CHAPTER 7

SOIL-STRUCTURE INTERACTION OF TUBULAR FRAMED STRUCTURE

7.1 INTRODUCTION

Under the earthquake excitations, the responses of structures, as well as soil mass in which the structure embedded, are not independent. The process in which the response of the soil mass influences the motion of the structure and the motion of the structure influences the response of the soil mass is termed as soil-structure interaction [Elnashai and Sarno 2008]. The structural systems have a dynamic response on soil systems that depends on inertia, stiffness and damping of the structures in general. The common dynamic analysis method is to determine the free-field ground motion at the site of the structure and then apply the motion at the base of the structure assuming that the base is fixed. This may be true in cases where the structure is founded on bedrock. However, if the structure is founded on soil bed, the earthquake motion at the base of the structure is not likely to be identical to the free-field ground motion. The presence of the structure will modify the free-field motions because the soil and structure interaction that creates a dynamic system quite different from the free-field condition.

The soil-structure interaction will result in a structural response that may be different from the structural response computed from a fixed base structure subjected to a free-field ground motion in following ways such as to increases fundamental period; adds to damping; increases peak displacement while reducing damage to structural components;

it may be detrimental in some cases like moderately flexible structures in soft soils; overloading of soil may cause excess foundation deformation that damage the basements. The response of soil to earthquake excitations is highly complex and depends on a large range of factors, many of which cannot be established with any certainty. It is common in the field of structural engineering to rigidly fix the foundation of a structure to the ground while carrying out design calculations. This is done to make calculations easier and to deliver quick solutions for static load cases and design combinations. For such analysis, fixed approach is usually acceptable. However, during earthquakes, fixed-ground calculations do not attribute to the actual behavior of the structure.

A solution of such problems requires an idealization of the behavior of the structure, soil mass and boundary conditions of the interface. For the majority of common building structures, the effects of soil-structure interaction (SSI) tend to be beneficial, since they reduce the bending moments and shear forces in the various members of the superstructure. According to Eurocode 8, part 5, the effect of SSI needs to be considered in structures with massive or deep-seated foundations, such as bridge piers, offshore caissons, and silos; slender tall structures such as towers and chimneys and the structures supported on very soft soils. Depending on the relative stiffness of the soil and structure, SSI can have an impact on the response of the structure. Thus, it becomes imperative to understand the effect of soil properties on the response of structures during the earthquake for seismic analysis. As the result of dynamic soil-structure interaction, the seismic response of a flexibly supported structure, i.e., a structure founded on deformable ground, differs in several ways from that of the same structure founded on fixed base and subjected to an identical free-field excitation. Probably the reasons behind this are stated as (i) the foundation motion of the flexibly-supported structure will differ from the free field motion and may include an important rocking component of the fixed-base structure; (ii) the fundamental period of vibration of the flexibly-supported structure will be longer than that of the fixed-base structure; (iii) the natural periods, mode shapes and modal participation factors of the flexibly supported structure will be different from those of the fixed-base structure; (iv) the overall damping of the flexibly-supported structure will include both the propagation of waves and the internal damping generated at the soil-foundation interface, in addition to the damping associated with the superstructure [Thusoo 2015]

7.2 TYPE OF ANALYSIS

The buildings, in the regions where earthquakes pose a serious threat to infrastructure, in some way designed elastically. Apart from the basic soil and structure models, proper consideration of SSI requires inclusion of following important elements; (i) proper soil properties for evaluation of soil free-field motion, (ii) transfer function or contact element between soil and foundation and (iii) elements for conversion of free- field motion to foundation input motion. The methods used to analyze the buildings subjected to earthquakes are given below.

