Seismic Response of Self Supported Stacks - Considering Foundation Compliance

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SUMMARY:
Self-supporting stacks both RCC and Steel, plays an important role in dispersion of flue gases to the atmosphere in power, petrochemical and steel industry. The present state of the art for design of such structure is still restricted to analyzing the same as fixed base cantilever though a number of researchers have argued that the response varies considerably when foundation compliance is taken into cognizance. In the present paper a mathematical model based on modal analysis considering dynamic soil structure interaction has been proposed to cater to the effect of foundation compliance and study its effect on the overall response. The model considers both multi flue and tapering stacks. The results are also compared with standard analysis package like STAAD Pro to check on the variations.

Keywords: Multi-flue, Single flue, DSSI, dynamic moments and shears, modal analysis, shape functions.

1. INTRODUCTION

With environmental legislations getting tougher every day, reinforced concrete chimneys that are mostly used to discharge effluent gases to atmosphere from various industries are getting progressively taller to reduce concentration of pollution at ground level. This is thus posing new challenge to structural engineers to design them safely against the demanding natural forces like wind, earthquake etc whose effects increase significantly with height. In the process of design of such tall chimneys, till recent past, the practice has been to design the stack considering it to be fixed at foundation level, ignoring effect of foundation compliance a shown in Fig. 1.1. It is only recently a number of chimneys have been built (H>220m) in various power plants and has created a significant debate among chimney analysts and designer as to whether to consider dynamic soil structure interaction (DSSI) for dynamic analysis for such chimneys.

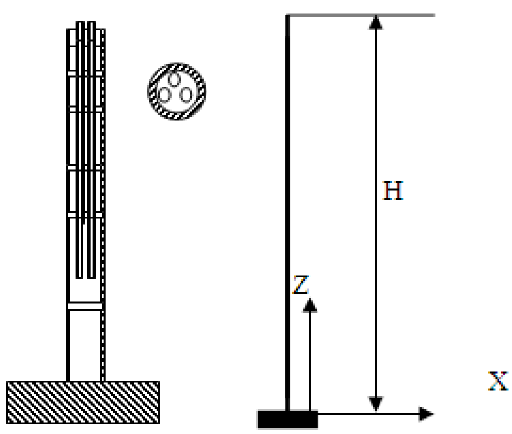

Figure 1.1. A typical Multi-flue RCC stack

A number of researchers have advocated consideration of DSSI while performing the dynamic analysis of chimney. Ghosh and Batavyal (1985), Luco (1986), Navarro (1992), Sadeghpour & Chowdhury (2008), Jaya et al (2009), Chowdhury (2010) have all suggested to consider DSSI for chimney design, each proposing different techniques to be adapted for dynamic analysis of such tall RCC chimneys. All researchers have concluded that fixed base moment and shear of chimney varies considerably when effect of foundation compliance is considered in the overall analysis.

Latest IS-1893(2005) Part IV has now provided allowance for DSSI analysis by furnishing translational and rocking spring of soil-foundation system based on Hall (1967), however the modus operandi of analysis is left open to the discretion of user. Though IS code has furnished coefficients D_v and D_m to cater to the DSSI effect, however it appears that time period values for which coefficient S_a/g is to be considered is fixed base time period even for DSSI case and this is fallacious. As time period will get modified due to foundation soil springs the S_a/g values should correspond to this modified time period and not the fixed base case.

2. PROPOSED METHOD

We start with analysis of a multi flue chimney of uniform cross section as shown in Fig. 1.1. Since outer core of a multi-flue chimney (usually termed as wind shield) is almost of uniform cross section or with very limited variation of cross sectional area along its height, we consider it as a cantilever beam fixed at base of uniform cross section. Free vibration equation of such beam is given by the expression (Hurty and Rubenstein 1967) in Eqn. 2.1.

