonry structures in Sri Lanka and they are more vulnerable to damage due to dynamic loadings induced by ground vibrations.

In the current study, levels of ground vibration induced by construction activities and its effect on structural responses have been investigated. Ground vibrations induced by pile driving were measured, considering the hammer type, pile length and pile size. In addition, ground vibration induced by soil compaction was measured. Characteristics of the measured vibration were compared and evaluated with respected to the guidelines given by local (i.e., Central Environmental Authority: CEA) and international (i.e., BS 7385 -1) standards. Finite Element Model (FEM) analysis has been
conducted by using a simple wall panel which is more vulnerable to damage due to dynamic loadings, to identify the structural response and possible techniques to reduce the damage on structures.

2.0 Methodology

To investigate the characteristics of ground vibration induced by pile driving and soil compaction, and dynamic performances of structures due to construction vibration, the following methodologies were used.

- Experimental measurements of ground vibration induced by construction activities.
- FEM analysis of a selected structure subjected to measured ground vibrations.

2.1 Experimental Measurements

Initial site investigations were performed so as to determine appropriate locations to install instrument and to measure ground vibration. Four channel seismograph (Figure 1(a)) capable of measuring vibration in three directions (i.e., Longitudinal, Transverse and Vertical) and air pressure was used to measure ground vibration. Seismograph measures the ground velocity in a range from 0 mm/s to 31.7 mm/s with a sampling rate of 1024. The seismograph was placed at the site in varied distances from the vibration source, while the arrow mark of the vibration receiver was pointed towards to the source. The ground spikes were placed on the ground as shown in Figures 1 (b) and (c).

Figure 1: (a) Seismograph, (b) Installing the transducer using the Ground Spikes, (c) Final installation with Ground Spikes inserted fully into the ground

Ground vibrations were measured at two different sites: Pile driving site and soil compacting site

**Pile Driving Site**

In this site, a precast pile driving was performed using a diesel hammer (Figure 2 (a)). The diesel hammer took about 30 minutes to drive a single pile. The length of a pile was 9 m and the cross section was 300 mm x 300 mm square.

In this site, soil type was more like peat and the average depth to the bedrock is 15m. Therefore, two 9 m concrete piles were welded at the edge of each pile, using steel plates. In this study, ground
vibration was monitored at 20 m (Location 1), 35 m (Location 2) and 50 m (Location 3) distance to the pile driving machine (i.e., the source) (Figure 2 (b)).

![Figure 2: (a) Pile driving using Diesel hammer, (b) Ground vibration monitoring at three locations (Location 1: at 20 m, Location 2: at 35 m, Location 3: at 50 m from the source)](image)

**Soil Compacting Site**

Ground vibrations induced by soil compaction were measured for two different compacting equipments: smooth wheel roller and a rammer. In this site, a rammer was used to compact the soil in building area while a smooth wheel roller was used to compact the soil nearby the retaining wall (Figure 3(b)). Ground vibration was monitored at short distances from the source in order to make sure a reasonable magnitude of ground vibration (i.e. PPV) would be recorded in the seismograph. Figure 3 (b) illustrates the locations where ground vibration was monitored: Location 4: 10m away from the rammer, Locations 5 and 6: 15 m away from the rammer, and Locations 7, 8 and 9 at roller compacting area near the retaining wall.

In addition, floor vibration levels induced by soil compaction were measured at the each level (Figure 3 (a)) of the nearby 4 storey building (i.e., first floor, second floor and the third floor levels). Minimum distance to the building from the source was about 5m.

The PPV values measured using the seismograph was retrieved using Blastware 10. The Fast Fourier Transformation (FFT) reports of the measured ground vibration were determined and dominant frequency for the event was determined. Dominant frequency and the PPV value were assessed based on the range recommended by BS 7385 and CEAs standards.
2.2 FEM modelling and analysis

The structural response to a dynamic load was investigated by using time history analysis. In this study, a wall was used as a simple structure and was modelled by using SAP2000 (ver16). The model is a two-dimensional, linear, isotropic finite element model. The geometry and the FE idealization, element types, material properties, boundary condition, loads and analysis procedures are described in this section.

Geometry and the FE Idealization

In the current study, the geometry of the wall was determined according to the average wall sizes in Sri Lankan dwellings. The selected wall size was 3m x 5m (i.e., height x length), having the thickness of 225mm, as in Sri Lanka, almost of domestic and commercial buildings are constructed using 225mm wall thickness.

Two FEM models were analyzed to determine structural responses and the effectiveness of the control technique. The first wall was modelled as a normal wall panel and was considered as the control model (Figure 4(a)) and the second wall was modelled considering a concrete frame having 225mm x 225mm concrete columns on right and left sides and 225mm x 300mm concrete beam on top (Figure 4(b)). In both cases, it was assumed that the wall remains as a linear elastic material until cracking. Concrete frame dimensions were selected according to the average frame dimensions used for Sri Lankan reinforced concrete buildings.
Figure 4: (a) Geometry of the wall panel without concrete frame, (b) Geometry of the wall panel with concrete frame

**Element Types**

Two types of elements were used in the model: frame elements and shell elements. In both models, the wall panel was modelled using ‘shell’ elements. In the second model (i.e., wall with the concrete frame), the frames were modelled using the ‘frame’ element. In the control model, there is only one element type (i.e., shell element) and no frame element. All elements were selected from the SAP2000 (ver16) built in library and the material properties were assigned (Documentation SAP2000, 2013).