7.2.1 Quasi-Static (QS) Analysis

The quasi-static method relatively simple and it requires only static analysis and estimates the response of the structure for an ensemble of earthquakes. It is based on the determination of seismic design forces. For the quasi-static method, the earthquake forces are divided by a behaviour factor (also known as a structural response factor or response modification coefficient). This factor accounts for the reserve strength of the building after the formation of the first plastic hinge and allows a pseudo inelastic design to be achieved without complicating the analysis. The extra requirement is to choose an appropriate building behaviour factor to account the inelastic behaviour. Typically, this is done by choosing a value from a table in a relevant earthquake code [IS 1893]. This is simple and reasonably effective, but it is overly conservative. The various ductility factors have been arrived at empirically based on past experience of structural behaviour during earthquakes and based on generalised analysis of simple models of various building types.

7.2.2 Time-History (TH) Analysis

The numerical integration method is usually referred to as time history analysis. It is required to get accurate responses of structure in the event of the earthquake with respect to time [Chen and Duan 1999]. Time histories theoretically contain complete information about the motion at the instrumental location, recording three traces or orthogonal records, two horizontal and one vertical. The TH analysis has a great advantage in fast solution times but also has two obvious drawbacks. First of all the methods of combining the scaled model results will always lead to final results which are all positive. The second drawback is that the analysis must be linear. A transient analysis does not have these limitations, but on the other side it is more costly in terms of solution times. Further, to run the earthquake analysis transient, it is necessary to artificially create the time-acceleration data in such a way that these data are compatible with the smoothed response spectra in the frequency plane.

7.2.3 Response Spectrum (RS) Analysis

Response spectrum analysis is a linear dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. The response spectrum method is identical to the quasi-static method except that it considers more than just the fundamental mode of vibration. Most codes require that enough modes of vibration are considered to account for 90% of the modal mass. Response spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of the structural period. It gives the maximum amplitude of responses. The maximum amplitude of record acceleration is termed the peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD), are the maximum respective amplitudes of velocity and displacement [Chen and Lui 2000]. Response spectra can also be used in assessing the response of linear systems with multiple modes of oscillation (multi-degree of freedom systems), although they are only accurate for low levels of damping. Modal analysis is performed to identify the modes, and the response in that mode can be picked from the response spectrum. This peak response is then combined to estimate a total response.

A typical combination method is the square root of the sum of the squares (SRSS) if the modal frequencies are not close. The result is typically different from that which would be calculated directly from an input since phase information is lost in the process of generating the response spectrum. The main limitation of response spectra is that they are only universally applicable for linear systems. Response spectra can be generated for non-linear systems, but are only applicable to systems with the same non-linearity, although attempts have been made to develop non-linear seismic design spectra with the wider structural application. The results of this cannot be directly combined for the multi-mode response.

7.4 EQUATION OF MOTION AND SOIL-STRUCTURE INTERACTION

Dynamic loading often results from vibration of the supports of the system rather than from dynamic external loads. To evaluate the response of such systems, it is necessary to develop an equation of motion for loading caused by base shaking. Fig. 7.1 explains where m denotes effective values of mass, k is the spring coefficient and c is the damping coefficient. The mass, spring, and damping are associated with the fundamental mode of vibration of the structure built in at its base, h = distance from the base to the centroid of the inertial forces [Wolf 1985]; u_t = total displacement; u_g = ground displacement; θ_g= ground rotation and u_{ed}= elastic deformation.