$$EI\left(\frac{\partial^4 w}{\partial z^4}\right) + \rho A\left(\frac{\partial^2 w}{\partial t^2}\right) = 0 \tag{2.1}$$

$$w(z,t) = Y(z).q(t) \tag{2.2}$$

$$EI\left(\frac{d^4 Y}{dz^4}\right) - \lambda^4 Y = 0 \text{ and } \lambda^4 = \frac{\rho A \omega^2}{EI} \tag{2.3}$$

$$Y = C_1 \sin \lambda z + C_2 \cos \lambda z + C_3 \sinh \lambda z + C_4 \cosh \lambda z \tag{2.4}$$

Based on Eqn. 2.2 and separation of variable technique, partial differential equation vide Eqn. 2.1 can be separated into two linear differential equation and one of which is given in Eqn. 2.3. The generic solution to this equation is given by Eqn. 2.4. Imposing the four boundary conditions given in Eqn. 2.5, we have the shape function solution as in Eqn. 2.6.

$$\text{1) } y=0 \text{ at z=0; 2) } \frac{dy}{dz} = 0 \text{ at } z = 0\text{; 3) } \frac{d^3 y}{dz^3} = 0 \text{ at } z = H\text{; 4) } \frac{d^2 y}{dz^2} = 0 \text{ at } z = H \tag{2.5}$$

$$Y_m = \sin\frac{\mu_m z}{H} - \sinh\frac{\mu_m z}{H} - \alpha_m\left(\cos\frac{\mu_m z}{H} - \cosh\frac{\mu_m z}{H}\right) \tag{2.6}$$

$$\mu_m = 1.875, 4.694, 7.855, \frac{2m-1}{2}\pi \text{ and } \alpha_m = \frac{\sin\mu_m + \sinh\mu_m}{\cos\mu_m + \cosh\mu_m} \tag{2.7}$$

Here m is the mode number 1, 2, 3 etc. Based on above it can be shown that the stiffness and mass matrix of such multi-flue stack can be expressed as (Meirovitch, 1967) in Eqn. 2.8 and Eqn. 2.9 respectively.

$$k_{ij} = EI\int_0^H \frac{d^2\phi_i(z)}{dz^2}\frac{d^2\phi_j(z)}{dz^2}dz \qquad (2.8)$$

and $$m_{ij} = \left[\frac{\gamma A}{g}\int_0^H \phi_i(z)\phi_j(z)dz\right] \qquad (2.9)$$

Considering the shape function as in Eqn. 2.10 and its double derivative Eqn. 2.11,

$$\phi_i = \sin\frac{\mu_i z}{H} - \sinh\frac{\mu_i z}{H} - \alpha_i\left(\cos\frac{\mu_i z}{H} - \cosh\frac{\mu_i z}{H}\right) \qquad (2.10)$$

$$\phi_i'' = \frac{\mu_i^2}{H^2}\left[-\sin\frac{\mu_i z}{H} - \sinh\frac{\mu_i z}{H} + \alpha_i\left(\cos\frac{\mu_i z}{H} + \cosh\frac{\mu_i z}{H}\right)\right] \qquad (2.11)$$

Before performing integration to find out k_{ij} and m_{ij}, we change the above to generalized co-ordinate by considering, $\xi = \frac{z}{H}$ when $d\xi = \frac{dz}{H}$ and as $z \to 0, \xi \to 0$ and as $z \to H, \xi \to 1$ based on above we can now express the double derivative as

$$f''(\xi)_i = \frac{\mu_i^2}{H^2}\left[-\sin\mu_i\xi - \sinh\mu_i\xi + \alpha_i\left(\cos\mu_i\xi + \cosh\mu_i\xi\right)\right] \qquad (2.12)$$

Thus stiffness and mass of the system can now be given by Eqn. 2.13 and Eqn. 2.14 respectively.

$$k_{ij} = \frac{EI\mu_i^2\mu_j^2}{H^3}\int_0^1 f''(\xi)_i f''(\xi)_j\, d\xi \qquad (2.13)$$

$$m_{ij} = \frac{\gamma AH}{g}\int_0^1 f(\xi)_i f(\xi)_j\, d\xi \qquad (2.14)$$

where i=j=1,2,3,………m.