In any FEM model, the mesh size is important to get the reliable results. However, decreasing the mesh size (increasing the number of nodes) may require higher computing power and more time. In the current study, 0.1m mesh was used for the masonry wall. This mesh size was selected to get the reliable results with the available computing power.

**Material Properties**

Masonry and Grade 25 concrete were used as materials in the models. In the current study, all the materials were considered as linear, elastic and isotropic.

For masonry, Elastic modulus assigned to the model was 4 GPa. This was based on average value of the Elastic modulus of masonry wall (i.e., 3.00GPa – 5.66GPa) found by Fuenteet al., (2010). Poisson’s ratio was considered as 0.17 for the masonry wall based on a previous study by Fuente et al., (2010). The ratio of Shear Modulus to the Elastic Modulus for masonry walls is about 0.4. According to this relationship, the Shear Modulus was determined as 1.6 GPa. Density of the masonry was considered as 1800 kg/m³ (Fuente et al., 2010).

For concrete, Poisson’s ratio is 0.2 and is not significantly varying with varying the compressive strength of concrete. However, the Elastic modulus of concrete is dependent on the compressive strength and, it can be predicted by using Equation (1) as stated in ACI 363 R 92 (1997) up to the compressive strength of 83 MPa.
\[ E_c = 3.65 \sqrt{f'_c} \]  

(1)

where,

\( E_c \) – Elastic modulus in GPa
\( f'_c \) – Compressive strength of concrete in MPa

For Grade 25 concrete, Elastic modulus was determined as 18.25GPa (Equation 1). Density of Grade 25 concrete was considered as 2400 kg/m\(^3\) as found by Bosiljkov (2000). Material properties assigned to the models are listed in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus (GPa)</th>
<th>Porosity</th>
<th>Density (kg/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry</td>
<td>4.00</td>
<td>0.17</td>
<td>1800</td>
</tr>
<tr>
<td>Concrete</td>
<td>18.25</td>
<td>0.20</td>
<td>2400</td>
</tr>
</tbody>
</table>

**Boundary Conditions and Applied Loads**

Boundary conditions considered for the analysis were “fixed” at the foundation level (i.e., the displacements, translational and rotational, were set to zero). In addition, for the wall with frame, all the nodes of the shell elements and all the nodes of the column at the foundation level are fixed.

After modeling the wall structure, the two models were analyzed for dynamic loading by using time history analysis. Displacement values, which were calculated by multiplying the measured velocity values from the time step (1/1024) for the duration of 1 s, were used for linear, modal, time history analysis of the structure. Stresses were obtained in X direction (i.e., direction along the wall) and Z direction (i.e., vertical direction) for the ground vibration measured in longitudinal direction at Location 2, which shows the maximum PPV of ground vibration induced by pile driving.

**3.0 Results and Discussion**

**3.1 Experimental Measurements**

FFT report for pile driving, which includes the frequency spectrum of the vibration in three measured directions is shown in Figure 5. Frequency spectrum shows that the most of the energy of the vibration is in a frequency range 7-12 Hz, and mostly concentrated at a frequency of 10 Hz.
PPV values of ground vibration induced by pile driving and soil compacting, are compared in Figure 6. The limiting PPV value given in CEA guidelines is 8 mm/s, while in BS 7385-1 (1990) it is 50 mm/s.

It can be seen from Figure 6, PPV value of ground vibration at Location 2 in pile driving site is 18 mm/s in all three directions. This exceeds the limiting value recommended by CEA guideline although it is within the recommended value in British Standards. This might possibly be caused by the boulders presented at Location 2 of the site. All other PPV of ground vibration induced by pile driving were less than 8 mm/s, implying that they were within the range recommended by CEA and BS standards.

In rammer and roller compacting, PPV values are in a range of 0.95 mm/s - 1.95 mm/s and 0.64 mm/s – 2.60 mm/s, respectively. Pile driving induced ground vibration having higher PPV values than that for the roller compacts and rammer compacts. Table 2 shows the frequency range of different vibration sources: pile driving, roller compaction and rammer compaction. It was found that pile driving induced vibration is in lower frequency range, compared to that of the roller compaction induced vibration, and rammer compaction induced vibration.
Table 2: Frequency range of different vibration sources

<table>
<thead>
<tr>
<th>Source of vibration</th>
<th>Frequency range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile driving</td>
<td>7Hz – 12Hz</td>
</tr>
<tr>
<td>Roller compaction</td>
<td>35Hz – 45Hz</td>
</tr>
<tr>
<td>Rammer compaction</td>
<td>30Hz – 150Hz</td>
</tr>
</tbody>
</table>

Comparison of Vibration Level in 4 Storey Building

Figure 6 shows the variation of PPV values of the floor vibration in 4 storey building due to the roller compaction. At the first, second and third floor levels of the building, PPV values in all three directions (i.e., longitudinal, vertical and transverse) were smaller than 0.6 mm/s. This is smaller value compared with the PPV of pile driving induced vibration (Figure 7).