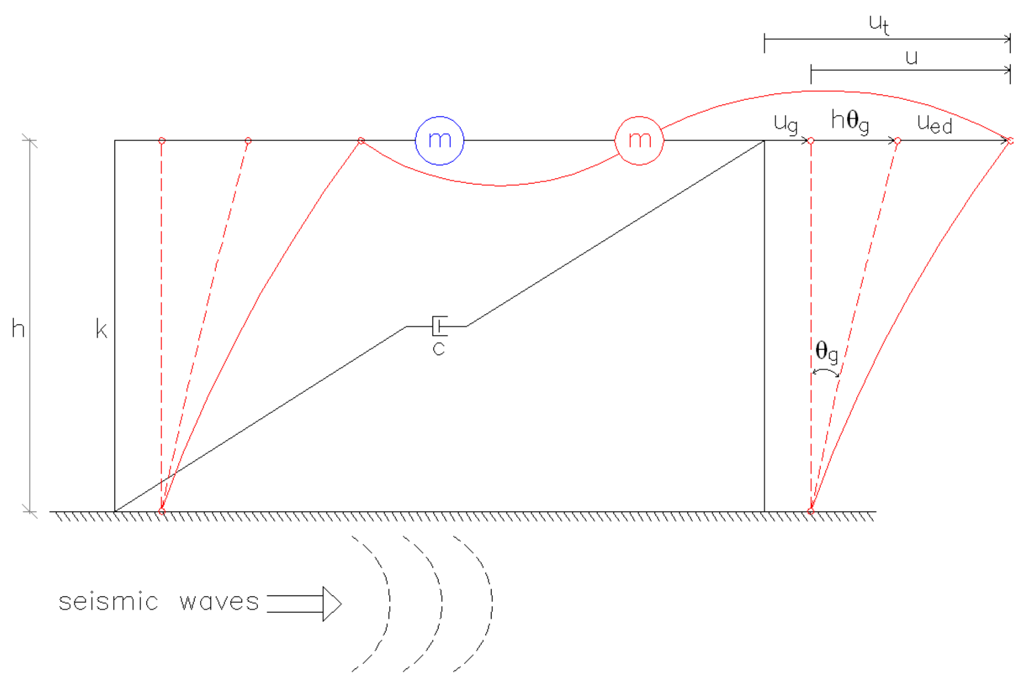


Fig. 7.1. Single degree of freedom systems subjected to base shaking (After Wolf 1985)

The impressions of the spring and damping are subject to displacement and velocity to the system with respect to the bottom of the system, but effects of the mass are dependent on the total acceleration of the system. For the fixed-base frequency of the structure ω = √(k/m) [Chopra 2001]. The effect of soil-structure interaction can be illustrated with the idealized model. The structure is modeled with mass m, a lateral stiffness with a spring coefficient k, damper with a coefficient c and the height of the structure h. The corresponding coefficients denoted as k_h and c_h in the horizontal direction, k_r (evaluated base on the modulus of subgrade reactions) and c_r in the rotational (rocking) direction. All spring and dampers have a length approaching zero. The foundation dashpots represent two sources of damping such as material damping caused by the inelastic behavior of the soil supporting the foundation, and radiation damping that occurs as dynamic forces in the structure causes the foundation to deform the soil, producing stress waves that travel away from the foundation. The magnitude of material damping will depend on the level of the strain induced in the soil; if the strains are high, material damping can be substantial, but if they are low, the material damping may be negligible. In contrast, radiation damping is a purely geometric effect that exists at low as well as high strain amplitudes. For typical foundation, radiation damping is often much greater than material damping [Kramer 1996]. If the structure is rigid, i.e., $k = \infty$ and the foundation is unable to rotate, i.e., $k_r = \infty$ the natural frequency for translational vibration, $\omega = \sqrt{(k_h/m)}$. If the structure is rigid ($k= \infty$) and the foundation is unable to translate ($k_h= \infty$), the natural frequency for rocking, $\omega = \sqrt{(k_r/m.h^2)}$.

7.5 STATEMENT OF THE PROBLEM

The present study focuses on understanding the behavior of tubular buildings under any given earthquake excitation. The responses of building models with fixed base and flexible base are evaluated for better understanding of the effect of Soil-Structure Interaction phenomena. The tubular tall buildings are idealized as cantilever box beam for the lateral loadings [Coull and Bose 1975, 1976; Coull and Ahmed 1978; Ha et al. 1978; Haji–Kazemi and Company 2002; Kwan 1994, 1996; Singh and Nagpal 1993]. The idealized cantilever beam has been assumed to resist the lateral load in shear mode. The uniform cantilever box beam, i.e., core/shear wall is used as a lateral load resisting system in the tubular tall building.

A shear beam, which replicates the tubular building, is modeled (Fig. 7.2) to analyze under the ground excitation. The main feature that may consider in the present chapter are (i) to evaluate the changes in various responses of the shear beam for fixed base and SSI considered model and (ii) to find the dynamic factors affecting the responses of the model.