For most of the chimneys it is found that first three modes are sufficient to predict the dynamic response, as modal mass participation is almost 100% by this. Thus for first three modes, stiffness matrix is given by

$$[K]_{ij} = \frac{EI}{H^3}\begin{bmatrix} \mu_1^4\int_0^1 f_1''(\xi)^2\,d\xi & & \text{Symmetric} \\ \mu_2^2\mu_1^2\int_0^1 f_2''(\xi)f_1''(\xi)d\xi & \mu_2^4\int_0^1 f_2''(\xi)^2\,d\xi & \\ \mu_3^2\mu_1^2\int_0^1 f_3''(\xi)f_1''(\xi)d\xi & \mu_3^2\mu_2^2\int_0^1 f_3''(\xi)f_2''(\xi)d\xi & \mu_3^4\int_0^1 f_3''(\xi)^2\,d\xi \end{bmatrix} \qquad (2.15)$$

and mass matrix is given by

$$[M]_{ij} = \frac{\gamma AH}{g}\begin{bmatrix} \int_0^1 f_1(\xi)^2 d\xi & & \text{Symmetric} \\ \int_0^1 f_2(\xi) f_1(\xi) d\xi & \int_0^1 f_2(\xi)^2 d\xi & \\ \int_0^1 f_3(\xi) f_1(\xi) d\xi & \int_0^1 f_3(\xi) f_2(\xi) d\xi & \int_0^1 f_3(\xi)^2 d\xi \end{bmatrix} \tag{2.16}$$

Integral functions in Eqns. 2.15 and 2.16 can very easily be solved based on Simpson's 1/3rd rule between limits 1 to 0 when we have stiffness and mass matrix is expressed as

$$[K]_{3X3} = \frac{EI}{H^3}\begin{bmatrix} 22.936 & -0.002 & 0.006 \\ -0.002 & 468.044 & -0.11 \\ 0.006 & -0.11 & 3812.81 \end{bmatrix} \tag{2.17}$$

$$[M]_{3X3} = \frac{W}{g}\begin{bmatrix} 1.855 & 0 & 0 \\ 0 & 0.964 & 0 \\ 0 & 0 & 1.002 \end{bmatrix} \tag{2.18}$$

Converting above into standard eigen-value form of $\mathbf{A\varphi} = \boldsymbol{\lambda\varphi}$ and applying the generalized Jacobi technique (Bathe & Wilson, 1980) we have (Chowdhury, 2008)

$$[\lambda] = \frac{EIg}{WH^3}\begin{bmatrix} 12.362 & 0 & 0 \\ 0 & 485.519 & 0 \\ 0 & 0 & 3806.5468 \end{bmatrix} \tag{2.19}$$

The Eigen vectors are given as in Eqn. 2.20 and are shown in Fig. 2.1.

$$[\boldsymbol{\varphi}] = \begin{bmatrix} -1.0 & 2.278\times10^{-6} & 8.528\times10^{-7} \\ 4.384\times10^{-6} & -1 & -3.437\times10^{-5} \\ 1.579\times10^{-6} & -3.307\times10^{-5} & 1 \end{bmatrix}\begin{bmatrix} f_1(\xi) \\ f_2(\xi) \\ f_3(\xi) \end{bmatrix} \tag{2.20}$$

Since $[\lambda] = \omega^2$ and $T = \frac{2\pi}{\omega}$ we have

$$[T] = \begin{bmatrix} 1.787 & 0 & 0 \\ 0 & 0.285 & 0 \\ 0 & & 0.102 \end{bmatrix}\sqrt{\frac{WH^3}{EIg}} \tag{2.21}$$

Here W total weight of chimney including all its appurtenance including 50 % of Live load during operation, I moment of inertia of the chimney cross section at base of the shell (i.e. top of chimney raft), E modulus of elasticity of the stack material, g acceleration due to gravity @ 9.81 m/sec^2.