In the first floor level of the 4 storey building, PPV values of floor vibration were comparatively higher; 0.56 mm/s in transverse direction while PPV values in the longitudinal and vertical direction are about 0.25 mm/s. In the third floor level, PPV value in vertical direction is 0.6 mm/s. In the second floor level of the building, PPV values: 0.06 mm/s, 0.13 mm/s and 0.04 mm/s in longitudinal, vertical and transverse directions, respectively. These were comparatively lesser than that in other floor levels.

The frequency levels of measured ground vibration in each floor level are in a same frequency range around 33 Hz - 43 Hz, although these results are not presented. Frequency did not change significantly with the floor level or with the direction of the vibration. This might be caused by the building vibrates at a common vibration mode around 33-43 Hz which matches with the frequencies of ground vibration induced by roller compaction (Table 2).
3.2 FEM Analysis

Modal analysis shows that, in the first mode, wall without frame has a natural frequency of 64 Hz while the wall with frame has a natural frequency of 62 Hz. Direct stress determined from FEM analysis of walls are summarized in Table 3.

Figures 8 and 9 show the variation of direct stresses along the wall (i.e., X direction), $S_{11}$, and direct stresses in the vertical direction, (i.e., Z direction), $S_{22}$, respectively, at time step 0.14s, which gives highest stresses, in the wall without the frame. It can be seen from Figure 8(a) that the maximum stress in X direction is 0.019 N/mm² and developed at the bottom-left corner and the top-right edge of the wall. This implies that the wall vibrates in sway motion, which would be the first mode.

It can be seen from Figure 9(a) that the maximum stress in Z direction is 0.112 N/mm² and developed at left bottom edge of the wall. Figure 9(b) shows the variation of direct stresses $S_{22}$ in the wall with frame.
When introducing the concrete frame to the wall (i.e., wall with frame), the maximum direct stress, S11, reduced to 0.005 N/mm² (Figure 8(b)). The highest stress found for the beam is 0.225 N/mm². It seems that although the concrete frame has higher stresses, they are not transferred to the wall panel and resulting to have comparably lower stress on masonry wall. Larger portion of the stresses induced by the dynamic loads were bared by the concrete frame structure.

After applying the concrete frame, the maximum direct stress in vertical direction, S22, was reduced from 0.112 N/mm² to 0.031 N/mm² implying that addition of frame affect on reducing the stresses in masonry in vertical direction.

Figure 9: Direct stresses in vertical direction, S22, (a) Wall without frame, (b) Wall with frame
4.0 Conclusions

Characteristics of ground vibration induced by construction activities and a possible technique to reduce the structural damages were investigated by using experimental measurements of ground vibration and time history analysis of a FEM model developed by using a commercially available software, SAP 2000 (ver 16).

It was found that ground vibrations (induced by construction activities) in longitudinal direction are greater than that in the vertical and transverse directions. The magnitudes of ground vibrations induced by pile driving process are greater than the ground vibration induced by soil compacting by using rammer and roller compactors. Peak Particle Velocities (PPV) of ground vibrations induced by pile driving and soil compacting were found to be 1.32 mm/s - 18.10 mm/s and 0.64 mm/s - 2.60 mm/s, respectively, satisfying the PPV values recommended by BS standards, although PPV of 18.1 mm/s exceeded the values recommended by the Central Environmental Authority (CEA) . The frequency components in ground vibrations induced by pile driving: 7 Hz – 12 Hz, are also less than the frequency components in ground vibration induced by roller compacting and rammer compacting: 35 Hz – 45 Hz and 30 Hz – 150 Hz, respectively.

Floor vibration (induced by soil compaction) at 4 storey building is significantly less than the ground vibration induced by pile driving. In longitudinal, transverse and vertical directions, PPV of floor vibrations were less than 1mm/s and occurred at frequency range of 33 Hz – 43 Hz.

From the FEM analysis, it was found that introduction of a frame to simple masonry wall reduced the direct stresses by about 75%: along the wall and in the vertical direction, direct stresses were 0.019 N/mm² and 0.112 N/mm², respectively, while they were 0.005N/mm² and 0.031N/mm², respectively for the wall with the frame, implying that the adding of a frame would be an effective method to reduce the stresses developed in wall structures.

Introducing concrete frames to masonry structures is an effective technique to reduce damages in masonry. In addition, continuous monitoring of ground vibrations induced by construction activities will help to maintain the existing vibration limits and reduce public complaints.

References