7.6 MATERIAL AND METHODOLOGY

7.6.1 Model Specifications

A shear beam replica of a tubular framed structure constructed with hollow steel section (HSS) of grade 202. The specifications of the shear beam are: column and beam size-10x10mm; total height-1200mm; floor height-115mm (ground floor height-120mm); column spacing-100mm and plan area 400x400mm [Kwan 1996]. The model is fixed in soil with the help of chair which have the top plate and strut placed directly beneath the column of the model. The length and diameter of the strut is 115mm and 12 mm respectively. The strut act as pile and the plate of the chair bolted with the base plate of the model, acted jointly as the tie beam (Fig. 7.2).

7.6.2 Soil Specifications

A disturbed soil, excavated from 1.5 m below ground level, was filled in a container of dimension 850 mm x 660 mm x 225 mm up to 150 mm from the base level. The filled soil is normally compacted to a bulk density 17 kN/m^3 from bulk density11.76 kN/m^3 in loose condition.

(a) Model with fixed base

(b) Model with base fix in soil

Fig.7.2. Model with fixed base and with base fix in soil

The filled soil is low compressible type (CL). An estimate of in situ dry density of the soil obtained in laboratory test was 14.4 kN/m^3 corresponding to a moisture content of 18.1 %.

7.6.3 Shake Table Specifications

The specification of the shake table is shown in Table 7.1.

Table 7.1. Specification of shake table

Horizontal shake table specifications	
Motion	Horizontal
Maximum pay load	200 Kgs
Top label size	1500 mm x 1200 mm
Frequency range	0 - 20 Hz
Frequency control	Within + / - 5%
Amplitude	+ / - 50 mm or Total 100 mm
Amplitude resolution	5 mm
Type of harmonics	SHM
Tentative 'g' value	0.1 g – 3g
Maximum height of model	1500 mm
Motor rating	10 HP, 3 Phase, 440 Volt input
Control panel input voltage	4 Wire, 3 Phase, 440 Volt input

7.6.4 Data Analysis

The specification of the equipment used in data recording and analysis are given in Table 7.2 and Fig. 7.3. The response of the shear beam recorded by using 6 sensors (accelerometers) placed at the height 400 mm, 800 mm and 1200 mm from base level at the central column and at corner column symmetrically. The response is drawn for dynamically analyzed lab model using FFT Spectrum Averaging analyzer (B and K PULSE Lab Shop Version 18.1.0.28 - 2013-11-23).

(a) Input module (b) Accelerometers

Fig. 7.3. Input module and accelerometer

Table 7.2. Specification of the equipments for data recording and analysis

Specification of the equipment for data recording and analysis	
Accelerometers type	4507
Input module type	6 Channel, 50KHz, 3050-A-060
Header size	79
Pulse version	80
Running pulse version	PULSE Lab Shop Version 18.1.0.28 - 2013-11-23
Data type	Real
Analyzer	FFT Spectrum Averaging
Analyzer name	FFT

The following responses were output of dynamic analysis of the model:

1. Displacement vs Time

2. Velocity vs Time

3. Acceleration vs Time

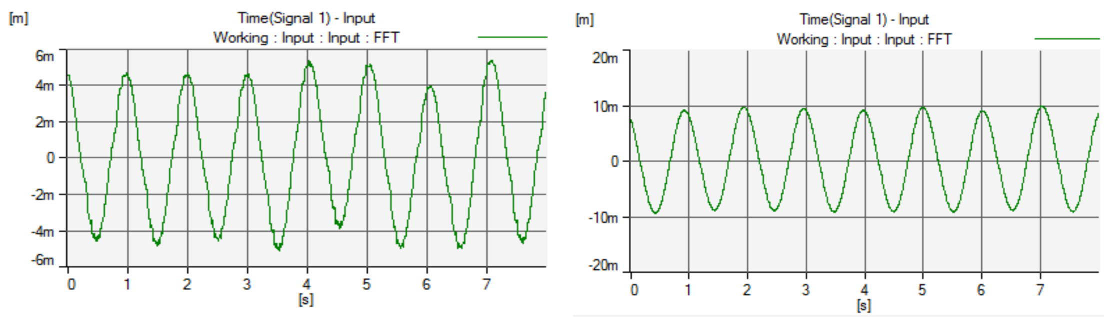
The model analysed at different frequency as input for shake table, i.e., 0.25, 0.5, 0.75 and 1.0 Hz and amplitude 5mm and 10mm. First, the model was analysed with fixing the base plate with shake table. For the soil structure interaction, the model is fixed in a soil mass with the help of chair as specified in section 7.6.1.