2.1 Effect of Foundation Compliance

To assess the phenomenon we start with the model as proposed by Veletsos (1974) and Wolf (1985) as shown in Fig. 2.2.

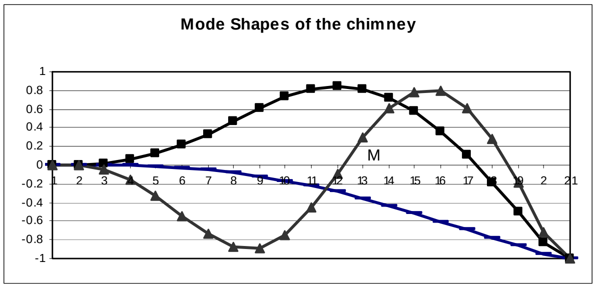

Figure 2.1. First three mode shapes of the chimney

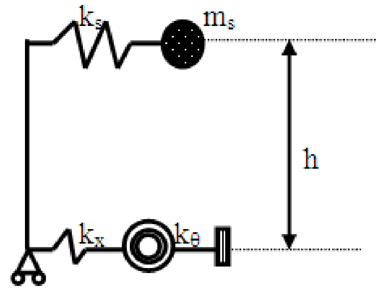

Figure 2.2. Mathematical Model of an oscillator considering the effect of DSSI

Based on the above mathematical model Veletsos (1974) has proposed the expression

$$T_e = T\sqrt{1 + \frac{k_s}{k_x} + \frac{k_s h^2}{k_\theta}} \tag{2.22}$$

Here T_e Equivalent Time period of the structure considering DSSI
T Fixed based time period of the structure
k_s Fixed based stiffness of the structure
k_x Lateral stiffness of the soil foundation system
k_θ Rocking stiffness of the soil foundation system

The foundation in this case is considered as a mass-less rigid footing resting on a homogenous elastic half-space. Eqn. 2.22 is an established expression in the realms of DSSI analysis and has been recommended by FEMA (2000), NEHRP etc in USA and is routinely used to cater to the DSSI effect of any structure under seismic load. For seismic analysis, usual procedure is to determine the effective time period based on Eqn. 2.22 when this would mostly show an elongation. This mostly results in an attenuation in seismic response, giving rise to the popular belief that for most cases DSSI reduces the response and thus designing a structure as fixed base model gives a conservative result (not to overlook the fact that the analysis thus conveniently becomes much simpler as the effect of soil is ignored from the analysis). Before we try to re- asses the above concept we modify Eqn. 2.22 as shown here after.

Squaring Eqn. 2.22 and considering $T = 2\pi\sqrt{M/K}$ we have Eqn. 2.23. Here k_e is equivalent stiffness of the structure considering the DSSI effect. Other nomenclatures are as shown in Fig. 2.3. Now considering $\omega = \sqrt{K/M}$, Eqn. 2.23 can further be expressed as Eqn. 2.24.

$$m_s/k_e = m_s/k_s + m_s/k_x + m_s h^2/k_\theta \tag{2.23}$$

$$\frac{1}{\omega_e^{\,2}} = \frac{1}{\omega_s^{\,2}} + \frac{1}{\omega_x^{\,2}} + \frac{1}{\omega_\theta^{\,2}} \tag{2.24}$$

Eqn. 2.24 has also been derived by Kramer (2003) for seismic force and Wolf (1985) who had further extended this to vertical mode also and has shown that DSSI coupling of this nature is valid for frames subjected to harmonic loads too. On careful examination of Eqn. 2.24 it shows that it is the classical expression derived by Dunkerley (1894) for determining the natural frequency of a vibrating shaft mounted with a number of discs. Wherein Dunkerley showed that on determination of the frequency of each of the individual discs in isolation the coupled frequency can be determined by the expression

$$\frac{1}{\omega_e^2} = \frac{1}{\omega_1^2} + \frac{1}{\omega_2^2} + \frac{1}{\omega_3^2} + ... + \frac{1}{\omega_n^2} \tag{2.25}$$

Here *n* is the number of discs mounted on the shaft. Based on Dunkerley's principle Fig. 2.2 can be broken down into three separate systems as shown in Fig. 2.3.

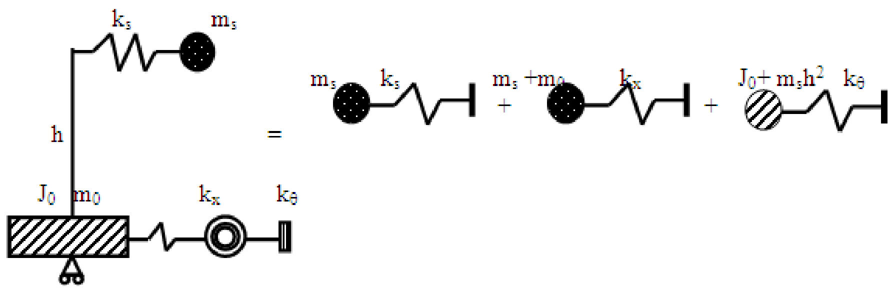

Figure 2.3. Formulation of chimney - considering inertial effect of the foundation

It is apparent from Fig. 2.3 that the coupled single degree oscillator can be broken down into three systems and on finding out the individual frequencies of each of the system the coupled natural frequency may be determined as per Eqn. 2.23. Multiplying Eqn. 2.23 by the factor *g*, acceleration due to gravity we have

$$\frac{m_s g}{k_e} = \frac{m_s g}{k_s} + \frac{m_s g}{k_x} + \frac{m_s h^2 g}{k_\theta} \tag{2.26}$$

$$\Rightarrow \delta_e = \delta_s + \delta_x + \delta_\theta \tag{2.27}$$

Eqn. 2.27 shows that displacement of three separate systems can be superimposed to arrive at the total displacement of the system provided of course the system is linear. Concepts as furnished in Fig. 2.3 and Eqn. 2.27 shall be used subsequently to derive a mathematical model and do a parametric study for DSSI under earthquake force. Extending the concept as shown in Fig. 2.2, mathematical model perceived for considering the inertial effect of foundation is as shown in Fig. 2.3. Usually the inertial effect of foundation is ignored in the analysis. Following Dunkerley's principle and Fig. 2.3 we have

$$\frac{1}{\omega_e^2} = \frac{m_s}{k_s} + \frac{m_0 + m_s}{k_x} + \frac{J_0 + m_s h^2}{k_\theta} \tag{2.28}$$

$$\frac{m_s + m_0}{k_e} = \frac{m_s}{k_s} + \frac{m_0 + m_s}{k_s} + \frac{J_0 + m_s h^2}{k_\theta} \tag{2.29}$$

2.2 Seismic Modal Analysis of Chimney

For seismic analysis, we again start with Eqn. 2.27 i.e. $\delta_e = \delta_s + \delta_x + \delta_\theta$. In terms of modal mass analysis this can be expressed as per Clough (2003) as

$$S_{de} = \frac{S_{as}}{\omega_s^2} + \frac{S_{ax}}{\omega_x^2} + \frac{S_{a\theta}}{\omega_\theta^2} \tag{2.30}$$

Thus in terms of code Eqn. 2.30 can be expressed as per IS 1893 (Part 1):2002 as given in Eqn. 2.31 which can be further expressed as Eqn. 2.32.

$$u_e = \kappa_i \left(\frac{ZI}{2R}\right)\left(\frac{S_{as}}{\omega_s{}^2} + \frac{S_{ax}}{\omega_x{}^2} + \frac{S_{a\theta}}{\omega_\theta{}^2}\right) \quad (2.31)$$

$$\Rightarrow u_e = \kappa_i \left(\frac{ZI}{2R}\right)\frac{m_s S_{as}}{k_s}\left[1 + \left(\frac{m_0 + m_s}{m_s}\right)\left(\frac{k_s}{k_x}\right)\left(\frac{S_{ax}}{S_{as}}\right) + \left(1 + \frac{J_0}{m_s h^2}\right)\left(\frac{S_{a\theta}}{S_{as}}\right)\left(\frac{k_s}{k_\theta}\right)h^2\right] \quad (2.32)$$