7.7 RESULT AND DISCUSSIONS

The model was analysed at different frequency and amplitude. The results for 1 Hz are presented in Fig. 7.4- 7.9. However, the results corresponding to all frequencies and amplitude considered are tabulated in Table 7.3.

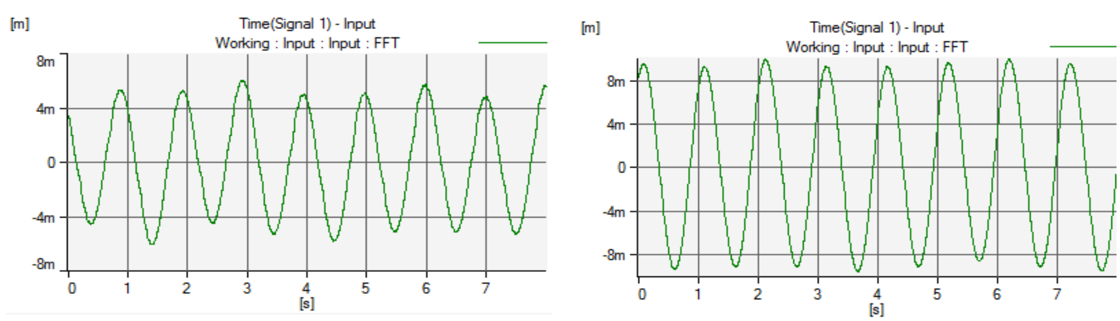
7.7.1 Displacement vs Time

It is clear from Table 7.3 that as the excitation frequency increases the peak model displacement (PMD) with fixed base, increases for both the input amplitudes of 5mm and 10mm. However, the rate of amplification is high for the lower input frequency.



(a) 1 Amplitude 5 mm (b) Amplitude 10 mm

Fig. 7.4. Displacement response for 1Hz frequency for fixed base



(a) Amplitude 5 mm (b) Amplitude 10 mm

Fig. 7.5. Displacement response for 1 Hz frequency and with soil structure interaction

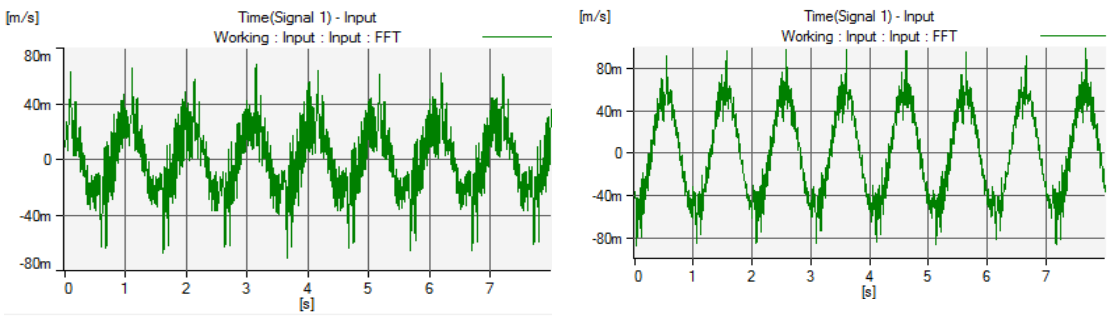
As the input frequency increases the amplification rate decreases (Table 7.3), however, for the input amplitude of 10mm, the PMD changes rapidly. It is approximately twice as compared to the input amplitude 5mm. Also, it is observed that the amplification, for the model under soil structure interaction, is higher as compared to the model with fixed base. However, it follows the same trend of the fixed base case corresponding to each input frequency.