In Eqn. 2.32 it is observed that the terms within the second parenthesis are dimensionless terms, and when k_x and $k_\theta \rightarrow \infty$ it converges to the fixed base response of the chimney in fundamental mode. Thus from Eqn. 2.32 we can clearly say that due to soil-foundation effect the fixed base response gets modified by the factor within the parenthesis which is nothing but the Amplification Factor (*AF*) due to DSSI (Chowdhury & Singh, 2010) given in Eqn. 2.33. For multi degree freedom system the amplification factor can be expressed as (Chowdhury & Dasgupta, 2011) in Eqn. 2.34.

$$AF = \left[1 + \left(\frac{m_0 + m_s}{m_s}\right)\left(\frac{k_s}{k_x}\right)\left(\frac{S_{ax}}{S_{as}}\right) + \left(1 + \frac{J_0}{m_s h^2}\right)\left(\frac{S_{a\theta}}{S_{as}}\right)\left(\frac{k_s}{k_\theta}\right)h^2\right] \quad (2.33)$$

$$AF_i = [I] + \frac{S_{axi}}{S_{asi}}\frac{[\lambda si]}{[\lambda_{xi}]} + \frac{S_{a\theta i}}{S_{asi}}\frac{[\lambda si]}{[\lambda_{\theta i}]} \quad (2.34)$$

Here [*I*] an identity matrix having non diagonal terms as zero, S_{asi} modal spectral acceleration due to the eigen values [λ_ι] as obtained from Eqn. 2.19, [$\lambda_{x\iota}$]= $K_x / \left(m_0 + m_{s\,i}\right)$,and [$\lambda_{\theta\iota}$]= $K_\theta / \left(J_0 + m_{s\,i} h_i{}^2\right)$.

Here m_{si} diagonal term of the mass matrix as shown in Eq. 2.18 for the i^{th} mode

- m_0 mass of the circular raft foundation supporting the chimney
- J_0 mass moment of inertia of the circular raft ($0.25 m_0 r^2$)
- h_i centroid height of the mass per mode from foundation top
- K_x Frequency independent translational spring for the circular foundation as $8Gr/(2-\nu)$
- K_θ Frequency independent rocking spring for the circular foundation as $8Gr^3/[3(1-\nu)]$
- C_x Translational radiation damping for the circular foundation expressed as $[4.6/(2-\nu)]\rho V_s r^2$
- C_θ Translational radiation damping for the circular foundation expressed as $[0.4/(1-\nu)]\rho V_s r^4$
- S_{axi}, $S_{a\theta i}$ Spectral accelerations due to the eigen values [$\lambda_{x\iota}$],[$\lambda_{\theta\iota}$] scaled up/down due to the modal damping corresponding to C_x and C_θ
- G Dynamic Shear modulus of soil @ ρV_s^2
- ρ Mass density of soil
- V_s Shear wave velocity of soil
- h_i modal C.G. as $h_i = \left[\int_0^1 \xi f_i(\xi) \Big/ \int_0^1 f_i(\xi)\right] H$

2.3 Calculation of Dynamic Amplitude, Moments and Shears

In terms of response spectrum analysis displacement S_d is given by, $S_d = S_a / \omega^2$. In terms of codal formulation, considering DSSI we may express it as

$$S_{di} = \kappa_i \frac{ZI}{2R} AF_i \frac{S_{ai}}{\omega_i{}^2} \quad (2.35)$$

Where, κ_i modal participation factor and is given by $\sum_{i=1}^{n} m_i\phi_i / \sum_{i=1}^{n} m_i\phi_i^2$ (2.36)

For an element of length dz, above can be expressed as

$$\kappa_i = \int_0^1 f_i(\xi)d\xi \bigg/ \int_0^1 f_i(\xi)^2 d\xi \tag{2.37}$$

Integration of mass participation factor within limits 1 to 0 for the first three modes gives κ_i as 0.569, 0.427 and 0.308 and height of modal CG h_i as 0.726H, 0.209H and 0.128H respectively for first three modes. Now considering, $\beta = \frac{ZI}{2R}$, a code factor, we can write the time dependent function of displacement as

$$S_{di} = \kappa_i \beta [AF_i] \frac{S_{ai}}{\omega_i^2} [\phi]^T f_i(\xi) \tag{2.38}$$

From Eqn. 2.20 observing that cross modal Eigen vector terms having negligible effect, Eqn. 2.38 can be rewritten as Eqn. 2.39. The bending moment and shear force along stack including the effect of DSSI can be expressed as in Eqns. 2.40 and 2.41. Where I_z moment of inertia of the chimney cross section at a height z from bottom and could be varying in case the chimney is tapered.

$$S_{di} = \kappa_i \beta [AF_i] \frac{S_{ai} T_i^2}{4\pi^2} f_i(\xi) \tag{2.39}$$

$$M_{zi} = \frac{EI_z}{4\pi^2 H^2} \kappa_i \beta [AF_i] \mu_i^2 S_{ai} T_i^2 f_i''(\xi) \tag{2.40}$$

$$V_{zi} = \frac{EI_z}{4\pi^2 H^3} \kappa_i \beta [AF_i] \mu_i^3 S_{ai} T_i^2 f_i'''(\xi) \tag{2.41}$$

3. RESULTS AND DISCUSSIONS

To evaluate the proposed method a real life chimney having following data is analyzed and presented.

Design Data

Height of chimney 278.825*m*, Inside diameter of chimney at bottom 29.9*m*, Inside diameter at top 17 *m*; Shell thickness at bottom 800 *mm* (Constant up to 30 *m* height from base), Shell thickness at top 500mm (Varying linearly from 30*m* to 175*m* and then constant to the top); Seismic zone is III as per IS-1893(2002), zone factor (Z) 0.16, importance factor (I) 1.75, response reduction factor (R) 3.0; Grade of concrete M40 for superstructure and M30 for substructure, total weight of shell including *DL* of internal and external platform and 50% of Live load 460905 *kN*; Outside Radius of base raft 22.5*m*; bearing capacity of soil 550 kN/m^2; dynamic shear modulus of soil 590000 kN/m^2; density of soil 20kN/m^3, Poisson's ratio 0.4, damping ratio of concrete considered 3%, weight of foundation 145350 *kN*.

Based on above design data it is apparent that DSSI amplifies the dynamic response of tall chimneys and will have more profound effect as the strength of soil reduces. Irrespective of the chimney having taper or otherwise the geometry has little effect on the global time period of the system where the overall weight and the base moment of inertia finally govern(refer time period based on proposed method and numerical analysis carried out in STAAD considering variation in Table 3.1. The amplification factor due to DSSI though high for higher modes (compared to first mode) yet lower value of modal participation factor κ_i and reduced centroidal height (h_i) attenuates the higher mode response considerably. As such the fundamental mode finally dominates the overall response. Many codes (for instance IS-1893(2005) Part *IV*) recommends chimney to be analyzed based on first mode only which is not correct, for higher mode participation do have significant effect irrespective of the chimney being considered as a fixed base or otherwise. Finally for the present case the chimney has been resting on rock having allowable bearing capacity of 550 kN/m^2 and shear wave velocity V_s 537 m/s, which is slightly less than 600 m/s – "the cut off mark" when foundations can usually be considered as fixed. Even with such high value of V_s the amplification due to DSSI is significant and it would have been conceptually wrong to assume the foundation acting as a fixed based one. In a nutshell, this reflects the importance of considering dynamic soil structure interaction effect for analysis of such tall chimneys under seismic force. The bending moment and shear force diagram of the chimney considering the effect of DSSI and fixed based condition are shown in Figs. 3.1 to 3.4.