Table 7.3. Average response corresponding to the different frequency and amplitude for fixed base and with SSI

Average Response	Input Frequency (Hz)	Input Amplitude			
		5mm		10mm	
		Fixed base	SSI	Fixed base	SSI
Displacement (mm)	0.25	0.98	1.02	1.21	1.52
	0.5	3.54	3.6	6.57	6.72
	0.75	4.80	5.06	8.72	8.76
	1	5.23	6.05	9.61	9.86
Velocity (m/s)	0.25	0.028	0.007	0.017	0.008
	0.5	0.029	0.016	0.029	0.025
	0.75	0.052	0.036	0.060	0.061
	1	0.070	0.054	0.093	0.086
Acceleration (m/s^2)	0.25	3.17	1.04	2.14	0.83
	0.5	3.87	1.14	2.73	1.82
	0.75	5.01	2.29	3.85	3.01
	1	6.19	2.77	4.59	2.31

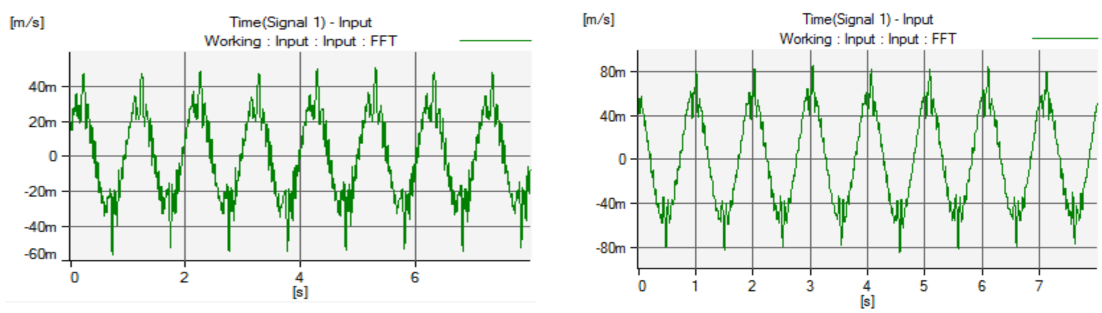
7.7.2 Velocity vs Time

As the input frequency and amplitude increases, the peak velocity of the model (PMV) increase accordingly, in both cases fixed base as well as with SSI (Table 7.3). The PMV have a lower magnitude in SSI when compared with the fixed base indicating that the magnitude of PMV falls down with consideration of SSI. For input amplitude of 5 mm, for the lower input frequency the reduction is approximately 40 to 50 % and for the higher input frequency it is approximately 20 to 30%. Similar for input amplitude 10 mm, the reduction of PMV varies from 10 to 50 %. Thus, it can be noted that the reduction in PMV is higher for lower frequencies.



(a) 0.25 Hz (b) 0.50 Hz

Fig. 7.6. Velocity response for 1Hz frequency and fixed base



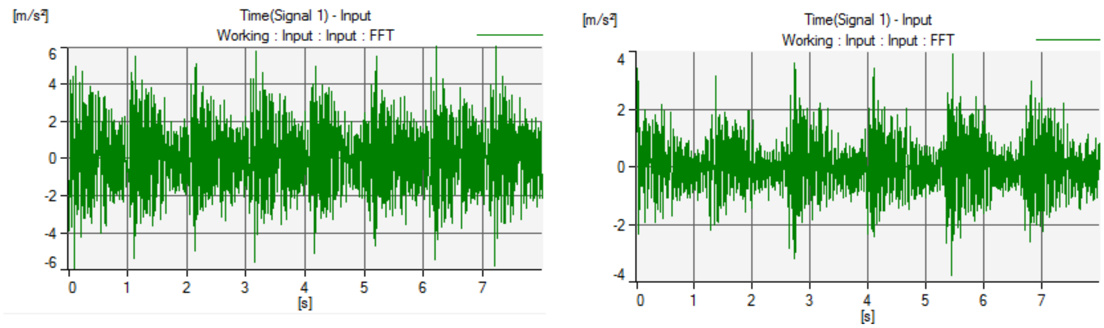
(a) 0.25 Hz (b) 0.50 Hz

Fig. 7.7. Velocity response for 1Hz frequency and with soil structure interaction

7.7.3 Acceleration vs Time

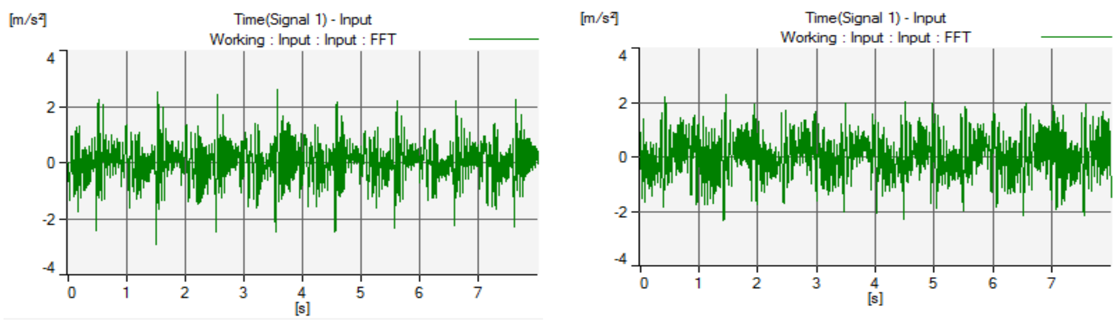
The peak model acceleration (PMA) has the same trend of variation with frequency and amplitude as PMV. For the increased magnitude of input frequency and amplitude, PMA is higher in both cases fixed base as well as with SSI. The PMA have a lower magnitude in SSI as compared with the fixed base. The magnitude of PMA also reduced with consideration of SSI. For input amplitude 5mm, the reduction is approximately 50 to 70 %

whereas for the 10mm input frequency, the reduction of PMA is 20 to 65 %. The reduction in PMA is higher for lower frequencies



(a) 0.25 Hz (b) 0.50 Hz

Fig. 7.8. Acceleration response for 1Hz frequency and fixed base



(a) 0.25 Hz (b) 0.50 Hz

Fig. 7.9. Acceleration response for 1Hz frequency and with soil structure interaction

The present experimental study has following limitations, i.e., (i) the model height is limited to 1500 mm; (ii) the motion is unidirectional and sinusoidal; (iii) the payload capacity 200 Kgs; (iv) the maximum dimension is limited to 1500 mm x 1200 mm.

Due to the limitation of the height, weight and size of the model, the appropriate lump mass dimensions were not maintained. However, the results obtained are significant in terms of the parameters studied.

The above results are reported for the corner columns. The displacement such as displacement, velocity and accelerations were also measured at the central column simultaneously. It was also observed that the corner column and central column vibrating in different phases. Since there is no relevant experimental study on the dynamic response of tubular structure and effect on the shear lag phenomenon (SLP) and effect of SSI on SLP in existing literature, the result present study assumes a distinct significance. However, the trend observed here needs re-investigations as the future scope of studies.

7.8 SUMMARY AND CONCLUDING REMARKS

It is observed that the support condition may have a profound effect on the global dynamic response of the shear beam. In particular, it is found that the influence of the soil-structure interaction may increase the maximum overall displacement of the shear beam significantly. On the basis of results and discussion, it is concluded that (i) The peak model displacement (PMD), increases significantly as the stiffness of the base decreases, (ii) The peak model velocity (PMV) of shear beam decreases with decreasing base stiffness (iii) The peak spectral acceleration response of the model (PMA) changes significantly as stiffness of base decreases.