Table 3.1. Salient features of analysis

	Mode		
	1	2	3
Fixed Base time period Chimney (Proposed)	3.27	0.53	0.19
Fixed Base time period Chimney based on STAAD	3.37	0.67	0.24
Time period foundation (translation)	0.246	0.191	0.192
Time period foundation (rocking)	2.173	0.458	0.286
Damping ratio for chimney	3%	3%	3%
Translational damping ratio of raft**	35%	44%	44%
Rocking damping ratio of raft**	5.3%	6.6%	7.6%
Spectral accelerations of fixed base chimney, S_{as}	0.017g	0.11g	0.143g
Spectral accelerations of foundation translation, S_{ax}	0.05g	0.05g	0.05g
Spectral accelerations of foundation rocking, Sa_θ	0.02g	0.109g	0.092g
Amplification Factors (AF) for DSSI effect	1.545	1.808	2.841

** This includes 5% material damping of soil in translational & rocking mode

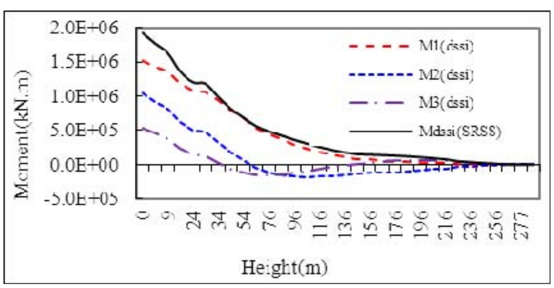

Figure 3.1. Bending Moment diagram of chimney for first three modes considering DSSI

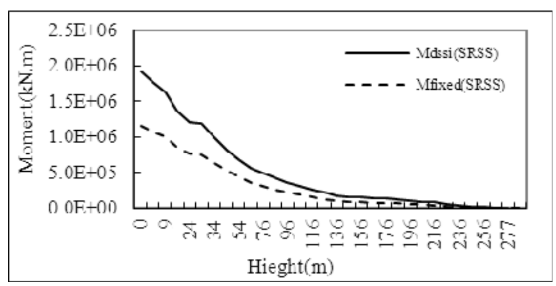

Figure 3.2. Bending Moment diagram of chimney for DSSI versus fixed

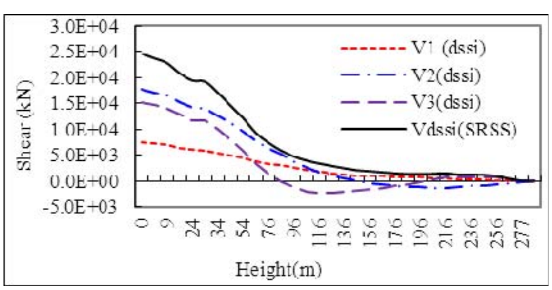

Figure 3.3. Shear force of chimney first three modes considering DSSI

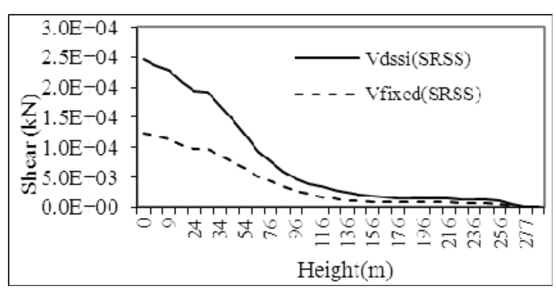

Figure 3.4. Shear force of chimney fixed base versus DSSI

4. CONCLUSION

A comprehensive analytical technique is proposed herein that can take care of DSSI effect as well as higher mode participation for dynamic analysis of tall chimneys, and can very well be developed in a spread sheet. It does not require an elaborate FEM software as an analytical tool. The proposed method also highlights some of the improvements that IS code committee may consider for amelioration of design engineering for such chimneys.

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